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Investigation of Performance of Skewed Reinforcing in Inverted-T Bridge Caps

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16. Abstract

Reinforced concrete inverted-T bridge caps (ITBCs) have been widely used in the bridges in the United States as they are aesthetically pleasing and offer a practical means to increase vertical clearance. Many of the ITBCs are skewed when two roads are not aligned perpendicularly and exceed the 45-degree angle of the construction requirements. The Texas ITBCs are designed using the traditional empirical procedures outlined in the TxDOT Bridge Design Manual (TxDOT BDM) LRFD that conform to the AASHTO LRFD (2014) Bridge Design Specifications. There are no precise calculation methods or guidelines given in the AASHTO LRFD (2014) or TxDOT BDM-LRFD (2015) to design skew ITBCs. However, any kind of improper detailing can cause poor placement of concrete and cracks within the concrete structure, which would reduce the load-carrying capacity and increase future maintenance costs. Faster and easier construction can be obtained if the skew transverse reinforcing throughout ITBCs is utilized. According to the results of lab tests, skewed transverse reinforcement will yield the same load capacity as the traditional design. In addition, using skewed transverse reinforcement throughout ITBCs will result in fewer cracks and smaller crack widths when compared to the traditional design. The Research Team selected three bent caps from an existing bridge to perform the preliminary FE analysis using ABAQUS. The analysis indicated that the critical locations to paste the strain gauges and attach LVDTs are the cantilever end faces of the bent caps. All the bent caps with skewed transverse reinforcing were observed to be safe under service and ultimate state loading. Three cases of reinforcement design for ITBCs are investigated. The parametric FE simulation of 96 specimens and the cost-benefit analysis results yielded these conclusions: (1) The skew transverse reinforcement (Case 1) achieves better structural performance compared to traditional transverse reinforcement (Case 2 and Case 3) with notably reduced construction cost. Therefore, the skewed transverse reinforcement can well be used for the design of skewed ITBCs. (2) The increase of the S Bar area notably enhances the stiffness and ultimate strength. In addition, the increase of the S Bar area also reduces the crack width. The increase of the S Bar area will contribute notably to the construction cost. Based on the parametric simulation results, the current design of the S bar area is adequate for structural safety and crack resistance. (3) The increase of the G Bar area notably reduces the maximum crack width with a negligible influence on the stiffness, ultimate strength, and construction cost. The current design of the G Bar (No. 7 Bars) is adequate for crack control. (4) When the concrete strength increases from 5 ksi to 7 ksi, the ultimate strength and the stiffness of ITBCs increase with reduced crack width. In addition, the influence of concrete strength on the construction cost is negligible. Updates from AASHTO (2010) to AASHTO (2017) are summarized in Appendix 1.

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ABSTRACT

In the past several decades, reinforced concrete inverted-T bridge caps (ITBCs) have been widely used in the bridges in Texas and the United States as they are aesthetically pleasing and offer a practical means to increase vertical clearance. Many of the ITBCs are skew when two roads are not aligned perpendicularly and exceed the angle of 45 degrees based on the construction requirements. The ITBCs in Texas are designed using the traditional empirical procedures outlined in the TxDOT Bridge Design Manual (TxDOT BDM) LRFD that conform to the AASHTO (American Association of State Highway and Transportation Officials) LRFD (2014) Bridge Design Specifications. There are no precise calculation methods or guidelines given in the AASHTO LRFD (2014) or TxDOT BDM-LRFD (2015) to design skew ITBCs. For a skew ITBC, the TxDOT Manual states that hanger and ledge reinforcement should be placed perpendicular to the centerline of the skew bent and the detailing of the skew ends of the bent should be done with a section of skewed stirrups and ledge reinforcements. Typically, the transition of perpendicular bars to the skew bars is carried out over column support, where the transverse reinforcement spacing is less critical. The designer of ITBC flares the bars out to match the skew angle while trying to maintain a minimum and maximum spacing based on the outcome of the design calculations. Such detailing of transverse reinforcements creates unequal spacing on both sides of the web, producing congestion of reinforcements on one side. The traditional method of flaring the transverse reinforcement out in skew ITBCs brings in significant complexity in design and during the construction process. In addition, the detailing of the transverse reinforcement has a profound influence on the overall shear capacity of the bent cap as well as the performance of the support ledge. Therefore, any kind of improper detailing can cause poor placement of concrete and cracks within the concrete structure, which would reduce the load-carrying capacity and increase future maintenance costs. Faster and easier construction can be obtained if the skew transverse reinforcing throughout ITBCs is utilized, and it can provide an alternative approach that will significantly reduce the design complexities and construction period. According to the results of lab tests (TxDOT Project 0-6905), using skewed transverse reinforcement throughout ITBCs will have the same load capacity as the traditional design. In addition, it is found that using skewed transverse reinforcement throughout ITBCs will have less number of cracks and smaller crack widths when compared to the traditional design.

Skewed transverse reinforcement has been applied to the design of ITBCs in TxDOT bridges because of its advantages. The Research Team (RT) selected Bent Cap 2, Bent Cap 6 and Bent Cap 7 of the bridge on Donigan Road over IH 10 to perform the preliminary FE analysis using ABAQUS. Once the overall structural behavior of actual ITBCs with skewed transverse reinforcement is better understood, the critical loading patterns during the load tests and crucial strain gage locations can be determined. Later, the developed numerical models will be calibrated against the field test results for the numerical simulation, considering unexplored parameters. From the preliminary FE analysis, it was observed that the critical locations to paste the strain gauges and attach LVDTs are the cantilever end faces of the bent caps. Moreover, it was also observed that all the bent caps with skewed transverse reinforcing are safe under service and ultimate state loading.

Due to the construction delays, a task (named Task 9a) is added and completed. In Task 9a, three cases of reinforcement design for ITBCs are investigated to cover the majority of the design detailing in Texas bridges. Based on the parametric FE simulation of 96 specimens and the cost-benefit analysis results, the

conclusions are summarized as follows: (1) The skew transverse reinforcement (Case 1) achieves better structural performance compared to traditional transverse reinforcement (Case 2 and Case 3) with notably reduced construction cost. Therefore, the skewed transverse reinforcement can well be used for the design of skewed ITBCs. (2) The increase of the S Bar area notably enhances the stiffness and ultimate strength. In addition, the increase of the S Bar area also reduces the crack width. The increase of the S Bar area will contribute notably to the construction cost. Based on the parametric simulation results, the current design of the S bar area is adequate for structural safety and crack resistance. (3) The increase of the G Bar area notably reduces the maximum crack width with a negligible influence on the stiffness, ultimate strength, and construction cost. The current design of the G Bar (No. 7 Bars) is adequate for crack control. (4) When the concrete strength increases from 5 ksi to 7 ksi, the ultimate strength and the stiffness of ITBCs increase with reduced crack width. In addition, the influence of concrete strength on the construction cost is negligible.

With skewed transverse reinforcement, the RT presents four design examples of ITBCs with skew angles of 0, 30, 45, and 60 degrees by using AASHTO (2017) and TxDOT (2020). The design examples are based on the TxDOT Inverted Tee Bent Cap Design Example (2010), which follows the AASHTO LRFD Bridge Design Specifications, 5th Ed. (2010), as prescribed by TxDOT Bridge Design Manual -LRFD (May 2009). The design steps of skewed ITBCs are also illustrated. In addition, the updates from AASHTO (2010) to AASHTO (2017) are also summarized in Appendix 1 of R1A, including the section number, the equations, and the tables, which are required to design an ITBC.

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CHAPTER 1: INTRODUCTION

1.1 PROJECT OVERVIEW

The Inverted-T Bridge Caps (ITBCs) are widely adopted in many bridges in Texas and all over the United States to reduce the beam height. In addition to the increased vertical clearance of the bridges, the ITBCs minimize the visible size of transverse bent caps and presents an aesthetically pleasing design. Another significant advantage of the ITBC system is its usage of precast beams, which can be quickly assembled on-site without any extra formwork (Synder et al., 2011). The precast components also enable higher quality and reduced construction periods. Figure 1.1 shows the component details and reinforcement details of the ITBCs. Unlike traditional rectangular bridge girders, the cross-section of the ITBC consists of the web and the ledge. The web is the primary section to transfer the shear forces, while the ledge serves as brackets to transfer girder load to the web. In order to transfer the vertical load, two types of reinforcements have been introduced in the ITBC, including the web shear reinforcements and the ledge reinforcements. The web shear reinforcements are web vertical stirrups that transfer the ledge load from the bottom of the web to the top of the web, and the ledge reinforcements are horizontal stirrups that help the cantilevered ledge to resist flexural tension forces in the transverse direction.

(a) Components of ITBC (Zhou et al., 2020) **(a) Reinforcing Details (Roy, 2018)**

Figure 1.1 Design Detail of ITBC

The skewed ITBCs serve as beam elements with concentrated loads applied to the bottom ledge (Coletti et al., 2011). Unlike traditional top-loaded beam structure, the force transfer mechanism of the skewed ITBC is as follows: (1) the loads are transferred from the ledge to the web in the transverse direction through the vertical hanger reinforcements; (2) the loads are transferred into the web section and reach the supports in the longitudinal direction (Zhou et al., 2020). During this process, the unequal loading position on the cantilevered skewed ledge may induce a three-dimensional flexural-shear-torsional combined load and complex cracking problem. Several experimental studies were conducted on the ITBC. Furlong et al. (1971) first investigated and demonstrated the shear and anchorage behavior of the ITBC reinforcements and provided suggestions for the design procedures of the ITBC specimens. Mirza and Furlong (1983a; 1983b; 1985) first investigated the failure mechanisms and serviceability behavior of the reinforced concrete ITBC by testing 27 simply supported specimens at a scale ratio of 1/3. Six typical failure mechanisms were reported as (1) flexural failure, (2) flexural shear failure, (3) torsional failure, (4) hanger failure of shear reinforcement, (5) flange punching failure, and (6) flange shear friction failure. The first three failures are the main control modes, while the others are premature failures and should be avoided during the design. Zhu and Hsu (2003) investigated the crack control of ITBCs and predicted the diagonal crack widths observed in tests based on a two-dimensional analytical model. Ambare and Peterman (2006) performed a finite element (FE) simulation of inverted T bridge systems to check the effects of live loads distribution on the behavior of the inverted T bridge system. The results were also compared with AASHTO LRFD (2014) and AASHTO Standard Specifications (2002), which indicated that loading distribution patterns have a direct effect on the bridge system, and the code method was more conservative than the FE method.

In design practice, many bridges have to be skewed according to the landscaping or construction requirements. Some of the ITBCs in practice have the skew-angle over 45° based on the angle of the bridges crossing roadways, waterways, and railways. The ITBCs in Texas are widely designed using the traditional empirical procedures outlined in the TxDOT (Texas Department of Transportation) Bridge Design Manual-LRFD that conforms to the AASHTO LRFD 2014 Bridge Design Specifications. There are no precise calculation methods or guidelines given in the AASHTO LRFD or TxDOT Bridge Design Manual-LRFD to design skew ITBCs. The TxDOT Bridge Design Manual states only that hanger and ledge reinforcement should be placed perpendicular to the centerline of the skew bent. The detailing of the skew ends of the bent should be done with a section of skew stirrups and ledge reinforcements. Typically, the transition of straight bars to the skew bars is carried out over the column support, where the transverse reinforcement spacing is less critical. The designer of the ITBC flares the bars out to match the skew angle while trying to maintain a minimum and maximum spacing based on the outcome of the design calculations. Such detailing of transverse reinforcement in skew ITBCs brings complexity to the design and construction process. This transverse reinforcement has a profound influence on the shear capacity of the bent cap and the performance of the support ledge. Therefore, any kind of improper detailing can cause poor placement of concrete and cracks within the concrete structure, which may reduce the load-carrying capacity and increase future maintenance costs. In addition, the provision of end face reinforcement to control the displacement at the free end of the ITBCs is necessary. Faster and easier construction can be obtained if skew transverse reinforcing steel is utilized, and it can provide an alternative approach that will significantly reduce the design complexities and construction period.

To understand the structural behavior of skewed ITBCs, Project 0-6905 started in 2016 with the following eight tasks included:

- Task 1: Literature Review
- Task 2: Parametric Study
- Task 3: Examination of Diverse Design Methodology
- Task 4: Design, Fabrication, and Testing of 1/2-Scale Skewed Inverted-T Bent Caps
- Task 5: Analysis of Task 4 Experimental Results
- Task 6: Advanced Numerical Analysis
- Task 7: Development of Details for Skewed Reinforcing Steel
- Task 8: Preparation of Final Report & Close Out Meeting

According to the results of lab tests (TxDOT Project 0-6905), using skewed transverse reinforcement throughout ITBCs will have the same load capacity as the traditional design. In addition, it is found that using skewed transverse reinforcement throughout ITBCs will have less number of cracks and smaller crack widths when compared to the traditional design. Because of the advantages of skewed transverse reinforcement, skewed transverse reinforcement has been applied to the design of ITBCs in TxDOT bridges. The Research Team (RT) has selected Bent Cap 2, Bent Cap 6 and Bent Cap 7 of the bridge on Donigan Road over IH 10 to perform the preliminary FE analysis using ABAQUS. After these eight tasks were completed and the final report was submitted, the project was extended in February 2019 with the following tasks:

- Task 9: Development of Preliminary Finite Element (FE) Models of the Significant ITBCs
- Task 10: Instrumentation of the Significant Skewed ITBCs to Conduct the Load Test
- Task 11: Analysis of Experimental Results
- Task 12: Calibration of the FE Models Developed in Task 9 with the Measured Load Test Data
- Task 13: Design Recommendations

Due to the construction delays, after Task 9, a new task was added to improve the knowledge on design methods and reinforcement detailing in the design of the skewed ITBCs:

- Task 9a: Development of Preliminary FE Models of the Significant ITBCs

Because of the environmental issues in the construction site, the project 0-6905 was decided to be on pause by the end of October 2020. Starting from Task 10, the tasks will be completed under a new project when the site becomes available.

From the experimental and analytical studies in Tasks 4 and 6, the following observations were made:

- The peak load-carrying capacity of the ITBC with skew reinforcing is almost equal to the traditional one.
- The number of cracks observed is fewer in the case of the ITBC with skew reinforcing; the observed maximum crack width is smaller in the case of skew reinforcing.
- The design and construction complexities can be significantly reduced, and a faster and easier construction process can be achieved when skew reinforcing is used.

Based on the above observations, implementation of the skew transverse reinforcing in inverted-T bridge caps was suggested; hence the project extension was proposed to implement the research findings to the actual full scale skewed ITBC in the bridge system. For the implementation task (Task 10), a seven-span bridge is proposed, which is under construction on Donigan Road over IH 10 near Brookshire in Waller County. The primary reasons for selecting this bridge for instrumentations and load tests are:

- Proximity to the UH research lab
- In agreement with the TxDOT project team
- Easy accessibility to bent caps and field equipment (lower bent heights)
- Limited traffic control required to instrument the bent caps and perform controlled load tests

A controlled load test will be performed on this bridge to investigate the performance of the skew ITBCs with skew reinforcing. Three bent caps are selected for instrumentation and load tests based on the severity and criticality of the loading condition. The primary features of these three bent caps are provided in Table 1-1. Strain gauges and other necessary sensors will be attached at the critical locations of rebars during the fabrication stage of the selected bent caps based on the analytical results in Task 9. Once the bridge construction is completed, the controlled load tests will be carried out based on standard procedure. During the load tests, transverse rebar stresses and bent deflections will be measured under known loading conditions. A wireless data acquisition system will be developed and used to monitor and record the data as it requires less on-site setup time than traditional wired systems and significantly minimizes traffic control time and disruptions to traffic. Each load test will continue for 5-20 minutes. In Task 9, the Research Team (RT) performed the preliminary FE analysis of the selected skewed inverted-T bridge caps using ABAQUS to understand the overall structural behavior of skewed reinforcement in actual large-scale ITBCs and to determine critical loading patterns during the load tests and crucial strain gauge locations. Later, the developed numerical models will be calibrated against the field test results for the numerical simulation assigned in Task 12, considering unexplored parameters. Based on the literature review, the FE simulation and the cost-benefit analysis for the ITBCs have not been reported (Bhargava 2009). The parametric FE modeling and cost estimation can be effectively used in the engineering design (Yazdani et al. 2017). The scope of the added Task 9a will significantly leverage the impact of this project and solve the dearth of reliable design methods and reinforcement detailing in the design of the skewed ITBCs.

Table 1.1. Details of the Bent Caps for the Instrumentation

Description	Bent 2	Bent 6	Bent 7
Skew angle	43^0	33^0	33^0
Loading condition	unsymmetrical dead loading	symmetrical dead loading	unsymmetrical dead loading
Elevation from ground level	18 ft	19 ft	19 ft
Span length	100 ft (back station) / 135 ft (forward station)	125 ft (back station) / 135 ft (forward station)	135 ft (back station) / 115 ft (forward station)
No. of girders	9 (back station) / 15 (forward station)	11 (back station) / 15 (forward station)	15 (back station) / 9 (forward station)

1.2 PROJECT OBJECTIVES

The objectives of this project are summarized as follows:

1. To understand the overall structural behavior of skewed reinforcement in actual large-scale ITBCs and to determine critical loading patterns during the load tests and crucial strain gage locations.
2. To compare and evaluate the structural performance of skew transverse reinforcement with traditional reinforcement in ITBCs regarding strength criteria.
3. To compare and evaluate the structural performance of skew transverse reinforcement with traditional transverse reinforcement in ITBCs in terms of serviceability criteria considering the cracking widths and stiffness.
4. To compare and evaluate the structural performance of skewed ITBCs with end bars and skewed ITBCs without end bars.

5. To compare and evaluate the cost-benefit analysis of skew transverse reinforcement with traditional reinforcement in ITBCs regarding design and construction cost.
6. The ITBC test specimens will be modeled in finite element software ABAQUS.
7. The general design recommendations and changes to the TxDOT practice to design skewed reinforcements in ITBCs will be proposed.

1.3 PROJECT SIGNIFICANCE

This project will provide the following benefits to the TxDOT and other stakeholders:

1. By replacing a traditional transverse reinforcement with a skewed one, proper placement of concrete and less complex fabrication of reinforcement could be ensured. As a result, the construction costs involved would be reduced.
2. Skewed reinforcement would reduce the congestion in the skew region of the bent cap. As a result, proper placement of concrete could be achieved. It would reduce the complexity in detailing the skew region of the bent cap by providing uniform spacing and the same size reinforcing bars. Therefore, lesser working hours and laborers would be required for the fabrication/construction of the ITBC with skewed reinforcement.
3. So far, no significant research has been undertaken to study the performance of skew transverse reinforcement in ITBC. A lack of experimental research has thwarted the use of skew reinforcing. Therefore, there are no specific design guidelines for the design of skew reinforcements in inverted-T bent caps, which makes the design unreliable with increased risks of failure. By providing proper design guidelines for different skew angles, high levels of lifetime uncertainties and risks of failure could be prevented. The skew reinforcement approach could reduce the replacement cost and increase the reliability, thereby benefiting the TxDOT and other stakeholders financially.

1.4 ORGANIZATION

This report is divided into five chapters. Chapter 1 introduces an overview and the objectives of the research in addition to an outline of this report. Chapter 2 presents the analytical results of the three skewed ITBCs (Task 9), that are shown in Table 1.1, to understand the overall structural behavior of skew reinforcement in actual ITBCs. Chapter 3 shows the cases of parametric study and finite element analysis results (Task 9a) for different design parameters to compare the cost-benefit analysis results of skew transverse reinforcement with those of traditional transverse reinforcement. Following the finite element analysis results, the design recommendations for skewed ITBCs are presented in Chapter 4. Moreover, to explain the step-by-step design procedures, four skewed ITBCs design examples are presented. All findings and conclusions of the research program are summarized in Chapter 5.

CHAPTER 2: DEVELOPMENT OF PRELIMINARY FINITE ELEMENT MODELS OF THE SIGNIFICANT ITBCs

2.1 INTRODUCTION

In this chapter, the preliminary finite element (FE) analysis of the selected skew inverted-T bridge caps is performed using ABAQUS to understand the overall structural behavior of skew reinforcement in actual large-scale ITBCs and to determine critical loading patterns during the load tests and crucial strain gauge locations. As significant ITBCs, Bent Cap 2, Bent Cap 6, and Bent Cap 7 of a seven-span bridge, which is under construction on Donigan Road over IH 10 near Brookshire in Waller County, are selected. The primary features of these three bent caps are provided in Table 1.1. Figure 2.1 shows the Google Map image of the proposed new bridge location and the existing old bridge.

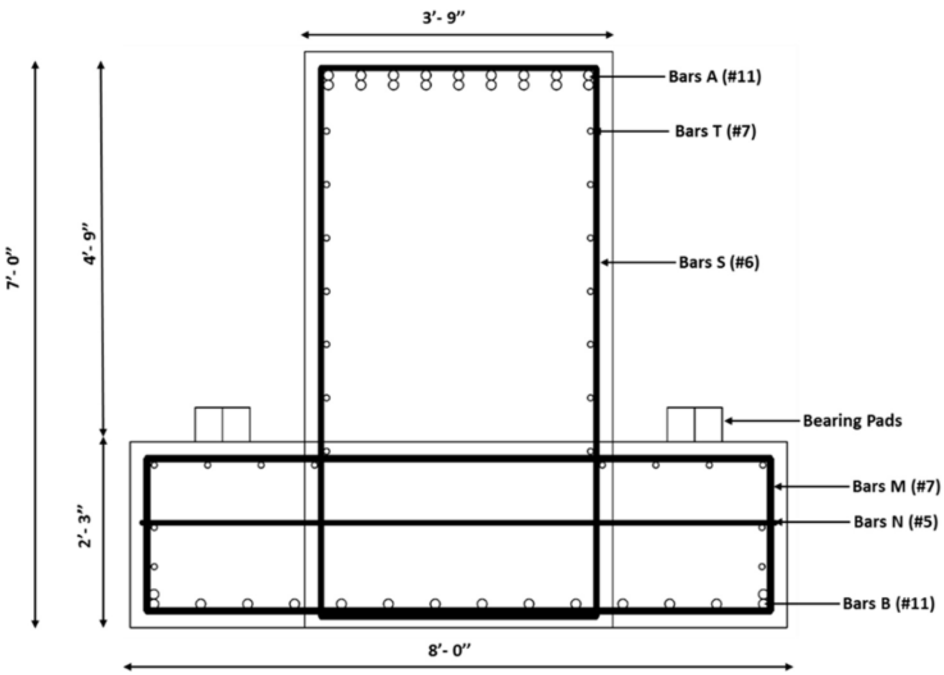
(**a**) Proposed new bridge location

(b) Existing old skewed bridge

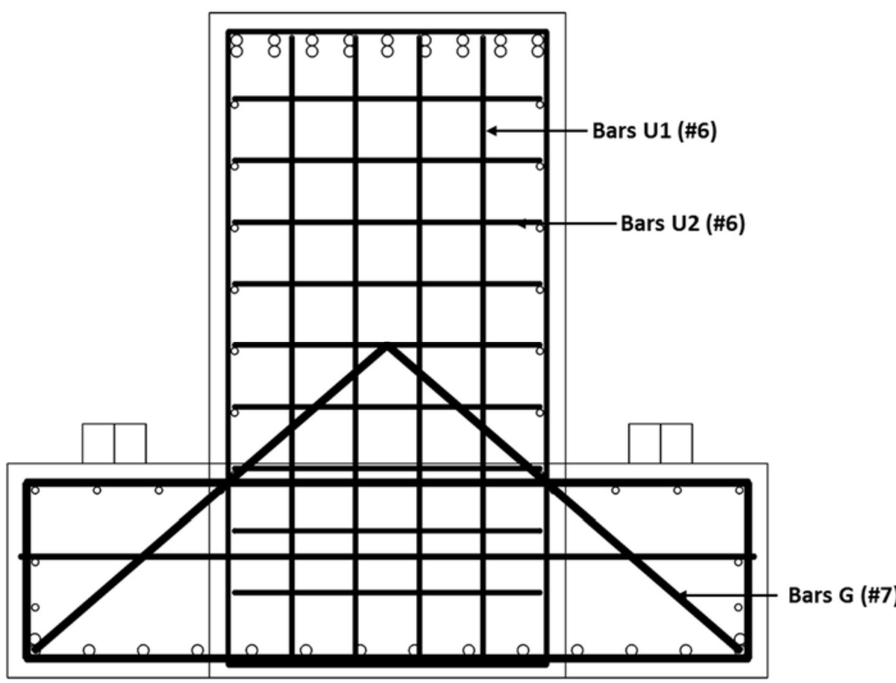
Figure 2.1 Proposed bridge on Donigan Road over IH 10 near Brookshire in Waller County

2.2 FINITE ELEMENT MODELING OF BENT CAPS IN ABAQUS

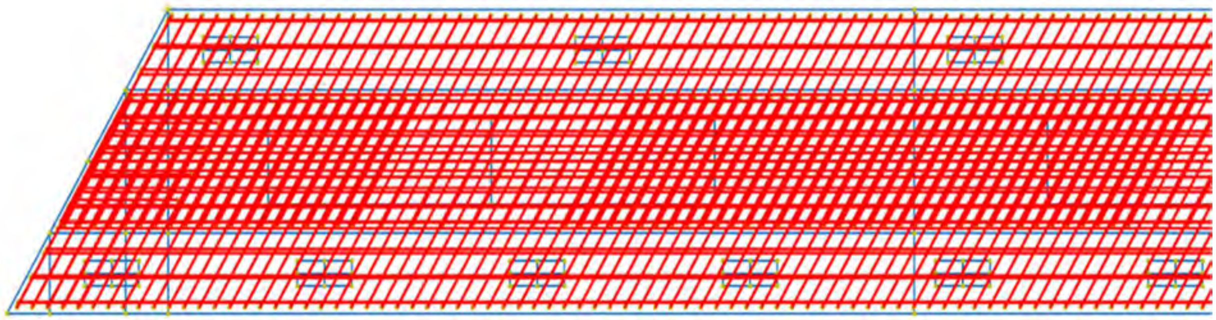
The finite element models of the actual ITBCs were developed using ABAQUS (2014). Figure 2.2(a) and Figure 2.2(b) show the typical cross-sectional view with reinforcing details of all the bent caps at the inner and end face locations, respectively. A partial plan view of the bent caps showing the transverse rebar details is shown in Figure 2.2(c). The 3D FE model of the bent caps depicting a cross-section view at the end face is shown in Figure 2.3. The typical FE mesh of a partial bent cap is provided in Figure 2.4. The concrete of the ITBCs is modeled using an eight-node, reduced integration, hourglass control solid element (C3D8R). A two-node linear three-dimensional (3-D) truss element (T3D2) was used to model the reinforcement because it is only subjected to axial force. The fours square rigid supports representing columns under the bridge bent cap were fixed at the bottom faces. There is a total of 24, 26 and 24 loading pads tied on top of the ledges of Bent Cap 2, Bent Cap 6, and Bent Cap 7, respectively. The superstructure loads from bridge girders are transferred to the bridge bent caps through these loading pads. The analysis was performed with two loading cases. The first one is the service load, which includes dead load and live load with the load combination factor equal to one. The second loading case is the factor load.



(a) Typical Bent Cap Cross Section

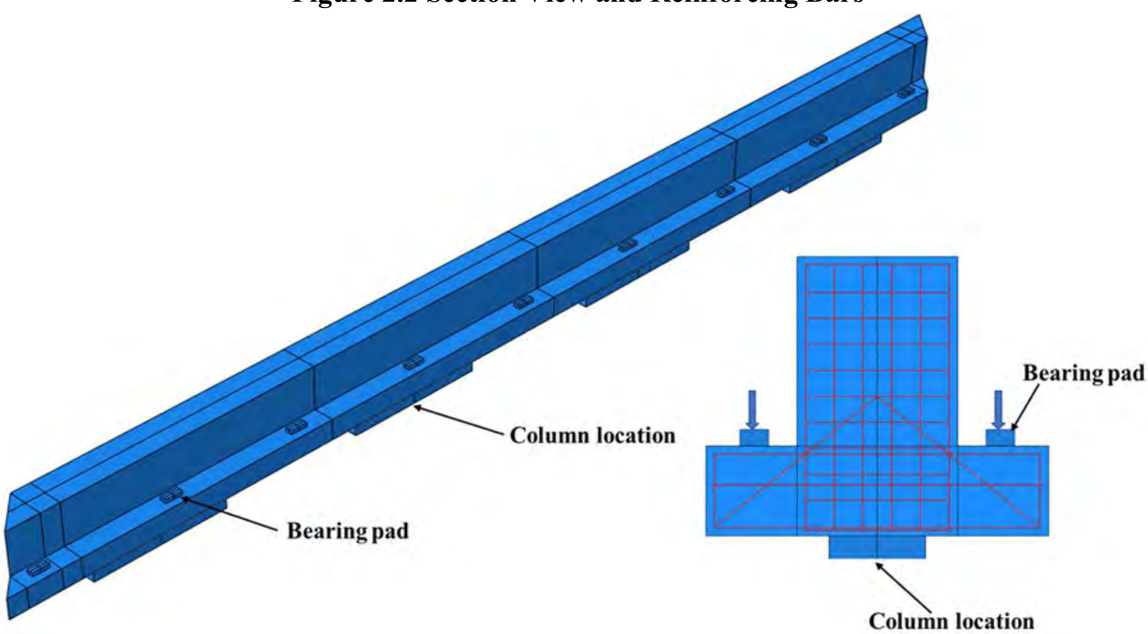


(b) Typical Bent Cap Cross Section at End Face

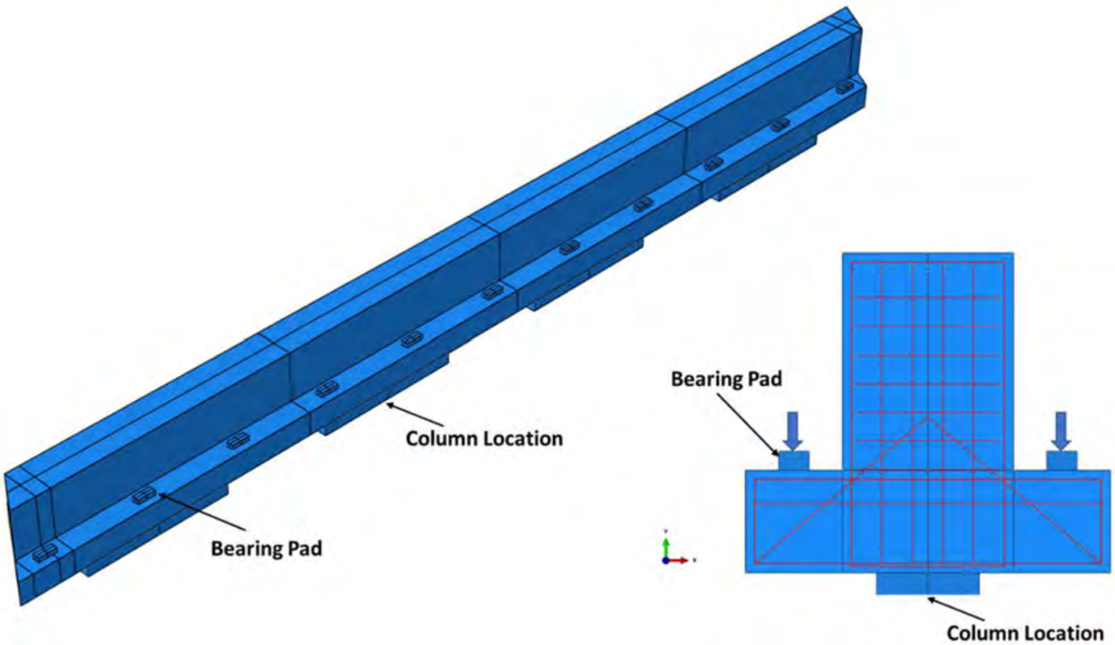


(c) Plan View of Reinforcing in ABAQUS

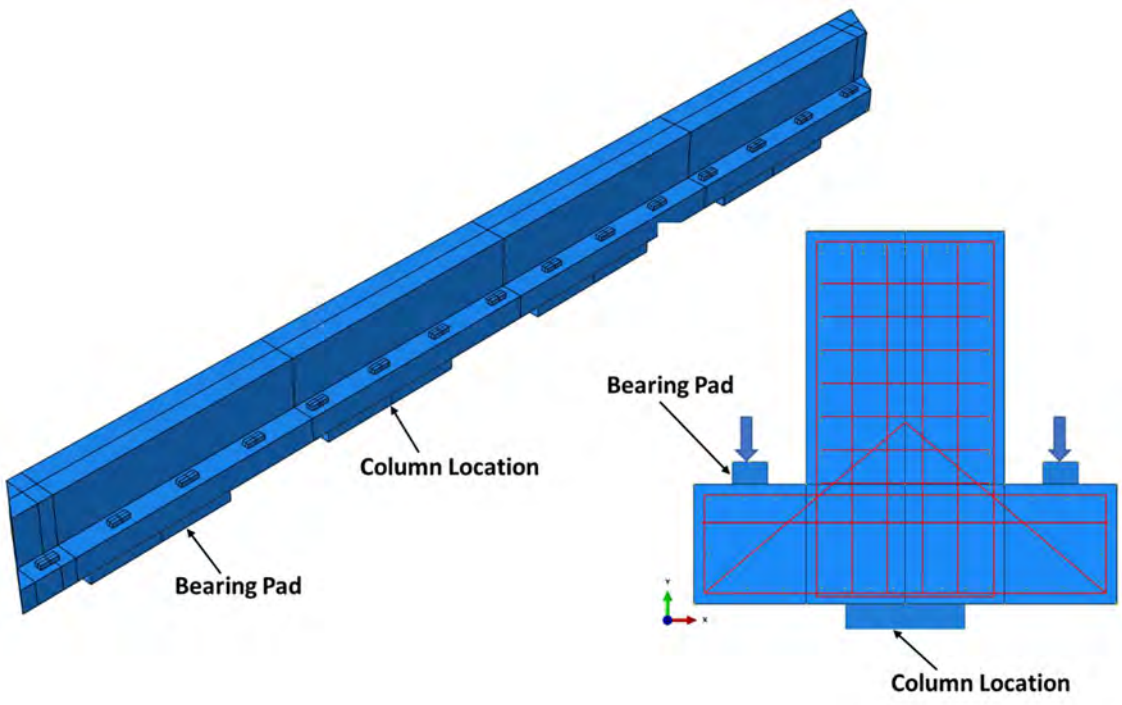
Figure 2.2 Section View and Reinforcing Bars



(a) Finite Element Model of Bent Cap 2 with Skew Angle 43^0



(b) Finite Element Model of Bent Cap 6 with Skew Angle 33^0



(c) Finite Element Model of Bent Cap 7 with Skew Angle 33^0

Figure 2.3 3D FE Model of Bent Caps in ABAQUS

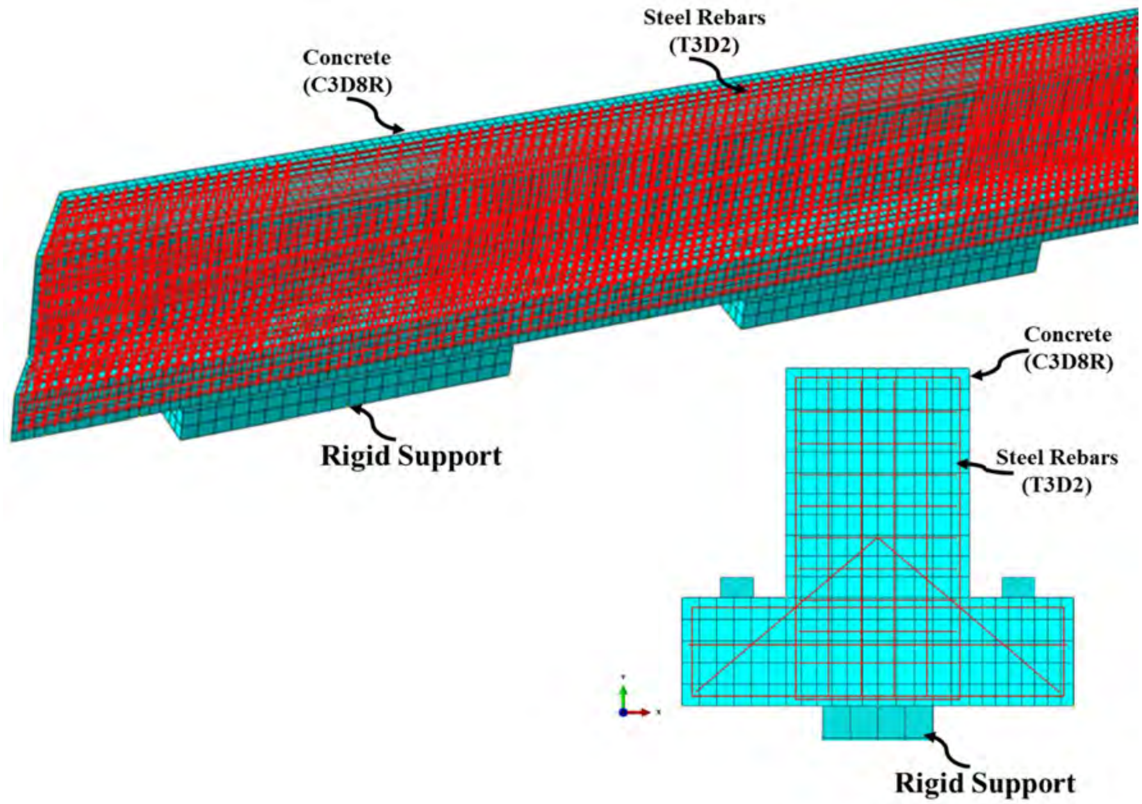


Figure 2.4 Partial 3D Finite Element Mesh of a Bent Cap
(C3D8R Solid Element for Concrete and T3D2 Truss Element for Reinforcements)

2.3 MATERIAL MODELS

The Concrete Damaged Plasticity (CDP) model was used as the constitutive model of concrete in the FEM model (Lee and Fenves, 1998). The CDP model requires the definition of uniaxial behavior in compression and tension. The stress-strain curves of concrete considered in the constitutive model are adopted from the book "Unified Theory of Concrete Structures" by Hsu and Mo (2010).

The uniaxial compression stress-strain behavior of concrete can be defined using the parabolic stress-strain model as shown in Figure 2.5. Equation 2-1 is used to develop the compression stress-strain curve.

$$\sigma_c = f_c' \left[\frac{2\varepsilon_c}{\varepsilon_0} - \left(\frac{\varepsilon_c}{\varepsilon_0}\right)^2\right] \text{ (psi)} \qquad \text{(Eq. 2-1)}$$

In ABAQUS, the model of concrete (Lubliner et al., 1989) requires the definitions of initial elastic modulus E_c and Poisson ratio ν. The initial elastic modulus E_c can be calculated using the AASHTO empirical equation (AASHTO 2014):

$$E_c = 57000\sqrt{f_c'} \text{ (psi)} \qquad \text{(Eq. 2-2)}$$

The Poisson ratio of concrete under uniaxial compressive stress ranges from about 0.15 to 0.22, with a representative value of 0.19 or 0.2 (AASHTO). In this report, the Poisson ratio of concrete is assumed to be ν = 0.2.

The uniaxial tension stress-strain behavior of smeared (average) concrete was proposed by Belarbi and Hsu (1994), as shown in Figure 2.5. Equations 2-3 and 2-4 are used to develop the tensile stress-strain curve.

Ascending branch:

$$\sigma_c = E_c\varepsilon_c \varepsilon_c \leq \varepsilon_{cr} \quad \text{(Eq. 2-3)}$$

Descending branch:

$$\sigma_c = f_{cr}\left(\frac{\varepsilon_{cr}}{\varepsilon_c}\right)^{0.4} \varepsilon_c > \varepsilon_{cr} \quad \text{(Eq. 2-4)}$$

where E_c = the elastic modulus of concrete, ε_{cr} = the cracking strain of concrete taken as 0.00008, and f_{cr} = the cracking stress of concrete taken as $0.00008E_c$.

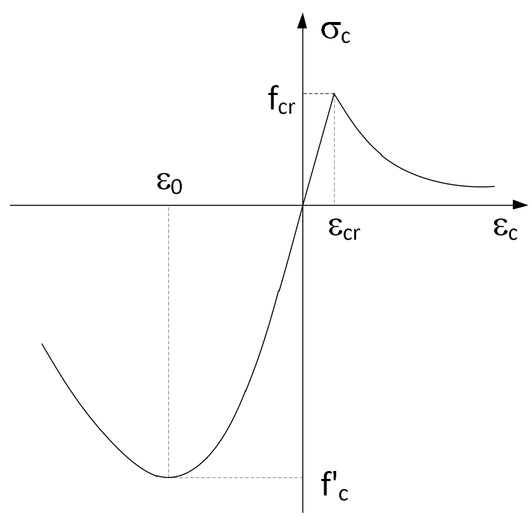


Figure 2.5 Stress-Strain Curves of Concrete in Tension and Compression

The stress-strain curve of the reinforcing bar is assumed to be elastic and perfectly plastic, as shown in Figure 2.6. In the ABAQUS program, the bond-slip effect between concrete and steel is not considered. In order to properly model the steel bars, the cross-section area, position, and orientation of each steel bar within the concrete element need to be specified.

Elastic branch:

$$f_s = E_s\varepsilon_s \;\; \varepsilon_s \leq \varepsilon_y \quad \text{(Eq. 2-5)}$$

Plastic branch:

$$f_s = f_y \varepsilon_s > \varepsilon_y \quad \text{(Eq. 2-6)}$$

where E_s = the elastic modulus of steel taken as 29000 ksi and ε_y =the yielding strain of steel.

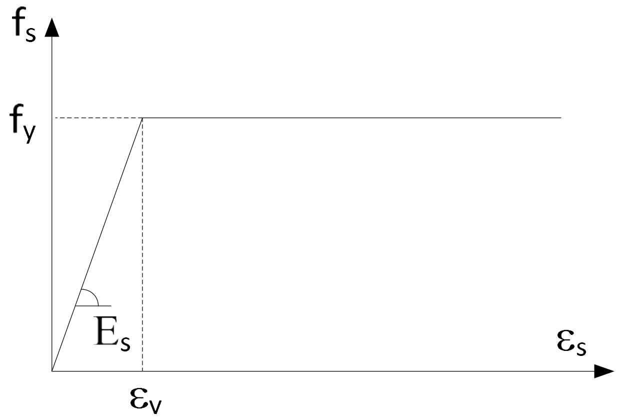


Figure 2.6 Stress-Strain Curve of Mild Steel

The details of the material parameters of the concrete damaged plasticity model for full-scale bent caps are listed in Table 2.1.

Table 2.1 Material Parameters for the Concrete Damaged Plasticity Model

Specimen designation	Young's modulus (ksi)	Poisson's ratio	Compressive strength (ksi)	Tensile strength (ksi)	Dilation angle (°)	Flow potential eccentricity	*K*
Bent 2	4031	0.2	5.0	0.325	31	0.1	0.67
Bent 6	4031	0.2	5.0	0.325	31	0.1	0.67
Bent 7	4031	0.2	5.0	0.325	31	0.1	0.67

2.4 3D FINITE ELEMENT RESULTS OF BENT CAPS

The analysis is performed for service load, which includes dead load and live load with the load combination factor equal to one. The ultimate load (strength limit state 1) is calculated by multiplying a factor of 1.25 with dead load, 1.75 with live load and 1.5 with overlay.

2.4.1 Stresses in Transverse Rebars at Service Load

The service loads for each of the interior girder locations and all the exterior girder locations of each bent cap are described in Table 2.2. Figure 2.7,Figure 2.8, and Figure 2.9 illustrate the contour plot of tensile stresses in the transverse reinforcement of skewed Bent Caps 2, 6, and 7, respectively, corresponding to skew angles of 43°, 33°, and 33°. As shown in Figure 2.7 the maximum tensile stress in the rebars of Bent Cap 2 is 9.08 ksi, which is within the stress limit prescribed by TxDOT and occurs in the transverse rebars at the end face (marked in the circle). Hence, the bent cap is safe in the service load condition. Similarly, as shown in Figure 2.8 and Figure 2.9, the maximum tensile stress in the rebars of Bent Cap 6 and Bent Cap 7 is 7.56 ksi and 9.73 ksi, respectively. The rebar stresses in Bent Cap 7 are higher than those in Bent Caps 2 and 6, due to the higher service load. It is evident that the stresses in rebars of all the bent caps under the service load are low and hence safe.

Table 2.2 Service Loading for Bent Caps

Bent	Service Load at Interior Bearing Pads (kips)	Service Load at Exterior Bearing Pads (kips)
Bent 2	222.48	240.19
Bent 6	226.64	238.86
Bent 7	244.52	258.00

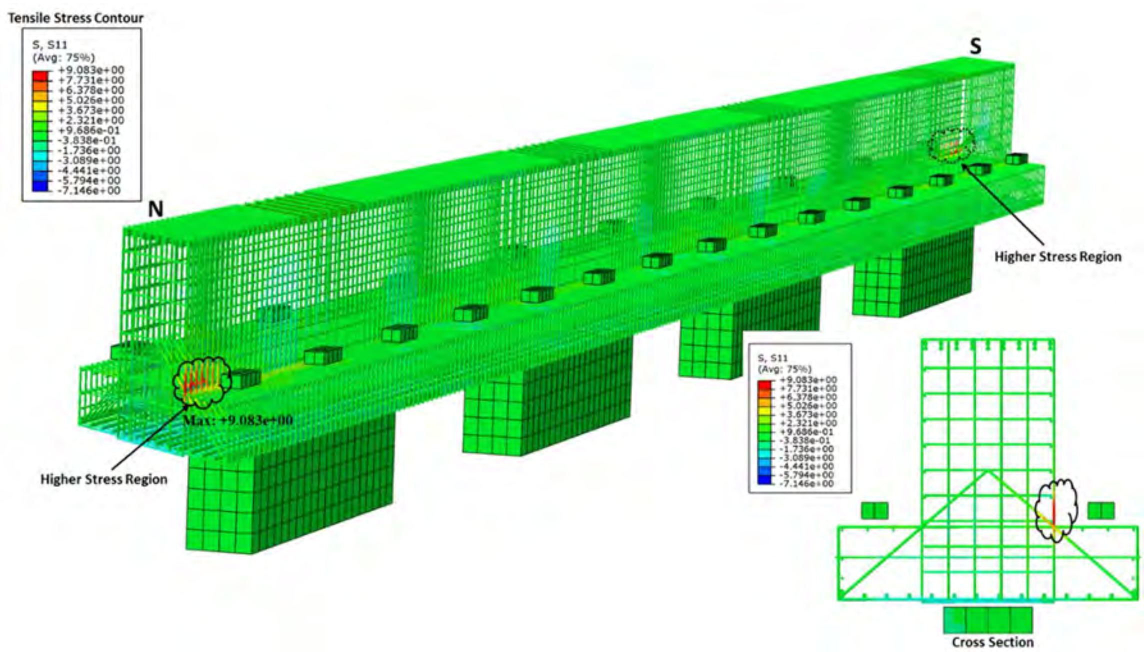


Figure 2.7 Tensile Stress Contour at Service Load of Bent Cap 2

[S11 = Tensile stresses in ksi in Rebars]

[Top (Red in color): Maximum stress, Bottom (Blue in color): Minimum stress]

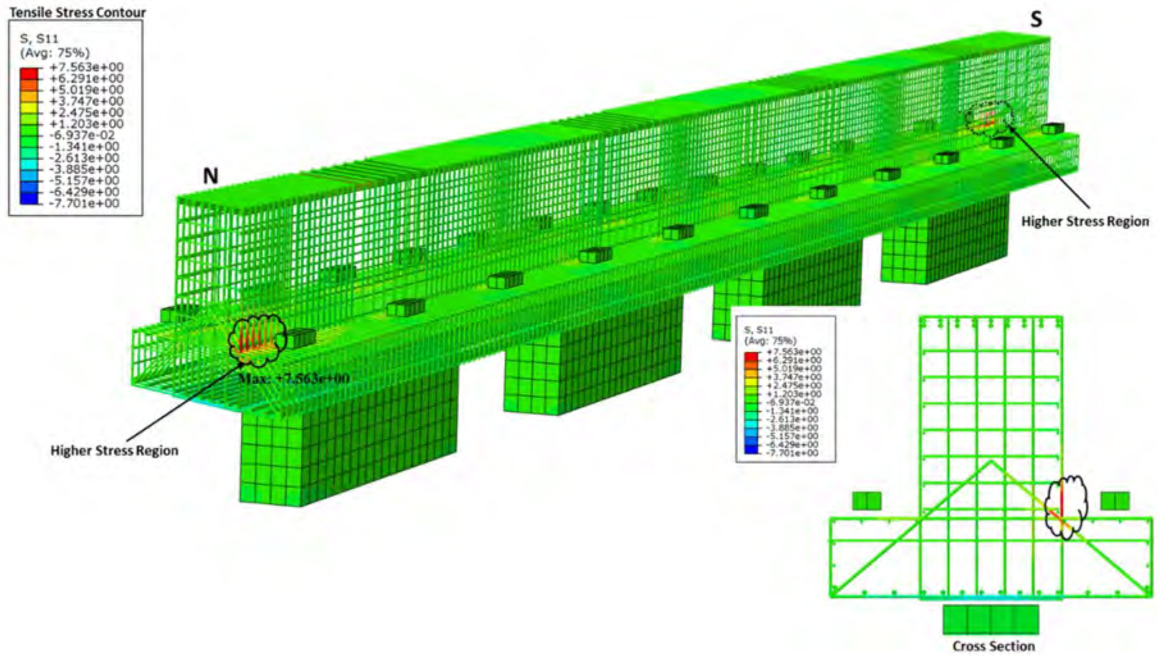


Figure 2.8 Tensile Stress Contour at Service Load of Bent Cap 6

[S11 = Tensile stresses in ksi in Rebars]

[Top (Red in color): Maximum stress, Bottom (Blue in color): Minimum stress]

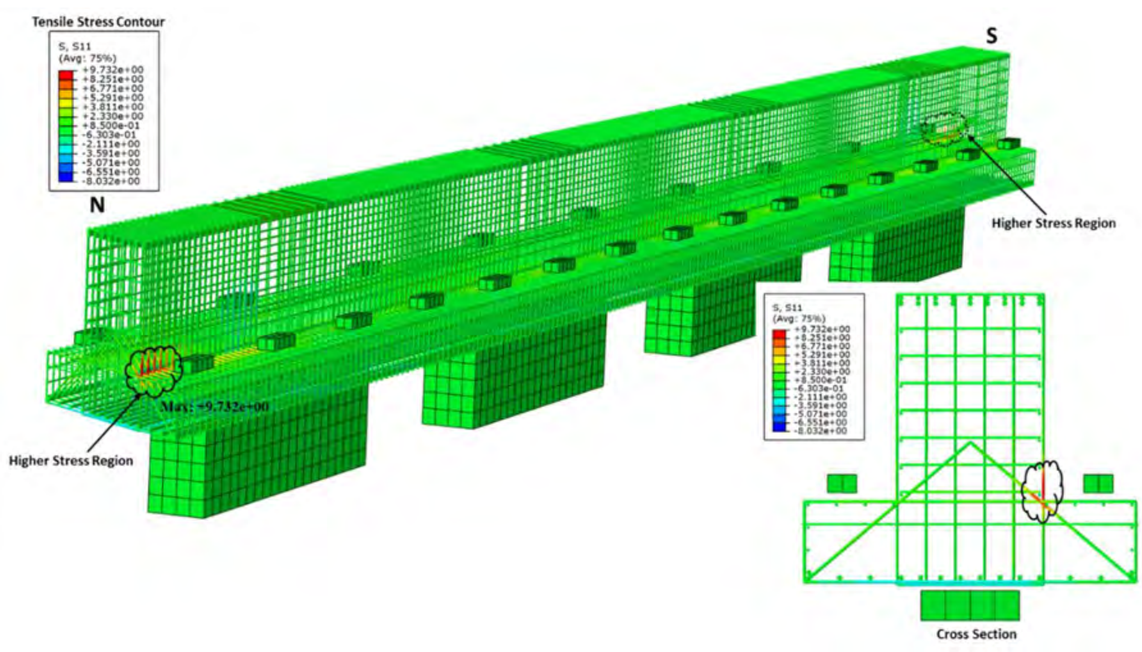


Figure 2.9 Tensile Stress Contour at Service Load of Bent Cap 7

[S11 = Tensile stresses in ksi in Rebars]

[Top (Red in color): Maximum stress, Bottom (Blue in color): Minimum stress]

2.4.2 Stresses in Transverse Rebars at Strength Limit State

The strength limit state loads for each of the interior girder locations and all the exterior girder locations of each bent cap are described in Table 2.3. Ultimate load (strength limit state 1) is calculated by multiplying a factor of 1.25 with dead load, 1.75 with live load and 1.5 with overlay. Figure 2.10, Figure 2.11, and Figure 2.12 illustrate the contour plot of tensile stresses in the transverse reinforcement of the skewed Bent Caps 2, 6, and 7, respectively, corresponding to skew angles of 43°, 33°, and 33°. As shown in Figure 2.10, the maximum tensile stress in the rebars of Bent Cap 2 is 24.20 ksi, which is within the stress limit prescribed by TxDOT. Hence, the bent cap is safe at the ultimate load condition.

Similarly, as shown in Figure 2.11 and Figure 2.12 the maximum tensile stress in the rebars of Bent Caps 6 and 7 is 23.25 ksi and 26.95 ksi, respectively. The rebar stresses in Bent Cap 7 is higher than those of Bent Caps 2 and 6, due to the higher ultimate load demand as shown in Table 2.3. It is evident that the stresses in rebars of all the bent caps under the ultimate load are lower than the yielding stress of steel rebars, which is considered to be 60 ksi and hence safe.

Table 2.3 Strength Limit State Loading for Bent Caps

Bent	Strength Limit State Load at Interior Bearing Pads (kips)	Strength Limit State Load at Exterior Bearing Pads (kips)
Bent 2	334.84	365.82
Bent 6	335.83	357.22
Bent 7	365.23	388.82

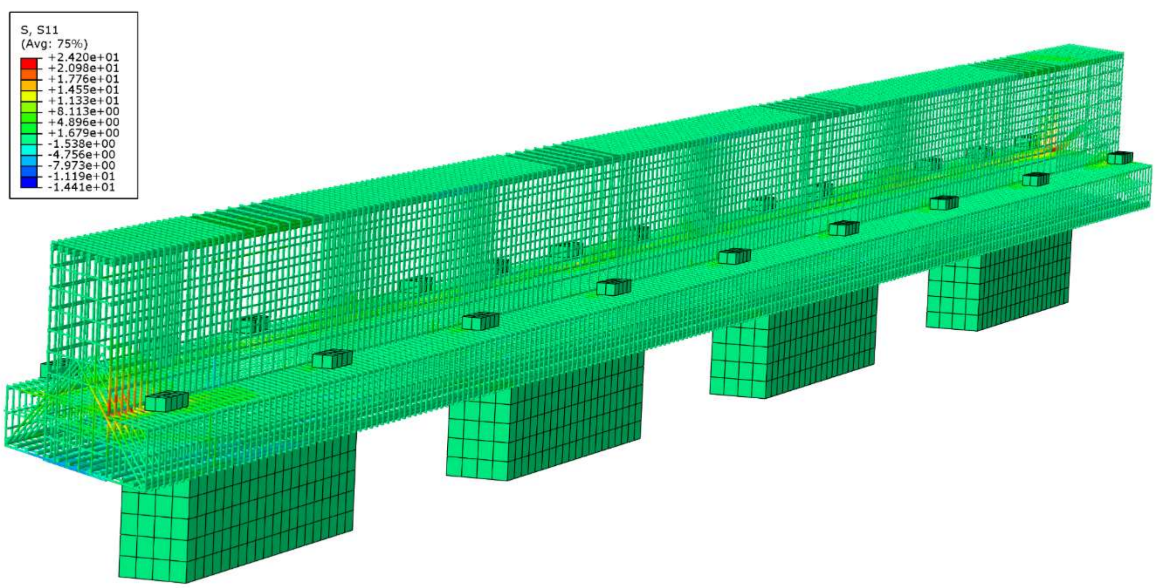


Figure 2.10 Tensile Stress Contour at Strength Limit State of Bent Cap 2

[S11 = Tensile stresses in ksi in Rebars]

[Top (Red in color): Maximum stress, Bottom (Blue in color): Minimum stress]

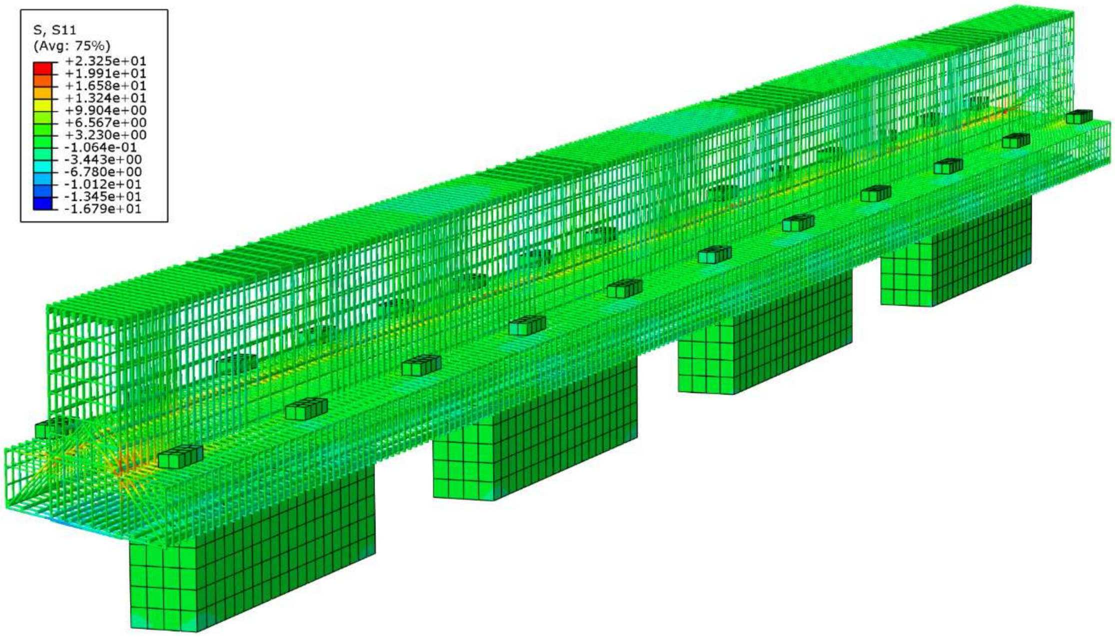


Figure 2.11 Tensile Stress Contour at Strength Limit State of Bent Cap 6

[S11 = Tensile stresses in ksi in Rebars]

[Top (Red in color) : Maximum stress, Bottom (Blue in color): Minimum stress]

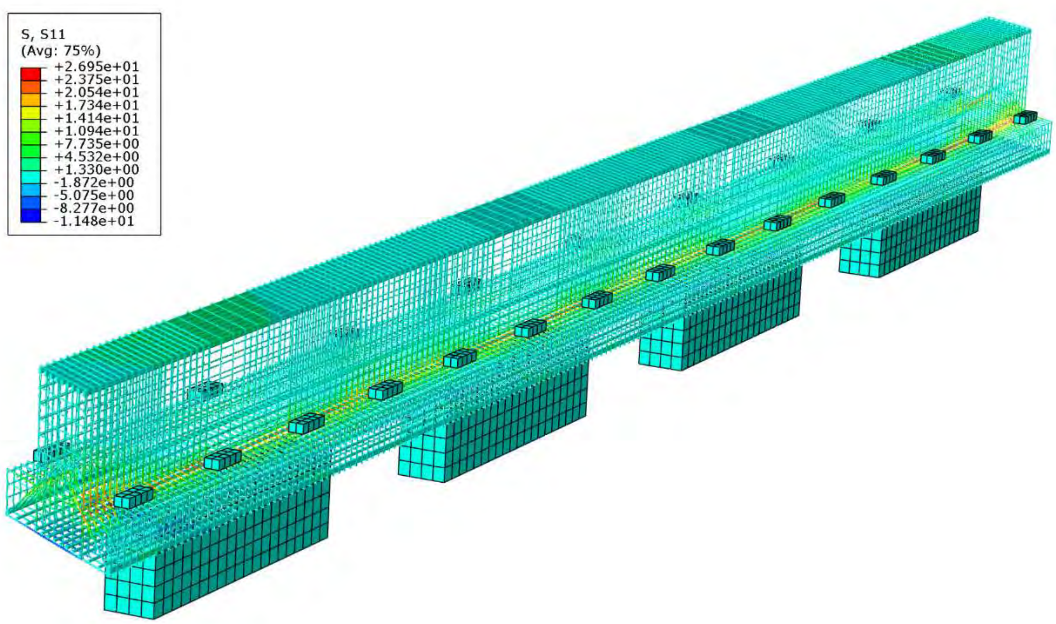


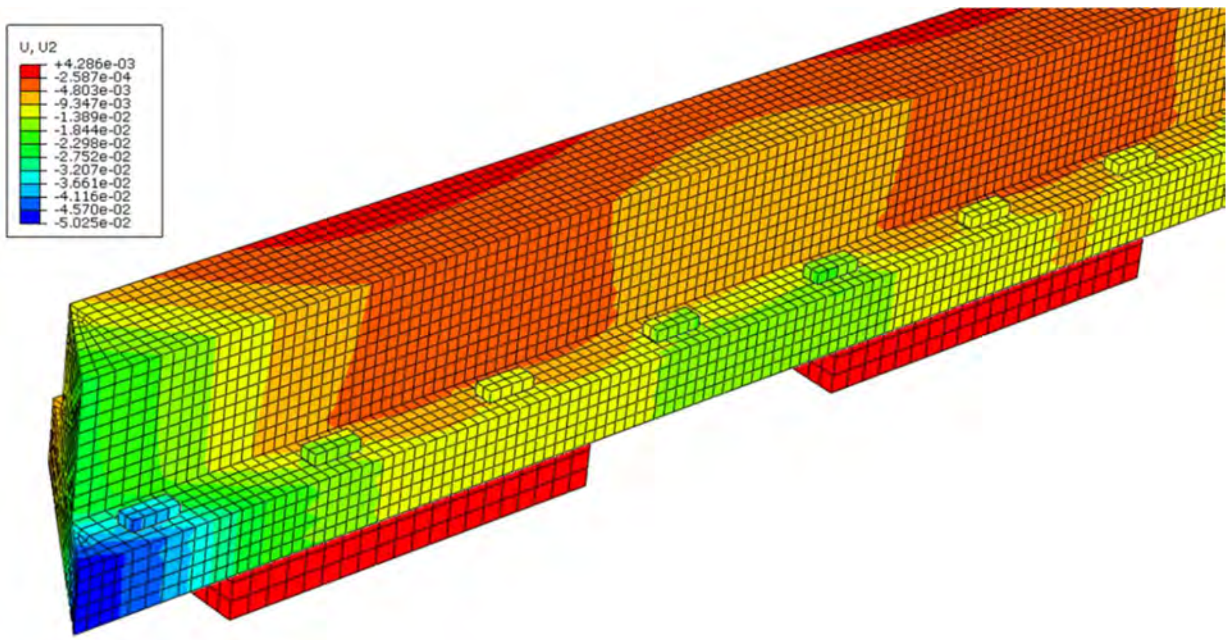
Figure 2.12 Tensile Stress Contour at Strength Limit State of Bent Cap 7

[S11 = Tensile stresses in ksi in Rebars]

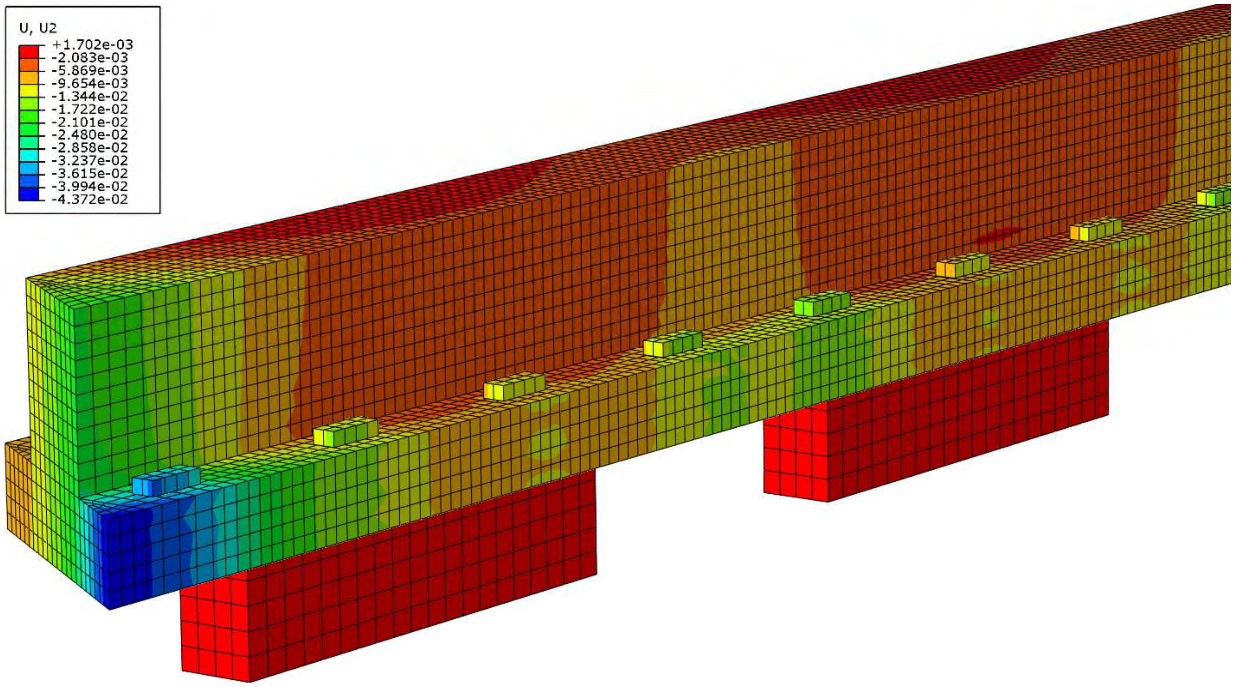
[Top (Red in color) : Maximum stress, Bottom (Blue in color): Minimum stress]

2.4.3 Comparison of Displacements at Service Load

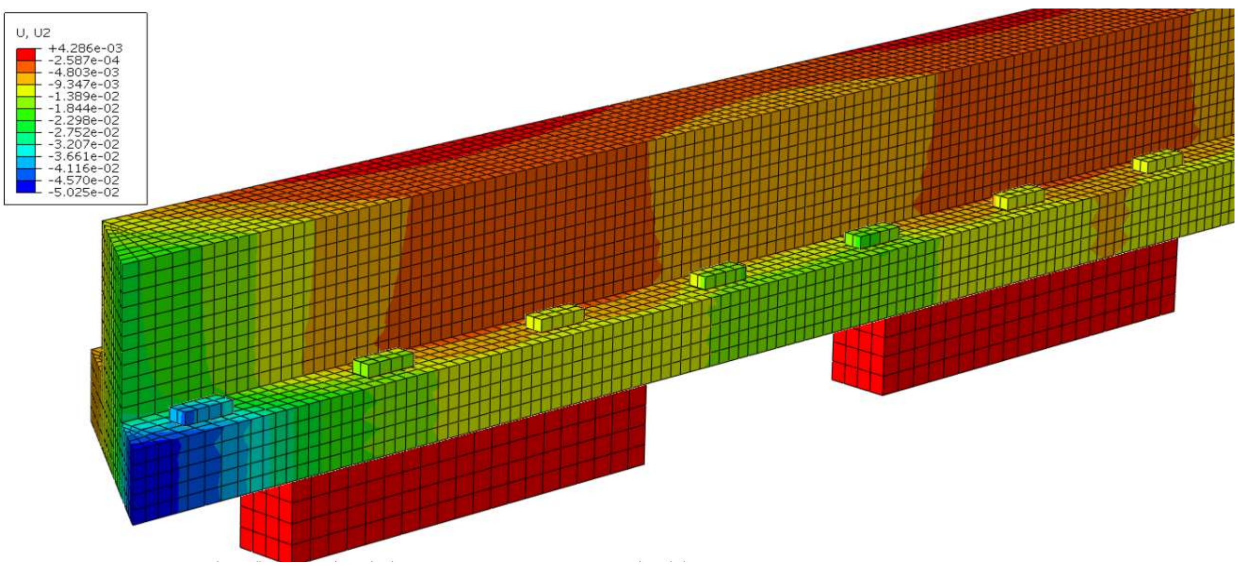
Figure 2.13 shows the magnitude of the deformations of three bent caps at the service loading. As can be seen from the figure, for Bent 2 (43-degree skew case) there is a maximum deformation of 0.05 inch. This deformation is in a downward direction and occurs at the acute angle skew end location (blue color). Similarly, for Bent Caps 6 and 7, the maximum observed deformation is 0.043 inch and 0.05 inch, respectively. The maximum deformation in the bent cap under service loading always occurs at the acute angle skew end, and the net deflection is in the downward direction. Though Bent Caps 6 and 7 have the same skewed angle, the magnitude of deformation is more in Bent Cap 7 because of the higher demand for service load. The maximum displacement is shown in the deep blue color contour, and the negative sign indicates that the displacement is downward. The larger deformation at the end face can be attributed to torsion generated by the unsymmetrical locations of the bearing pads on the ledges of the bridge cap. This deformation pattern will be verified during the load tests.



(a) Bent Cap 2



(b) Bent Cap 6

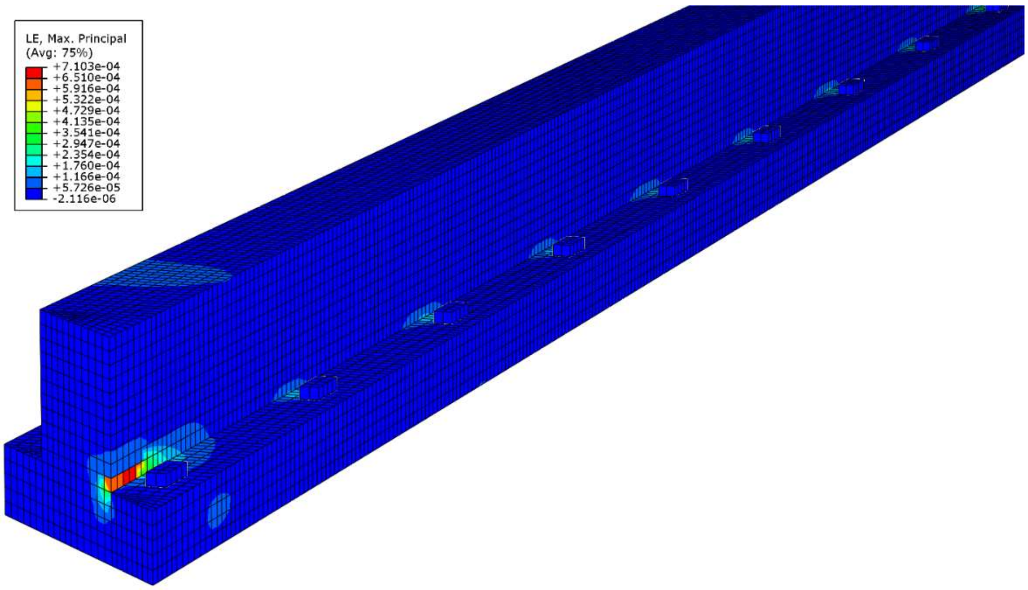


(c) Bent Cap 7

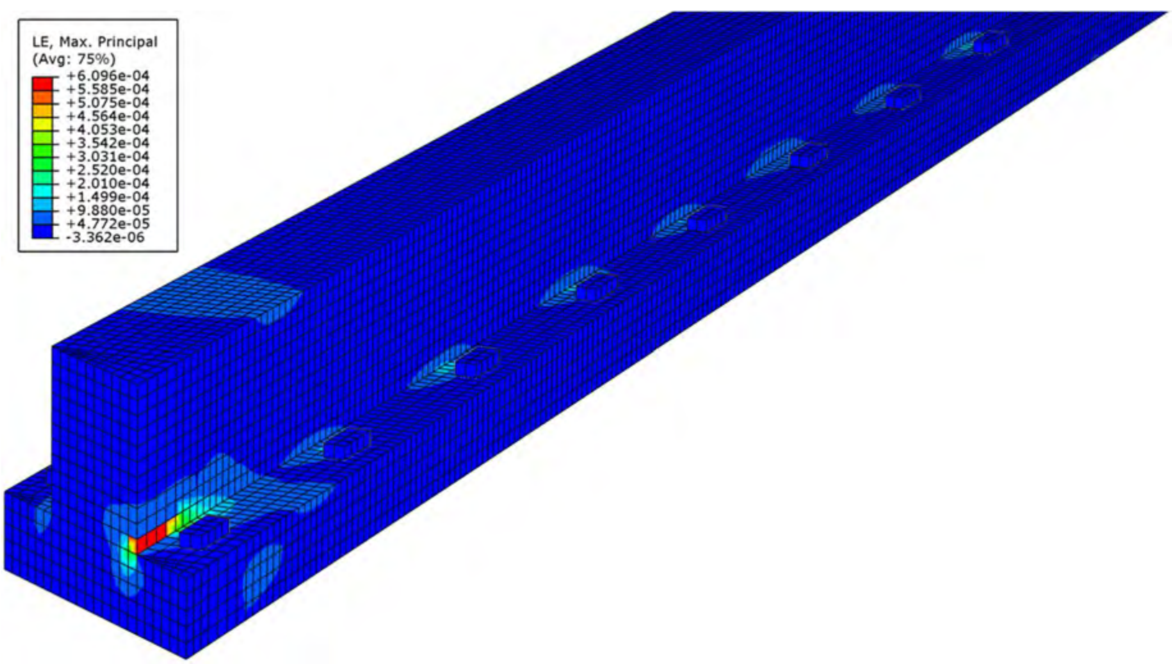
Figure 2.13 Displacement at Service Load for Bent Caps

2.4.4 Comparison of Principal Tensile Strains

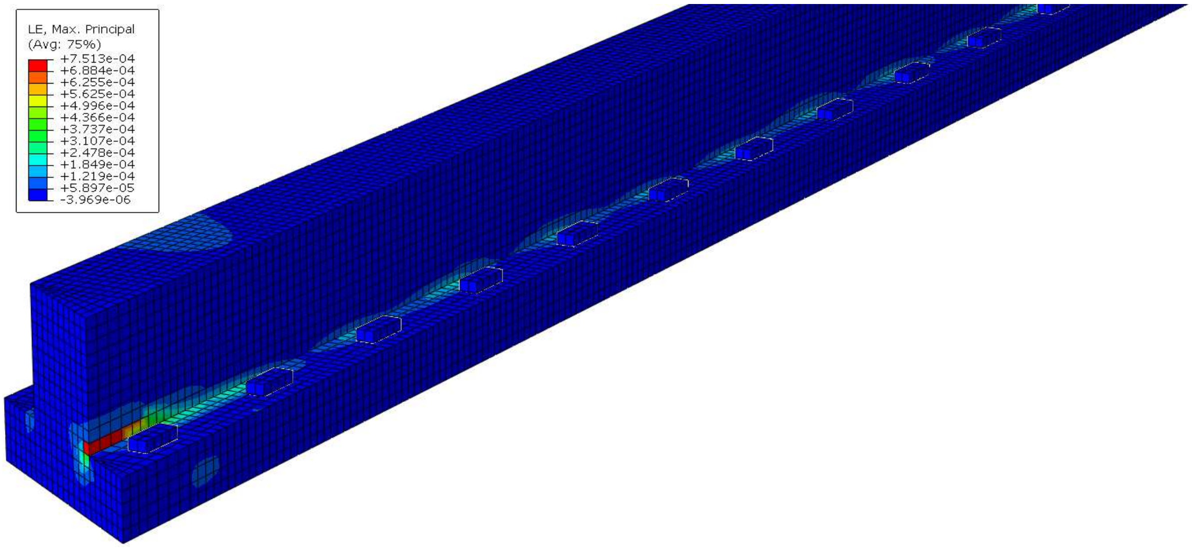
Figure 2.14 shows the FE analysis results which address the comparison of the cracking among all the three bent caps. In the figure, the contour of the principal tensile strain in concrete is illustrated. To show the cracking zone, a lower limit of the principal strain (i.e., 0.00008) was defined so that the regions at which principal strain is less than cracking strain have a different color than the cracked regions. The other regions with different colors, therefore, represent the higher tensile strains. As can be seen from the figure, the tensile strains in most of the parts of bent caps are much lower than the cracking strain. These regions are represented by deep blue color. Locations near loading pads and the re-entrant corner between ledge and web have higher tensile strain, which is represented by light blue and red colors. Hence, under the application of service load, no cracks should be observed in most of the regions of the bent caps. There may be some microcrack formations in some local regions of the bent caps. The principal tensile strain of Bent Cap 7 is observed to be higher because of higher service load.



(a) Bent Cap 2



(b) Bent Cap 6



(c) Bent Cap 7

Figure 2.14 Comparison of Principal Tensile Strain at Service load

2.5 SUMMARY

Because of the advantages of skewed transverse reinforcement, skewed transverse reinforcement has been applied to the design of ITBCs in TxDOT bridges. The Research Team (RT) has selected Bent Cap 2, Bent Cap 6 and Bent Cap 7 of the bridge on Donigan Road over IH 10 to perform the preliminary FE analysis using ABAQUS. Once the overall structural behavior of actual ITBCs with skewed transverse reinforcement is better understood, the critical loading patterns during the load tests and crucial strain gage locations can be determined. Later, the developed numerical models will be calibrated against the field test results for the numerical simulation, considering unexplored parameters. From the preliminary FE analysis, it was observed that the critical locations to paste the strain gauges and attach LVDTs are the cantilever end faces of the bent caps. Moreover, it was also observed that all the bent caps with skewed transverse reinforcing are safe under service and ultimate state loading.

CHAPTER 3: DEVELOPMENT OF PRELIMINARY FINITE ELEMENT MODELS OF THE SIGNIFICANT ITBCs

3.1 INTRODUCTION

In this chapter, the preliminary finite element (FE) analysis of the selected three bent caps (explained in Chapter 2) are performed using ABAQUS to conduct the cost-benefit analysis of skew ITBCs considering different parameters (Task 9a). Due to the construction delays, a task (named Task 9a) was added. Based on the literature review, the FE simulation and the cost-benefit analysis for the ITBCs have not been reported (Bhargava 2009). The parametric FE modeling and cost estimation can be effectively used in the engineering design (Yazdani et al. 2017). In cost-benefit analysis, stiffness of the bent caps under the service load, maximum crack width under the service load, and the ultimate strength of the bent caps are compared as structural behavior. The design parameters, FE Modeling, and the cost-benefit analysis of the bent caps are explained in the following sections.

3.2 CASES OF PARAMETRIC STUDY

The parametric study on the full-scale was performed on Bent 2, Bent 6, and Bent 7 of the bridge on Donigan Road over IH 10, including Case 1, Case 2, and Case 3 for each bent. For the detailing of transverse reinforcement, the following three cases of reinforcement design for the ITBCs have been investigated to cover the majority of the design detailing in Texas bridges.

(1) Case 1: the skew transverse reinforcement is applied, and the U1 Bars, U2 Bars, U3 Bars, and G Bars are also applied at both ends of the bent cap. This case is the same as that presented in Task 9. However, in Task 9, only critical locations were determined from the analytical results. In this additional task, detailed analyses in Case 1 have been completed, including the investigation of the effect of the G Bars and S Bars on the structural performance of the ITBCs. Figure 3.1,Figure 3.2, and Figure 3.3 show the skew reinforcements (Case 1) for Bent 2, Bent 6, and Bent 7, respectively.

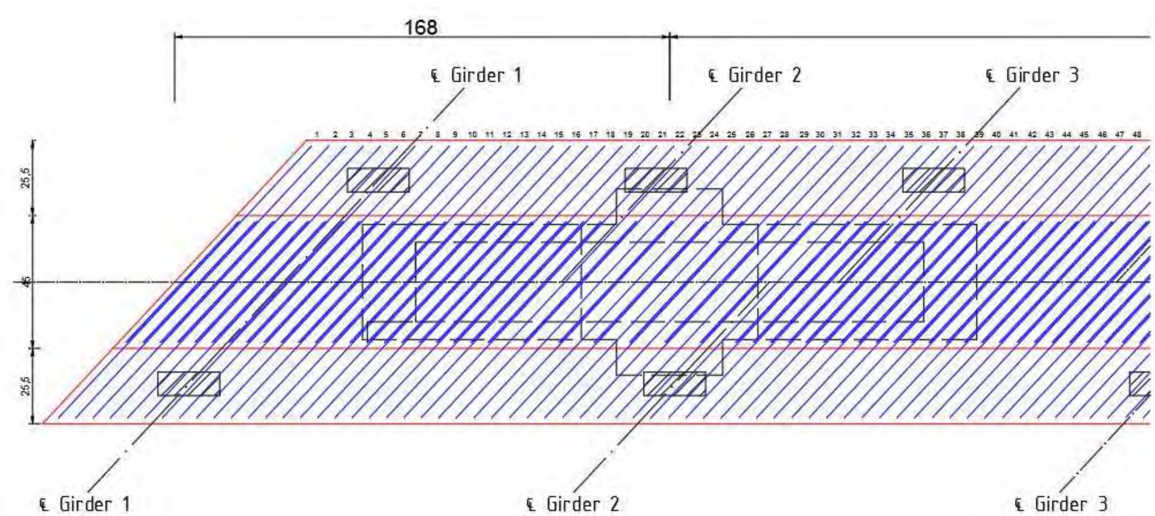


Figure 3.1 Case 1 for Bent 2 (Current Design of Skew Reinforcement, unit: inch)

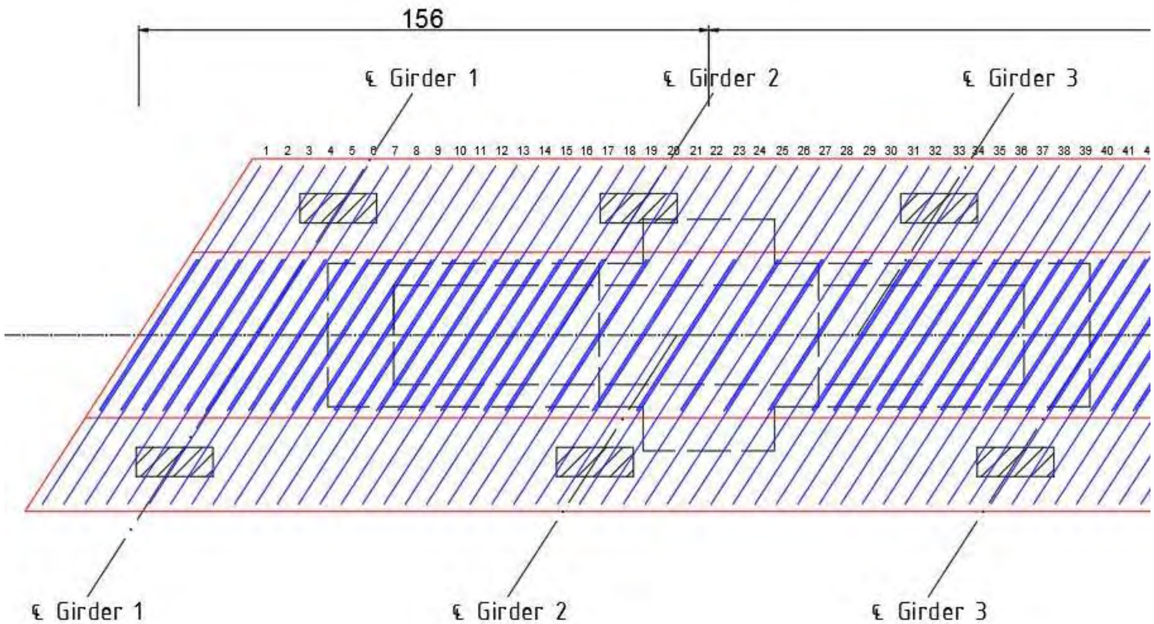


Figure 3.2 Case 1 for Bent 6 (Current Design of Skew Reinforcement, unit: inch)

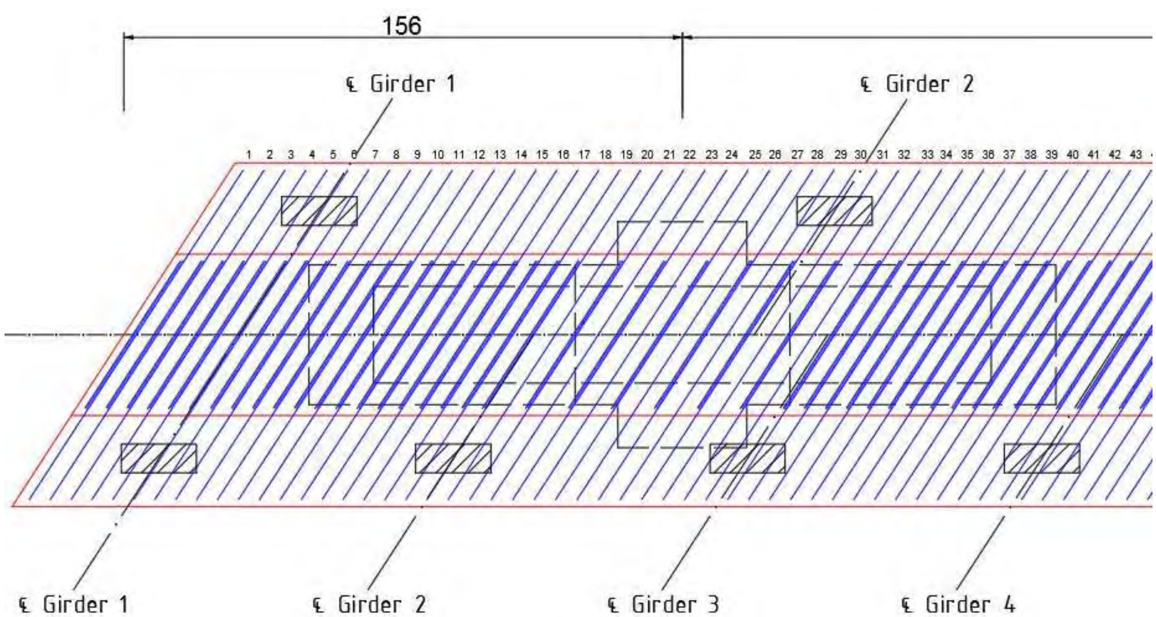


Figure 3.3 Case 1 for Bent 7 (Current Design of Skew Reinforcement, unit: inch)

(2) Case 2: the traditional method of flaring the transverse reinforcement out in skew ITBCs is adopted. Figure 3.4, Figure 3.5, and Figure 3.6 show the traditional detailing of reinforcement without end bars (Case 2) for Bent 2, Bent 6, and Bent 7, respectively. Figure 3.7 shows the sectional and elevation end view of Bent 2 without end bars.

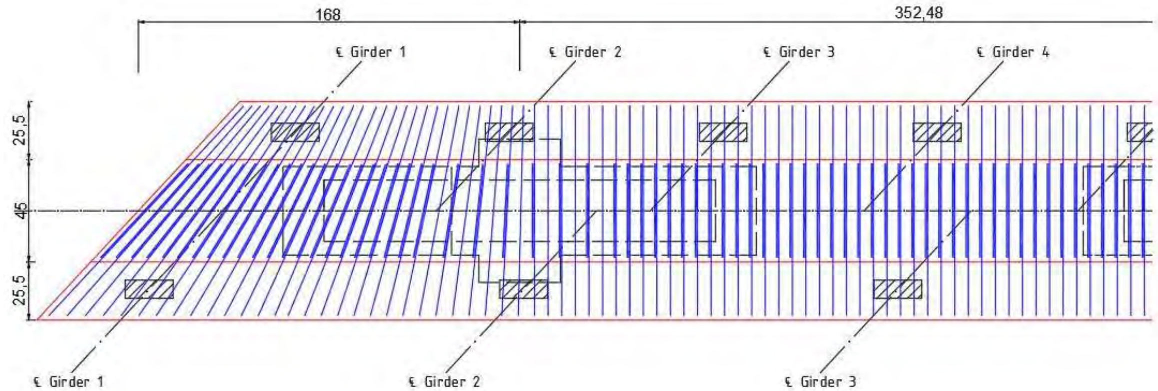


Figure 3.4 Case 2 for Bent 2 (Traditional Detailing of Reinforcement without End Bars, unit: inch)

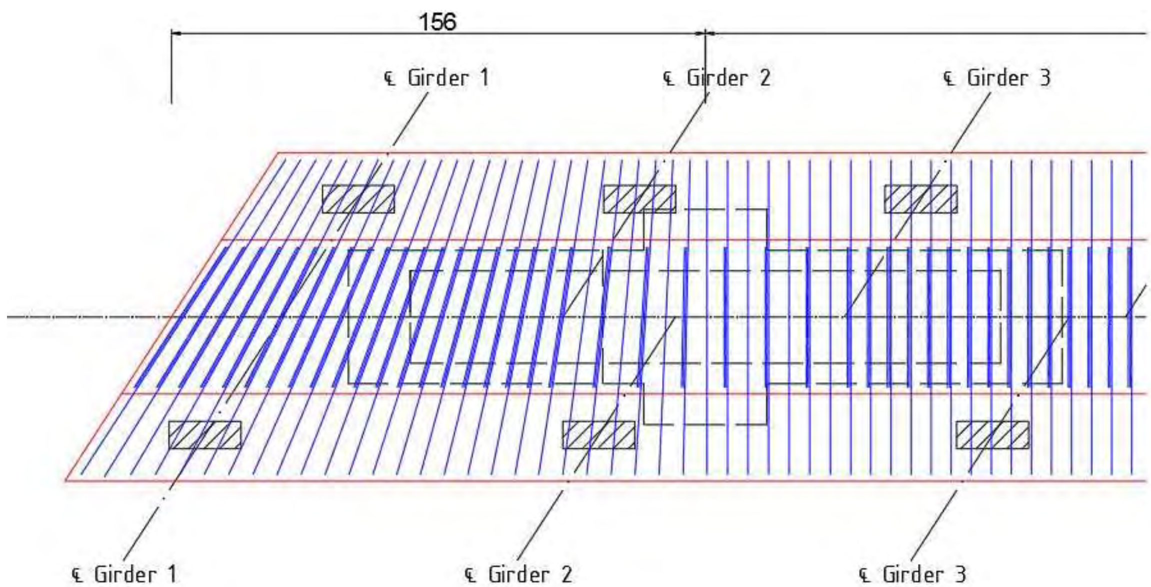


Figure 3.5 Case 2 for Bent 6 (Traditional Detailing of Reinforcement without End Bars, unit: inch)

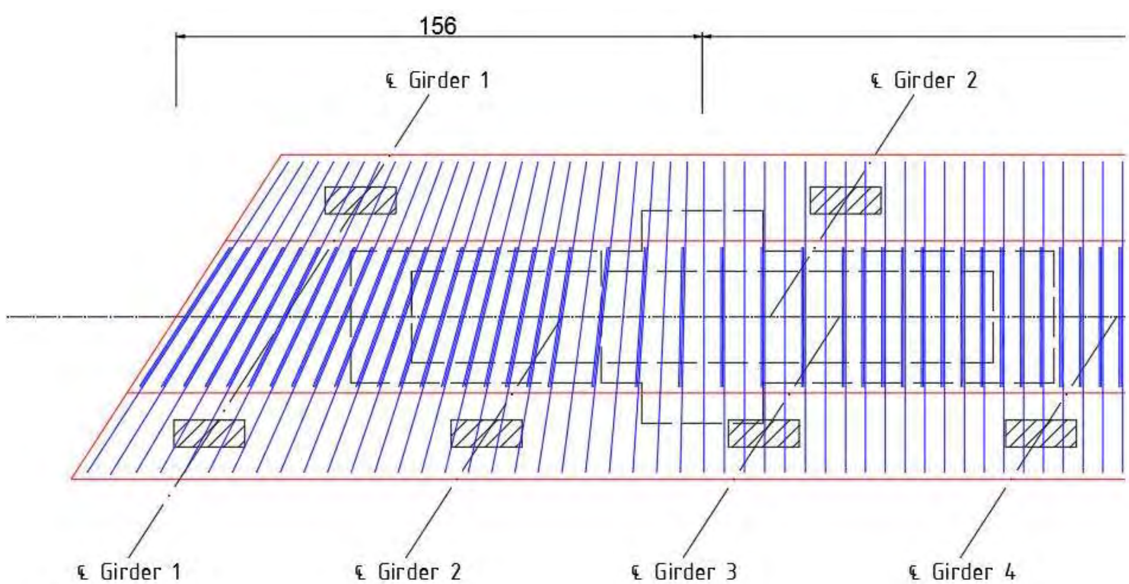
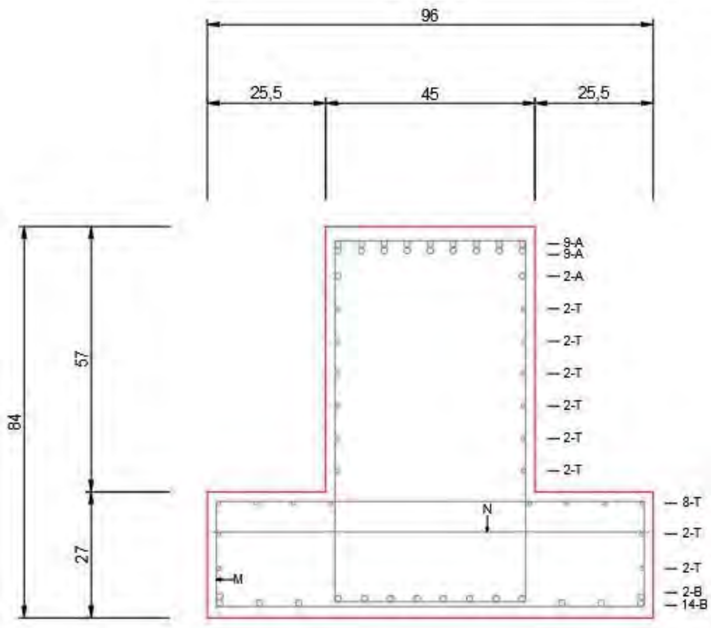
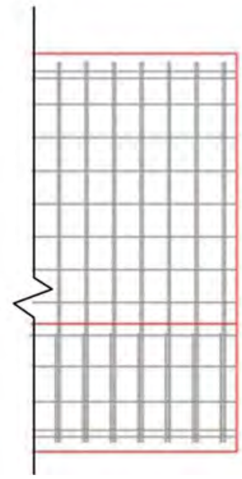


Figure 3.6 Case 2 for Bent 7 (Traditional Detailing of Reinforcement without End Bars, unit: inch)



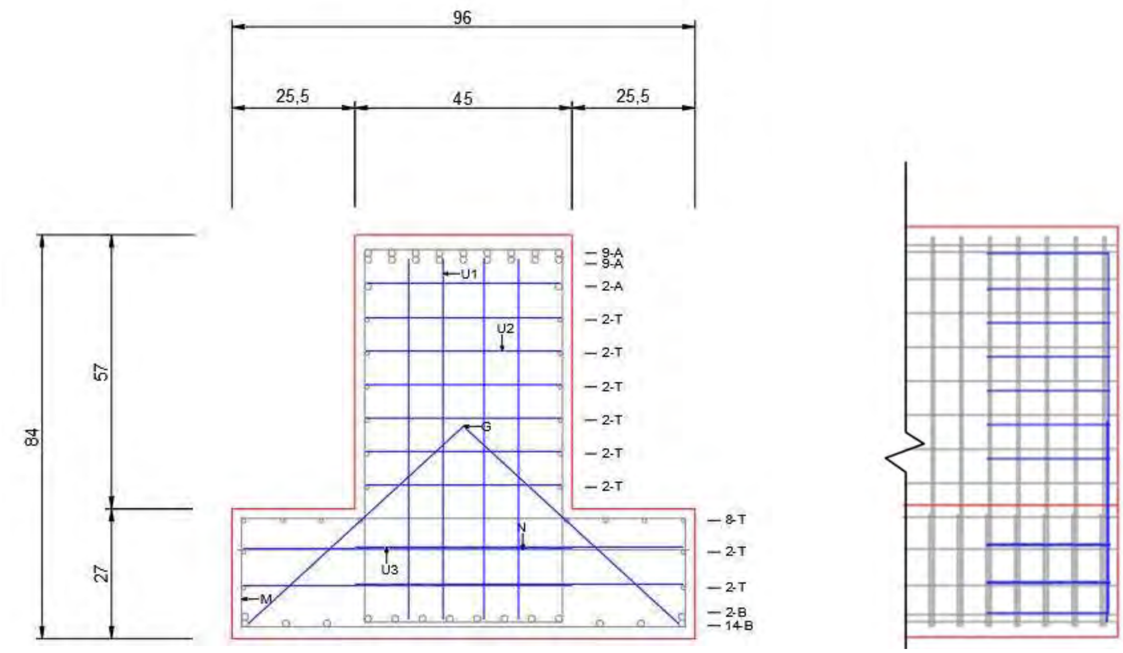
(a) Sectional view



(b) Elevation view

Figure 3.7 Bent 2-End View of Traditional Design Without End Bars in Case 2 (unit: inch)

(3) Case 3: in addition to the traditional detailing of flaring transverse reinforcement in Case 2, the U1 bars, U2 bars, U3 Bars, and G bars are applied at both ends of the bent cap. Figure 3.8 shows the sectional and elevation end view of Bent 2 with end bars.



(a) Sectional View **(b) Elevation View**

Figure 3.8 Bent 2-End View of Traditional Design with End Bars in Case 3 (unit: inch).

Table 3.1 shows the specimens for the parametric FE simulation. The defined nomenclature of the specimens is as follows: For Specimen C3B2C5Smin, the first "C" denotes Case (1, 2, or 3) for the transverse reinforcement detailing; the second character "B" denotes Bent (2, 6, or 7); the third character "C" denotes the concrete strength (5 or 7 ksi); the last character "S" denotes S Bar area [minimum (i.e. 26% less than current design), 0% more (i.e. current design), 20% more or 40% more than current design]. In order to investigate the minimum reinforcement design of the AASHTO (American Association of Highway and Transportation Officials) LRFD (2014) Bridge Design Specifications, the RT calculated the minimum reinforcement area of S Bars for each bent based on the design service load and the AASHTO specifications to serve as the reference group and denote it as "Smin.," which is 26% less than the current design. If "G3" to "G6" are used at the end of the nomenclature, they denote the size of G Bars (No. 3 to No. 6 bars). Specimens C1B2C5S0, C1B6C5S0, and C1B7C5S0 denote the current design of Bent 2, Bent 6, and Bent 7, respectively.

Table 3.1 Specimens of Parametric Finite Element Simulation

No.	Name	Case	Bent Cap			Concrete Strength (ksi)		Transverse Reinforcement Detailing			Amount of Transverse Rebar				G Bar Size				
			Bent 2	Bent 6	Bent 7	5	7	Skew w/ end bars	Traditional w/o end bars	Traditional w/ end bars	Minimum (M)	Current Design	20% higher than current design	40% higher than current design	#3	#4	#5	#6	#7
1	C1B2C5Smin	1	X			X		X			X								X
2	**C1B2C5S0**	**1**	**X**			**X**		**X**				**X**							**X**
3	C1B2C5S20	1	X			X		X					X						X
4	C1B2C5S40	1	X			X		X						X					X
5	C1B2C7Smin	1	X				X	X			X								X
6	C1B2C7S0	1	X				X	X				X							X
7	C1B2C7S20	1	X				X	X					X						X
8	C1B2C7S40	1	X				X	X						X					X
9	C1B6C5Smin	1		X		X		X			X								X
10	C1B6C5S0	1		X		X		X				X							X
11	C1B6C5S20	1		X		X		X					X						X
12	C1B6C5S40	1		X		X		X						X					X
13	C1B6C7Smin	1		X			X	X			X								X
14	C1B6C7S0	1		X			X	X				X							X
15	C1B6C7S20	1		X			X	X					X						X
16	C1B6C7S40	1		X			X	X						X					X
17	C1B7C5Smin	1			X	X		X			X								X
18	C1B7C5S0	1			X	X		X				X							X
19	C1B7C5S20	1			X	X		X					X						X
20	C1B7C5S40	1			X	X		X						X					X
21	C1B7C7Smin	1			X		X	X			X								X
22	C1B7C7S0	1			X		X	X				X							X
23	C1B7C7S20	1			X		X	X					X						X
24	C1B7C7S40	1			X		X	X						X					X
25	C1B2C5G3	1	X			X		X				X			X				
26	C1B2C5G4	1	X			X		X				X				X			
27	C1B2C5G5	1	X			X		X				X					X		
28	C1B2C5G6	1	X			X		X				X						X	
29	C1B6C5G3	1		X		X		X				X			X				

No.	Name	Case	Bent Cap			Concrete Strength (ksi)		Transverse Reinforcement Detailing			Amount of Transverse Rebar				G Bar Size				
			Bent 2	Bent 6	Bent 7	5	7	Skew w/ end bars	Traditional w/o end bars	Traditional w/ end bars	Minimum (M)	Current Design	20% higher than current design	40% higher than current design	#3	#4	#5	#6	#7
30	C1B6C5G4	1		X		X		X				X				X			
31	C1B6C5G5	1		X		X		X				X					X		
32	C1B6C5G6	1		X		X		X				X						X	
33	C1B7C5G3	1			X	X		X				X			X				
34	C1B7C5G4	1			X	X		X				X				X			
35	C1B7C5G5	1			X	X		X				X					X		
36	C1B7C5G6	1			X	X		X				X						X	
37	C2B2C5Smin	2	X			X			X		X								
38	**C2B2C5S0**	2	X			X			X			X							
39	C2B2C5S20	2	X			X			X				X						
40	C2B2C5S40	2	X			X			X					X					
41	C2B2C7Smin	2	X				X		X		X								
42	C2B2C7S0	2	X				X		X			X							
43	C2B2C7S20	2	X				X		X				X						
44	C2B2C7S40	2	X				X		X					X					
45	C2B6C5Smin	2		X		X			X		X								
46	C2B6C5S0	2		X		X			X			X							
47	C2B6C5S20	2		X		X			X				X						
48	C2B6C5S40	2		X		X			X					X					
49	C2B6C7Smin	2		X			X		X		X								
50	C2B6C7S0	2		X			X		X			X							
51	C2B6C7S20	2		X			X		X				X						
52	C2B6C7S40	2		X			X		X					X					
53	C2B7C5Smin	2			X	X			X		X								
54	C2B7C5S0	2			X	X			X			X							
55	C2B7C5S20	2			X	X			X				X						
56	C2B7C5S40	2			X	X			X					X					
57	C2B7C7Smin	2			X		X		X		X								
58	C2B7C7S0	2			X		X		X			X							
59	C2B7C7S20	2			X		X		X				X						

No.	Name	Case	Bent Cap			Concrete Strength (ksi)		Transverse Reinforcement Detailing			Amount of Transverse Rebar				G Bar Size				
			Bent 2	Bent 6	Bent 7	5	7	Skew w/ end bars	Traditional w/o end bars	Traditional w/ end bars	Minimum (M)	Current Design	20% higher than current design	40% higher than current design	#3	#4	#5	#6	#7
60	C2B7C7S40	2			X		X		X					X					
61	C3B2C5Smin	3	X			X				X	X								X
62	**C3B2C5S0**	3	X			X				X		X							X
63	C3B2C5S20	3	X			X				X			X						X
64	C3B2C5S40	3	X			X				X				X					X
65	C3B2C7Smin	3	X				X			X	X								X
66	C3B2C7S0	3	X				X			X		X							X
67	C3B2C7S20	3	X				X			X			X						X
68	C3B2C7S40	3	X				X			X				X					X
69	C3B6C5Smin	3		X		X				X	X								X
70	C3B6C5S0	3		X		X				X		X							X
71	C3B6C5S20	3		X		X				X			X						X
72	C3B6C5S40	3		X		X				X				X					X
73	C3B6C7Smin	3		X			X			X	X								X
74	C3B6C7S0	3		X			X			X		X							X
75	C3B6C7S20	3		X			X			X			X						X
76	C3B6C7S40	3		X			X			X				X					X
77	C3B7C5Smin	3			X	X				X	X								X
78	C3B7C5S0	3			X	X				X		X							X
79	C3B7C5S20	3			X	X				X			X						X
80	C3B7C5S40	3			X	X				X				X					X
81	C3B7C7Smin	3			X		X			X	X								X
82	C3B7C7S0	3			X		X			X		X							X
83	C3B7C7S20	3			X		X			X			X						X
84	C3B7C7S40	3			X		X			X				X					X
85	C3B2C5G3	3	X			X				X		X			X				
86	C3B2C5G4	3	X			X				X		X				X			
87	C3B2C5G5	3	X			X				X		X					X		
88	C3B2C5G6	3	X			X				X		X						X	
89	C3B6C5G3	3		X		X				X		X			X				
90	C3B6C5G4	3		X		X				X		X				X			

No.	Name	Case	Bent Cap			Concrete Strength (ksi)		Transverse Reinforcement Detailing			Amount of Transverse Rebar				G Bar Size				
			Bent 2	Bent 6	Bent 7	5	7	Skew w/ end bars	Traditional w/o end bars	Traditional w/ end bars	Minimum (M)	Current Design	20% higher than current design	40% higher than current design	#3	#4	#5	#6	#7
91	C3B6C5G5	3		X		X				X		X					X		
92	C3B6C5G6	3		X		X				X		X						X	
93	C3B7C5G3	3			X	X				X		X			X				
94	C3B7C5G4	3			X	X				X		X				X			
95	C3B7C5G5	3			X	X				X		X					X		
96	C3B7C5G6	3			X	X				X		X						X	

3.3 3D FINITE ELEMENT MODELING OF BENT CAPS IN ABAQUS

The FE models of three different cases (Case 1, Case 2, and Case 3) of ITBCs were developed using ABAQUS (2020). 3D FE modeling of large-scale ITBCs are described in "2.2. FINITE ELEMENT MODELING OF BENT CAPS IN ABAQUS". To model the specimens in this chapter, the same method is followed. The same material model is used for the concrete and the steel in the ABAQUS models as defined in "2.3. MATERIAL MODELS". Table 3.2 shows the details of the material parameters of the concrete damaged plasticity model for full-scale bent caps for 5 ksi and 7 ksi concrete.

Table 3.2 Material Parameters for the Concrete Damaged Plasticity Model

Concrete grade	Young's modulus (ksi)	Poisson's ratio	Tensile strength (ksi)	Density (lb/ft^3)	Dilation angle (°)	Flow potential eccentricity	*K*
5 ksi	4031	0.2	0.325	150	31	0.1	0.67
7 ksi	4770	0.2	0.382	150	31	0.1	0.67

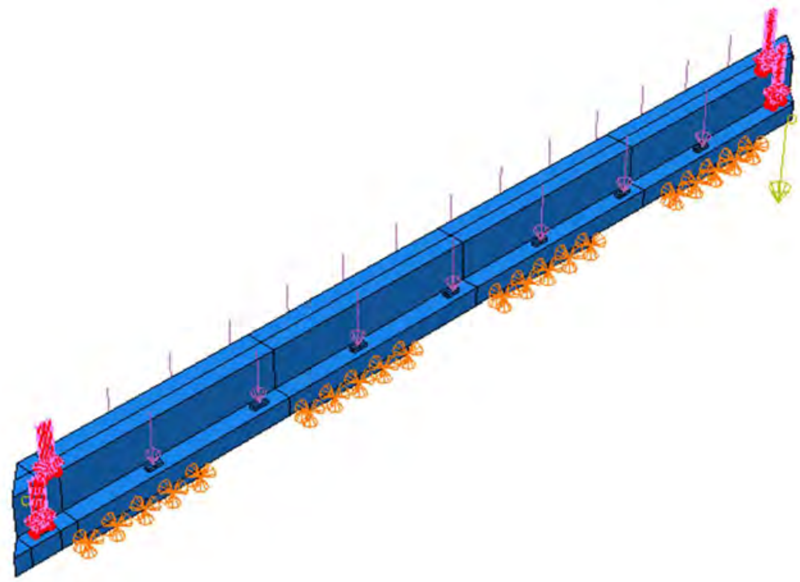
There is a total of 24, 26, and 24 bearing pads tied on top of the ledges of Bent Cap 2, Bent Cap 6, and Bent Cap 7, respectively. The superstructure loads are transferred from the bridge girders to the bridge bent caps through these bearing pads. The analysis was performed with two loading cases. The first loading case is the service load, which includes dead load and live load with the load combination factor equal to one. The second loading case is the ultimate load.

3.3.1 Boundary Conditions at Service Load

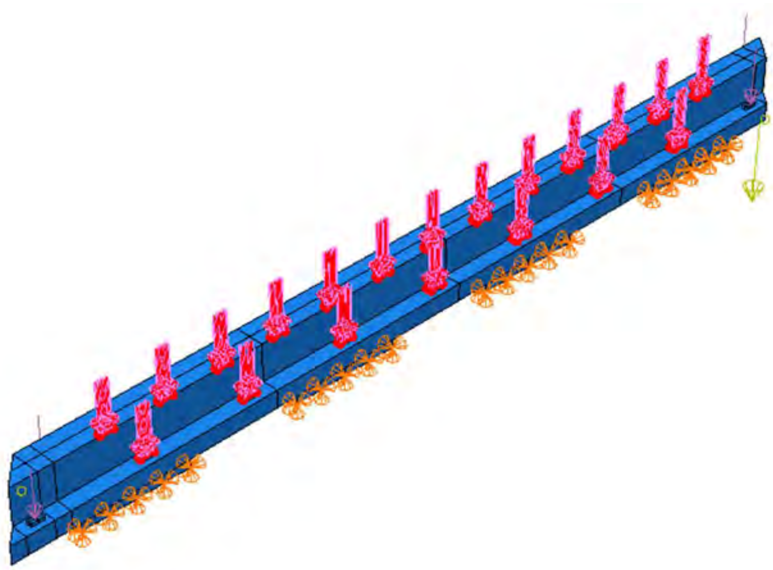
The service load for the bent caps is calculated following the AASHTO LRFD Bridge Design Specifications, 8th Ed. (2017) as prescribed by the TxDOT Bridge Design Manual – LRFD (2020). According to this specification, the service load is applied differently on the exterior and interior bearing pads. Figure 3.9 shows the surfaces for exterior and interior bearing pads in ABAQUS models. The calculated service load is applied as a uniform pressure to these surfaces. The service loads for Bent Cap 2, Bent Cap 6, and Bent Cap 7 are shown in Table 3.3.

Table 3.3. Service Load for Bent Caps

Bent	Service Load at Interior Bearing Pads (kips)	Service Load at Exterior Bearing Pads (kips)
Bent 2	222.48	240.19
Bent 6	226.64	238.86
Bent 7	244.52	258.00



(a) Exterior Bearing Pads



(b) Interior Bearing Pads

Figure 3.9 Loads on the Bearing Pads in ABAQUS Models

3.3.2 Boundary Conditions at Ultimate Load

To calculate the ultimate load capacities of the bent caps, the uniform and equal loads are applied to all bearing pads. This load is provided through a reference point assigned to the top of the bent caps. Figure 3.10 shows the coupling constraint between the reference point and the bearing pads for calculating ultimate capacity. As shown in Figure 3.10, a coupling constraint is defined between the reference point and all bearing pads. Subsequently, a deflection of two inches is applied to the reference point in order to provide the load.

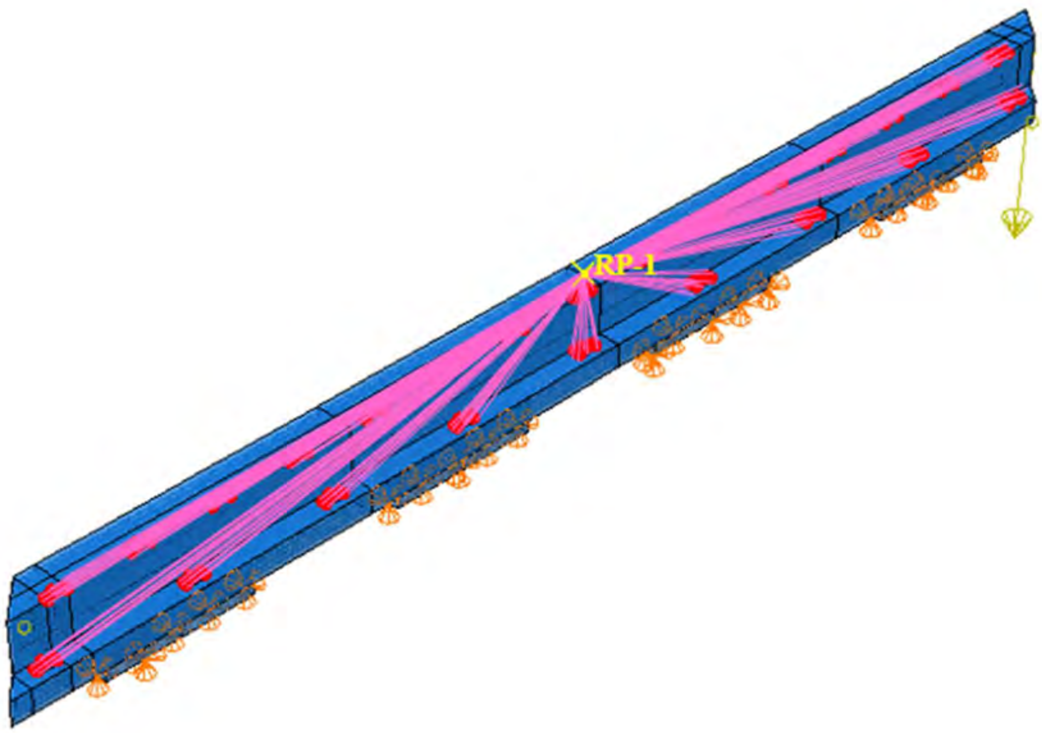


Figure 3.10 Coupling Constraint between the Reference Point and Bearing Pads for Ultimate Loads

3.4 3D FINITE ELEMENT ANALYTICAL RESULTS OF BENT CAPS

The 96 specimens are modeled in ABAQUS in order to investigate structural performances of ITBCs under the service load and ultimate load. Design parameters are skew angle (43° or 33°), detailing of transverse reinforcements (skew transverse reinforcement or traditional transverse reinforcement), end bars (with or without U1 Bars, U2 Bars, U3 Bars, and G Bars), size of S Bars (minimum, current design, 20% more or 40% more than current design), size of G Bars (No. 3 to No. 7 bars), and concrete strength (5 or 7 ksi). Based on these parameters, the displacement and the stiffness at the service load, the principal tensile strain of concrete and crack widths at the service load, and the ultimate capacities of the bent caps are investigated.

3.4.1 Displacement and Stiffness Comparisons at Service Load

The deflections at the midpoints of the two ends of the bent caps, named as D1 and D2 as shown in Figure 3.11, are obtained by the FE simulation results. To calculate the stiffness, the total vertical load is divided by each of both the deflections at these points. Table 3.4 shows the deflection results of each specimen under the service load.

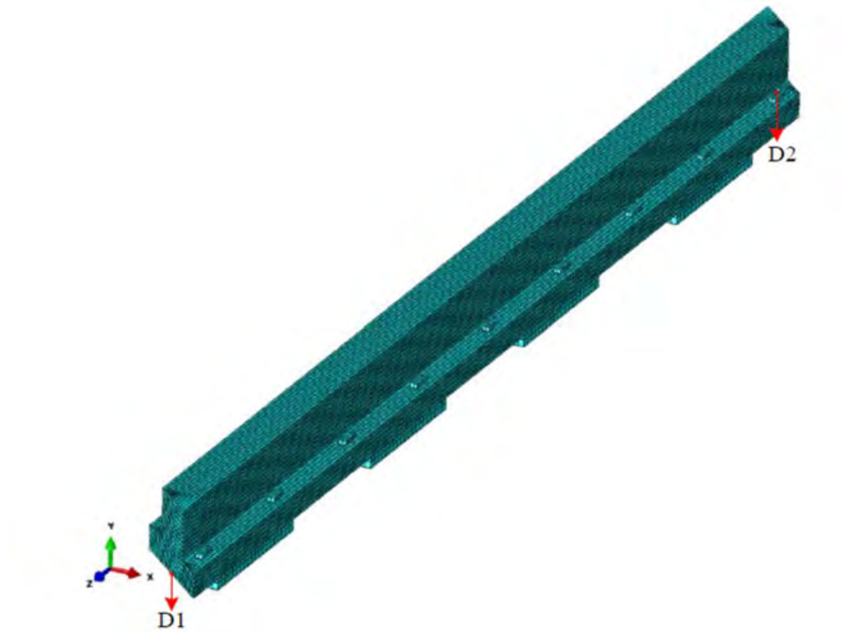


Figure 3.11 Location of Mid-Points of Both Ends D1 and D2

Table 3.4 Deflection Results at Points D1 and D2 under the Service Load

No.	Name	Deflection @ D1 (in.)	Deflection @ D2 (in.)
1	C1B2C5Smin	-0.0179	-0.0190
2	C1B2C5S0	-0.0177	-0.0188
3	C1B2C5S20	-0.0176	-0.0187
4	C1B2C5S40	-0.0176	-0.0187
5	C1B2C7Smin	-0.0151	-0.0161
6	C1B2C7S0	-0.0151	-0.0160
7	C1B2C7S20	-0.0150	-0.0159
8	C1B2C7S40	-0.0150	-0.0159
9	C1B6C5Smin	-0.0153	-0.0160
10	C1B6C5S0	-0.0152	-0.0159
11	C1B6C5S20	-0.0152	-0.0158
12	C1B6C5S40	-0.0151	-0.0158
13	C1B6C7Smin	-0.0130	-0.0135
14	C1B6C7S0	-0.0129	-0.0134
15	C1B6C7S20	-0.0129	-0.0134
16	C1B6C7S40	-0.0128	-0.0134
17	C1B7C5Smin	-0.0176	-0.0164
18	C1B7C5S0	-0.0174	-0.0163
19	C1B7C5S20	-0.0172	-0.0162

No.	Name	Deflection @ D1 (in.)	Deflection @ D2 (in.)
20	C1B7C5S40	-0.0172	-0.0161
21	C1B7C7Smin	-0.0147	-0.0138
22	C1B7C7S0	-0.0146	-0.0138
23	C1B7C7S20	-0.0145	-0.0137
24	C1B7C7S40	-0.0145	-0.0137
25	C1B2C5G3	-0.0179	-0.0190
26	C1B2C5G4	-0.0178	-0.0189
27	C1B2C5G5	-0.0178	-0.0189
28	C1B2C5G6	-0.0178	-0.0189
29	C1B6C5G3	-0.0154	-0.0160
30	C1B6C5G4	-0.0153	-0.0160
31	C1B6C5G5	-0.0153	-0.0159
32	C1B6C5G6	-0.0152	-0.0159
33	C1B7C5G3	-0.0176	-0.0164
34	C1B7C5G4	-0.0175	-0.0164
35	C1B7C5G5	-0.0175	-0.0164
36	C1B7C5G6	-0.0174	-0.0163
37	C2B2C5Smin	-0.0182	-0.0194
38	C2B2C5S0	-0.0180	-0.0192
39	C2B2C5S20	-0.0179	-0.0191
40	C2B2C5S40	-0.0177	-0.0190
41	C2B2C7Smin	-0.0154	-0.0166
42	C2B2C7S0	-0.0153	-0.0165
43	C2B2C7S20	-0.0152	-0.0164
44	C2B2C7S40	-0.0151	-0.0163
45	C2B6C5Smin	-0.0150	-0.0158
46	C2B6C5S0	-0.0148	-0.0156
47	C2B6C5S20	-0.0148	-0.0154
48	C2B6C5S40	-0.0147	-0.0153
49	C2B6C7Smin	-0.0125	-0.0131
50	C2B6C7S0	-0.0125	-0.0130
51	C2B6C7S20	-0.0125	-0.0130
52	C2B6C7S40	-0.0125	-0.0129
53	C2B7C5Smin	-0.0170	-0.0162
54	C2B7C5S0	-0.0166	-0.0158
55	C2B7C5S20	-0.0165	-0.0156
56	C2B7C5S40	-0.0164	-0.0155
57	C2B7C7Smin	-0.0140	-0.0135
58	C2B7C7S0	-0.0139	-0.0132
59	C2B7C7S20	-0.0138	-0.0132
60	C2B7C7S40	-0.0138	-0.0131
61	C3B2C5Smin	-0.0180	-0.0192
62	C3B2C5S0	-0.0178	-0.0190
63	C3B2C5S20	-0.0177	-0.0189
64	C3B2C5S40	-0.0176	-0.0189
65	C3B2C7Smin	-0.0152	-0.0162
66	C3B2C7S0	-0.0151	-0.0161

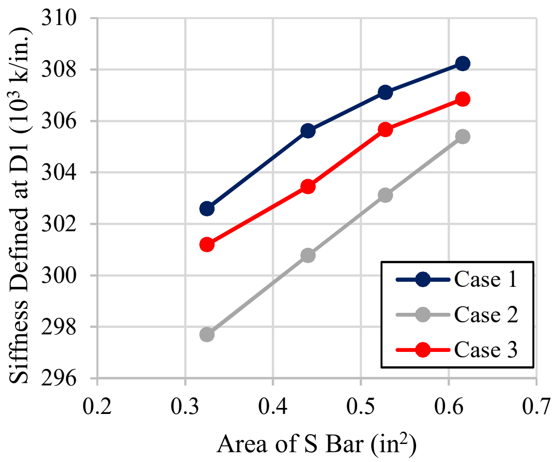
No.	Name	Deflection @ D1 (in.)	Deflection @ D2 (in.)
67	C3B2C7S20	-0.0151	-0.0161
68	C3B2C7S40	-0.0150	-0.0160
69	C3B6C5Smin	-0.0147	-0.0155
70	C3B6C5S0	-0.0146	-0.0153
71	C3B6C5S20	-0.0146	-0.0152
72	C3B6C5S40	-0.0146	-0.0152
73	C3B6C7Smin	-0.0124	-0.0130
74	C3B6C7S0	-0.0124	-0.0129
75	C3B6C7S20	-0.0124	-0.0129
76	C3B6C7S40	-0.0124	-0.0129
77	C3B7C5Smin	-0.0164	-0.0157
78	C3B7C5S0	-0.0163	-0.0155
79	C3B7C5S20	-0.0162	-0.0154
80	C3B7C5S40	-0.0162	-0.0154
81	C3B7C7Smin	-0.0138	-0.0132
82	C3B7C7S0	-0.0137	-0.0131
83	C3B7C7S20	-0.0137	-0.0131
84	C3B7C7S40	-0.0137	-0.0131
85	C3B2C5G3	-0.0179	-0.0191
86	C3B2C5G4	-0.0179	-0.0191
87	C3B2C5G5	-0.0179	-0.0191
88	C3B2C5G6	-0.0179	-0.0190
89	C3B6C5G3	-0.0147	-0.0155
90	C3B6C5G4	-0.0147	-0.0154
91	C3B6C5G5	-0.0147	-0.0154
92	C3B6C5G6	-0.0146	-0.0154
93	C3B7C5G3	-0.0165	-0.0157
94	C3B7C5G4	-0.0164	-0.0157
95	C3B7C5G5	-0.0164	-0.0156
96	C3B7C5G6	-0.0163	-0.0156

The total vertical load is the summation of the service load on the interior and exterior bearing pads and is calculated as 5413 lb, 5950 lb, and 5920 lb for Bent Cap 2, Bent Cap 6, and Bent Cap 7, respectively. The stiffness is calculated by the following equation.

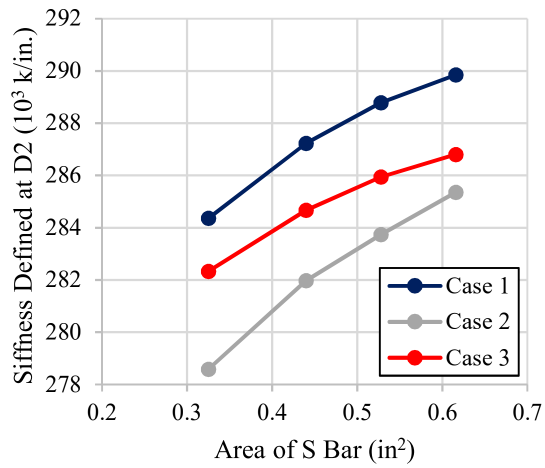
$$k = \frac{F}{\Delta} \tag{Eq. 3-1}$$

where F is the total vertical load, and Δ is the deflection.

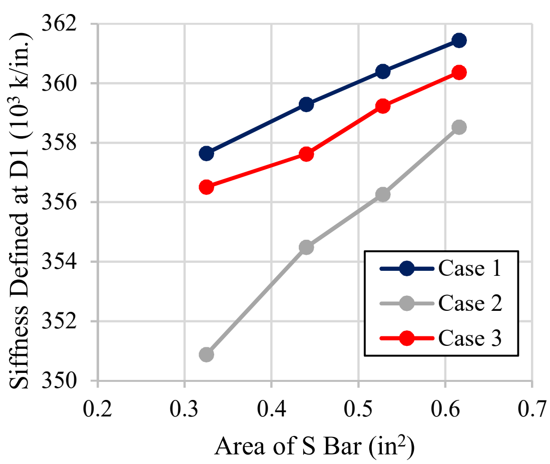
Figure 3.12 shows the comparison of stiffness values of the specimens for each bent cap at points D1 and D2. Based on the FE analysis results, the stiffness slightly increases with increasing the S Bar area because the S Bars reduce the tensile strain of the bent caps. In addition, increasing the concrete compressive strength from 5 ksi to 7 ksi significantly enhances the stiffness, which is attributed to the higher tensile strength and elastic modulus of higher strength concrete. As shown in Figure 3.12, the stiffness values of specimens in Case 2 are lower than that of specimens in Case 3 with end bars. Therefore, the end bars (U1 Bars, U2 Bars, U3 Bars, and G Bars) have a significant influence on the stiffness since they reduce the deflection at the bent cap ends. Moreover, the stiffness increases with respect to the G Bar area.



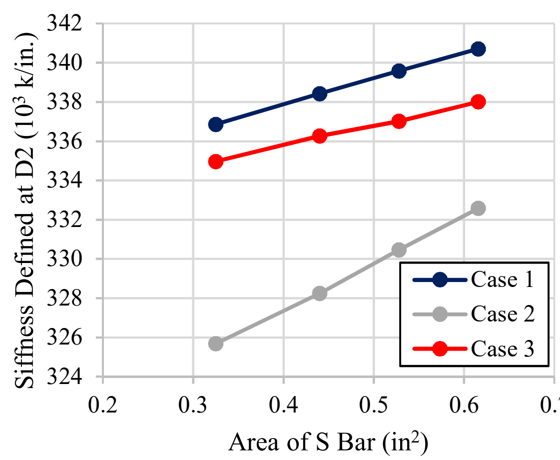
(a) Influence of S Bar Area on Bent 2 with 5 ksi Concrete at D1



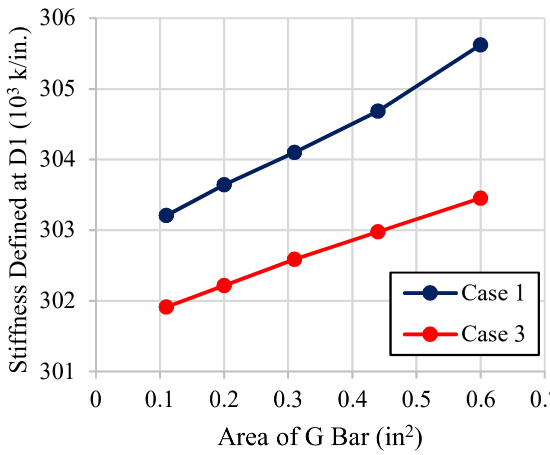
(b) Influence of S Bar Area on Bent 2 with 5 ksi Concrete at D2



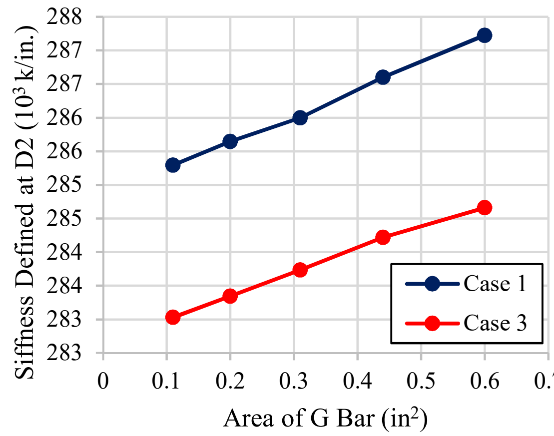
(c) Influence of S Bar Area on Bent 2 with 7 ksi Concrete at D1



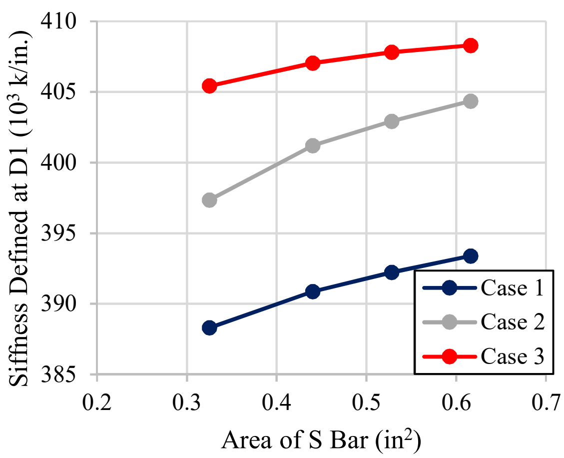
(d) Influence of S Bar Area on Bent 2 with 7 ksi Concrete at D2



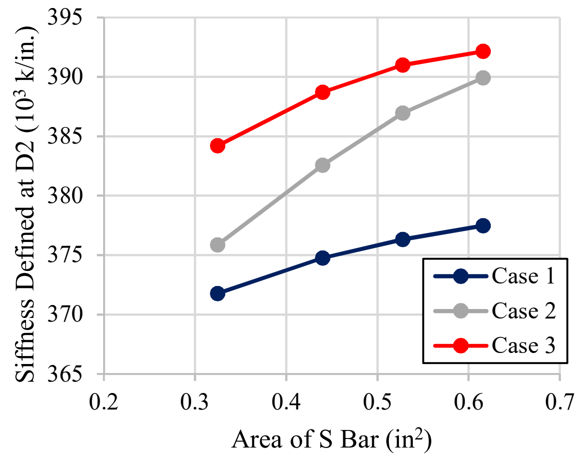
(e) Influence of G Bar Area on Bent 2 with 5 ksi Concrete at D1



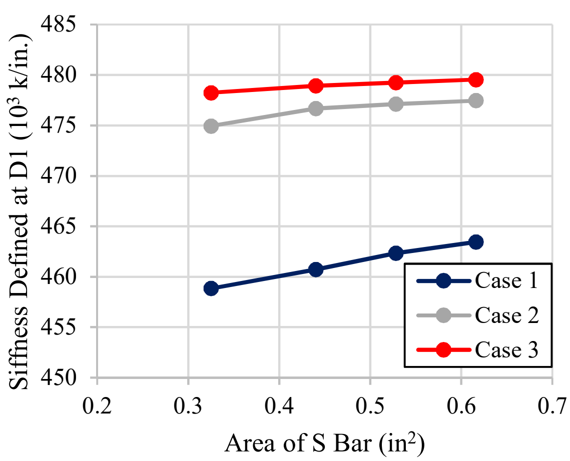
(f) Influence of G Bar Area on Bent 2 with 5 ksi Concrete at D2



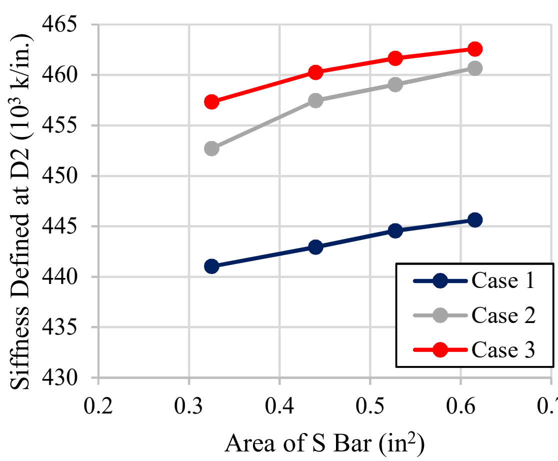
(g) Influence of S Bar Area on Bent 6 with 5 ksi Concrete at D1



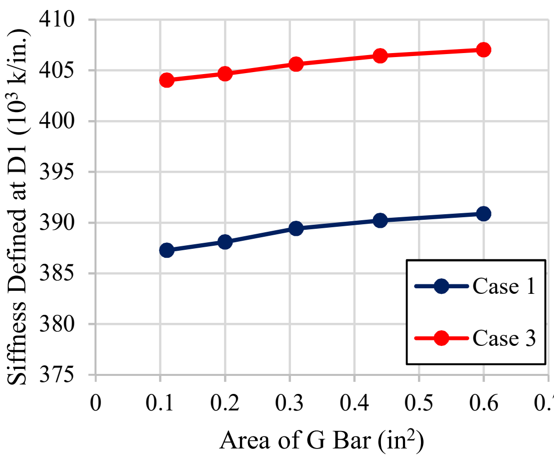
(h) Influence of S Bar Area on Bent 6 with 5 ksi Concrete at D2



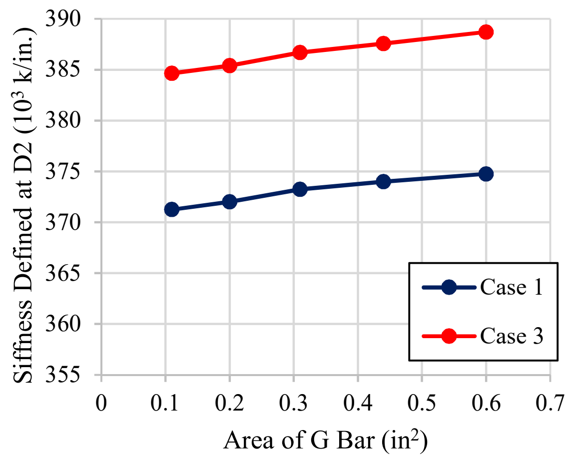
(i) Influence of S Bar Area on Bent 6 with 7 ksi Concrete at D1



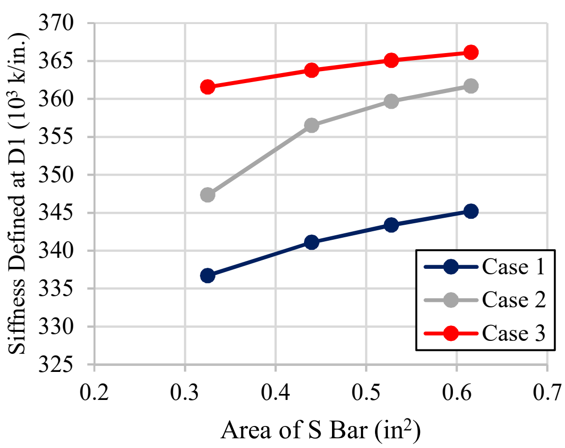
(j) Influence of S Bar Area on Bent 6 with 7 ksi Concrete at D2



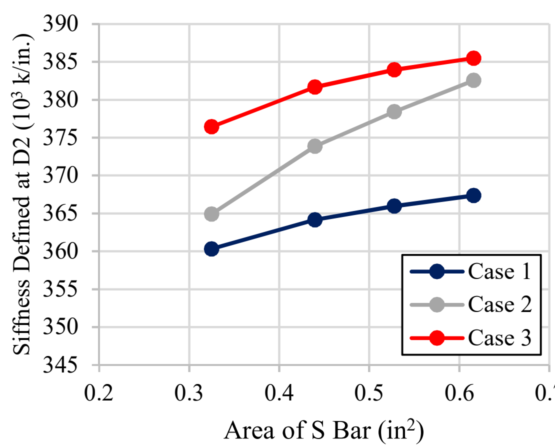
(k) Influence of G Bar Area on Bent 6 with 5 ksi Concrete at D1



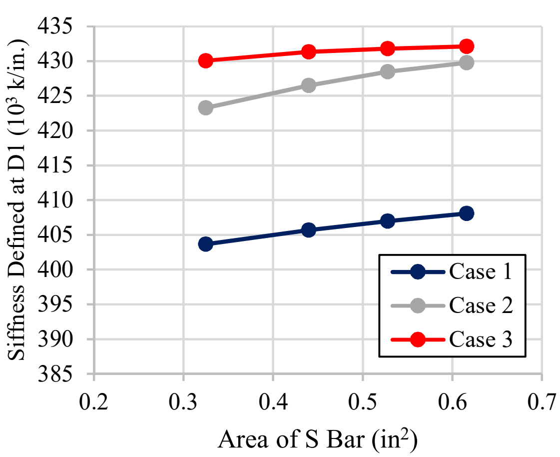
(l) Influence of G Bar Area on Bent 6 with 5 ksi Concrete at D2



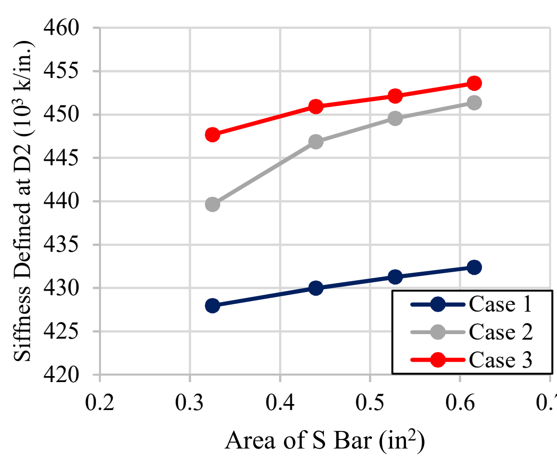
(m) Influence of S Bar Area on Bent 7 with 5 ksi Concrete at D1



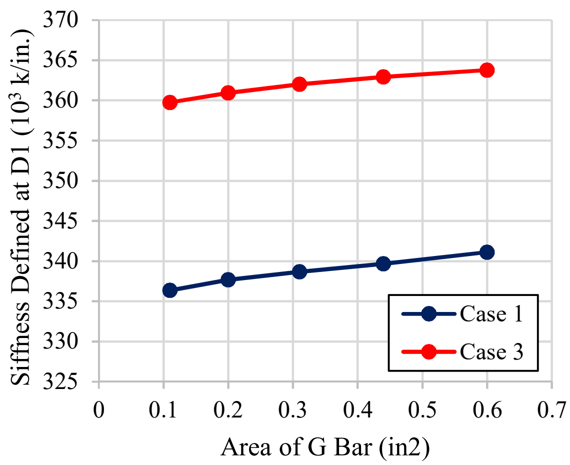
(n) Influence of S Bar Area on Bent 7 with 5 ksi Concrete at D2

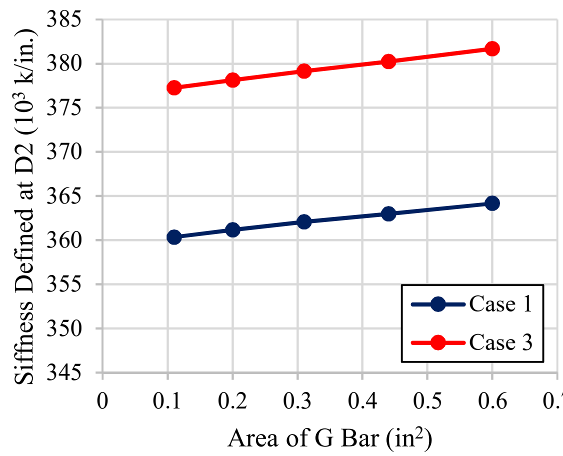


(o) Influence of S Bar Area on Bent 7 with 7 ksi Concrete at D1



(p) Influence of S Bar Area on Bent 7 with 7 ksi Concrete at D2





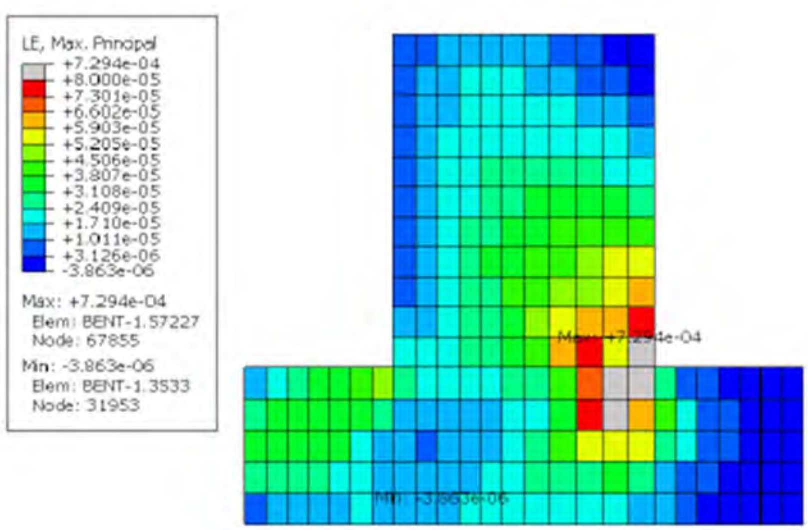
(q) Influence of G Bar Area on Bent 7 with 5 ksi Concrete at D1

(r) Influence of G Bar Area on Bent 7 with 5 ksi Concrete at D2

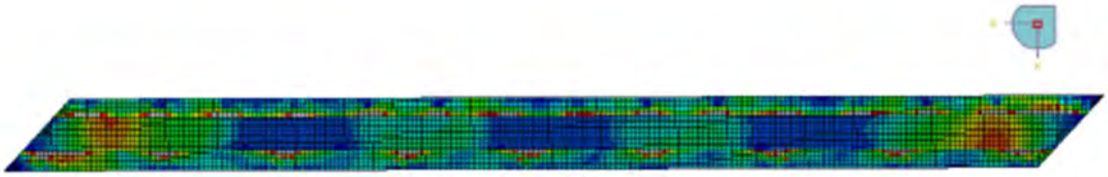
Figure 3.12 Comparison of Stiffness at the Service Load

3.4.2 Principal Tensile Strain and Crack Width Comparisons at Service Load

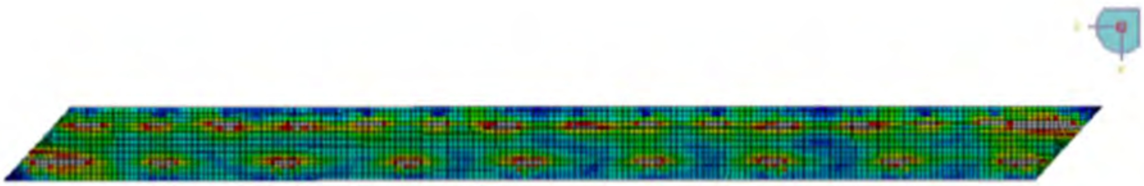
Based on the concrete damaged plasticity model in ABAQUS, the cracking behavior of each specimen at the service load is investigated. Cracks are generally observed at the interface between the ledge and the web, and cracking is generally developed in horizontal crack surfaces. The vertical load, applied from the girders to the ledge, is transferred through the S Bars the bent cap. Since no prestress is applied to the S Bars, the bent cap is prone to micro-cracking under the concentrated loads under the service load. Figure 3.13 shows the location of micro-cracks of Specimen C3B2C5S0. As shown in Figure 3.13, most of the microcracks are observed at the interface between the ledge and the web, close to the end of the bent cap.



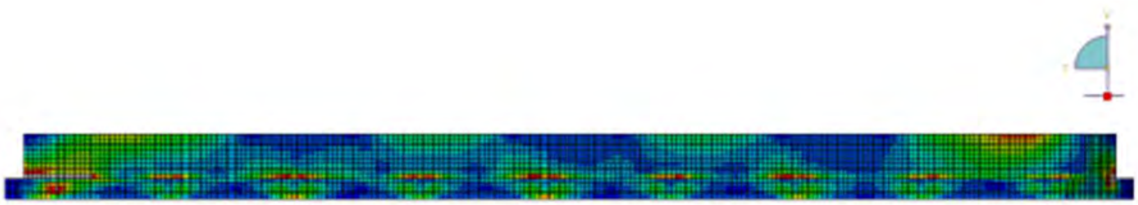
(a) Sectional View of Principal Tensile Strain



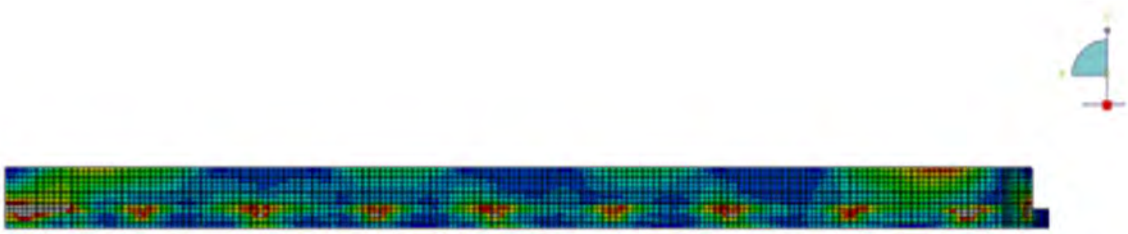
(b) Plan View of Principal Tensile Strain



(c) Cutting Plan View of Principal Tensile Strain



(d) Side View of Principal Tensile Strain



(e) Cutting side view of principal tensile strain

Figure 3.13 Principal Tensile Strains in Current Design of Bent 2 at the Service Load (Specimen C3B2C5S0)

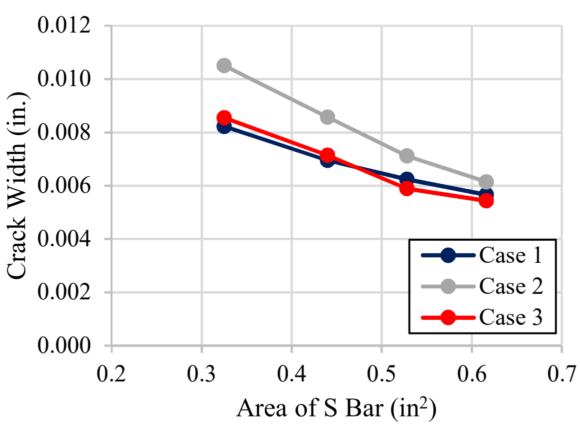
The principal tensile strain is obtained from the FE analyses to calculate the crack width. The maximum principal tensile strain of the concrete section for each specimen is shown in Table 3.5. The maximum cracking strain, ε_{cr}, is calculated by subtracting the maximum tensile strain obtained from ABAQUS simulation results by the crack strain. The average crack spacing, L_m, is calculated as recommended by ACI Committee 224 (ACI, 2001). The crack width is calculated by multiplying the maximum cracking strain, ε_{cr}, with the average crack spacing, L_m. Both traditional and skewed design causes microcracking, which is difficult to see with the naked eye and will generally not affect the structural behavior. Therefore, the structural serviceability of the current design at the service load is verified. Figure 3.14 shows the comparison of the crack width of each specimen for all bent caps. Because the location of the maximum crack width is at the end of the ITBCs, the end bars (U1 Bars, U2 Bars, U3 Bars, and G Bars) have a significant influence on crack width. Besides, maximum crack width significantly decreases with the increasing G Bar area. Increasing the S Bar area and the compressive strength of concrete notably decreases the crack width.

Table 3.5 Principal Tensile Strain and Maximum Crack Width of Concrete at Service Load

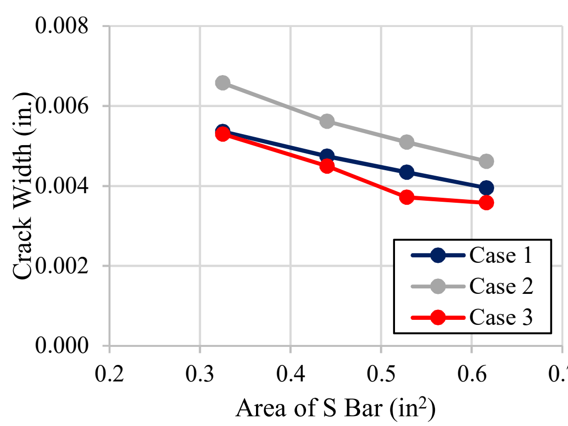
No.	Name	Maximum Tensile Strain	Maximum Crack Width (in.)
1	C1B2C5Smin	0.000833	0.0082
2	C1B2C5S0	0.000711	0.0069
3	C1B2C5S20	0.000644	0.0062
4	C1B2C5S40	0.000589	0.0057
5	C1B2C7Smin	0.000571	0.0054
6	C1B2C7S0	0.000511	0.0047
7	C1B2C7S20	0.000473	0.0043
8	C1B2C7S40	0.000436	0.0040
9	C1B6C5Smin	0.000700	0.0068
10	C1B6C5S0	0.000609	0.0058
11	C1B6C5S20	0.000557	0.0053
12	C1B6C5S40	0.000512	0.0048
13	C1B6C7Smin	0.000478	0.0043
14	C1B6C7S0	0.000426	0.0038
15	C1B6C7S20	0.000380	0.0033
16	C1B6C7S40	0.000339	0.0029
17	C1B7C5Smin	0.000876	0.0087
18	C1B7C5S0	0.000751	0.0074
19	C1B7C5S20	0.000683	0.0067
20	C1B7C5S40	0.000630	0.0061
21	C1B7C7Smin	0.000606	0.0057
22	C1B7C7S0	0.000544	0.0051
23	C1B7C7S20	0.000506	0.0047
24	C1B7C7S40	0.000474	0.0044
25	C1B2C5G3	0.000910	0.0091
26	C1B2C5G4	0.000867	0.0087
27	C1B2C5G5	0.000822	0.0082
28	C1B2C5G6	0.000771	0.0076
29	C1B6C5G3	0.000818	0.0081
30	C1B6C5G4	0.000766	0.0076
31	C1B6C5G5	0.000711	0.0069
32	C1B6C5G6	0.000664	0.0064
33	C1B7C5G3	0.001054	0.0107
34	C1B7C5G4	0.000969	0.0098
35	C1B7C5G5	0.000892	0.0089
36	C1B7C5G6	0.000826	0.0082

No.	Name	Maximum Tensile Strain	Maximum Crack Width (in.)
37	C2B2C5Smin	0.001042	0.0105
38	C2B2C5S0	0.000859	0.0086
39	C2B2C5S20	0.000724	0.0071
40	C2B2C5S40	0.000633	0.0061
41	C2B2C7Smin	0.000682	0.0066
42	C2B2C7S0	0.000590	0.0056
43	C2B2C7S20	0.000541	0.0051
44	C2B2C7S40	0.000495	0.0046
45	C2B6C5Smin	0.001058	0.0107
46	C2B6C5S0	0.000878	0.0088
47	C2B6C5S20	0.000724	0.0071
48	C2B6C5S40	0.000641	0.0062
49	C2B6C7Smin	0.000662	0.0064
50	C2B6C7S0	0.000527	0.0049
51	C2B6C7S20	0.000475	0.0044
52	C2B6C7S40	0.000450	0.0041
53	C2B7C5Smin	0.001239	0.0127
54	C2B7C5S0	0.001025	0.0104
55	C2B7C5S20	0.000885	0.0089
56	C2B7C5S40	0.000800	0.0080
57	C2B7C7Smin	0.000813	0.0080
58	C2B7C7S0	0.000665	0.0064
59	C2B7C7S20	0.000599	0.0057
60	C2B7C7S40	0.000571	0.0055
61	C3B2C5Smin	0.000863	0.0086
62	C3B2C5S0	0.000729	0.0071
63	C3B2C5S20	0.000613	0.0059
64	C3B2C5S40	0.000569	0.0054
65	C3B2C7Smin	0.000565	0.0053
66	C3B2C7S0	0.000488	0.0045
67	C3B2C7S20	0.000416	0.0037
68	C3B2C7S40	0.000402	0.0036
69	C3B6C5Smin	0.000785	0.0077
70	C3B6C5S0	0.000636	0.0061
71	C3B6C5S20	0.000565	0.0054
72	C3B6C5S40	0.000556	0.0053
73	C3B6C7Smin	0.000501	0.0046
74	C3B6C7S0	0.000418	0.0037
75	C3B6C7S20	0.000416	0.0037
76	C3B6C7S40	0.000412	0.0037

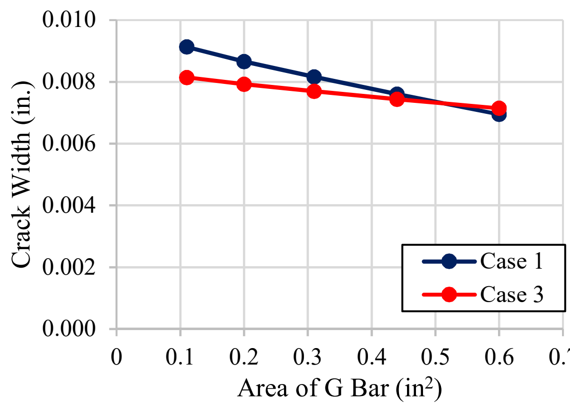
No.	Name	Maximum Tensile Strain	Maximum Crack Width (in.)
77	C3B7C5Smin	0.000866	0.0086
78	C3B7C5S0	0.000713	0.0070
79	C3B7C5S20	0.000677	0.0066
80	C3B7C5S40	0.000659	0.0064
81	C3B7C7Smin	0.000588	0.0055
82	C3B7C7S0	0.000523	0.0049
83	C3B7C7S20	0.000516	0.0048
84	C3B7C7S40	0.000507	0.0047
85	C3B2C5G3	0.000820	0.0081
86	C3B2C5G4	0.000800	0.0079
87	C3B2C5G5	0.000779	0.0077
88	C3B2C5G6	0.000756	0.0074
89	C3B6C5G3	0.000817	0.0081
90	C3B6C5G4	0.000779	0.0077
91	C3B6C5G5	0.000728	0.0071
92	C3B6C5G6	0.000686	0.0067
93	C3B7C5G3	0.000923	0.0093
94	C3B7C5G4	0.000886	0.0089
95	C3B7C5G5	0.000839	0.0084
96	C3B7C5G6	0.000783	0.0077



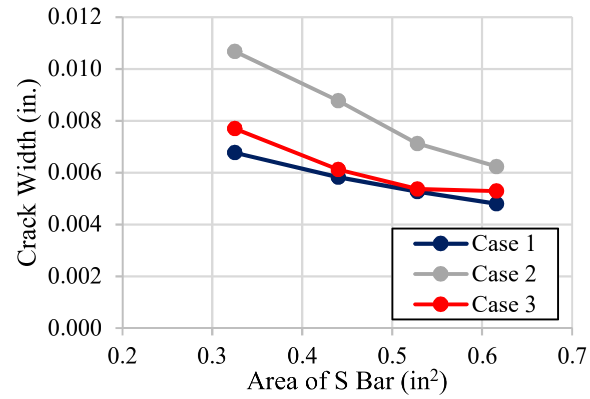
(a) Influence of S Bar Area on Bent 2 with 5 ksi Concrete



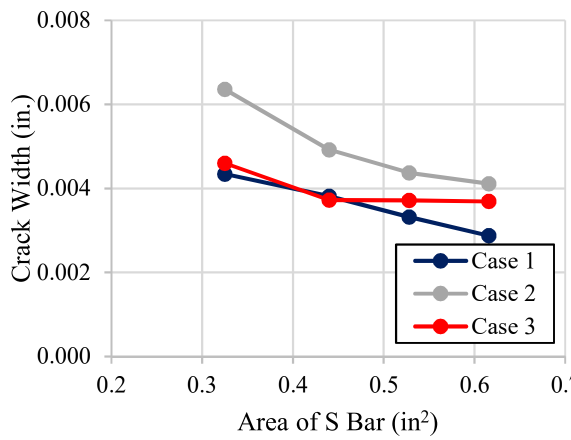
(b) Influence of S Bar Area on Bent 2 with 7 ksi Concrete



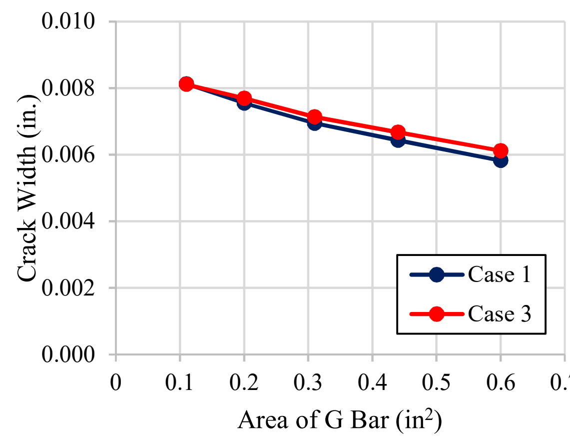
(c) Influence of G Bar Area on Bent 2 with 5 ksi Concrete



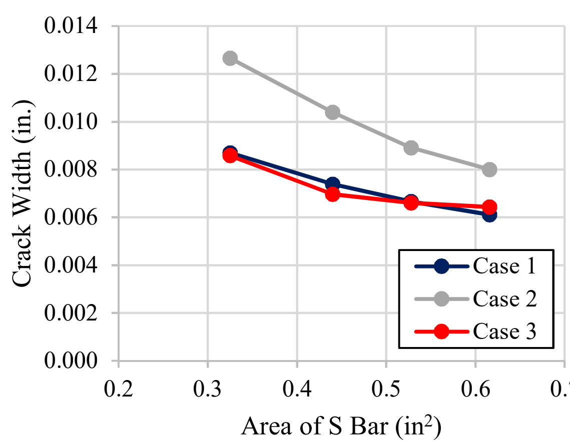
(d) Influence of S Bar Area on Bent 6 with 5 ksi Concrete



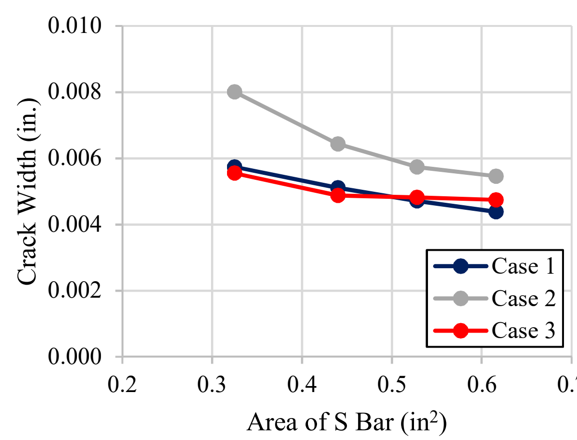
(e) Influence of S Bar area on Bent 6 with 7 ksi Concrete



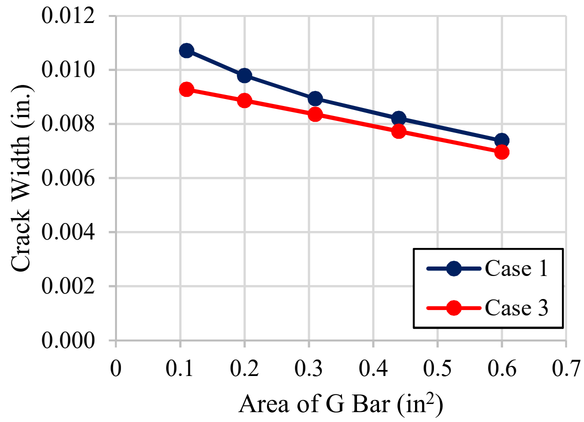
(f) Influence of G Bar area on Bent 6 with 5 ksi Concrete



(g) Influence of S Bar area on Bent 7 with 5 ksi Concrete



(h) Influence of S Bar area on Bent 7 with 7 ksi Concrete

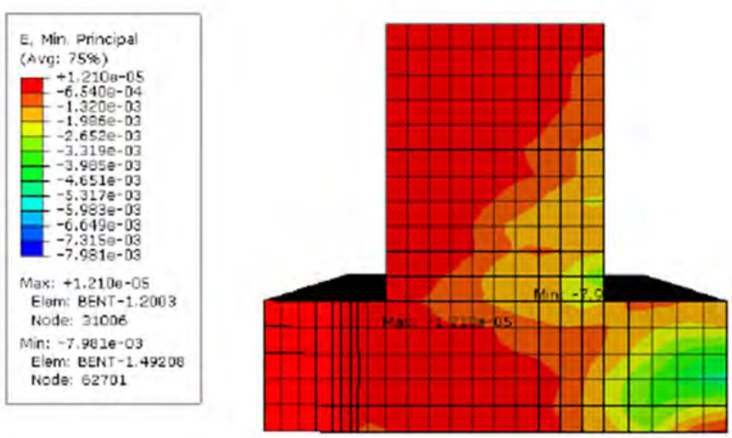


(i) Influence of G Bar area on Bent 7 with 5 ksi Concrete

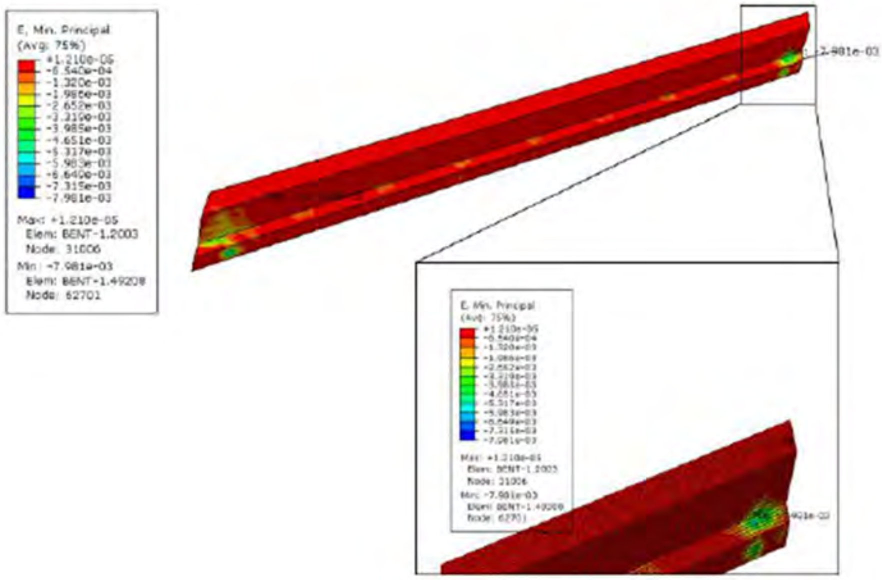
Figure 3.14 Comparison of Crack Width at the Service Load

3.4.3 Comparisons of Ultimate Capacity

To calculate the ultimate capacity of bent caps, the vertical force is uniformly applied at each bearing pad. Based on the FE analyses results, the deflections at point D1 as defined in Figure 3.11 are obtained and the load-displacement curve is defined for each specimen. The principal compressive strain of concrete at the ultimate capacity is obtained from the FE analyses. Figure 3.15 shows the principal compressive strain of concrete for specimen C3B2C5S0. As shown in Figure 3.15(a)–(c), the compressive softening of concrete material is localized around both ends of the specimen. The S Bars yielded at both ends of the specimen at the peak load, as shown in Figure 3.15(d). In addition, Figure 3.15(e) shows that the sectional view of reinforcement stress was not symmetrical, indicating the failure mode of Bent 2 is attributed to the combination of shear force and torsional moment instead of the shear failure.

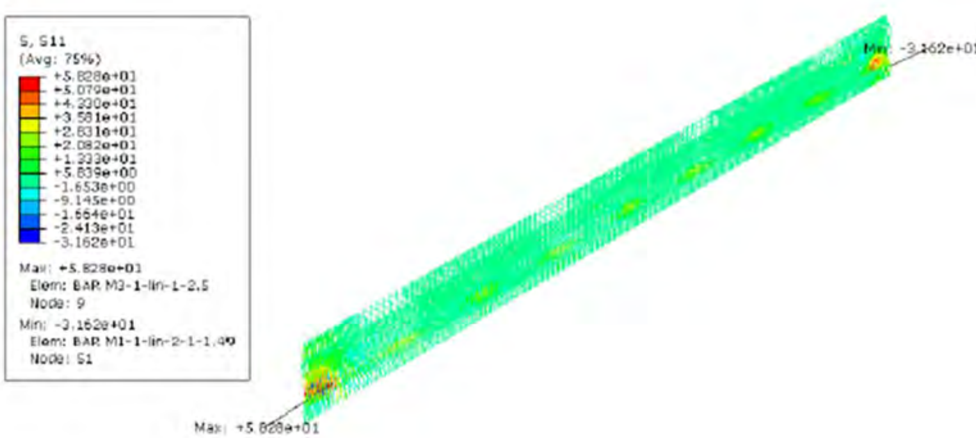


(a) Sectional View of Principal Compressive Strain of Concrete

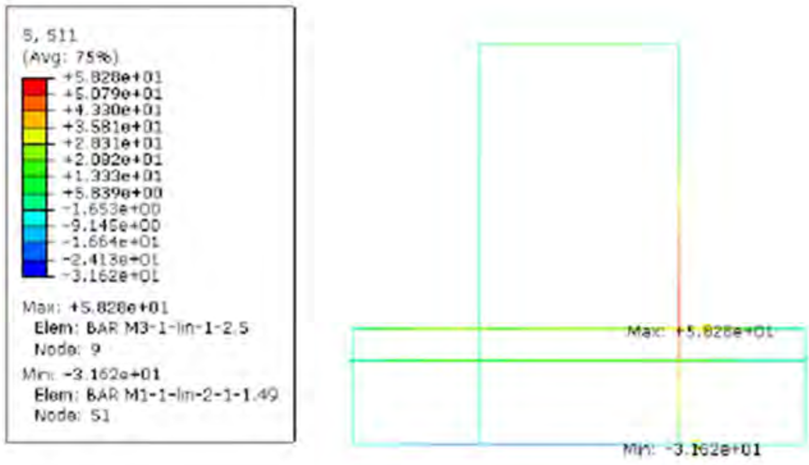


(b) Local View of Principal Compressive Strain of Concrete

(c) Plan View of Principal Compressive Strain of Concrete



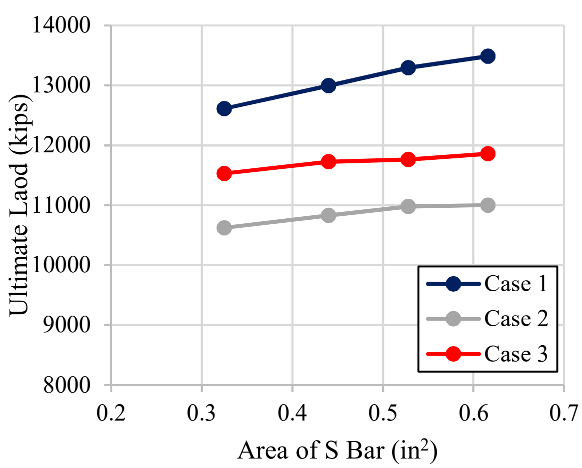
(d) Reinforcement Stress at the Peak Load



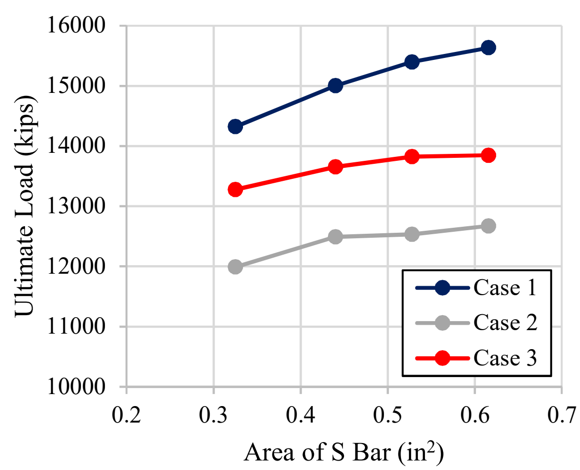
(e) Sectional View of Reinforcement Stress at the Peak Load

Figure 3.15 Stress and Strain Contours in Specimen C3B2C5S0 at the Ultimate Load

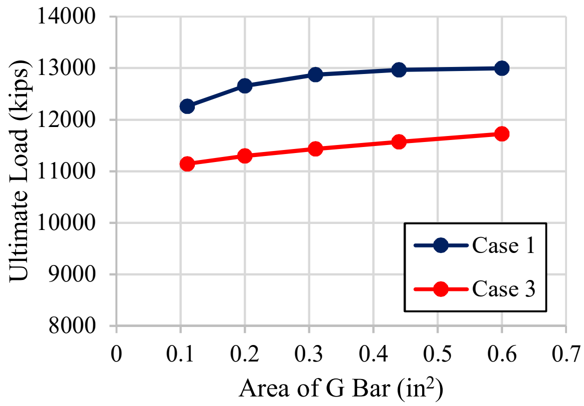
The ultimate capacity of specimens is compared in Figure 3.16. The ultimate capacity of specimens notably increases with the increase of the S Bar area and concrete compressive strength. In addition, the capacity of Case 2 and Case 3 are notably lower than Case 1, which indicates the rebar detailing has a significant influence on the ultimate capacity. For all bent caps, skew transverse reinforcement is better than the traditional transverse reinforcement. The dramatic difference between the specimens of Case 2 and Case 3 shows that end bars (U1, U2, U3, and G Bars) have a notable effect on the ultimate capacity. Moreover, the ultimate capacity of the ITBCs considerably increases with increasing the G Bar area.



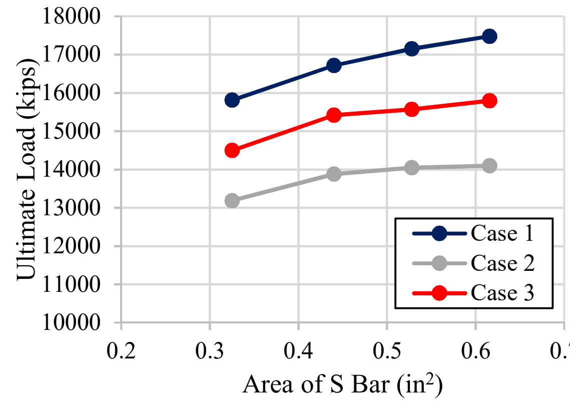
(a) Influence of S Bar Area on Bent 2 with 5 ksi Concrete



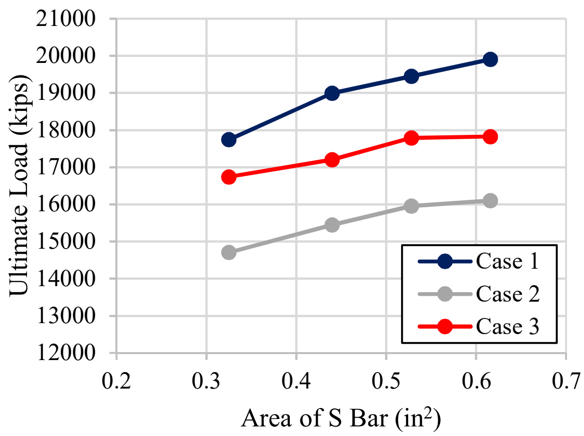
(b) Influence of S Bar Area on Bent 2 with 7 ksi Concrete



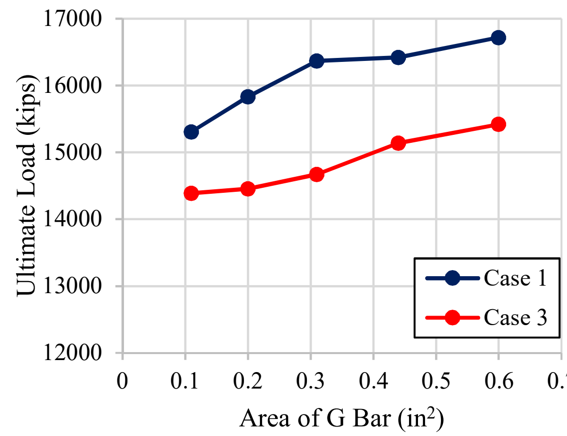
(c) Influence of G Bar Area on Bent 2 with 5 ksi Concrete



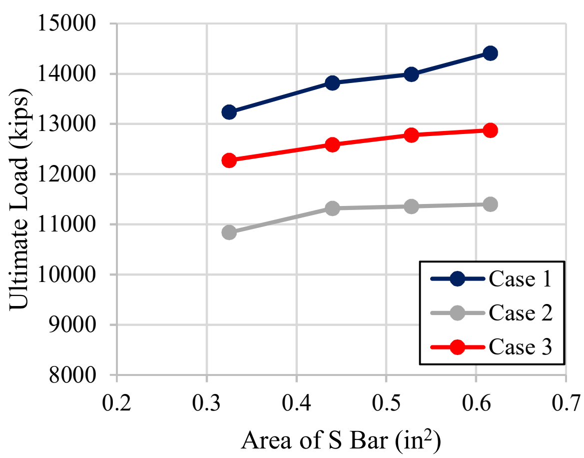
(d) Influence of S Bar Area on Bent 6 with 5 ksi Concrete



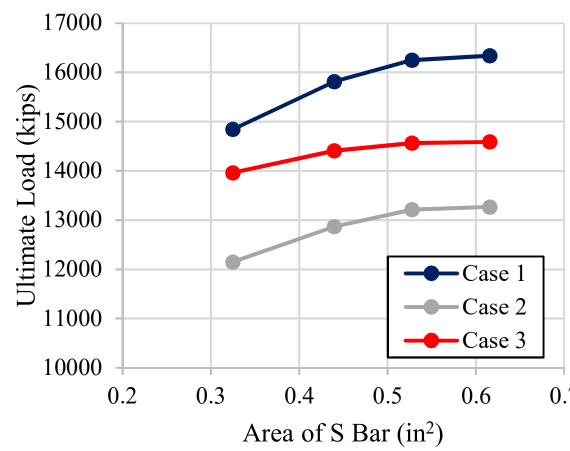
(e) Influence of S Bar Area on Bent 6 with 7 ksi Concrete



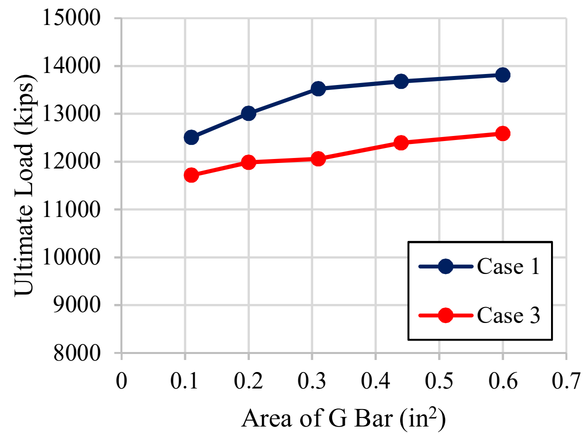
(f) Influence of G Bar Area on Bent 6 with 5 ksi Concrete



(g) Influence of S Bar Area on Bent 7 with 5 ksi Concrete



(h) Influence of S Bar Area on Bent 7 with 7 ksi Concrete



(i) Influence of G Bar area on Bent 7 with 5 ksi Concrete

Figure 3.16 Comparison of Ultimate Capacity

3.5 COST-BENEFIT ANALYSIS

A literature review is conducted on the cost analysis of bridges in Texas. The RT consulted many bridge engineers about the design and construction cost in bridge construction in conducting the cost-benefit analysis. In this analysis, only the direct costs of construction and design are considered. In this section, basic assumptions on cost estimation of ITBCs, and comparison of costs and benefits of the specimens are clarified.

3.5.1 Basic Assumptions

In cost estimation, only the direct costs, which are the cost for material and labor, design man-hour, and construction time schedules, of ITBCs are considered. To calculate the direct material cost, the quantity takeoff is performed for the specimens. Table 3.6, Table 3.7, and Table 3.8 show the quantity takeoff and the amount of materials of Bent Cap 2 for Case 1, Case 2, and Case 3, respectively. As a material cost, only reinforcing bars and concrete are included. The formwork, shoring tower placement, and removal are not included because these do not depend on the reinforcement detailing and concrete strength. As can be seen from Table 3.6,Table 3.7, and Table 3.8, the only difference in the material cost between the cases is the amount of M Bars, N Bars, S Bars, and the end bars (U1 Bars, U2 Bars, U3 Bars, and G Bars). The amount of the reinforcement bars for each specimen is estimated following the same steps. The total amount of concrete is calculated as 155 cubic yards for Bent Cap 2 and 135.4 cubic yards for Bent Cap 6 and Bent Cap 7. The influence of concrete strength on the cost is negligible. Therefore, the unit material cost and casting cost of 5 ksi concrete and 7 ksi concrete are assumed to be the same.

Table 3.6 Quantity Takeoff for Specimen C1B2C5S0

Reinforcement Bars					
Bar	**No.**	**Size**	**Area (in2)**	**Length (in.)**	**Weight (lbs)**
A	20	# 11	1.56	1389	12329
B	16	# 11	1.56	1389	9863
T	24	# 7	0.6	1389	5690
D	8	1 1/4"	1.23	20	56
M	234	# 7	0.6	329	13142
N	234	# 5	0.31	127	2621
S	388	# 6	0.44	299	14522
G	15	# 7	0.6	150	384
U1	12	# 6	0.44	157	236
U2	21	# 6	0.44	134	352
U3	12	# 6	0.44	171	257
Total					**59453**

Concrete		
Item	**Strength (psi)**	**Volume (cy)**
Class "F" Concrete (Cap)	5000	155

Table 3.7 Quantity Takeoff for Specimen C2B2C5S0

Reinforcement Bars					
Bar	**No.**	**Size**	**Area (in2)**	**Length (in.)**	**Weight (lbs)**
A	20	# 11	1.56	1389	12329
B	16	# 11	1.56	1389	9863
T	24	# 7	0.6	1389	5690
D	8	1 1/4"	1.23	20	56
M1	14	# 7	0.6	331.5	792
M2	2	# 7	0.6	323.7	111
M3	2	# 7	0.6	316.5	108
M4	2	# 7	0.6	311	106
M5	2	# 7	0.6	305	104
M6	2	# 7	0.6	297	101
M7	2	# 7	0.6	292	100
M8	2	# 7	0.6	287	98
M9	2	# 7	0.6	282	96
M10	2	# 7	0.6	277	95
M11	2	# 7	0.6	273	93
M12	2	# 7	0.6	270	92
M13	2	# 7	0.6	268	91
M14	2	# 7	0.6	266	91
M15	2	# 7	0.6	265	90

Reinforcement Bars					
Bar	**No.**	**Size**	**Area (in2)**	**Length (in.)**	**Weight (lbs)**
M16	192	# 7	0.6	262	8587
Total M	**234**	**# 7**	**0.6**	**#varies**	**10756**
N1	14	# 5	0.31	127	157
N2	2	# 5	0.31	124	22
N3	2	# 5	0.31	120	21
N4	2	# 5	0.31	117	21
N5	2	# 5	0.31	114	20
N6	2	# 5	0.31	110	19
N7	2	# 5	0.31	107	19
N8	2	# 5	0.31	105	19
N9	2	# 5	0.31	102	18
N10	2	# 5	0.31	100	18
N11	2	# 5	0.31	98	17
N12	2	# 5	0.31	97	17
N13	2	# 5	0.31	96	17
N14	2	# 5	0.31	95	17
N15	2	# 5	0.31	94	17
N16	192	# 5	0.31	93	1575
Total N	**234**	**# 5**	**0.31**	**#varies**	**1993**
S1	28	# 6	0.44	299	1048
S2	4	# 6	0.44	296	148
S3	4	# 6	0.44	293	147
S4	4	# 6	0.44	290	145
S5	4	# 6	0.44	287	144
S6	4	# 6	0.44	284	142
S7	4	# 6	0.44	282	141
S8	4	# 6	0.44	280	140
S9	4	# 6	0.44	277	139
S10	4	# 6	0.44	276	138
S11	4	# 6	0.44	274	137
S12	4	# 6	0.44	273	137
S13	4	# 6	0.44	272	136
S14	4	# 6	0.44	271	136
S15	4	# 6	0.44	270	135
S16	304	# 6	0.44	268	10199
Total S	**388**	**# 6**	**0.44**	**#varies**	**13212**
G	0	# 7	0.6	0	0
U1	0	# 6	0.44	0	0
U2	0	# 6	0.44	0	0

Reinforcement Bars					
Bar	**No.**	**Size**	**Area (in2)**	**Length (in.)**	**Weight (lbs)**
U3	0	# 6	0.44	0	0
Total					**53900**

Concrete		
Item	**Strength (psi)**	**Volume (cy)**
Class "F" Concrete (Cap)	5000	155

Table 3.8 Quantity Takeoff for Specimen C3B2C5S0

Case 3 / Bent Cap 2 Details					
Bar	**No.**	**Size**	**Area (in2)**	**Length (in.)**	**Weight (lbs)**
A	20	# 11	1.56	1389	12329
B	16	# 11	1.56	1389	9863
T	24	# 7	0.6	1389	5690
D	8	1 1/4"	1.23	20	56
M1	14	# 7	0.6	331.5	792
M2	2	# 7	0.6	323.7	111
M3	2	# 7	0.6	316.5	108
M4	2	# 7	0.6	311	106
M5	2	# 7	0.6	305	104
M6	2	# 7	0.6	297	101
M7	2	# 7	0.6	292	100
M8	2	# 7	0.6	287	98
M9	2	# 7	0.6	282	96
M10	2	# 7	0.6	277	95
M11	2	# 7	0.6	273	93
M12	2	# 7	0.6	270	92
M13	2	# 7	0.6	268	91
M14	2	# 7	0.6	266	91
M15	2	# 7	0.6	265	90
M16	192	# 7	0.6	262	8587
Total M	**234**	**# 7**	**0.6**	**#varies**	**10756**
N1	14	# 5	0.31	127	157
N2	2	# 5	0.31	124	22
N3	2	# 5	0.31	120	21
N4	2	# 5	0.31	117	21
N5	2	# 5	0.31	114	20
N6	2	# 5	0.31	110	19
N7	2	# 5	0.31	107	19
N8	2	# 5	0.31	105	19
N9	2	# 5	0.31	102	18

Case 3 / Bent Cap 2 Details					
Bar	**No.**	**Size**	**Area (in2)**	**Length (in.)**	**Weight (lbs)**
N10	2	# 5	0.31	100	18
N11	2	# 5	0.31	98	17
N12	2	# 5	0.31	97	17
N13	2	# 5	0.31	96	17
N14	2	# 5	0.31	95	17
N15	2	# 5	0.31	94	17
N16	192	# 5	0.31	93	1575
Total N	**234**	**# 5**	**0.31**	**#varies**	**1993**
S1	28	# 6	0.44	299	1048
S2	4	# 6	0.44	296	148
S3	4	# 6	0.44	293	147
S4	4	# 6	0.44	290	145
S5	4	# 6	0.44	287	144
S6	4	# 6	0.44	284	142
S7	4	# 6	0.44	282	141
S8	4	# 6	0.44	280	140
S9	4	# 6	0.44	277	139
S10	4	# 6	0.44	276	138
S11	4	# 6	0.44	274	137
S12	4	# 6	0.44	273	137
S13	4	# 6	0.44	272	136
S14	4	# 6	0.44	271	136
S15	4	# 6	0.44	270	135
S16	304	# 6	0.44	268	10199
Total S	**388**	**# 6**	**0.44**	**#varies**	**13212**
G	15	# 7	0.6	150	384
U1	12	# 6	0.44	157	236
U2	21	# 6	0.44	134	352
U3	12	# 6	0.44	171	257
Total					**55129**
Concrete					
Item	**Strength (psi)**				**Volume (cy)**
Class "F" Concrete (Cap)	5000				155

Table 3.9 shows the estimated construction time for skew and traditional reinforcement detailing in hours based on previous experiences. To estimate the values, the RT used a previous lab test where 6 laborers worked for 8 hours to prepare the caging of a skewed reinforcement detailing of a 20 ft bent cap. In addition, 6 laborers worked for 1 hour in pouring and vibrating the concrete of the same bent cap. For the 20 ft bent cap specimen with traditional reinforcement detailing, 4 more hours were spent than skewed

reinforcement to prepare the reinforcement cage, and 1 more hour was spent for casting concrete. The construction time for a 20 ft bent cap is scaled to predict the full-scale specimen with a length of 116 ft, and the total construction time is estimated as 310 hours for skewed reinforcement and 480 hours for traditional reinforcement.

Table 3.9 Estimated Construction Time

Item	Case		Unit
	Skewed	Traditional	
Rebar Preparation and Placement	280	420	hr
Concrete Casting	30	60	hr
Total	**310**	**480**	**hr**

The annual wage for rebar workers and concrete workers is obtained from the U.S. Bureau of Labor Statistics (Website, 2020) as $50,960 and $38,380, respectively. To determine the cost of employees, the payroll taxes, insurance, benefits, and supplies are also added to the annual wage. The hourly wage of rebar labor and concrete labor is calculated to be $30.81 and $24.30, respectively. Table 3.10 shows the items and amounts to calculate actual labor costs.

Table 3.10 Estimated Labor Wage

Item	Rebar Labors		Concrete Labors	
	Quantity	Unit	Quantity	Unit
Working Hour	2080	hr/year	2080	hr/year
Wage	24.5	$/hr	18.45	$/hr
Payroll Labor Cost	50960	$/yr	38380	$/yr
Payroll Taxes	4120	$/yr	3165	$/yr
Insurance	2000	$/yr	2000	$/yr
Benefits	2000	$/yr	2000	$/yr
Supplies	5000	$/yr	5000	$/yr
Total	64080	$/yr	50545	$/yr
Wage	**30.81**	**$/hr**	**24.30**	**$/hr**

Another item included in the cost analysis is the design procedure of bent caps. In this section, the design time is calculated, including engineering design, technical drawings, and review. It is assumed that a design engineer designs the bent cap, a draftsman does technical drawings, and a senior engineer reviews the project. After consulting with several bridge engineers, the design of traditional reinforcement detailing is estimated to require 40% more time than skew transverse reinforcement detailing. The design time and hourly wages of design are shown in Table 3.11 and Table 3.12, respectively

Table 3.11 Estimated Design Time

Item	Case		Unit
	Skewed	Traditional	
Engineering Design	30	42	hr
Drawing	60	84	hr
Review	4	6	hr

Table 3.12 Estimated Design Wage

Item	Quantity	Unit
Design Engineer	150	$/hr
Draftsman	120	$/hr
Senior Engineer	200	$/hr

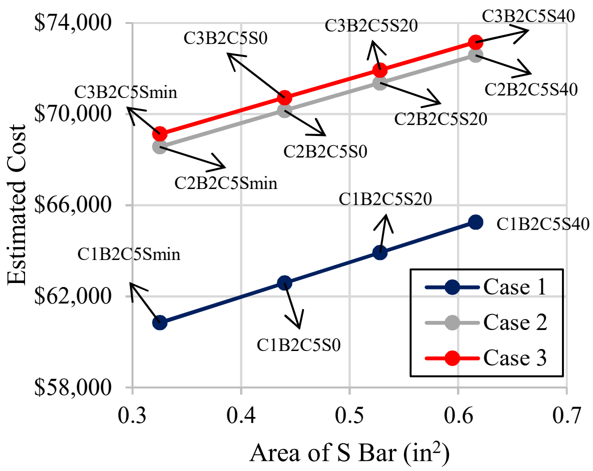
3.5.2 Comparison of Costs

The direct cost of ITBCs is calculated as the sum of the material cost, the labor cost, and the design cost. As an example, the estimated cost of Specimen C1B2C5S0 is shown in Table 3.13

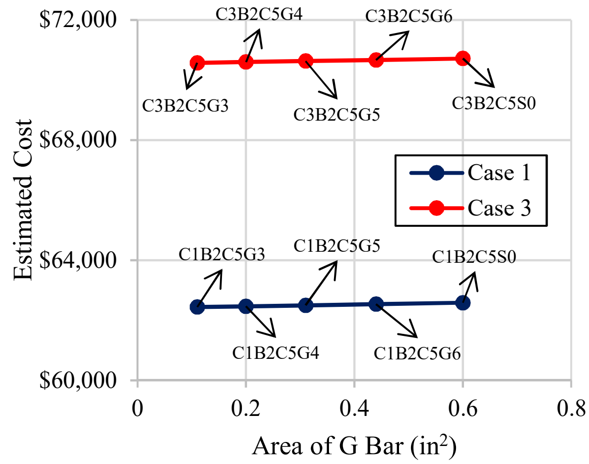
Table 3.13. The cost estimation is compared for Case 1, Case 2, and Case 3 in Figure 3.17. The cost analysis indicates that the cost of the specimens of Case 1 is 11% to 16% lower than the cost of the specimens of Case 3. The savings in cost are mainly attributed to the reduced construction hours and lower design costs. Therefore, the skew transverse reinforcement is notably effective in reducing the design and construction cost of skew ITBCs. In addition, the comparison in Figure 3.17 shows that adding G bars has very little influence on the direct cost while adding S bars has a larger influence on the direct cost. This is attributed to the fact that the G bars are only applied to both ends of the ITBCs while the S bars are applied uniformly in the ITBCs. Therefore, Figure 3.17 indicates that adding G bars is a more economical way of reducing the crack width observed at both ends of the ITBCs.

Table 3.13 Cost Estimation for Specimen C1B2C5S0

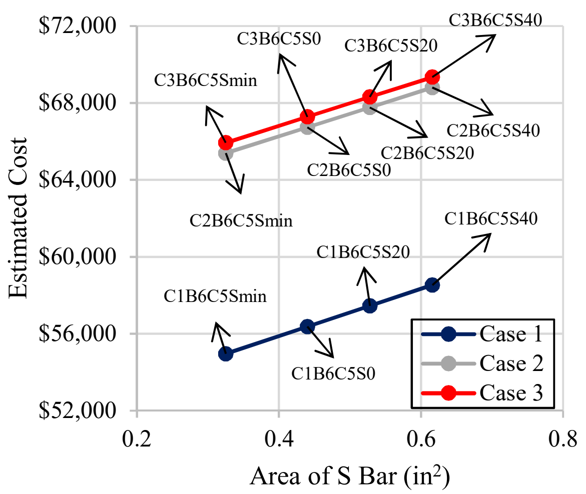
Item	Quantity	Unit	Unit Price	Total Price
Gr60 Reinforcing Bars	59453	lb	$0.46	$27,348.38
Class "F" Concrete (Cap)	155	cy	$86.35	$13,384.25
Design (Engineering)	30	hrs	$150.00	$4,500.00
Design (Technical Drawings)	60	hrs	$120.00	$7,200.00
Design (Reviewing)	4	hrs	$200.00	$800.00
Labor (Rebar)	280	hrs	$31	$8,624.00
Labor (Concrete)	30	hrs	$24	$729.00
Total				**$62,585.63**



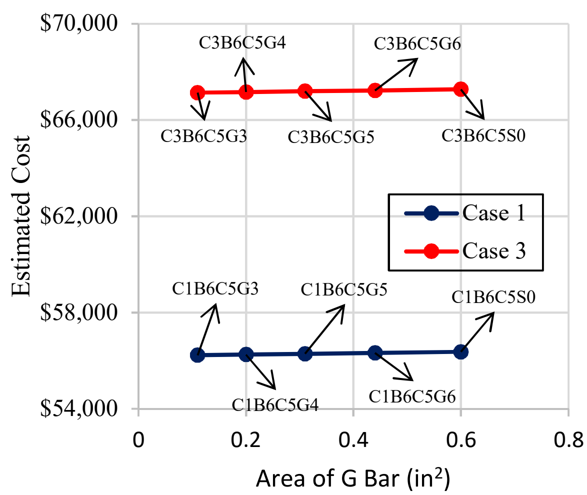
(a) Influence of S Bars on Cost for Bent 2



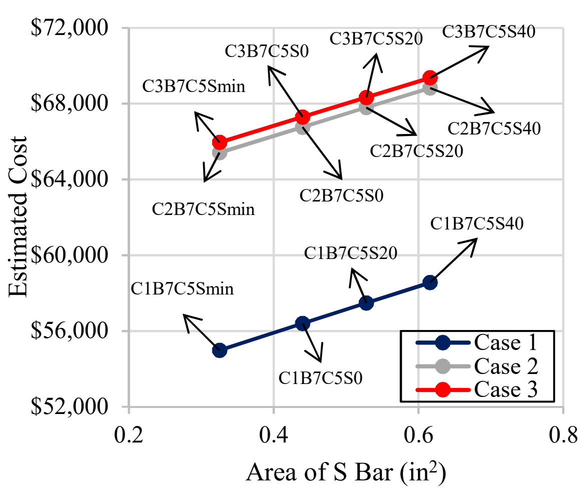
(b) Influence of G Bars on Cost for Bent 2



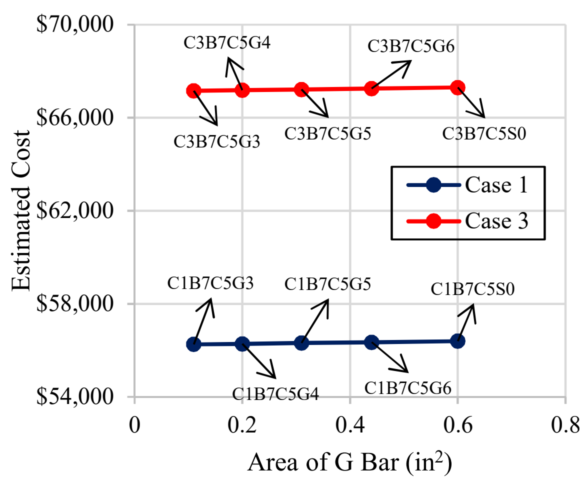
(c) Influence of S Bars on Cost for Bent 6



(d) Influence of G Bars on Cost for Bent 6



(e) Influence of S Bars on Cost for Bent 7



(f) Influence of G Bars on Cost for Bent 7

Figure 3.17 Comparison of Estimated Cost for Case 1, Case 2, and Case 3

3.5.3 Comparison of Benefits

Cost-benefit analysis is conducted for the specimens considering the stiffness, the crack widths, and the ultimate capacities. The FE analysis results presented in Section 3.4 "3D FINITE ELEMENT ANALYTICAL RESULTS OF BENT CAPS" are combined with the estimated costs. Table 3.14 shows all the calculated results of the cost-benefit analysis.

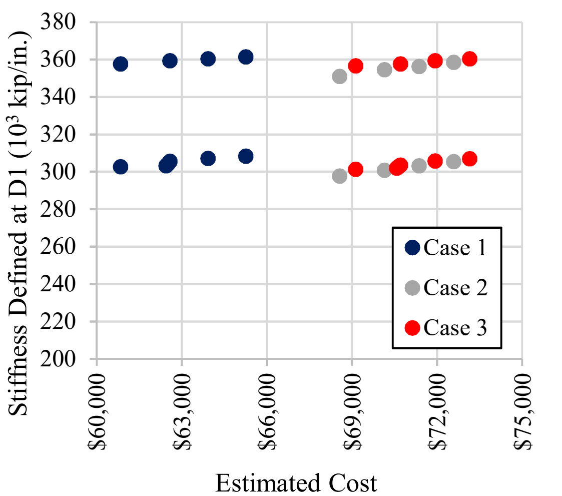
Table 3.14 Cost-Benefit Analysis Results

No.	Name	Cost	Stiffness defined at D1 (10^3 kip/in.)	Stiffness defined at D2 (10^3 kip/in.)	Crack Width (in.)	Ultimate Load (kips)
1	C1B2C5Smin	$60,839	302.6	284.4	0.0082	12613
2	C1B2C5S0	$62,585	305.6	287.2	0.0069	12997
3	C1B2C5S20	$63,921	307.1	288.8	0.0062	13293
4	C1B2C5S40	$65,257	308.2	289.8	0.0057	13488
5	C1B2C7Smin	$60,839	357.6	336.9	0.0054	14322
6	C1B2C7S0	$62,585	359.3	338.4	0.0047	15002
7	C1B2C7S20	$63,921	360.4	339.6	0.0043	15394
8	C1B2C7S40	$65,257	361.4	340.7	0.0040	15633
9	C1B6C5Smin	$54,954	388.3	371.8	0.0068	15812
10	C1B6C5S0	$56,368	390.9	374.8	0.0058	16719
11	C1B6C5S20	$57,450	392.2	376.3	0.0053	17152
12	C1B6C5S40	$58,532	393.4	377.5	0.0048	17480
13	C1B6C7Smin	$54,954	458.8	441.0	0.0043	17743
14	C1B6C7S0	$56,368	460.7	442.9	0.0038	18999
15	C1B6C7S20	$57,450	462.3	444.6	0.0033	19450
16	C1B6C7S40	$58,532	463.5	445.6	0.0029	19908
17	C1B7C5Smin	$54,980	336.7	360.3	0.0087	13237
18	C1B7C5S0	$56,394	341.1	364.2	0.0074	13816
19	C1B7C5S20	$57,476	343.4	365.9	0.0067	13990
20	C1B7C5S40	$58,558	345.2	367.4	0.0061	14415
21	C1B7C7Smin	$54,980	403.7	428.0	0.0057	14843
22	C1B7C7S0	$56,394	405.7	430.0	0.0051	15811
23	C1B7C7S20	$57,476	407.0	431.3	0.0047	16245
24	C1B7C7S40	$58,558	408.1	432.4	0.0044	16338
25	C1B2C5G3	$62,441	303.2	285.3	0.0091	12259
26	C1B2C5G4	$62,467	303.6	285.6	0.0087	12656
27	C1B2C5G5	$62,500	304.1	286.0	0.0082	12870
28	C1B2C5G6	$62,538	304.7	286.6	0.0076	12967
29	C1B6C5G3	$56,229	387.3	371.3	0.0081	15310
30	C1B6C5G4	$56,255	388.1	372.0	0.0076	15833
31	C1B6C5G5	$56,286	389.4	373.3	0.0069	16368
32	C1B6C5G6	$56,323	390.2	374.0	0.0064	16422
33	C1B7C5G3	$56,255	336.4	360.4	0.0107	12509
34	C1B7C5G4	$56,281	337.7	361.2	0.0098	13007
35	C1B7C5G5	$56,312	338.7	362.1	0.0089	13522
36	C1B7C5G6	$56,348	339.7	363.0	0.0082	13681
37	C2B2C5Smin	$68,563	297.7	278.6	0.0105	10623
38	C2B2C5S0	$70,152	300.8	282.0	0.0086	10830
39	C2B2C5S20	$71,367	303.1	283.7	0.0071	10978
40	C2B2C5S40	$72,583	305.4	285.4	0.0061	11002
41	C2B2C7Smin	$68,563	350.9	325.7	0.0066	11989
42	C2B2C7S0	$70,152	354.5	328.2	0.0056	12490

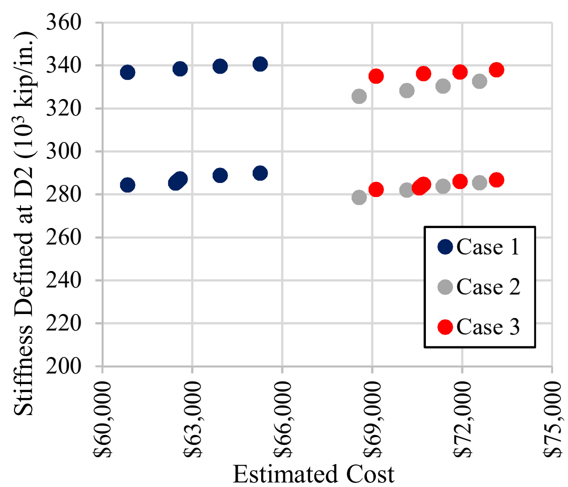
No.	Name	Cost	Stiffness defined at D1 (10^3 kip/in.)	Stiffness defined at D2 (10^3 kip/in.)	Crack Width (in.)	Ultimate Load (kips)
43	C2B2C7S20	$71,367	356.3	330.5	0.0051	12536
44	C2B2C7S40	$72,583	358.5	332.6	0.0046	12673
45	C2B6C5Smin	$65,387	397.4	375.9	0.0107	13190
46	C2B6C5S0	$66,736	401.2	382.6	0.0088	13881
47	C2B6C5S20	$67,768	402.9	387.0	0.0071	14045
48	C2B6C5S40	$68,800	404.4	389.9	0.0062	14098
49	C2B6C7Smin	$65,387	474.9	452.7	0.0064	14705
50	C2B6C7S0	$66,736	476.7	457.5	0.0049	15447
51	C2B6C7S20	$67,768	477.1	459.1	0.0044	15956
52	C2B6C7S40	$68,800	477.5	460.7	0.0041	16102
53	C2B7C5Smin	$65,413	347.3	364.9	0.0127	10840
54	C2B7C5S0	$66,762	356.5	373.9	0.0104	11317
55	C2B7C5S20	$67,794	359.7	378.4	0.0089	11357
56	C2B7C5S40	$68,826	361.7	382.6	0.0080	11400
57	C2B7C7Smin	$65,413	423.3	439.6	0.0080	12150
58	C2B7C7S0	$66,762	426.5	446.8	0.0064	12867
59	C2B7C7S20	$67,794	428.5	449.6	0.0057	13211
60	C2B7C7S40	$68,826	429.8	451.4	0.0055	13266
61	C3B2C5Smin	$69,129	301.2	282.3	0.0086	11530
62	C3B2C5S0	$70,717	303.5	284.7	0.0071	11725
63	C3B2C5S20	$71,933	305.7	285.9	0.0059	11764
64	C3B2C5S40	$73,148	306.9	286.8	0.0054	11859
65	C3B2C7Smin	$69,129	356.5	335.0	0.0053	13277
66	C3B2C7S0	$70,717	357.6	336.3	0.0045	13653
67	C3B2C7S20	$71,933	359.2	337.0	0.0037	13823
68	C3B2C7S40	$73,148	360.4	338.0	0.0036	13846
69	C3B6C5Smin	$65,927	405.4	384.2	0.0077	14496
70	C3B6C5S0	$67,275	407.0	388.7	0.0061	15421
71	C3B6C5S20	$68,308	407.8	391.0	0.0054	15572
72	C3B6C5S40	$69,340	408.3	392.1	0.0053	15798
73	C3B6C7Smin	$65,927	478.2	457.3	0.0046	16743
74	C3B6C7S0	$67,275	478.9	460.3	0.0037	17204
75	C3B6C7S20	$68,308	479.3	461.7	0.0037	17787
76	C3B6C7S40	$69,340	479.5	462.6	0.0037	17828
77	C3B7C5Smin	$65,952	361.6	376.4	0.0086	12277
78	C3B7C5S0	$67,301	363.8	381.7	0.0070	12589
79	C3B7C5S20	$68,333	365.1	384.0	0.0066	12782
80	C3B7C5S40	$69,366	366.1	385.5	0.0064	12874
81	C3B7C7Smin	$65,952	430.1	447.7	0.0055	13962
82	C3B7C7S0	$67,301	431.4	450.9	0.0049	14408
83	C3B7C7S20	$68,333	431.8	452.1	0.0048	14566
84	C3B7C7S40	$69,366	432.1	453.6	0.0047	14589
85	C3B2C5G3	$70,573	301.9	283.0	0.0081	11144
86	C3B2C5G4	$70,599	302.2	283.3	0.0079	11294
87	C3B2C5G5	$70,632	302.6	283.7	0.0077	11432
88	C3B2C5G6	$70,670	303.0	284.2	0.0074	11568
89	C3B6C5G3	$67,137	404.0	384.7	0.0081	14389
90	C3B6C5G4	$67,162	404.7	385.4	0.0077	14458
91	C3B6C5G5	$67,193	405.6	386.7	0.0071	14669
92	C3B6C5G6	$67,230	406.4	387.6	0.0067	15141
93	C3B7C5G3	$67,163	359.7	377.3	0.0093	11717

No.	Name	Cost	Stiffness defined at D1 (10^3 kip/in.)	Stiffness defined at D2 (10^3 kip/in.)	Crack Width (in.)	Ultimate Load (kips)
94	C3B7C5G4	$67,188	361.0	378.1	0.0089	11983
95	C3B7C5G5	$67,219	362.0	379.1	0.0084	12058
96	C3B7C5G6	$67,256	362.9	380.2	0.0077	12391

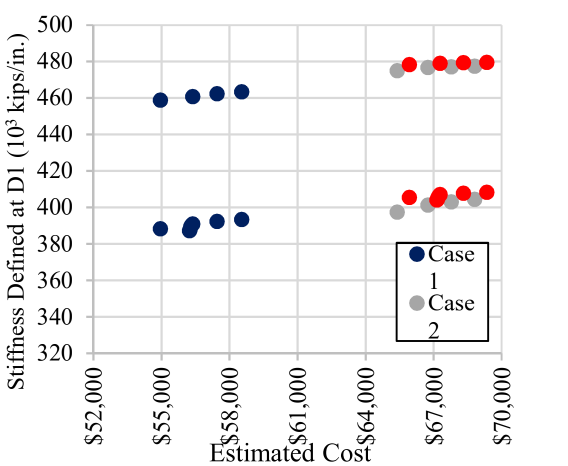
Figure 3.18 shows the cost and stiffness comparison of the specimens. In Figure 3.18, each point stands for the result of a specimen in the parametric analysis. Case 1 is marked by blue, Case 2 is marked by gray, and Case 3 is marked by red. For Bent Cap 2, the stiffness value of Case 1 is slightly higher than that of both Case 2 and Case 3. For Bent Cap 6 and Bent Cap 7, the stiffness value of Case 1 is slightly lower than that of both Case 2 and Case 3. The cost of Case 1 is notably lower than that of both Case 2 and Case 3.



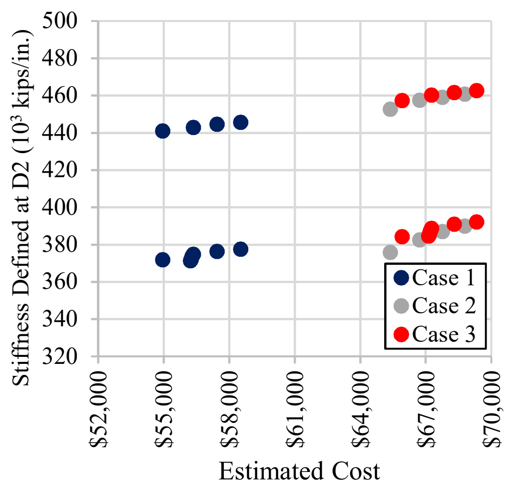
(a) Cost versus stiffness of Bent 2 defined at D1



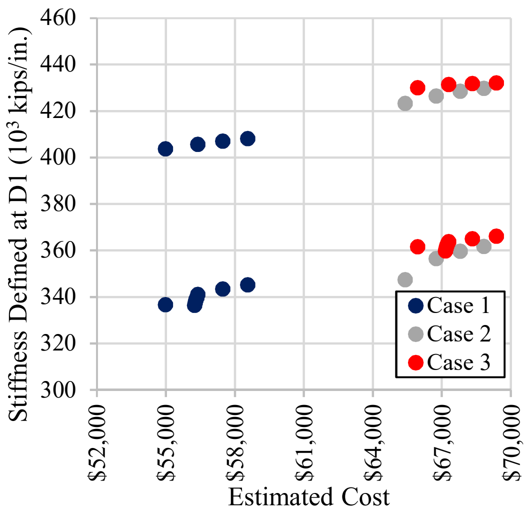
(b) Cost versus stiffness of Bent 2 defined at D2



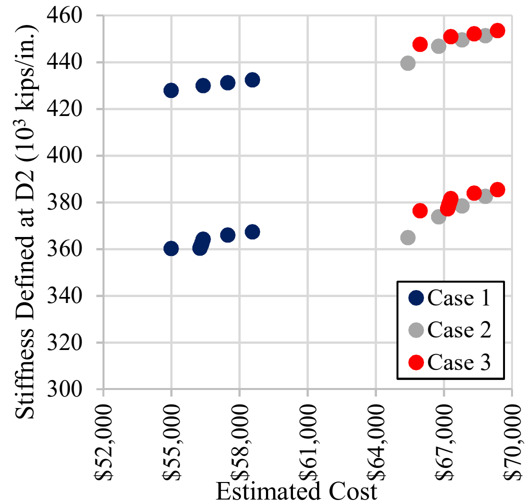
(c) Cost versus stiffness of Bent 6 defined at D1



(d) Cost versus stiffness of Bent 6 defined at D2



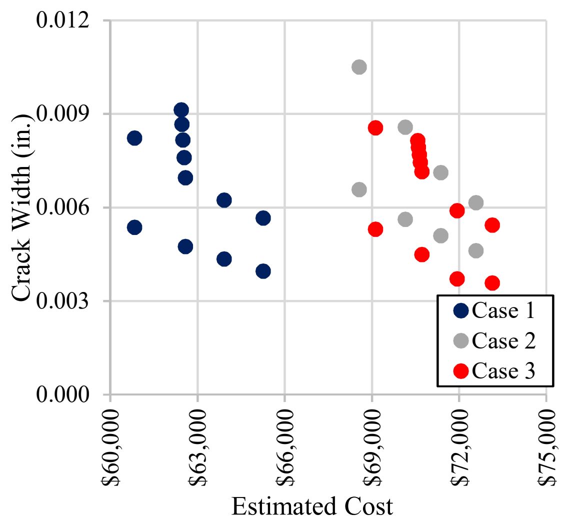
(e) Cost versus stiffness of Bent 7 defined at D1



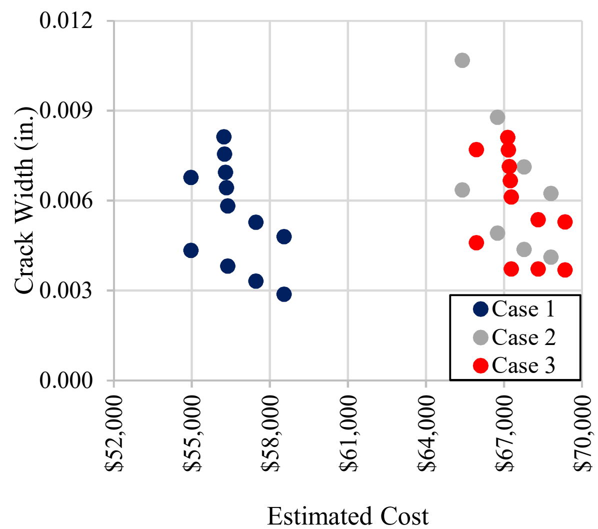
(f) Cost versus stiffness of Bent 7 defined at D2

Figure 3.18 Cost and Stiffness Comparison of Bent 2, Bent 6, and Bent 7

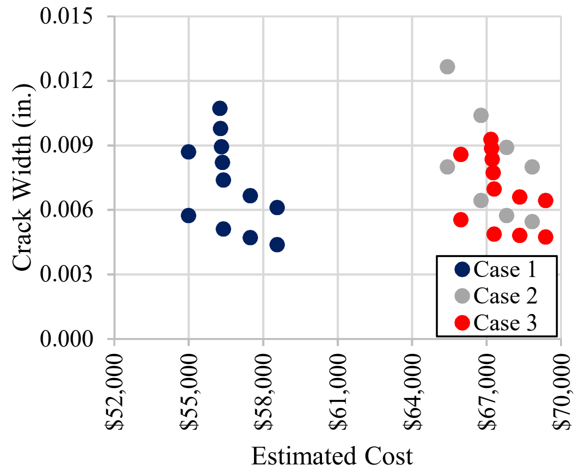
Figure 3.19 shows the cost and crack width comparisons of the specimens. Case 2 has the largest crack widths for all bent caps. For Bent Cap 2, the result of Case 1 and Case 3 are almost equal. For Bent Cap 6 and Bent Cap 7, specimens in Case 1 always have a smaller crack width than Case 3.



(a) Bent 2



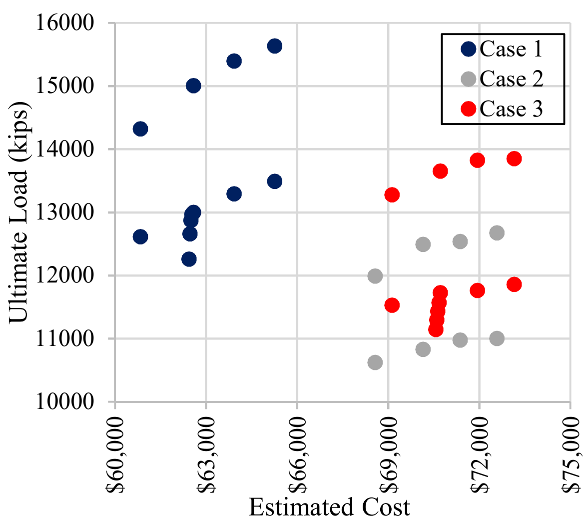
(b) Bent 6



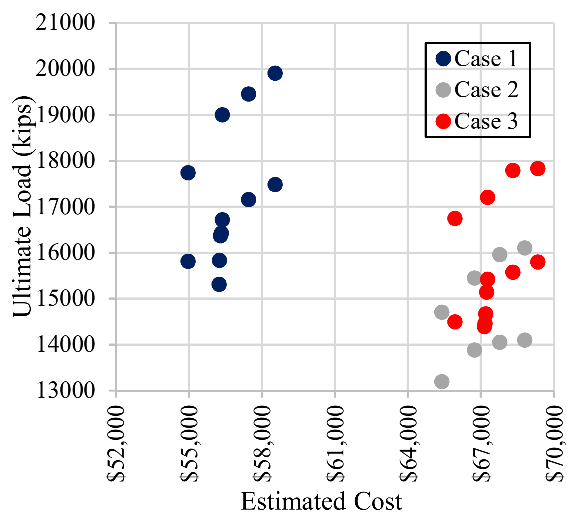
(c) Bent 7

Figure 3.19 Cost and Crack Width Comparisons of Bent 2, Bent 6, and Bent 7

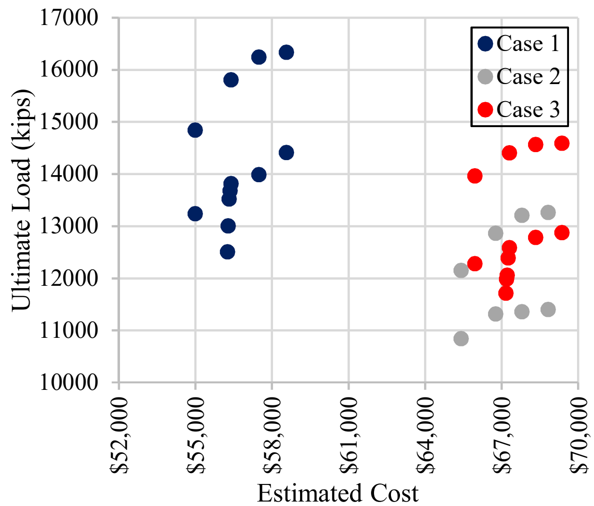
Figure 3.20 shows the cost and ultimate capacity comparisons of the specimens. As shown in Figure 3.20, Case 1 has a notably enhanced ultimate capacity than Case 2 and Case 3.



(a) Bent 2



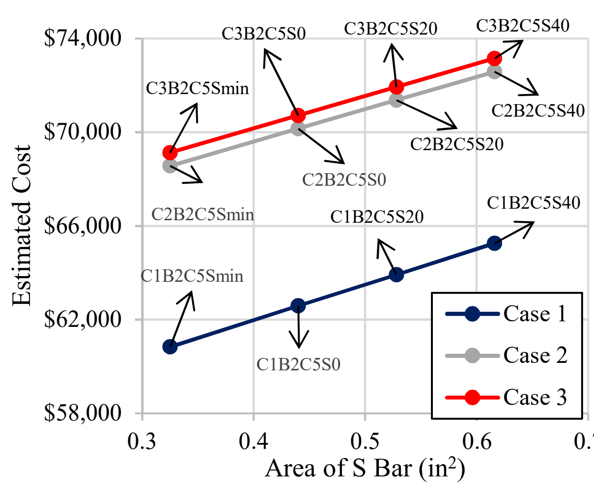
(b) Bent 6



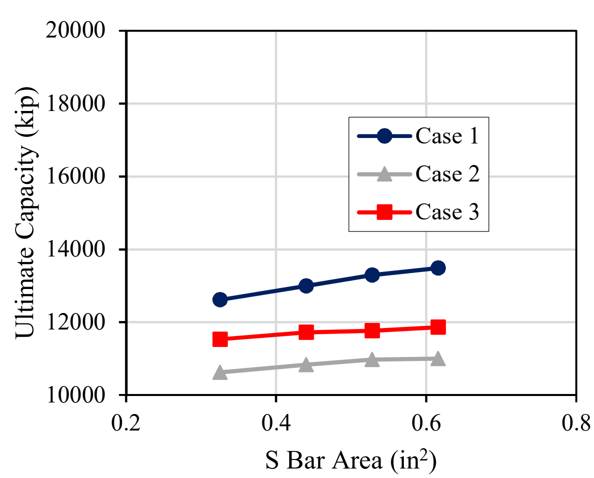
(c) Bent 7

Figure 3.20 Cost and Ultimate Load Comparisons of Bent 2, Bent 6, and Bent 7

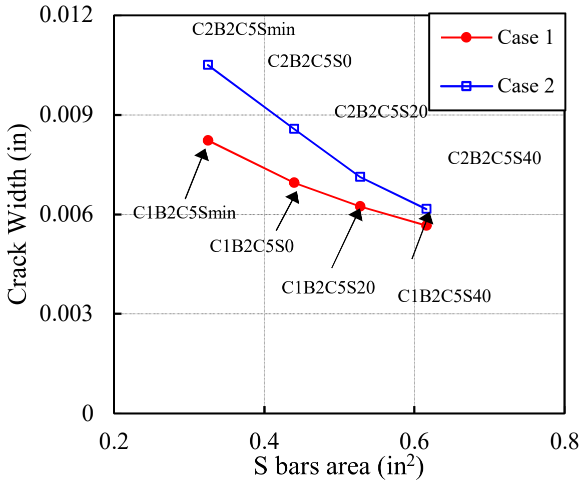
Figure 3.21 shows the influence of the S Bar area on the cost and performance of Bent 2 with 5 ksi concrete. As shown in Figure 3.21(a), the increase of the S Bar area contributes to the construction cost. As shown in Figure 3.21(b), the FE simulation results show that the stiffness notably increases with the S Bar area. As shown in Figure 3.21(c) and Figure 3.21(d), increasing the S Bar area reduces the maximum crack width significantly. As shown in Table 3.14, based on the parametric simulation results, the calculated maximum crack width of 0.0127 in. was observed in Specimen C2B7C5Smin. As recommended by the Article 5.6.7 of AASHTO LRFD Specifications (2017), the limit for crack width is 0.017 in. for Class 1 exposure condition and 0.013 in. for Class 2 exposure condition. Therefore, the minimum reinforcement area of S Bars based on the design service load and the AASHTO specifications (2014), which is 26% lower than the current design, is adequate for crack control. Based on the parametric simulation results, the current design of the S Bar area is adequate for structural safety and crack resistance.



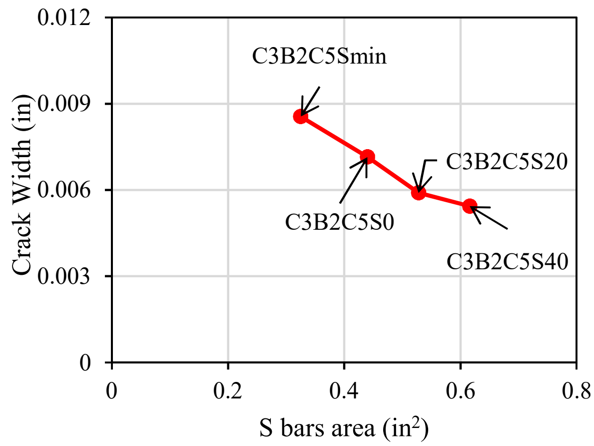
(a) Influence of S Bar Area on Cost



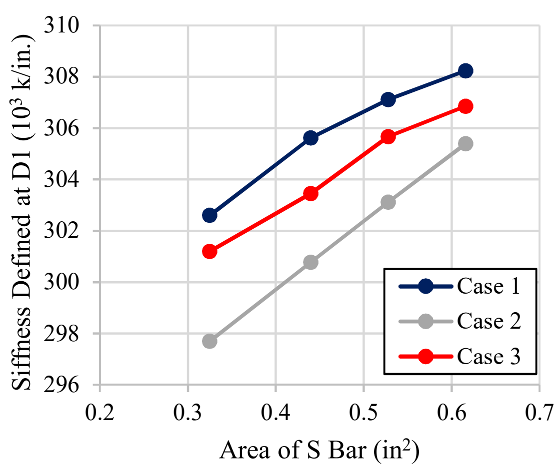
(b) Influence of S Bar Area on Ultimate Capacity



(c) Influence of S bar Area on Crack width of Case 1 and Case 2



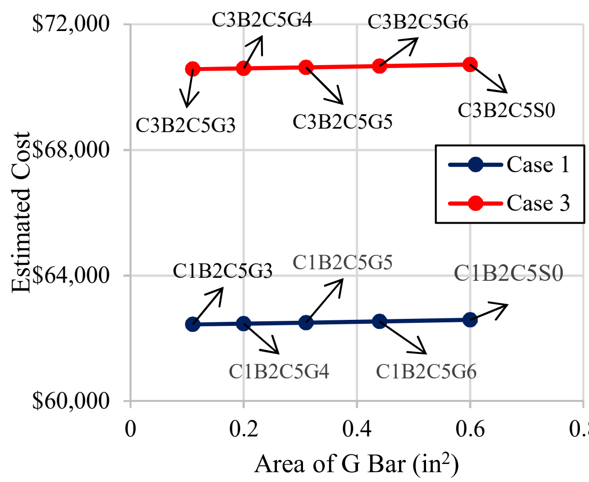
(d) Influence of S Bar Area on Crack width of Case 3



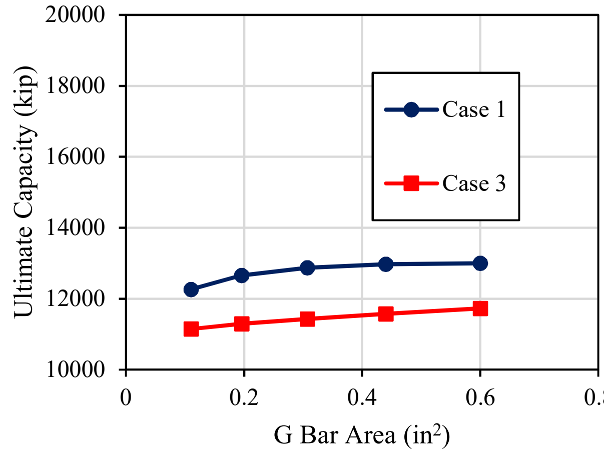
(e) Influence of S Bar Area on Stiffness

Figure 3.21 Influence of S Bar Area on Cost and Performance of Bent 2 with 5 ksi concrete

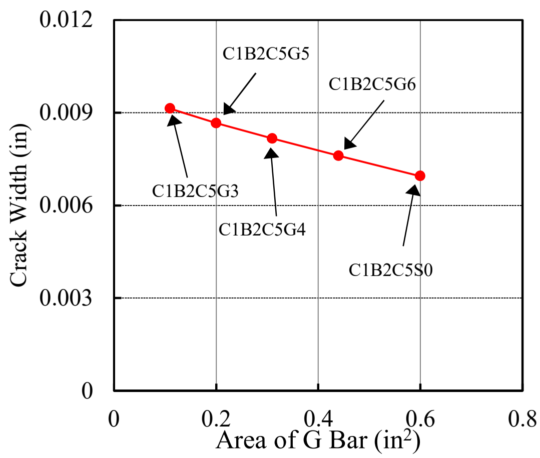
Figure 3.22 shows the influence of the G Bar area on the cost and performance of Bent 2 with 5 ksi concrete. As shown in Figure 3.22(a), the increase of the G Bar area has little influence on the construction cost. As shown in Figure 3.22(b), the FE analysis results show that the G Bar area has little influence on the ultimate capacity. As shown in Figure 3.22(c) and Figure 3.22(d), increasing the G Bar area reduces the maximum crack width significantly. Based on the comparison between Figure 3.21 and Figure 3.22, the S Bar area has a more notable influence on the crack width than the G Bar area. As shown in Table 3.14, the maximum crack width of all specimens with the current design of G Bar (No. 7 Bars) is 0.0127 in. (Specimen C2B7C5Smin), which meets the AASHTO (2017) requirements for both Class 1 and Class 2 exposure conditions. In conclusion, the current design of G Bar (No. 7 Bars) is adequate for crack control.



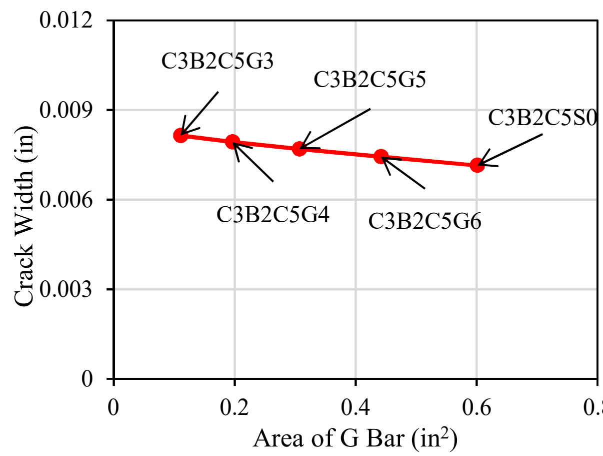
(a) Influence G Bar Area on Cost



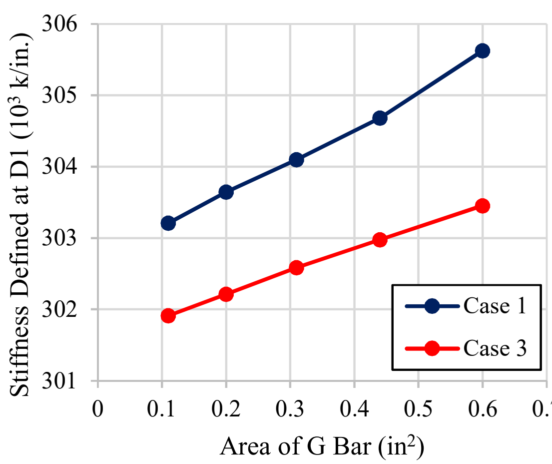
(b) Influence of G Bar Area on the Ultimate Capacity



(c) Influence of G Bar Area on Crack Width of Case 1



(d) Influence of G Bar area on crack width of Case 3



(e) Influence of G Bar area on Stiffness

Figure 3.22 Influence of G Bar Area on Cost and Performance of Bent 2 with 5 ksi concrete

3.6 SUMMARY

In Chapter 3 (Task 9a), three cases of reinforcement design for ITBCs are investigated to cover the majority of the design detailing in Texas bridges. Based on the parametric FE simulation of 96 specimens and the cost-benefit analysis results, the conclusions are summarized as follows:

(1) The skew transverse reinforcement (Case 1) achieves better structural performance compared to traditional transverse reinforcement (Case 2 and Case 3) with notably reduced construction cost. Therefore, the skew transverse reinforcement can well be used for the design of skewed ITBCs.

(2) For skew reinforcing, smaller number of cracks and smaller crack width will be achieved.

(3) The increase of the S Bar area notably enhances the stiffness and ultimate strength. In addition, the increase of the S Bar area also reduces the crack width. The increase of the S Bar area will contribute notably to the construction cost. Based on the parametric simulation results, the current design of the S bar area is adequate for structural safety and crack resistance.

(4) The increase of the G Bar area notably reduces the maximum crack width with a negligible influence on the stiffness, ultimate strength, and construction cost. The current design of the G Bar (No. 7 Bars) is adequate for crack control.

(5) When the concrete strength increases from 5 ksi to 7 ksi, the ultimate strength and the stiffness of ITBCs increase with reduced crack width. In addition, the influence of concrete strength on the construction cost is negligible.

Task 9a will significantly leverage the impact of this project and solve the dearth of reliable design methods and reinforcement detailing in the design of skewed ITBCs.

CHAPTER 4: DESIGN RECOMMENDATIONS AND DESIGN EXAMPLES

Finite element models of the significant ITBCs explained in Chapter 2 and Chapter 3, show that all the bent caps with skew transverse reinforcing are safe under service and limit state loading. Moreover, from the cost-benefit analysis, it is observed that the skew transverse reinforcement achieves better structural performance compared to traditional transverse reinforcement with notably reduced construction cost. Therefore, the skew transverse reinforcement can well be used for the design of skewed ITBCs.

In this chapter, design recommendations for skewed ITBCs are explained and four different design examples are presented following AASHTO LRFD Bridge Design Specifications, 8th Ed. (2017) and TxDOT Bridge Manual - LRFD (January 2020). The previous ITBC design example published by TxDOT is in accordance with the AASHTO LRFD Bridge Design Specifications, 5th Ed. (2010) as prescribed by TxDOT Bridge Design Manual - LRFD (May 2009). The updates from AASHTO LRFD 2010 to AASHTO LRFD 2017 are provided in Appendix 1.

4.1 DESIGN RECOMMENDATIONS

According to AASHTO LRFD (2017), TxDOT BDM (2020), and finite element analysis results of the significant ITBCs (Task 9 and Task 9a), the design recommendations for skew reinforcing bars are suggested below:

1. It is recommended to use skew transverse reinforcement for the design of skewed ITBCs. As explained in detail in Chapter 3, the skew transverse reinforcement achieves better structural performance compared to traditional transverse reinforcement with notably reduced construction cost.

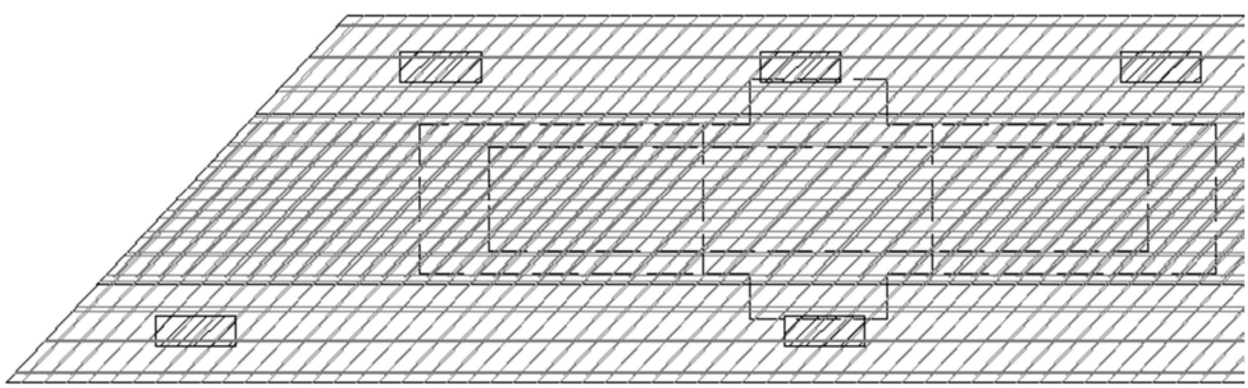


Figure 4.1 Skewed Transverse Reinforcement in skewed ITBCs

2. It is recommended to design double S Bars throughout the bent cap. The spacing of S Bars can be increased at the location of column support, no greater than 12”.
3. For skewed ITBCs design, M Bars and N Bars are paired together with equal spacing, which needs to be equal to or an integer multiple of the spacing of S Bars.

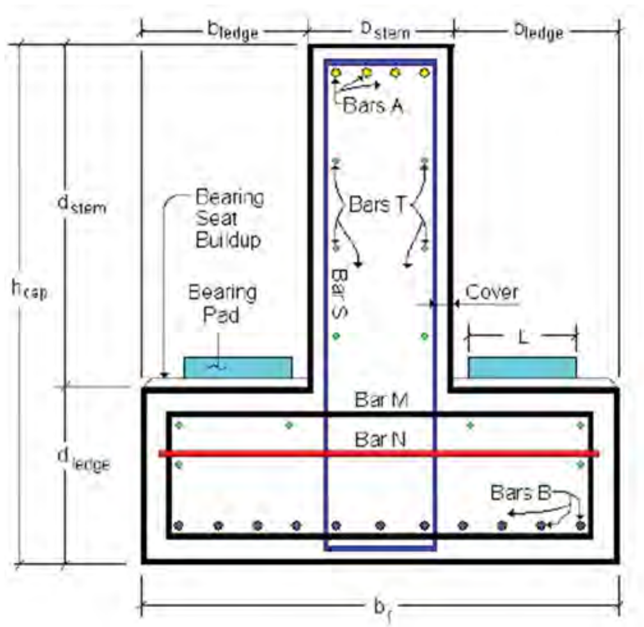


Figure 4.2 Typical Section View of ITBCs

4. The stem width (b_{stem}) is at least 3" wider than the column diameter.
5. As a general rule of thumb, ledge depth (d_{ledge}) is greater than or equal to 2'-3", which is the depth at which a bent from a typical bridge will pass the punching shear check.
6. The distance from the face of the stem to center of bearing pad is 12" for TxGirders.
7. The end bars (U1 Bars, U2 Bars, U3 Bars, and G Bars) notably reduces the maximum crack width. It is recommended to place #6 U1 Bars, U2 Bars, and U3 Bars at the end faces and #7 G Bars at approximately 6in. spacing at the first 30" to 35" of the end of the bent cap. U1 Bars are vertical end reinforcements, U2 Bars, and U3 Bars are horizontal end reinforcements at the stem and the ledge, respectively. G Bars are the diagonal end reinforcement.
8. TxDOT Bridge Design Manual – LRFD Ch. 4, Sect. 5 limits the minimum concrete compressive strength as $f_c' = 3.6$ ksi. However, finite element models in Task 9a shows that concrete strength notably increases the ultimate strength and the stiffness of ITBCs and reduces crack width. Therefore, it is recommended to have concrete compressive strength at least $f_c' = 5$ ksi.

4.2 INVERTED-T BENT CAP DESIGN EXAMPLE 1 (0° SKEW ANGLE)

Design example is in accordance with the AASHTO LRFD Bridge Design Specifications, 8th Ed. (2017) as prescribed by TxDOT Bridge Manual - LRFD (January 2020).

4.2.1 Design Parameters

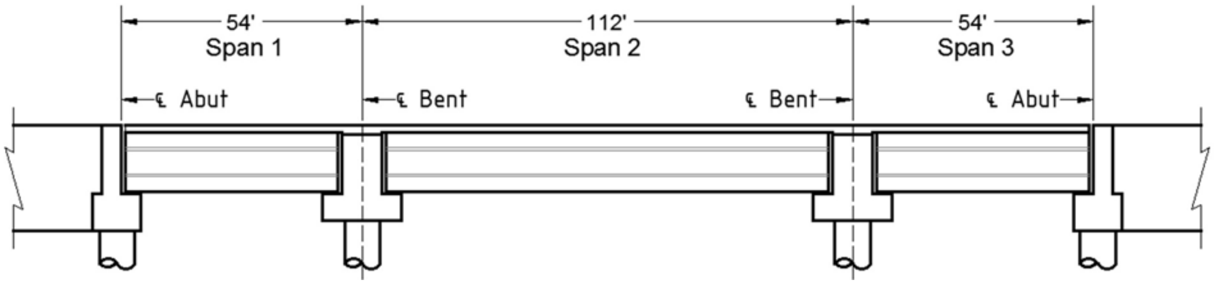


Figure 4.3 Spans of the Bridge with 0 Degree Skewed ITBC

Span 1

54' Type TX54 Girders (0.851 k/ft)

6 Girders Spaced @ 8.00' with 3' overhangs

2" Haunch

Span 2

112' Type TX54 Girders (0.851 k/ft)

6 Girders Spaced @ 8.00' with 3' overhangs

3.75" Haunch

Span 3

54' Type TX54 Girders (0.851 k/ft)

6 Girders Spaced @ 8.00' with 3' overhangs

2" Haunch

All Spans

Deck is 46 ft wide

Type T551 Rail (0.382 k/ft)

8" Thick Slab (0.100 ksf)

Assume 2" Overlay @ 140 pcf (0.023 ksf)

Use Class "C" Concrete

$f'_c = 5$ ksi

$w_c = 150$ pcf (for weight)

$w_c = 145$ pcf (for Modulus of Elasticity calculation)

"AASHTO LRFD" refers to the ASSHTO LRFD Bridge Design Specification, 8th Ed. (2017)..

"BDM-LRFD" refers to the TxDOT Bridge Design Manual - LRFD (January 2020).

"TxSP" refers to TxDOT guidance, recommendations, and standard practice.

"Furlong & Mirza" refers to "Strength and Serviceability of Inverted T-Beam Bent Caps Subject to Combined Flexure, Shear, and Torsion", Center for Highway Research Research Report No. 153-1F, The University of Texas at Austin, August 1974.

The basic bridge geometry can be found on the Bridge Layout located in the Appendices.

(BDM-LRFD, Ch. 4, Sect. 5, Materials)

Grade 60 Reinforcing

$F_y = 60$ ksi *(BDM-LRFD, Ch. 4, Sect. 5, Materials)*

<u>Bents</u>

Use 36" Diameter Columns (Typical for Type TX54 Girders)

Define Variables

<u>Back Span</u>	**<u>Forward Span</u>**	
Span1 = 54ft	Span2 = 112ft	*Span Length*
GdrSpa1 = 8ft	GdrSpa2 = 8ft	*Girder Spacing*
GdrNo1 = 6	GdrNo2 = 6	*Number of Girders in Span*
GdrWt1 = 0.851klf	GdrWt2 = 0.851klf	*Weight of Girder*
Haunch1 = 2in	Haunch2 = 3.75in	*Size of Haunch*

<u>Bridge</u>

Skew = 0deg *Skew of Bents*

BridgeW = 46ft *Width of Bridge Deck*

RdwyW = 44ft *Width of Roadway*

GirderD = 54in *Depth of Type TX54 Girder*

BrgSeat = 1.5in *Bearing Seat Buildup*

BrgPad = 2.75in *Bearing Pad Thickness*

SlabThk = 8in *Thickness of Bridge Slab*

OverlayThk = 2in *Thickness of Overlay*

RailWt = 0.372klf *Weight of Rail*

$w_c = 0.150$kcf *Unit Weight of Concrete for Loads*

$w_{Olay} = 0.140$kcf *Unit Weigh of Overlay*

<u>Bents</u>

$f_c = 5$ksi *Concrete Strength*

$w_{cE} = 0.145$kcf *Unit Weight of Concrete for E_c*

$E_c = 33000 \cdot w_{cE}^{1.5} \cdot \sqrt{f_c}$ $E_c = 4074$ ksi *Modulus of Elasticity of Concrete (AASHTO LRFD Eq. C5.4.2.4-2)*

$f_y = 60$ksi *Yield Strength of Reinforcement*

$E_s = 29000$ksi *Modulus of Elasticity of Steel*

$D_{column} = 36$in *Diameter of Columns*

Other Variables

$\text{IM} = 33\%$ *Dynamic Load Allowance (AASHTO LRFD Table 3.6.2.1-1)*

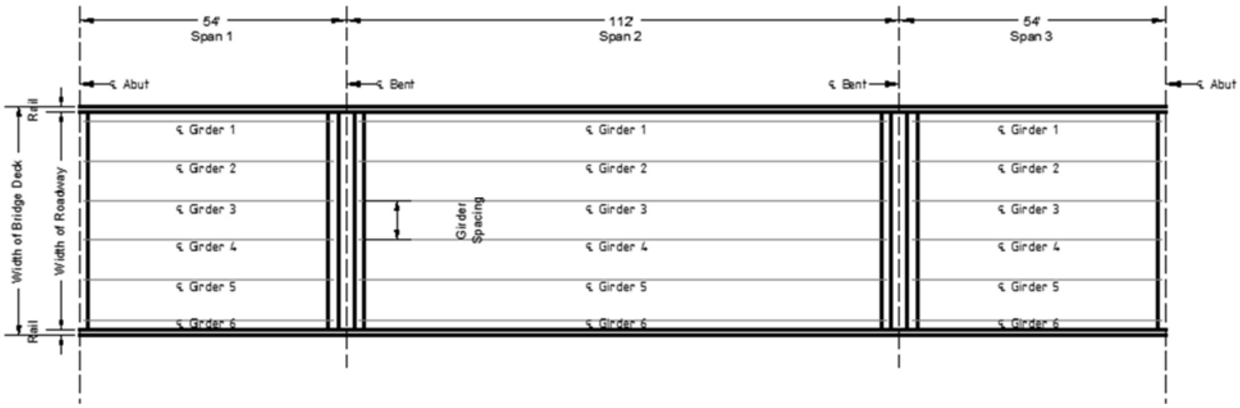


Figure 4.4 Top View of the 0 Degree Skewed ITBC with Spans and Girders

4.2.2 Determine Cap Dimensions

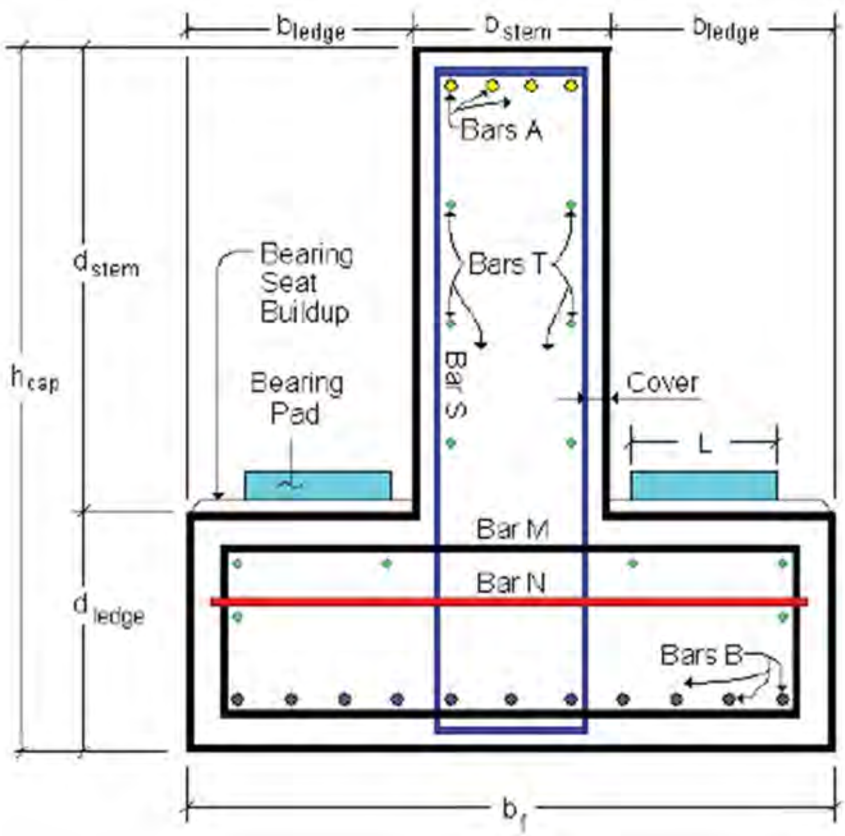


Figure 4.5 Section View of 0 Degree Skewed ITBC

4.2.2.1 Stem Width

$b_{stem} = D_{column} + 3in$ $\qquad$ $b_{stem} = 39 \text{ in}$

The stem is typically at least 3" wider than the Diameter of the Column (36") to allow for the extension of the column reinforcement into the Cap. (TxSP)

4.2.2.2 Stem Height

Haunch2 is the larger of the two haunches.

Distance from Top of Slab to Top of Ledge:

$$D_{Slab\_to\_Ledge} = SlabThk + Haunch2 + GirderD + BrgPad + BrgSeat$$

$$D_{Slab\_to\_Ledge} = 70.00 \text{ in}$$

$StemHaunch = 3.75 \text{ in}$

The top of the stem must be 2.5" below the bottom of the slab. (BDM-LRFD, Ch. 4, Sect. 5, Geometric Constraints)

Accounting for the 1/2" of bituminous fiber, the top of the stem must have at least 2" of haunch on it, but the haunch should not be less than either of the haunches of the adjacent spans.

$d_{stem} = D_{Slab\_to\_Ledge} - SlabThk - StemHaunch - 0.5in$

The stem must accommodate ½" of bituminous fiber.

$d_{stem} = 57.75\ in$

Use: $d_{stem} = 57\ in$

Round the Stem Height down to the nearest 1". (TxSP)

4.2.2.3 Ledge Width

The Ledge Width must be adequate for Bar M to develop fully.

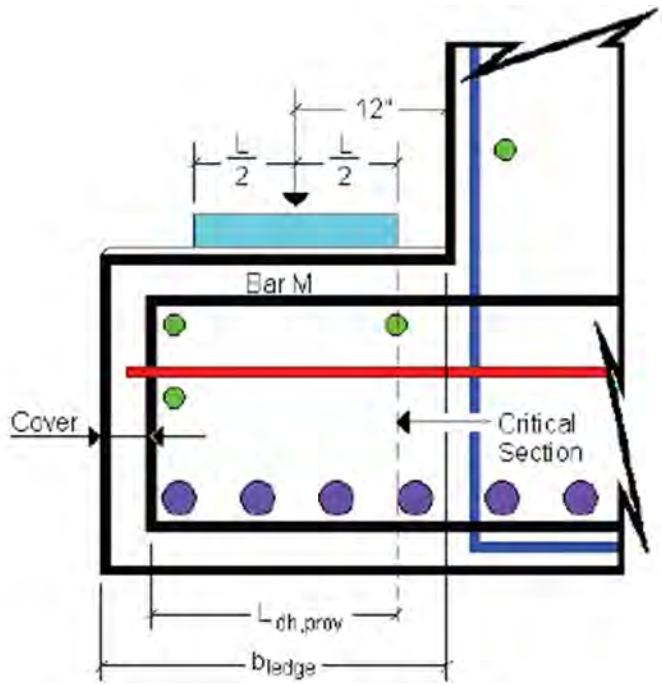


Figure 4.6 Ledge Section of 0 Degree ITBC

"$L_{dh,prov}$" must be greater than or equal to "$L_{dh,req}$" for Bar M.

"cover" is measured from the center of the transverse bars.

"L" is the length of the Bearing Pad along the girder. A typical type TX54 bearing pad is 8" × 21" as shown in the IGEB standard.

$cover = 2.5\ in$

$L = 8\ in$

Determine the Required Development Length of Bar M:

Try # 6 Bar for Bar M.

$d_{bar\_M} = 0.750\ in$

$A_{bar\_M} = 0.44\ in^2$

Basic Development Length

$L_{dh} = \frac{38.0 \cdot d_{bar\_M}}{60} \cdot \left(\frac{f_y}{\sqrt{f_c}}\right)$ $\qquad L_{dh} = 12.75\ in$ *(AASHTO LRFD Eq. 5.10.8.2.4a-2)*

Modification Factors for L_{dh}: *(AASHTO LRFD 5.10.8.2.4b)*

Is Top Cover greater than or equal to 2.5", and Side Cover greater than or equal to 2"?

$$\text{SideCover} = \text{cover} - \frac{d_{bar\_M}}{2} = 2.13 \text{ in}$$

$$\text{TopCover} = \text{cover} - \frac{d_{bar_M}}{2} = 2.13 \text{ in}$$

"Side Cover" and "Top Cover" are the clear cover on the side and top of the hook respectively. The dimension "cover" is measured from the center of Bar M.

No. Reinforcement Confinement Factor, $\lambda_{rc} = 1.0$

Coating Factor, $\lambda_{cw} = 1.0$

Excess Reinforcement Factor, $\lambda_{er} = 1.0$

Concrete Density Modification Factor, $\lambda = 1.0$

(AASHTO LRFD 5.4.2.8)

The Required Development Length:

(AASHTO LRFD 5.10.8.2.4a)

$$L_{dh\_req} = \max(L_{dh} \cdot \left(\frac{\lambda_{rc} \cdot \lambda_{cw} \cdot \lambda_{er}}{\lambda}\right), 8 \cdot d_{bar\_M}, 6\text{in.})$$

Therefore,

$$L_{dh\_req} = 12.75 \text{ in}$$

$b_{ledge\_min} = L_{dh\_req} + \text{cover} + 12\text{in} - \frac{L}{2}$ $\qquad$ $b_{ledge\_min} = 23.25 \text{ in}$

The distance from the face of the stem to the center of bearing is 12" for TxGirders (IGEB).

Use:

$$b_{ledge} = 24 \text{ in}$$

Width of Bottom Flange:

$b_f = 2 \cdot b_{ledge} + b_{stem}$ $\qquad$ $b_f = 87 \text{ in}$

4.2.2.4 Ledge Depth

As a general rule of thumb, Ledge Depth is greater than or equal to 2'-3". This is the depth at which a bent from a typical bridge will pass the punching shear check.

Use a Ledge Depth of 28".

$$d_{ledge} = 28 \text{ in}$$

Total Depth of Cap:

$h_{cap} = d_{stem} + d_{ledge}$ $\qquad$ $h_{cap} = 85 \text{ in}$

4.2.2.5 Summary of Cross-Sectional Dimensions

$b_{stem} = 39 \text{ in}$

$d_{stem} = 57 \text{ in}$

$b_{ledge} = 24 \text{ in}$

$d_{ledge} = 28 \text{ in}$

$h_{cap} = 85 \text{ in}$

4.2.2.6 Length of Cap

First define Girder Spacing and End Distance:

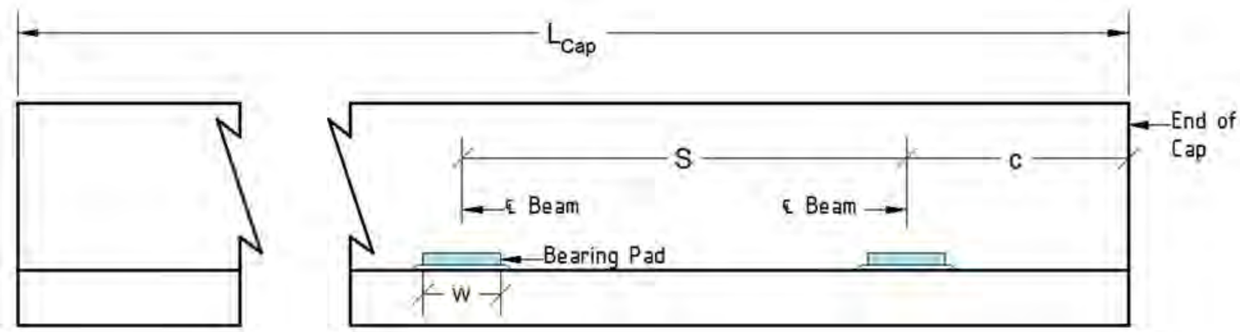


Figure 4.7 Elevation View of 0 Degree ITBC

$S = 8\ \text{ft}$ — *Girder Spacing*

$c = 2\ \text{ft}$ — *"c" is the distance from the Center Line of the Exterior Girder to the Edge of the Cap measured along the Cap.*

$L_{Cap} = S \cdot (GdrNo1 - 1) + 2c$ $\quad L_{Cap} = 44\ \text{ft}$ — *Length of Cap*

TxDOT policy is as follows, "The edge distance between the exterior bearing pad and the end of the inverted T-beam shall not be less than 12in." (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria) replacing the statement in AASHTO LRFD 5.13.2.5.5 stating it shall not be less than d_f. Preferably, the stem should extend at least 3" beyond the edge of the bearing seat.

Bearing Pad Dimensions: — *(IGEB standard)*

$L = 8\ \text{in}$ — *Length of Bearing Pad*

$W = 21\ \text{in}$ — *Width of Bearing Pad*

4.2.3 Cross Sectional Properties of Cap

$A_g = d_{ledge} \cdot b_f + d_{stem} \cdot b_{stem}$ $\quad A_g = 4659\text{in}^2$

$ybar = \frac{d_{ledge} \cdot b_f \cdot \left(\frac{1}{2} d_{ledge}\right) + d_{stem} \cdot b_{stem} \cdot \left(d_{ledge} + \frac{1}{2} d_{stem}\right)}{A_g}$ $\quad ybar = 34.3\ \text{in}$ — *Distance from bottom of the cap to the center of gravity of the cap*

$$I_g = \frac{b_f \cdot d_{ledge}^3}{12} + b_f \cdot d_{ledge} \cdot \left(ybar - \frac{1}{2} d_{ledge}\right)^2 + \frac{b_{stem} \cdot d_{stem}^3}{12} + \cdots$$

$$b_{stem} \cdot d_{stem} \cdot \left[ybar - \left(d_{ledge} + \frac{1}{2} d_{stem}\right)\right]^2 \qquad I_g = 2.86 \times 10^6\ \text{in}^4$$

4.2.4 Cap Analysis

4.2.4.1 Cap Model

Assume:

4 Columns Spaced @ 12'-0"

The cap will be modeled as a continuous beam with simple supports using TxDOT's CAP18 program.

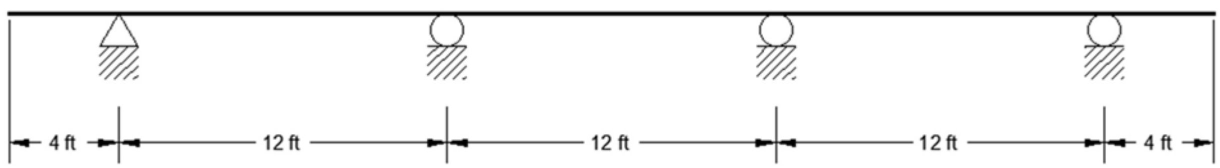


Figure 4.8 Continuous Beam Model for 0 Degree ITBC

TxDOT does not consider frame action for typical multi-column bents.

(BDM-LRFD, Ch. 4, Sect. 5, Structural Analysis).

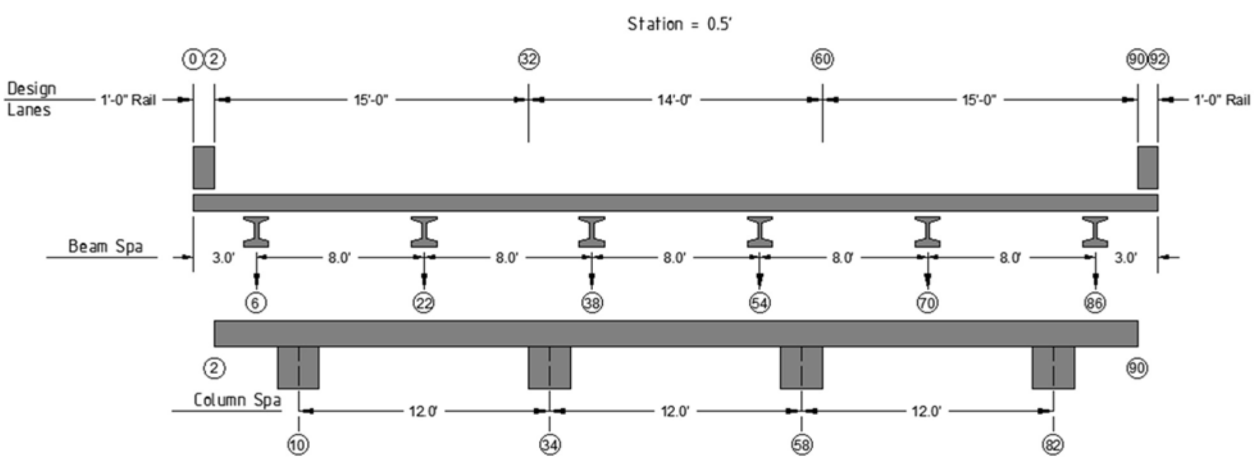


Figure 4.9 Cap 18 Model of 0 Degree ITBC

The circled numbers in Figure 4.9 are the stations that will be used in the CAP 18 input file. One station is 0.5 ft in the direction perpendicular to the pgl, not parallel to the bent.

$$\text{station} = 0.5 \text{ ft}$$ *Station increment for CAP 18*

Recall:

$$E_c = 4074 \text{ ksi} \qquad I_g = 2.86 \times 10^6 \text{ in}^4$$

$$E_c I_g = 1.165 \times 10^{10} \text{ kip} \cdot \text{in}^2 / \left(12 \frac{\text{in}}{\text{ft}}\right)^2 \qquad E_c I_g = 8.09 \times 10^7 \text{kip} \cdot \text{ft}^2$$

4.2.4.1.1 *Dead Load*

Values used in the following equations can be found on "4.2.1 Design Parameters"

SPAN 1

$$\mathrm{Rail1} = \frac{2 \cdot \mathrm{RailWt} \cdot \frac{\mathrm{Span1}}{2}}{\min(\mathrm{GdrNo1},6)} \qquad \mathrm{Rail1} = 3.44\,\frac{\mathrm{kip}}{\mathrm{girder}}$$

Rail Weight is distributed evenly among stringers, up to 3 stringers per rail (TxSP).

$$\mathrm{Slab1} = w_c \cdot \mathrm{GdrSpa1} \cdot \mathrm{SlabThk} \cdot \frac{\mathrm{Span1}}{2} \cdot 1.10 \qquad \mathrm{Slab1} = 23.76\,\frac{\mathrm{kip}}{\mathrm{girder}}$$

Increase slab DL by 10% to account for haunch and thickened slab ends.

$$\mathrm{Girder1} = \mathrm{GdrWt1} \cdot \frac{\mathrm{Span1}}{2} \qquad \mathrm{Girder1} = 22.98\,\frac{\mathrm{kip}}{\mathrm{girder}}$$

$$\mathrm{DLRxn1} = (\mathrm{Rail1} + \mathrm{Slab1} + \mathrm{Girder1}) \qquad \mathrm{DLRxn1} = 50.17\,\frac{\mathrm{kip}}{\mathrm{girder}}$$

Overlay is calculated separetely, because it has different load factor than the rest of the dead loads.

$$\mathrm{Overlay1} = w_{\mathrm{Olay}} \cdot \mathrm{GdrSpa1} \cdot \mathrm{OverlayThk} \cdot \frac{\mathrm{Span1}}{2} \qquad \mathrm{Overlay1} = 5.04\,\frac{\mathrm{kip}}{\mathrm{girder}}$$

Design for future overlay.

SPAN 2

$$\mathrm{Rail2} = \frac{2 \cdot \mathrm{RailWt} \cdot \frac{\mathrm{Span2}}{2}}{\min(\mathrm{GdrNo2},6)} \qquad \mathrm{Rail2} = 7.13\,\frac{\mathrm{kip}}{\mathrm{girder}}$$

$$\mathrm{Slab2} = w_c \cdot \mathrm{GdrSpa2} \cdot \mathrm{SlabThk} \cdot \frac{\mathrm{Span2}}{2} \cdot 1.10 \qquad \mathrm{Slab2} = 49.28\,\frac{\mathrm{kip}}{\mathrm{girder}}$$

$$\mathrm{Girder2} = \mathrm{GdrWt1} \cdot \frac{\mathrm{Span2}}{2} \qquad \mathrm{Girder2} = 47.66\,\frac{\mathrm{kip}}{\mathrm{girder}}$$

$$\mathrm{DLRxn2} = (\mathrm{Rail2} + \mathrm{Slab2} + \mathrm{Girder2}) \qquad \mathrm{DLRxn2} = 104.07\,\frac{\mathrm{kip}}{\mathrm{girder}}$$

$$\mathrm{Overlay2} = w_{\mathrm{Olay}} \cdot \mathrm{GdrSpa2} \cdot \mathrm{OverlayThk} \cdot \frac{\mathrm{Span2}}{2} \qquad \mathrm{Overlay2} = 10.45\,\frac{\mathrm{kip}}{\mathrm{girder}}$$

CAP

$$\mathrm{Cap} = w_c \cdot A_g = 4.853\,\frac{\mathrm{kip}}{\mathrm{ft}} \cdot \frac{0.5\mathrm{ft}}{\mathrm{station}} \qquad \mathrm{Cap} = 2.427\,\frac{\mathrm{kip}}{\mathrm{station}}$$

4.2.4.1.2 *Live Load* *(AASHTO LRFD 3.6.1.2.2 and 3.6.1.2.4)*

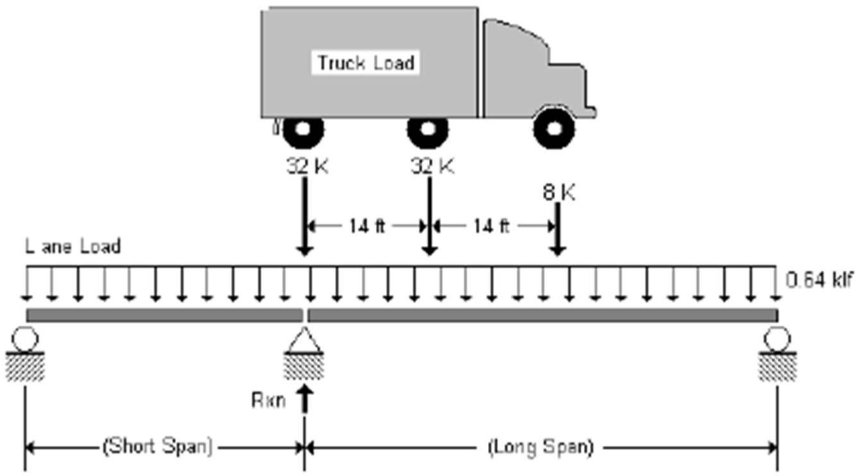


Figure 4.10 Live Load Model of 0 Degree ITBC

$\text{LongSpan} = \max(\text{Span1}, \text{Span2})$ $\text{LongSpan} = 112\ \text{ft}$

$\text{ShortSpan} = \min(\text{Span1}, \text{Span2})$ $\text{ShortSpan} = 54\ \text{ft}$

$\text{IM} = 0.33$

$$\text{Lane} = 0.64\text{klf} \cdot \left(\frac{\text{LongSpan}+\text{ShortSpan}}{2}\right)$$

$$\text{Lane} = 53.12\,\frac{\text{kip}}{\text{lane}}$$

$$\text{Truck} = 32\text{kip} + 32\text{kip} \cdot \left(\frac{\text{LongSpan}-14}{\text{LongSpan}}\right) + \cdots$$
$$8\text{kip} \cdot \left(\frac{\text{LongSpan}-2}{\text{LongSpan}}\right)$$

$$\text{Truck} = 66.00\,\frac{\text{ki}}{\text{lane}}$$

Use HL-93 Live Load. For maximum reaction at interior bents, "Design Truck" will always govern over "Design Tandem". For the maximum reaction when the long span is more than twice as long as the short span, place the rear (32 kip) axle over the support and the middle (32 kip) and front (8 kip) axles on the long span. For the maximum reaction when the long span is less than twice as long as the short span, place the middle (32 kip) axle over the support, the front (8 kip) axle on the short span and the rear (32 kip) axle on the long span.

$$\text{LLRxn} = \text{Lane} + \text{Truck} \cdot (1 + \text{IM})$$

$$\text{LLRxn} = 140.90\,\frac{\text{kip}}{\text{lane}}$$

Combine "Design Truck" and "Design Lane" loadings (AASHTO LRFD 3.6.1.3). Dynamic load allowance, IM, does not apply to "Design Lane." (AASHTO LRFD 3.6.1.2.4)

$P = 16.0kip \cdot (1 + IM)$

$P = 21.28\,kip$

$w = \frac{LLRxn \quad (2 \cdot P)}{10ft}$

$w = 9.83 \frac{kip}{ft} \cdot \frac{0.5ft}{station}$

$w = 4.92 \frac{kip}{station}$

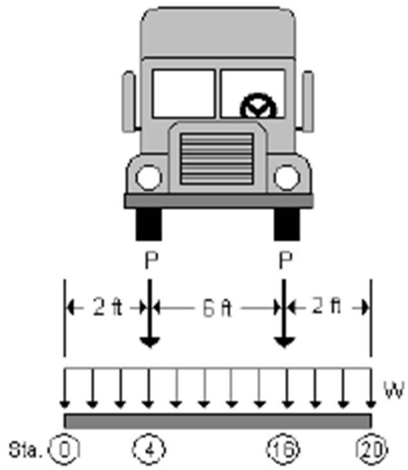


Figure 4.11 Live Load Model of 0 Degree Skewed ITBC for CAP18

The Live Load is applied to the slab by two 16 kip wheel loads increased by the dynamic load allowance with the reminder of the live load distributed over a 10 ft (AASHTO LRFD 3.6.1.2.1) design lane width. (TxSP)

The Live Load applied to the slab is distributed to the beams assuming the slab is hinged at each beam except the outside beam. (BDM-LRFD, Ch. 4, Sect. 5, Structural Analysis)

4.2.4.1.3 *Cap 18 Data Input*

Input "Multiple Presence Factors" into CAP18 as "Load Reduction Factors".

Multiple Presence Factors, m (AASHTO LRFD Table 3.6.1.1.2-1)

No. of Lanes	Factor "m"
1	1.20
2	1.00
3	0.85
>3	0.65

Limit States (AASHTO LRFD 3.4.1)

The cap design need only consider Strength I, Service I, and Service I with DL (TxSP).

Strength I

Live Load and Dynamic Load Allowance	LL+IM = 1.75
Dead Load Components	DC = 1.25
Dead Load Wearing Surface (Overlay)	DW = 1.50

TxDOT allows the Overlay Factor to be reduced to 1.25 (TxSP), since overlay is typically used in design only to increase the safety factor, but in this example we will use DW=1.50.

Service I

Live Load and Dynamic Load Allowance	LL+IM = 1.00
Dead Load and Wearing Surface	DC & DW = 1.00

Dead Load

TxDOT considers Service level Dead Load only with a limit reinforcement stress of 22 ksi to minimize cracking. (BDM-LRFD, Chapter 4, Section 5, Design Criteria)

4.2.4.1.4 *Cap 18 Output*

These loads are the maximum loads from the CAP 18 Output File Located in the Appendices.

	Max +M	**Max -M**
Dead Load:	$M_{posDL} = 249.2\ kip \cdot ft$	$M_{negDL} = -378.5\ kip \cdot ft$
Service Load:	$M_{posServ} = 491.6\ kip \cdot ft$	$M_{negServ} = -590.0\ kip \cdot ft$
Factored Load:	$M_{posUlt} = 740.6\ kip \cdot ft$	$M_{negUlt} = -851.0\ kip \cdot ft$

4.2.4.2 Girder Reactions on Ledge

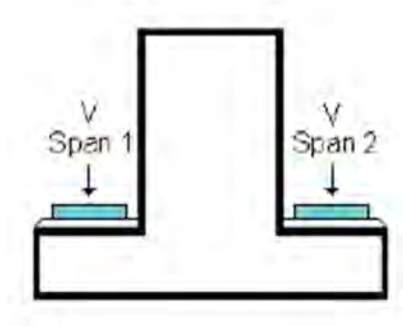


Figure 4.12 Girder Reactions on the Ledge of 0 Degree Skewed ITBC

Dead Load

$\mathrm{DLSpan1} = \mathrm{Rail1} + \mathrm{Slab1} + \mathrm{Girder1}$ $\qquad \mathrm{DLSpan1} = 50.17\,\frac{\mathrm{kip}}{\mathrm{girder}}$

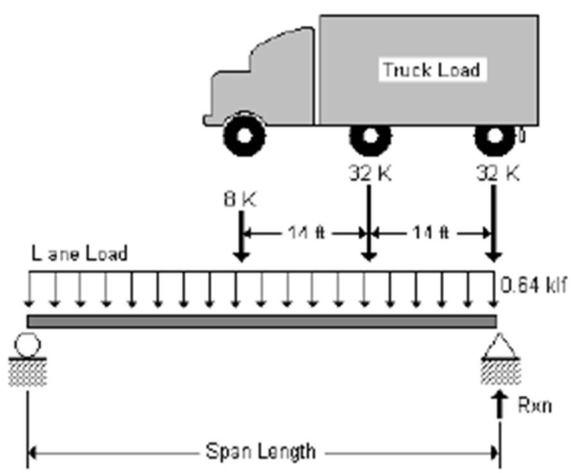
$\mathrm{Overlay1} = 5.04\,\frac{\mathrm{kip}}{\mathrm{girder}}$

$\mathrm{DLSpan2} = \mathrm{Rail2} + \mathrm{Slab2} + \mathrm{Girder2}$ $\qquad \mathrm{DLSpan2} = 104.07\,\frac{\mathrm{kip}}{\mathrm{girder}}$

$\mathrm{Overlay2} = 10.45\,\frac{\mathrm{kip}}{\mathrm{girder}}$

Live Load

Loads per Lane:



Use HL-93 Live Load. For maximum reaction at interior bents, "Design Truck" will always govern over "Design Tandem" for Spans greater than 26ft. For the maximum reaction, place the back (32 kips) axle over the support.

Figure 4.13 Live Load Model of 0 Degree Skewed ITBC for Girder Reactions on Ledge

$$\mathbf{LaneSpan1 = 0.64klf \cdot \left(\frac{Span1}{2}\right)} \qquad \mathbf{LaneSpan1 = 17.28\,\frac{kip}{lane}}$$

$$\mathbf{LaneSpan2 = 0.64klf \cdot \left(\frac{Span2}{2}\right)} \qquad \mathbf{LaneSpan2 = 35.84\,\frac{kip}{lane}}$$

$$\mathbf{TruckSpan1 = 32kip + 32kip \cdot \left(\frac{Span1-14ft}{Span1}\right) + 8kip \cdot \left(\frac{Span1-28ft}{Span1}\right)}$$

$$\mathrm{TruckSpan1} = 59.56\,\frac{\mathrm{kip}}{\mathrm{lane}}$$

$$\mathbf{TruckSpan2 = 32kip + 32kip \cdot \left(\frac{Span2-14ft}{Span2}\right) + 8kip \cdot \left(\frac{Span2-28ft}{Span2}\right)}$$

$$\mathrm{TruckSpan2} = 66.00\,\frac{\mathrm{kip}}{\mathrm{lane}}$$

$\mathrm{IM} = 0.33$

$\mathrm{LLRxnSpan1} = \mathrm{LaneSpan1} + \mathrm{TruckSpan1} \cdot (1 + \mathrm{IM})$

$$\mathrm{LLRxnSpan1} = 96.49\,\frac{\mathrm{kip}}{\mathrm{lane}}$$

$\mathrm{LLRxnSpan2} = \mathrm{LaneSpan2} + \mathrm{TruckSpan2} \cdot (1 + \mathrm{IM})$

$$\mathrm{LLRxnSpan2} = 123.62\,\frac{\mathrm{kip}}{\mathrm{girder}}$$

Combine "Design Truck" and "Design Lane" loadings (AASHTO LRFD 3.6.1.3).

Dynamic load allowance, IM, does not apply to "Design Lane." (AASHTO LRFD 3.6.1.2.4).

The Live Load Reactions are assumed to be the Shear Live Load Distribution Factor multiplied by the Live Load Reaction per Lane. The Shear Live Load Distribution Factor is calculated using the "LRFD Live Load Distribution Factors" Spreadsheet found in the Appendices.

The Exterior Girders must have a Live Load Distribution Factor equal to or greater than the Interior Girders. This is to accommodate a possible future bridge widening. Widening the bridge would cause the exterior girders to become interior girders.

$gV_{Span1\_Int} = 0.814$

$gV_{Span1\_Ext} = 0.814$

$gV_{Span2\_Int} = 0.814$

$gV_{Span2\_Ext} = 0.814$

$\mathrm{LLSpan1Int} = gV_{Span1\_Int} \cdot \mathrm{LLRxnSpan1}$ $\qquad \mathrm{LLSpan1Int} = 78.54\,\frac{\mathrm{kip}}{\mathrm{girder}}$

$\mathrm{LLSpan1Ext} = gV_{Span1\_Ext} \cdot \mathrm{LLRxnSpan1}$ $\qquad \mathrm{LLSpan1Ext} = 78.54\,\frac{\mathrm{kip}}{\mathrm{girder}}$

$\mathrm{LLSpan2Int} = gV_{Span2\_Int} \cdot \mathrm{LLRxnSpan2}$ $\qquad \mathrm{LLSpan2Int} = 100.63\,\frac{\mathrm{kip}}{\mathrm{girder}}$

$\mathrm{LLSpan2Ext} = gV_{Span2\_Ext} \cdot \mathrm{LLRxnSpan2}$ $\qquad \mathrm{LLSpan2Ext} = 100.63\,\frac{\mathrm{kip}}{\mathrm{girder}}$

Span 1

Interior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{s\_Span1Int} = \mathrm{DLSpan1} + \mathrm{Overlay1} + \mathrm{LLSpan1Int}$$

$$V_{s\_Span1Int} = 134\ \mathrm{kip}$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{u\_Span1Int} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 + 1.75 \cdot LLSpan1Int$$

$$V_{u\_Span1Int} = 208 \text{ kip}$$

Exterior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{s\_Span1Ext} = DLSpan1 + Overlay1 + LLSpan1Ext$$

$$V_{s\_Span1Ext} = 134 \text{ kip}$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{u\_Span1Ext} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 + 1.75 \cdot LLSpan1Ext$$

$$V_{u\_Span1Ext} = 208 \text{ kip}$$

Span 2

Interior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{s\_Span2Int} = DLSpan2 + Overlay2 + LLSpan2Int$$

$$V_{s\_Span2Int} = 215 \text{ kip}$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{u\_Span2Int} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot LLSpan2Int$$

$$V_{u\_Span2Int} = 322 \text{ kip}$$

Exterior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{s\_Span2Ext} = DLSpan2 + Overlay2 + LLSpan2Ext$$

$$V_{s\_Span2Ext} = 215 \text{ kip}$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{u\_Span2Ext} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot LLSpan2Ext$$

$$V_{u\_Span2Ext} = 322 \text{ kip}$$

4.2.4.3 Torsional Loads

To maximize the torsion, the live load only acts on the longer span.

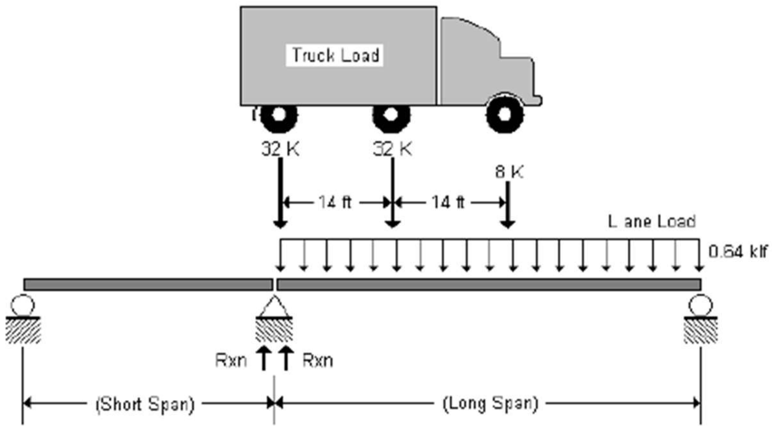


Figure 4.14 Live Load Model of 0 Degree Skewed ITBC for Torsional Loads

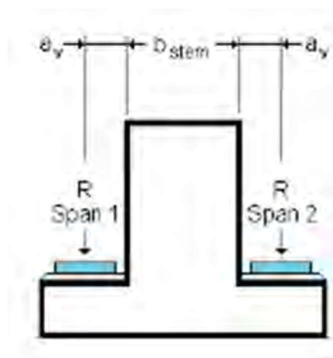


Figure 4.15 Loads on the Ledge of 0 Degree Skewed ITBC for Torsion

$a_v = 12 \text{ in}$

"a_v" is the value for the distance from the face of the stem to the center of bearing for the girders. 12" is the typical values for TxGirders on ITBC (IGEB). The lever arm is the distance from the center line of bearing to the centerline of the cap.

$b_{stem} = 39 \text{ in}$

$LeverArm = a_v + \frac{1}{2} b_{stem}$ $\quad$ $LeverArm = 31.5 \text{ in}$

Interior Girders

Girder Reactions

$$R_{u\_Span1} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1$$

$$R_{u\_Span1} = 70 \text{ kip}$$

$$R_{u\_Span2} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2\_Int} \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)]$$

$$R_{u\_Span2} = 322 \text{ kip}$$

Torsional Load

$$T_{u\_Int} = \left|R_{u\_Span1} - R_{u\_Span2}\right| \cdot LeverArm$$

$$T_{u\_Int} = 660\ kip \cdot ft$$

<u>Exterior Girders</u>

Girder Reactions

$$R_{u\_Span1} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1$$

$$R_{u\_Span1} = 70\ kip$$

$$R_{u\_Span2} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2\_Ext} \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)]$$

$$R_{u\_Span2} = 322\ kip$$

Torsional Load

$$T_{u\_Ext} = \left|R_{u\_Span1} - R_{u\_Span2}\right| \cdot LeverArm$$

$$T_{u\_Ext} = 660\ kip \cdot ft$$

Torsion on Cap

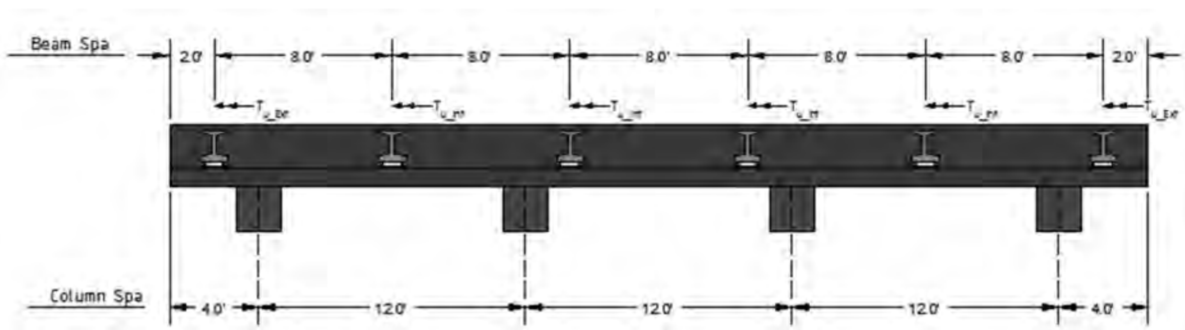


Figure 4.16 Elevation View of 0 Degree ITBC with Torsion Loads

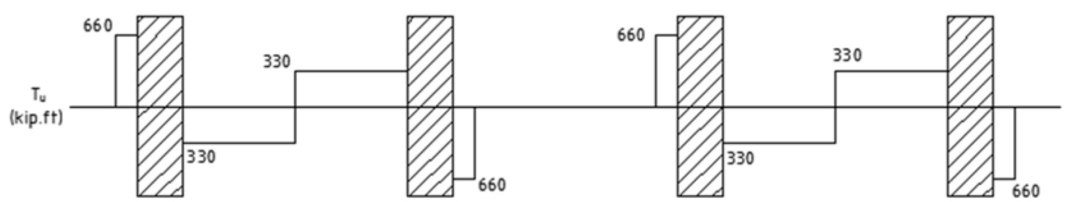


Figure 4.17 Torsion Diagram of 0 Degree ITBC

Analyzed assuming Bents are torsionally rigid at Effective Face of Columns.

$T_u = 660\ kip \cdot ft$ *Maximum Torsion on Cap*

4.2.4.4 Load Summary

Ledge Loads

Interior Girder

Service Load

$V_{s\_Int} = \max(V_{s\_Span1Int}, V_{s\_Span2Int})$ $V_{s\_Int} = 215.15\ \text{kip}$

Factored Load

$V_{u\_Int} = \max(V_{u\_Span1Int}, V_{u\_Span2Int})$ $V_{u\_Int} = 321.86\ \text{kip}$

Exterior Girder

Service Load

$V_{s\_Ext} = \max(V_{s\_Span1Ext}, V_{s\_Span2Ext})$ $V_{s\_Ext} = 215.15\ \text{kip}$

Factored Load

$V_{u\_Ext} = \max(V_{u\_Span1Ext}, V_{u\_Span2Ext})$ $V_{u\_Ext} = 321.86\ \text{kip}$

Cap Loads

Positive Moment (From CAP18)

Dead Load: $M_{posDL} = 249.2\ \text{kip} \cdot \text{ft}$

Service Load: $M_{posServ} = 491.6\ \text{kip} \cdot \text{ft}$

Factored Load: $M_{posUlt} = 740.6\ \text{kip} \cdot \text{ft}$

Negative Moment (From CAP18)

Dead Load: $M_{negDL} = -378.5\ \text{kip} \cdot \text{ft}$

Service Load: $M_{negServ} = -590.0\ \text{kip} \cdot \text{ft}$

Factored Load: $M_{negUlt} = -851.0\ \text{kip} \cdot \text{ft}$

Maximum Torsion and Concurrent Shear and Moment (Strength I)

$T_u = 660\ \text{kip} \cdot \text{ft}$

$V_u = 447.4\ \text{kip}$

$M_u = 334.5\ \text{kip} \cdot \text{ft}$

Located two stations away from centerline of column.

V_u and M_u values are from CAP18

4.2.5 Locate and Describe Reinforcing

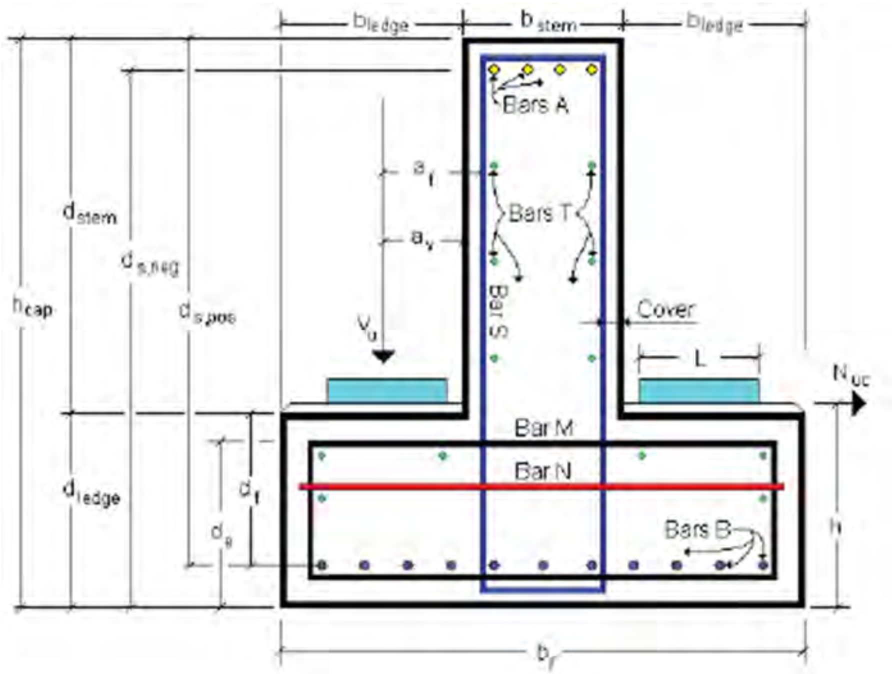


Figure 4.18 Section View of 0 Degree Skewed ITBC

Recall:

$b_{stem} = 39 \text{ in}$

$d_{stem} = 57 \text{ in}$

$b_{ledge} = 24 \text{ in}$

$d_{ledge} = 28 \text{ in}$

$b_f = 87 \text{ in}$

$h_{cap} = 85 \text{ in}$

$\text{cover} = 2.5 \text{ in}$ *Measured from Center of bar*

4.2.5.1 Describe Reinforcing Bars

Use # 11 bars for Bar A

$A_{bar\_A} = 1.56 \text{ in}^2$ $\quad$ $d_{bar\_A} = 1.410 \text{ in}$

Use # 11 bars for Bar B

$A_{bar\_B} = 1.56 \text{ in}^2$ $\quad$ $d_{bar\_B} = 1.410 \text{ in}$

Use # 6 bars for Bar M

$A_{bar\_M} = 0.44 \text{ in}^2$ $\quad$ $d_{bar\_M} = 0.75 \text{ in}$

Use # 6 bars for Bar N

$A_{bar\_N} = 0.44 \text{ in}^2$ $\quad$ $d_{bar\_N} = 0.75 \text{ in}$

Use # 6 bars for Bar S

$A_{bar\_S} = 0.44 \text{ in}^2$ $\quad$ $d_{bar\_S} = 0.75 \text{ in}$

Use # 6 bars for Bar T

$A_{bar\_T} = 0.44 \text{ in}^2$ $\quad$ $d_{bar\_T} = 0.75 \text{ in}$

In the calculation of b_{ledge}, # 6 Bar M was considered. Bar M must be # 6 or smaller to allow it fully develop.

To prevent confusion, use the same bar size for Bar N as Bar M.

4.2.5.2 Calculate Dimensions

$d_{s\_neg} = h_{cap} - cover - \frac{1}{2}d_{bar\_S} - \frac{1}{2}d_{bar\_A}$ $\quad$ $d_{s\_neg} = 81.42 \text{ in}$

$d_{s\_pos} = h_{cap} - cover - \frac{1}{2}\max(d_{bar\_S}, d_{bar\_M}) - \frac{1}{2}d_{bar\_B}$ $\quad$ $d_{s\_pos} = 81.42 \text{ in}$

$a_v = 12 \text{ in}$

$a_f = a_v + cover$ $\quad$ $a_f = 14.50 \text{ in}$

$d_e = d_{ledge} - cover$ $\quad$ $d_e = 25.50 \text{ in}$

$d_f = d_{ledge} - cover - \frac{1}{2}d_{bar\_M} - \frac{1}{2}d_{bar\_B}$ $\quad$ $d_f = 24.42 \text{ in}$

$h = d_{ledge} + BrgSeat$ $\quad$ $h = 29.50 \text{ in}$

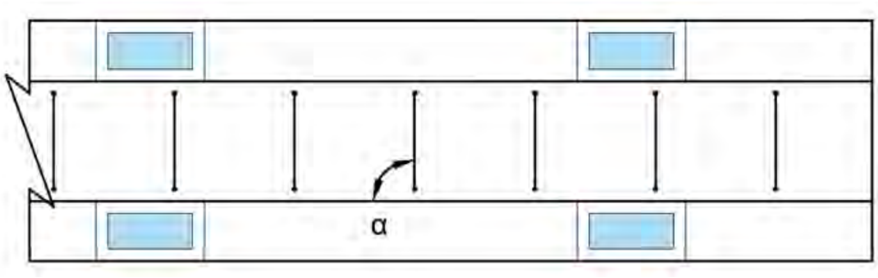


Figure 4.19 Plan View of 0 Degree Skewed ITBC

$\alpha = 90\ \text{deg}$ *Angle of Bars S*

Recall:

$L = 8\ \text{in}$

$W = 21\ \text{in}$ *Dimension of Bearing Pad*

4.2.6 Check Bearing

The load on the bearing pad propagates along a truncated pyramid whose top has the area A_1 and whose base has the area A_2. A_1 is the loaded area (the bearing pad area: L×W). A_2 is the area of the lowest rectangle contained wholly within the support (the Inverted Tee Cap). A_2 must not overlap the truncated pyramid of another load in either direction, nor can it extend beyond the edges of the cap in any direction.

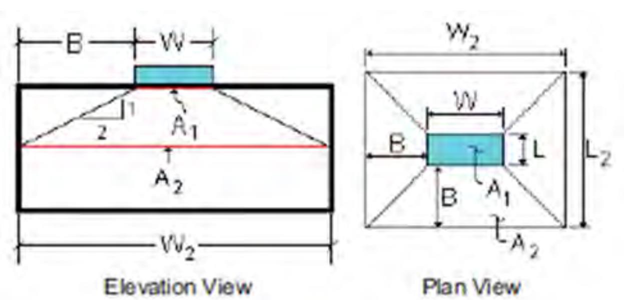


Figure 4.20 Bearing Check for 0-degree Skew Angle

Resistance Factor (ϕ) = 0.7 *(AASHTO LRFD 5.5.4.2)*

$A_1 = L \cdot W$ $\quad A_1 = 168\ \text{in}^2$ *Area under Bearing Pad*

Interior Girders

$$B = \min\left[\left(b_{\text{ledge}} - a_v\right) - \frac{1}{2}L, \left(a_v + \frac{1}{2}b_{\text{stem}}\right) - \frac{1}{2}L, 2d_{\text{ledge}}, \frac{1}{2}S - \frac{1}{2}W\right]$$

"B" is the distance from perimeter of A_1 to the perimeter of A_2 as seen in the above figure

$B = 8\ \text{in.}$

$L_2 = L + 2 \cdot B$ $\quad L_2 = 24.00\ \text{in}$

$W_2 = W + 2 \cdot B$ $\quad W_2 = 37.00\ \text{in}$

$A_2 = L_2 \cdot W_2$ $\quad A_2 = 888\ \text{in}^2$

Modification factor

$m = \min\left(\sqrt{\frac{A_2}{A_1}}, 2\right)$ = 2.29 and 2 $\quad m = 2$ *AASHTO LRFD Eq. 5.6.5-3*

$\phi V_n = \phi \;\; 0.85 \;\; f_c \;\; A_1 \;\; m$ $\quad \phi V_n = 999.6$ kips *AASHTO LRFD Eqs. 5.6.5-1 and 5.6.5-2.*

$V_{u\_Int} = 321.86 < \phi V_n$ BearingChk = "OK!" V_{u_int} *from "4.2.4.4Load Summary".*

Exterior Girders

$$B = \min\left[\left(b_{ledge} - a_v\right) - \frac{1}{2}L, \left(a_v + \frac{1}{2}b_{stem}\right) - \frac{1}{2}L, 2d_{ledge}, \frac{1}{2}S - \frac{1}{2}W, c - \frac{1}{2}W\right]$$

B= 8 in. *"B" is the distance from perimeter of A_1 to the perimeter of A_2 as seen in the above figure*

$L_2 = L + 2 \;\; B$ $\quad L_2 = 24.00$ in

$W_2 = W + 2 \;\; B$ $\quad W_2 = 37.00$ in

$A_2 = L_2 \;\; W_2$ $\quad A_2 = 888\ \text{in}^2$

Modification factor

$m = \min\left(\sqrt{\frac{A_2}{A_1}}, 2\right)$ = 2.29 and 2 $\quad m = 2$ *AASHTO LRFD Eq. 5.6.5-3*

$\phi V_n = \phi \;\; 0.85 \;\; f_c \;\; A_1 \;\; m$ $\quad \phi V_n = 999.6$ kips *AASHTO LRFD Eqs. 5.6.5-1 and 5.6.5-2:*

$V_{u\_ext} = 321.86\ \text{kips} < \Phi V_n$ BearingChk= "OK!" V_{u_ext} *from "4.2.4.4 Load Summary".*

4.2.7 Check Punching Shear

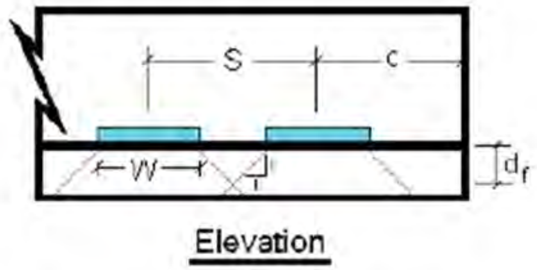


Figure 4.21 Punching Shear Check for 0-degree Skew Angle

*AASHTO LRFD **5.8.4.3.4**, the truncated pyramids assumed as failure surfaces for punching shear shall not overlap.*

Resistance Factor (ϕ) = 0.90 *AASHTO LRFD 5.5.4.2.*

Determine if the Shear Cones Intersect

Is $\frac{1}{2}S - \frac{1}{2}W \geq d_f$?

Yes. Therefore, shear cones do not intersect in the longitudinal direction of the cap.

$\frac{1}{2}S - \frac{1}{2}W = 37.5$ in

$d_f = 24.42$ in

TxDOT uses "d_f" instead of "d_e" for Punching Shear (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria). This is because "d_f" has traditionally been used for inverted tee bents and was sed in the Inverted Tee Research (Furiong % Mirza pg. 58).

Is $\frac{1}{2}b_{stem} + a_v - \frac{1}{2}L \geq d_f$?

Yes. Therefore, shear cones do not intersect in the transverse direction of the cap.

$\frac{1}{2}b_{stem} + a_v - \frac{1}{2}L = 27.5$ in

$d_f = 24.42$ in

Interior Girders

$V_n = 0.125 \cdot \lambda \sqrt{f'_c}\, b_o\, d_f$ $V_n = 585.91$ kips *AASHTO LRFD 5.8.4.3.4-3*

$b_o = W + 2L + 2d_f$ $b_o = 84.84$ in *AASHTO LRFD 5.8.4.3.4-4*

$\phi V_n = 527.32$ kips

$V_{u\_Int} = 321.86 \text{ kips} < \phi V_n$ PunchingShearChk= “OK!” *V_{u_int} from “4.2.4.4 Load Summary”.*

Exterior Girders

$V_n = \min[(0.125 \cdot \sqrt{f_c} \cdot \left(\frac{1}{2}W + L + d_f + c\right) * d_f, 0.125 \cdot \sqrt{f_c} \cdot (W + 2L + 2d_f) * d_f)]$ $V_n = 545.15$ kips *AASHTO LRFD 5.8.4.3.4-3 and 5.8.4.3.4-5*

$\phi V_n = 411.09$ kips

$V_{u\_ext} = 321.86 \text{ kips} < \phi V_n$ PunchingShearChk= "OK!" *V_{u_ext} from "4.2.4.4 Load Summary".*

4.2.8 Check Shear Friction

Resistance Factor (ϕ) =0.90 *AASHTO LRFD 5.5.4.2*

Determine the Distribution Width

<u>Interior Girders</u>

$b_{s\_Int} = \min(W + 4a_v, S)$ *"S" is the girder spacing.*

= min (69 in, 96 in)

$b_{s\_Int} = 69$ in

$A_{cv} = b_{s\_Int} \cdot d_e$ $A_{cv} = 1759.5$ in2

<u>Exterior Girders</u>

$b_{s\_Ext} = \min(W + 4a_v, S, 2c)$ *"S" is the girder spacing.*

= min [69, 96, 48]

= 48 in

$A_{cv} = b_{s\_ext} \cdot d_e$ $A_{cv} = 1224$ in2

Interior Girders

$V_n = \min(0.2 \cdot f_c \cdot A_{cv}, 0.8 \cdot A_{cv})$ $V_n = 1408$ kips *AASHTO LRFD 5.8.4.2.2-1 and 5.8.4.2.2-2*

= min (1759.5, 1408)

$\phi V_n = 1267$ kips

$V_{u\_Int} = 321.86 \text{ kips} < \phi V_n$ ShearFrictionChk= "OK!" *V_{u_int} from "4.2.4.4 Load Summary".*

Exterior Girders

$V_n = \min(0.2 \cdot f_c \cdot A_{cv}, 0.8 \cdot A_{cv})$ $V_n = 979.2$ kips *AASHTO LRFD 5.8.4.2.2-1 and 5.8.4.2.2-2*

= min (1224, 979.2)

$\phi V_n = 881$ kips

$V_{u\_ext} = 321.86 \text{ kips} < \phi V_n$ ShearFrictionChk= "OK!" *V_{u_ext} from "4.2.4.4 Load Summary".*

4.2.9 Flexural Reinforcement for Negative Bending (Bars A)

$M_{dl} = |M_{negDL}|$ — $M_{dl} = 378.5\ \text{kip} \cdot \text{ft}$

From Cap 18 Output.

$M_s = |M_{negServ}|$ — $M_s = 590.0\ \text{kip} \cdot \text{ft}$

$M_u = |M_{negUlt}|$ — $M_u = 851.0\ \text{kip} \cdot \text{ft}$

(AASHTO LRFD 5.6.3.3)

4.2.9.1 <u>Minimum Flexural Reinforcement</u>

Factored Flexural Resistance, M_r, must be greater than or equal to the lesser of $1.2M_{cr}$ (Cracking Moment) or $1.33M_u$ (Ultimate Moment).

$I_g = 2.86 \times 10^6\ \text{in}^4$ — *Gross Moment of Inertia*

$h_{cap} = 85\ \text{in}$ — *Depth of Cap*

$ybar = 34.3\ \text{in}$ — *Distance to the Center of Gravity of the Cap from the bottom of the Cap*

$f_r = 0.24\sqrt{f_c}$ — $f_r = 0.537\ \text{ksi}$ — *Modulus of Rupture (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)*

$y_t = h_{cap} - ybar$ — $y_t = 50.70\ \text{in}$ — *Distance from Center of Gravity to extreme tension fiber*

$S = \frac{I_g}{y_t}$ — $S = 5.64 \times 10^4\ \text{in}^3$ — *Section Modulus for the extreme tension fiber*

$M_{cr} = S \cdot f_r \cdot \frac{1\text{ft}}{12\text{in}}$ — $M_{cr} = 2523.9\ \text{kip} \cdot \text{ft}$ — *Cracking Moment (AASHTO LRFD Eq. 5.6.3.3-1)*

M_f = minimum of:

$1.2M_{cr} = 3028.7\ \text{kip} \cdot \text{ft}$

$1.33M_u = 1131.8\ \text{kip} \cdot \text{ft}$

Design the lesser of $1.2M_{cr}$ *or* $1.33M_u$ *when determining mininum area of steel required.*

Thus, M_r must be greater than $M_f = 1131.8\ \text{kip} \cdot \text{ft}$

4.2.9.2 Moment Capacity Design

Try, 6 ~ #11's Top		*Number of bars in tension*
$BarANo = 6$		*Diameter of main reinforcing bars*
$d_{bar\_A} = 1.410\ in$		
$A_{bar\_A} = 1.56\ in^2$		*Area of main reinforcing bars*
$A_s = BarANo \cdot A_{bar\_A}$	$A_s = 9.36\ in^2$	*Area of steel in tension*
$d_{stirrup} = d_{bar\_S}$	$d_{stirrup} = 0.75\ in$	*Diameter of shear reinforcing bars*
$d = d_{s\_neg}$	$d = 81.42\ in$	
$b = b_f$	$b = 87\ in$	
$f_c = 5.0\ ksi$		*Compressive Strength of Concrete*
$f_y = 60\ ksi$		*Yield Strength of Rebar*
$\beta_1 = 0.85 - 0.05(f_c - 4ksi)$		*(AASHTO LRFD 5.6.2.2)*
Bounded by: $0.65 \le \beta_1 \le 0.85$	$\beta_1 = 0.80$	
$c = \frac{A_s f_y}{0.85 f_c \beta_1 b}$	$c = 1.90\ in$	*Depth of Cross Section under Compression under Ultimate Load (AASHTO LRFD Eq. 5.6.3.1.2-4)*

This "c" is the distance from the extreme compression fiber to the neutral axis, not the distance from the center of bearing of the last girder to the end of the cap.

$a = c \cdot \beta_1$	$a = 1.52\ in$	*Depth of Equivalent Stress Block (AASHTO LRFD 5.6.2.2)*

Note: "a" is less than "d_{ledge}". Therefore the equivalent stress block acts over a rectangular area. If "a" was greater than "d_{ledge}", it would act over a Tee shaped area.

$M_n = A_s f_y \left(d - \frac{a}{2}\right) \cdot \frac{1ft}{12in}$	$M_n = 3774.9\ kip \cdot ft$	*Nominal Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.2-1)*
$\varepsilon_s = 0.003 \cdot \frac{d-c}{c}$	$\varepsilon_s = 0.126$	*Strain in Reinforcing at Ultimate*
$\varepsilon_s > 0.005$		
FlexureBehavior = "Tension Controlled"		*(AASHTO LRFD 5.6.2.1)*
$\Phi_M = 0.90$		*(AASHTO LRFD 5.5.4.2)*
$M_r = \Phi_M M_n$	$M_r = 3397.4\ kip \cdot ft$	*Factored Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.1-1)*
$M_f = 1131.8\ kip \cdot ft < M_r$	MinReinfChk = "OK!"	
$M_u = 851.0\ kip \cdot ft < M_r$	UltimateMom = "OK!"	

4.2.9.3 Check Serviceability

To find s_{max}:

For service loads, the stress on the cross-section is located as shown in Figure 4.22.

Modular Ratio:

$n = \frac{E_s}{E_c}$ $\quad n = 7.12$

Tension Reinforcement Ratio:

$\rho = \frac{A_s}{b \cdot d}$ $\quad \rho = 0.0013$

$k = \sqrt{(2\rho n) + (\rho n)^2} - (\rho n)$ $\quad k = 0.127$

$d \cdot k = 10.34 \text{ in} \; < \; d_{ledge} = 28 \text{ in}$

Therefore, the compression force acts over a rectangular area.

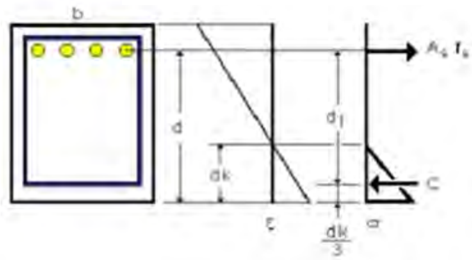


Figure 4.22 Stresses on the Cross Section for Service Loads of 0 Degree Skewed ITBC

$j = 1 - \frac{k}{3}$ $\quad j = 0.958$

If the compression force does not act over rectangular area, j will be different.

$f_{ss} = \frac{M_s}{A_s \cdot j \cdot d} \cdot \frac{12in}{1ft}$ $\quad f_{ss} = 9.70 \text{ ksi}$

Service Load Bending Stress in outer layer of the reinforcing.

$f_a = 0.6 f_y$ $\quad f_a = 36.00 \text{ ksi}$

Allowable Bending Stress (BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

$f_{ss} < f_a$ $\quad$ ServiceStress = "OK!"

$d_c = cover + \frac{1}{2} d_{stirrup} + \frac{1}{2} d_{bar\_A}$ $\quad d_c = 3.58 \text{ in}$

Exposure Condition Factor:

$\gamma_e = 1.00$

For Class 1 Exposure Conditions. For areas where deicing chenicals are frequently used, design for Class 2 Exposure ($\gamma_e = 0.75$). (BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

$\beta_s = 1 + \frac{d_c}{0.7(h_{cap} - d_c)}$ $\quad \beta_s = 1.06$

$s_{max} = \min\left(\frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c, 12in.\right)$ $\quad s_{max} = 12 \text{ in}$

(AASHTO LRFD Eq. 5.6.7-1)

$s_{Actual} = \frac{b_{stem} - 2d_c}{BarANo - 1}$ $\quad s_{Actual} = 6.37 \text{ in}$

$s_{Actual} < s_{max}$ $\quad$ ServiceabilityCheck = "OK!"

A good practice is to place a bar every 12 in along each surface of the bent. (TxSP)

4.2.9.4 Check Dead Load

Check allowable M_{dl}: $\quad f_{dl} = 22 \text{ ksi}$

TxDOT limits dead load stress to 22 ksi, which is set to limit observed cracking under dead load.

Allowable Dead Load Moment

$M_a = A_s \cdot d \cdot j \cdot f_{dl} \cdot \frac{1ft}{12in}$ $\quad M_a = 1338.5 \text{ kip} \cdot \text{ft}$

$M_{dl} = 378.5\ kip \cdot ft\ <\ M_a$ DeadLoadMom = "OK!"

4.2.10 Flexural Reinforcement for Positive Bending (Bars B)

$M_{dl} = M_{posDL}$ $M_{dl} = 249.2\ kip \cdot ft$

$M_s = M_{posServ}$ $M_s = 491.6\ kip \cdot ft$

$M_u = M_{posUlt}$ $M_u = 740.6\ kip \cdot ft$

4.2.10.1 Minimum Flexural Reinforcement

Factored Flexural Resistance, M_r, must be greater than or equal to the lesser of $1.2M_{cr}$ (Cracking Moment) or $1.33M_u$ (Ultimate Moment).

$I_g = 2.86 \times 10^6\ in^4$ *Gross Moment of Inertia*

$y_t = ybar$ $y_t = 34.3\ in$ *Distance to the Center of Gravity of the Cap from the top of the Cap*

$f_r = 0.24\sqrt{f_c}$ $f_r = 0.537\ ksi$ *Modulus of Rupture (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)*

$S = \frac{I_g}{y_t}$ $S = 8.34 \times 10^4\ in^3$ *Section Modulus for the extreme tension fiber*

$M_{cr} = S \cdot f_r \cdot \frac{1ft}{12in}$ $M_{cr} = 3732.2\ kip \cdot ft$ *Cracking Moment (AASHTO LRFD Eq. 5.6.3.3-1)*

M_f = minimum of: *Design the lesser of $1.2M_{cr}$ or $1.33M_u$ when determining mininum area of steel required.*

$1.2M_{cr} = 4478.6\ kip \cdot ft$

$1.33M_u = 985.0\ kip \cdot ft$

Thus, M_r must be greater than $M_f = 985.0\ kip \cdot ft$

4.2.10.2 Moment Capacity Design

Try, 11 ~ #11's Bottom		*Number of bars in tension*
$BarBNo = 11$		*Diameter of main reinforcing bars*
$d_{bar\_B} = 1.41\ in$		
$A_{bar\_B} = 1.56\ in^2$		*Area of main reinforcing bars*
$A_s = BarBNo \cdot A_{bar\_B}$	$A_s = 17.16\ in^2$	*Area of steel in tension*
$d = d_{s\_pos}$	$d = 81.42\ in$	
$b = b_{stem}$	$b = 39\ in$	
$f_c = 5.0\ ksi$		*Compressive Strength of Concrete*
$f_y = 60\ ksi$		*Yield Strength of Rebar*
$\beta_1 = 0.85 - 0.05(f_c - 4ksi)$		*(AASHTO LRFD 5.6.2.2)*
Bounded by: $0.65 \le \beta_1 \le 0.85$	$\beta_1 = 0.80$	
$c = \frac{A_s f_y}{0.85\ _c \beta_1 b}$	$c = 7.76\ in$	*Depth of Cross Section under Compression under Ultimate Load (AASHTO LRFD Eq. 5.6.3.1.2-4)*

This "c" is the distance from the extreme compression fiber to the neutral axis, not the distance from the center of bearing of the last girder to the end of the cap.

$a = c \cdot \beta_1$	$a = 6.21\ in$	*Depth of Equivalent Stress Block (AASHTO LRFD 5.6.2.2)*

Note: "a" is less than "d_{stem}". Therefore the equivalent stress block acts over a rectangular area. If "a" was greater than "d_{stem}", it would act over a Tee shaped area.

$M_n = A_s f_y \left(d - \frac{a}{2}\right) \cdot \frac{1ft}{12in}$	$M_n = 6719.4\ kip \cdot ft$	*Nominal Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.2-1)*
$\varepsilon_s = 0.003 \cdot \frac{d-c}{c}$	$\varepsilon_s = 0.028$	*Strain in Reinforcing at Ultimate*
$\varepsilon_s > 0.005$		
FlexureBehavior = "Tension Controlled"		*(AASHTO LRFD 5.6.2.1)*
$\Phi_M = 0.90$		*(AASHTO LRFD 5.5.4.2)*
$M_r = \Phi_M \cdot M_n$	$M_r = 6047.5\ kip \cdot ft$	*Factored Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.1-1)*

$M_f = 985.0\ kip \cdot ft < M_r$ MinReinfChk = "OK!"

$M_u = 740.6\ kip \cdot ft < M_r$ UltimateMom = "OK!"

4.2.10.3 Check Serviceability

To find s_{max}:

For service loads, the stress on the cross-section is located as shown in Figure 4.23.

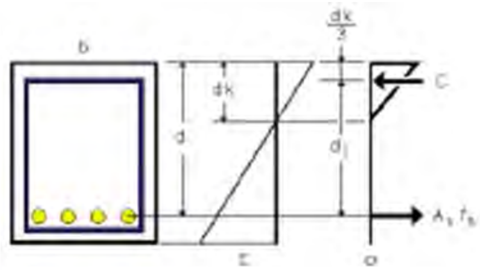


Figure 4.23 Stresses on the Cross Section for Bars B for Service Loads of 0 Degree Skewed ITBC

Modular Ratio:

$$n = \frac{E_s}{E_c} \qquad n = 7.12$$

Tension Reinforcement Ratio:

$$\rho = \frac{A_s}{b \cdot d} \qquad \rho = 0.0054$$

$$k = \sqrt{(2\rho n) + (\rho n)^2} - (\rho n) \qquad k = 0.242$$

$d \cdot k = 19.70\text{ in} < d_{stem} = 57.00\text{ in}$

Therefore, the compression force acts over a rectangular area.

$$j = 1 - \frac{k}{3} \qquad j = 0.919$$

If the compression force does not act over rectangular area, j will be different.

$$f_{ss} = \frac{M_s}{A_s \cdot j \cdot d} \cdot \frac{12\text{in}}{1\text{ft}} \qquad f_{ss} = 4.59\text{ ksi}$$

Service Load Bending Stress in outer layer of the reinforcing.

$$f_a = 0.6 f_y \qquad f_a = 36.00\text{ ksi}$$

Allowable Bending Stress (BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

$f_{ss} < f_a$ ServiceStress = "OK!"

$$d_c = \text{cover} + \frac{1}{2} d_{stirrup} + \frac{1}{2} d_{bar\_B} \qquad d_c = 3.58\text{ in}$$

"cover" is measured to center of shear reinforcement.

Exposure Condition Factor:

$$\gamma_e = 1.00$$

For Class 1 Exposure Conditions. For areas where deicing chenicals are frequently used, design for Class 2 Exposure ($\gamma_e = 0.75$). (BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

$$\beta_s = 1 + \frac{d_c}{0.7(h_{cap} - d_c)} \qquad \beta_s = 1.06$$

$$s_{max} = \min\left(\frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c, 12\text{in.}\right) \qquad s_{max} = 12\text{ in}$$

(AASHTO LRFD Eq. 5.6.7-1)

A good practice is to place a bar every 12 in along each surface of the bent. (TxSP)

Bars Inside Stirrup Bar S

Try: BarBInsideSNo = 5

Number of Bars B that are inside Stirrup Bar S.

$$s_{Actual} = \frac{b_{stem} - 2\left(\text{cover} + \frac{1}{2} d_{bar\_S} + \frac{1}{2} d_{bar\_B}\right)}{\text{BarBInsideSNo} -} \qquad s_{Actual} = 7.96\text{ in}$$

$s_{Actual} < s_{max}$ ServiceabilityCheck = "OK

Bars Outside Stirrup Bar S

$$\mathrm{BarBOutsideSNo} = 11 - \mathrm{BarBInsideSNo}$$

Number of Bars B that are inside Stirrup Bar S.

$$\mathrm{BarBOutsideSNo} = 6$$

$$s_{Actual} = \frac{2b_{ledge} + 2\left(\mathrm{cove}\quad \frac{1}{2}d_{bar\_S} + \frac{1}{2}d_{bar\_B} - \mathrm{cove}\quad \frac{1}{2}d_{bar\_M} - \frac{1}{2}d_{bar\_B}\right)}{\mathrm{BarBOutsideSNo}}$$

$$s_{Actual} = 8.0\ \mathrm{in} < s_{max}$$

ServiceabilityCheck = “OK

4.2.10.4 Check Dead Load

TxDOT limits dead load stress to 22 ksi. This is due to observed cracking under dead load.

Check allowable M_{dl}: $f_{dl} = 22\ \mathrm{ksi}$

Allowable Dead Load Moment

$$M_a = A_s \cdot d \cdot j \cdot f_{dl} \cdot \frac{1\mathrm{ft}}{12\mathrm{in}} \qquad M_a = 2354.00\ \mathrm{kip} \cdot \mathrm{ft}$$

$$M_{dl} = 249.2\ \mathrm{kip} \cdot \mathrm{ft} < M_a$$

DeadLoadMom = “OK!”

Flexural Steel Summary:

Use 6 ~ # 11 Bars on Top

& 11 ~ # 11 Bars on Bottom

4.2.11 Ledge Reinforcement (Bars M & N)

Try Bars M and Bars N at a 4.90" spacing.

$s_{bar\_M} = 4.90 \text{ in}$

$s_{bar\_N} = 4.90 \text{ in}$

Use trial and error to determine the spacing needed for the ledge reinforcing.

It is typical for Bars M & N to be paired together.

4.2.11.1 Determine Distribution Widths

These distribution widths will be used on the following pages to determine the required ledge reinforcement per foot of cap.

Distribution Width for Shear (AASHTO LRFD 5.8.4.3.2)

Interior Girders

$$b_{s\_Int} = \min(W + 4a_v, S)$$

$$b_{s\_Int} = 69.00 \text{ in}$$

Exterior Girders

$$b_{s\_Ext} = \min(W + 4a_v, 2c, S)$$

$$b_{s\_Ext} = 48.00 \text{ in}$$

Note: These are the same distribution widths used for the Shear Friction check.

"S" is the girder spacing.

"c" is the distance from the center of bearing of the outside beam to the end of the ledge.

Distribution Width for Bending and Axial Loads (AASHTO LRFD 5.8.4.3.3)

Interior Girders

$$b_{m\_Int} = \min(W + 5a_f, S)$$

$$b_{m\_Int} = 93.50 \text{ in}$$

Exterior Girders

$$b_{m\_Ext} = \min(W + 5a_f, 2c, S)$$

$$b_{m\_Ext} = 48.00 \text{ in}$$

4.2.11.2 Reinforcing Required for Shear Friction

AASHTO LRFD 5.7.4.1

$\phi = 0.90$ *(AASHTO LRFD 5.5.4)*

$\mu = 1.4$ $c_1 = 0\ \text{ksi}$ $P_c = 0\ \text{kip}$

"μ" is 1.4 for monolithically placed concrete. (AASHTO LRFD 5.7.4.4)

Recall: $d_e = 25.50\ \text{in}$

For clarity, the cohesion factor is labeled "c_1". This is to prevent confusion with "c", the distance from the last girder to the edge of the cap. c_1 is 0ksi for corbels and ledges. (AASHTO LRFD 5.7.4.4)

"P_c" is zero as there is no axial compression.

Minimum Reinforcing (AASHTO LRFD Eq. 5.7.4.2-1)

$$A_{vf\_min} = \frac{0.05\ \text{ksi} \cdot A_{cv}}{f_y}$$

$$A_{cv} = d_e \cdot b_s \quad \text{and} \quad a_{vf} = \frac{A_{vf}}{b_s}$$

$$a_{vf\_min} = \frac{0.05\text{ksi} \cdot d_e}{f_y} \qquad a_{vf\_min} = 0.26\frac{\text{in}^2}{\text{ft}}$$

Minimum Reinforcing required for Shear Friction

Interior Girders

$$A_{cv} = d_e \cdot b_{s\_Int} \qquad A_{cv} = 1759\ \text{in}^2$$

$$V_{u\_Int} = 322\ \text{kip}$$

From "4.2.4.4 Load Summaryry".

$$V_n = c_1 A_{cv} + \mu\left(A_{vf} f_y + P_c\right)$$

(AASHTO LRFD Eq. 5.7.4.3-3)

$$\phi V_n \geq V_u$$

$$\phi \cdot \left[c_1 A_{cv} + \mu\left(A_{vf} f_y + P_c\right)\right] \geq V_u$$

(AASHTO LRFD Eq. 5.7.4.3-1 & AASHTO LRFD Eq. 5.7.4.3-2)

$$A_{vf} = \frac{\frac{\frac{V_{u\_Int}}{\phi} - c_1 A_{cv}}{\mu} - P_c}{f_y} \qquad A_{vf} = 4.26\ \text{in}^2$$

Required Reinforcing for Shear Friction

$$a_{vf\_Int} = \frac{A_{vf}}{b_{s\_Int}} \qquad a_{vf\_Int} = 0.74\frac{\text{in}^2}{\text{ft}}$$

Required Reinforcing for Shear Friction per foot length of cap

Exterior Girders

$A_{cv} = d_e \cdot b_{s\_Ext}$ $A_{cv} = 1224\ in^2$

$V_{u\_Ext} = 322\ kip$ *From "Load Summary".*

$$V_n = c_1 A_{cv} + \mu(A_{vf} f_y + P_c)$$ *(AASHTO LRFD Eq. 5.7.4.3-3)*

$$\Phi V_n \geq V_u$$ *(AASHTO LRFD Eq. 5.7.4.3-1 & AASHTO LRFD Eq. 5.7.4.3-2)*

$$\Phi \cdot [c_1 A_{cv} + \mu(A_{vf} f_y + P_c)] \geq V_u$$

$A_{vf} = \dfrac{\dfrac{\frac{V_{u\_Ext}}{\Phi} - c_1 A_{cv}}{\mu} - P_c}{f_y}$ $A_{vf} = 4.26\ in^2$ *Required Reinforcing for Shear Friction*

$a_{vf\_Ext} = \dfrac{A_{vf}}{b_{s\_Ext}}$ $a_{vf\_Ext} = 1.06\ \frac{in^2}{ft}$ *Required Reinforcing for Shear Friction per foot length of cap*

4.2.11.3 Reinforcing Required for Flexure

AASHTO LRFD 5.8.4.2.1

Recall: $h = 29.50\ in$ $d_e = 25.50\ in$ $a_v = 12\ in$ *From "4.2.5.2 Calculate Dimensions"*

Interior Girders

$V_{u\_Int} = 322\ kip$ *From "4.2.4.4 Load Summary".*

$N_{uc\_Int} = 0.2 \cdot V_{u\_Int}$ $N_{uc\_Int} = 64.4\ kip$ *(AASHTO LRFD 5.8.4.2.1)*

$M_{u\_Int} = V_{u\_Int} \cdot a_v + N_{uc\_Int}(h - d_e)$ $M_{u\_Int} = 343.5\ kip \cdot ft$ *(AASHTO LRFD Eq. 5.8.4.2.1-1)*

Use the following equations to solve for A_f:

$$\Phi M_n \geq M_{u\_Int}$$ *(AASHTO LRFD Eq. 1.3.2.1-1)*

$$M_n = A_f f_y \left(d_e - \frac{a}{2}\right)$$ *(AASHTO LRFD Eq.5.6.3.2.2-1)*

$$c = \frac{A_f f_y}{\alpha_1 f_c \beta_1 b_{m\_Int}}$$ *(AASHTO LRFD Eq. 5.6.3.1.2-4)*

$$\alpha_1 = 0.85$$

$$\beta_1 = 0.80$$ *(AASHTO LRFD 5.6.2.2)*

$$a = c\beta_1$$

$$0.75 \leq \Phi = 0.65 + 0.15\left(\frac{d_e}{c} - 1\right) \leq 0.90$$ *(AASHTO LRFD 5.5.4.2)*

Solve for A_f: $A_f = 3.02\ in^2$ *Required Reinforcing for Flexure*

$a_{f\_Int} = \dfrac{A_f}{b_{m\_Int}}$ $a_{f\_Int} = 0.39\ \frac{in^2}{ft}$ *Required Reinforcing for Flexure per foot length of cap*

Exterior Girders

$V_{u\_Ext} = 322 \text{ kip}$ *From "4.2.4.4 Load Summary".*

$N_{uc\_Ext} = 0.2 \cdot V_{u\_Ext}$ $N_{uc\_Ext} = 64.4 \text{ kip}$ *(AASHTO LRFD 5.8.4.2.1)*

$M_{u\_Ext} = V_{u\_Ext} \cdot a_v + N_{uc\_Ext}(h - d_e)$ $M_{u\_Ext} = 343.5 \text{ kip} \cdot \text{ft}$ *(AASHTO LRFD Eq. 5.8.4.2.1-1)*

Use the following equations to solve for A_f:

$\Phi M_n \geq M_{u\_Ext}$ *(AASHTO LRFD Eq. 1.3.2.1-1)*

$M_n = A_f f_y \left(d_e - \frac{a}{2}\right)$ *(AASHTO LRFD Eq.5.6.3.2.2-1)*

$c = \frac{A_f f_y}{\alpha_1 f_c \beta_1 b_{m\_Ext}}$ *(AASHTO LRFD Eq. 5.6.3.1.2-4)*

$\alpha_1 = 0.85$

$\beta_1 = 0.80$ *(AASHTO LRFD 5.6.2.2)*

$a = c\beta_1$

$0.75 \leq \Phi = 0.65 + 0.15\left(\frac{d_e}{c} - 1\right) \leq 0.90$ *(AASHTO LRFD 5.5.4.2)*

Solve for A_f: $A_f = 3.05 \text{ in}^2$ *Required Reinforcing for Flexure*

$a_{f\_Ext} = \frac{A_f}{b_{m\_Ext}}$ $a_{f\_Ext} = 0.76 \frac{\text{in}^2}{\text{ft}}$ *Required Reinforcing for Flexure per foot length of cap*

4.2.11.4 Reinforcing Required for Axial Tension

(AASHTO LRFD 5.8.4.2.2)
AASHTO LRFD 5.5.4.2

$\Phi = 0.90$

Interior Girders:

$N_{uc\_Int} = 0.2V_{u\_Int}$ $N_{uc\_Int} = 64.4 \text{ kip}$

$A_n = \frac{N_{uc\_Int}}{\Phi f_y}$ $A_n = 1.19 \text{ in}^2$ *Required Reinforcing for Axial Tension*

$a_{n\_Int} = \frac{A_n}{b_{m\_Int}}$ $a_{n\_Int} = 0.15 \frac{\text{in}^2}{\text{ft}}$ *Required Reinforcing for Axial Tension per foot length of cap*

Exterior Girders:

$N_{uc\_Ext} = 0.2V_{u\_Int}$ $N_{uc\_Ext} = 64.4 \text{ kip}$

$A_n = \frac{N_{uc\_Ext}}{\Phi f_y}$ $A_n = 1.19 \text{ in}^2$ *Required Reinforcing for Axial Tension*

$a_{n\_Ext} = \frac{A_n}{b_{m\_Ext}}$ $a_{n\_Ext} = 0.30 \frac{\text{in}^2}{\text{ft}}$ *Required Reinforcing for Axial Tension per foot length of cap*

(AASHTO LRFD 5.8.4.2.1)

4.2.11.5 Minimum Reinforcing

$a_{s\_min} = 0.04 \frac{f_c}{f_y} d_e$ $a_{s\_min} = 1.02 \frac{in^2}{ft}$ *Minimum Required Reinforcing*

4.2.11.6 Check Required Reinforcing

Actual Reinforcing:

$a_s = \frac{A_{bar\_M}}{s_{bar\_M}}$ $a_s = 1.08 \frac{in^2}{ft}$ *Primary Ledge Reinforcing Provided*

$a_h = \frac{A_{bar\_N}}{s_{bar\_N}}$ $a_h = 1.08 \frac{in^2}{ft}$ *Auxiliary Ledge Reinforcing Provided*

Checks: $A_s \geq A_{s\_min}$ *(AASHTO LRFD 5.8.4.2.1)*

$A_s \geq A_f + A_n$ *(AASHTO LRFD 5.8.4.2.2)*

$A_s \geq \frac{2A_{vf}}{3} + A_n$ *(AASHTO LRFD Eq. 5.8.4.2.2-5)*

$A_h \geq 0.5(A_s - A_n)$ *(AASHTO LRFD Eq. 5.8.4.2.2-6)*

Check Interior Girders:

Bar M:

Check if: $a_s \geq a_{s\_min}$ *(AASHTO LRFD 5.8.4.2.1)*

$a_s \geq a_{f\_Int} + a_{n\_Int}$ *(AASHTO LRFD 5.8.4.2.2)*

$a_s \geq \frac{2a_{vf\_Int}}{3} + a_{n\_Int}$ *(AASHTO LRFD Eq. 5.8.4.2.2-5)*

$a_s = 1.26 \frac{in^2}{ft}$

$a_{s\_min} = 1.02 \frac{in^2}{ft} \quad < \quad a_s$

$a_{f\_Int} + a_{n\_Int} = 0.54 \frac{in^2}{ft} \quad < \quad a_s$

$\frac{2a_{vf\_Int}}{3} + a_{n\_Int} = 0.64 \frac{in^2}{ft} \quad < \quad a_s$

BarMCheck = “OK!”

Bar N:

Check if: $a_h \geq 0.5 \cdot (a_s - a_{n\_Int})$ *(AASHTO LRFD Eq. 5.8.4.2.2-6)*

$a_s =$ The maximum of:

$a_{f\_Int} + a_{n\_Int}$

$\frac{2a_{vf\_Int}}{3} + a_{n\_Int}$

$a_s = 0.64 \frac{in^2}{ft}$

"a_s" in this equation is the steel required for Bar M, based on the requirements for Bar M in AASHTO LRFD 5.8.4.2.2. This is derived from the suggestion that Ah should not be less than $A_f/2$ nor less than $A_{vf}/3$ (Furlong & Mirza pg. 73 & 74)

$$0.5 \cdot (a_s - a_{n\_Int}) = 0.25 \frac{in^2}{ft} < a_h$$

BarNCheck = "OK!"

Check Exterior Girders:

Bar M:

Check if: $a_s \geq a_{s\_min}$ *(AASHTO LRFD 5.8.4.2.1)*

$a_s \geq a_{f\_Ext} + a_{n\_Ext}$ *(AASHTO LRFD 5.8.4.2.2)*

$a_s \geq \frac{2a_{vf\_Ext}}{3} + a_{n\_Ext}$ *(AASHTO LRFD Eq. 5.8.4.2.2-5)*

$$a_s = 1.26 \frac{in^2}{ft}$$

$$a_{s\_min} = 1.02 \frac{in^2}{ft} < a_s$$

$$a_{f\_Ext} + a_{n\_Ext} = 1.06 \frac{in^2}{ft} < a_s$$

$$\frac{2a_{vf\_Ext}}{3} + a_{n\_Ext} = 1.01 \frac{in^2}{ft} < a_s$$

BarMCheck = "OK!"

Bar N:

Check if: $a_h \geq 0.5 \cdot (a_s - a_{n\_Ext})$ *(AASHTO LRFD Eq. 5.8.4.2.2-6)*

$a_s =$ The maximum of:

$a_{f\_Ext} + a_{n\_Ext}$

$\frac{2a_{vf\_Ext}}{3} + a_{n\_Ext}$

$$a_s = 1.06 \frac{in^2}{ft}$$

"a_s" in this equation is the steel required for Bar M, based on the requirements for Bar M in AASHTO LRFD 5.8.4.2.2. This is derived from the suggestion that Ah should not be less than $A_f/2$ nor less than $A_{vf}/3$ (Furlong & Mirza pg. 73 & 74)

$$0.5 \cdot (a_s - a_{n\_Ext}) = 0.38 \frac{in^2}{ft} < a_h$$

BarNCheck = "OK!"

Ledge Reinforcement Summary:

Use # 6 primary ledge reinforcing @ 4.90" maximum spacing

& # 6 auxiliary ledge reinforcing @ 4.90" maximum spacing

4.2.12 Hanger Reinforcement (Bars S)

Try Double # 6 Stirrups at a 7.80" spacing.

Use trial and error to determine the spacing needed for the hanger reinforcing.

$s_{bar\_S} = 7.80\ \text{in}$

$A_{hr} = 2\text{stirrups} \cdot A_{bar\_S}$ $\quad A_{hr} = 0.88\ \text{in}^2$

$A_v = 2\text{legs} \cdot A_{hr}$ $\quad A_v = 1.76\ \text{in}^2$

4.2.12.1 <u>Check Minimum Transverse Reinforcement</u>

$b_v = b_{stem}$ $\quad b_v = 39\ \text{in}$

$A_{v\_min} = 0.0316\lambda\sqrt{f_c}\frac{b_v \cdot s_{bar\_S}}{f_y}$ *(AASHTO LRFD Eq. 5.7.2.5-1)*

$\lambda = 1.0$ for normal weight concrete *(AASHTO LRFD 5.4.2.8)*

$A_{v\_min} = 0.36\ \text{in}^2$

$A_v > A_{v\_min}$ MinimumSteelCheck = "OK!"

4.2.12.2 <u>Check Service Limit State</u>

AASHTO LRFD 5.8.4.3.5 with notifications from BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

<u>Interior Girders</u>

TxDOT uses "2/3 f_y" from the original research (Furlong & Mirza Eq. 5.4) instead of "0.5 f_y" from AASHTO LRFD Eq. 5.8.4.3.5-1. (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)

$V_{all} =$ minimum of:

$$\frac{A_{hr} \cdot \left(\frac{2}{3}f_y\right)}{s_{bar\_S}} \cdot (W + 3a_v) = 217\ \text{kip}$$

Bounded by: $(W + 3a_v) \leq \min(S, 2c)$

$$\frac{A_{hr} \cdot \left(\frac{2}{3}f_y\right)}{s_{bar\_S}} \cdot S = 433\ \text{kip}$$

(BDM-LRFD Ch.4, Sect. 5, Design Criteria modified to limit the distribution width to the girder spacing. This will prevent distribution widths from overlapping)

$V_{all} = 217\ \text{kip}$

$V_{s\_Int} = 215\ \text{kip} < V_{all}$ ServiceCheck = "OK!"

Exterior Girders

TxDOT uses "2/3 f_y" from the original research (Furlong & Mirza Eq. 5.4) instead of "0.5 f_y" from AASHTO LRFD Eq. 5.8.4.3.5-1. (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)

V_{all} = minimum of:

V_{all} for the Interior Girder

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{bar\_S}} \cdot \left(\frac{W + 3a_v}{2} + c\right) = 217 \text{ kip}$$

Bounded by: $(W + 3a_v) \leq \min(S, 2c)$

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{bar\_S}} \cdot \left(\frac{S}{2} + c\right) = 325 \text{ kip}$$

(BDM-LRFD Ch.4, Sect. 5, Design Criteria Modified to limit the distribution width to half the girder spacing and the distance to the edge of the cap. This will prevent distribution widths from overlapping or extending over the edge of the cap.)

$V_{all} = 217$ kip

$V_{s\_Ext} = 215 \text{ kip} < V_{all}$ ServiceCheck = "OK!"

4.2.12.3 Check Strength Limit State

(AASHTO LRFD 5.8.4.3.5)

$\Phi = 0.90$ *(AASHTO LRFD Eq. 5.5.4.2)*

Interior Girders:

V_n = minimum of:

$$\frac{A_{hr} \cdot f_y}{s_{bar\_S}} \cdot S = 650 \text{ kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-2)*

$$\left(0.063\sqrt{f_c} \cdot b_f \cdot d_f\right) + \frac{A_{hr} \cdot f_y}{s_{bar\_S}} (W + 2d_f) = 772 \text{kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-3)*

$V_n = 650$ kip

$\Phi V_n = 585$ kip

$V_{u\_Int} = 322 \text{ kip} < \Phi V_n$ UltimateCheck = "OK!"

Exterior Girders:

V_n = minimum of:

V_n for the Interior Girder

$$\frac{A_{hr} \cdot f_y}{s_{bar\_S}} \cdot \left(\frac{S}{2} + c\right) = 487 \text{ kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-2)*

$$\left(0.063\sqrt{f_c} \cdot b_f \cdot d_f\right) + \frac{A_{hr} \cdot f_y}{s_{bar\_S}} \left(\frac{W + 2d_f}{2} + c\right) = 698 \text{ kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-3)*

(These equations are modified to limit the distribution width to the edge of the cap)

$V_n = 487$ kip

$\Phi V_n = 438$ kip

$V_{u\_Ext} = 322 \text{ kip} < \Phi V_n$ UltimateCheck = "OK!"

4.2.12.4 Check Combined Shear and Torsion

The following calculations are for Station 36. All critical locations must be checked. See the Concrete Section Shear Capacity spreadsheet in the appendices for calculations at other locations. Shear and Moment were calculated using the CAP 18 program.

$M_u = 334.5 \text{ kip} \cdot \text{ft}$ $\quad V_u = 447.4 \text{ kip}$ $\quad N_u = 0 \text{ kip}$ $\quad T_u = 660 \text{ kip} \cdot \text{ft}$

Recall:

$\beta_1 = 0.80$ $\quad f_y = 60 \text{ ksi}$

$f_c = 5.0 \text{ ksi}$ $\quad E_s = 29000 \text{ ksi}$

$b_f = 87 \text{ in}$ $\quad h_{cap} = 85 \text{ in}$ $\quad b_{stem} = 39 \text{ in}$ $\quad h = 29.50 \text{ in}$

$b_v = b_{stem}$ $\quad b_v = 39 \text{ in}$

Find d_v:

(AASHTO LRFD 5.7.2.8)

Shears are maximum near the column faces. In these regions the cap is in negative bending with tension in the top of the cap. Therefore, the calculations are based on the steel in the top of the bent cap.

$A_s = A_{bar\_A} \cdot BarANo$ $\quad A_s = 9.36 \text{ in}^2$

$c = \frac{A_s f_y}{0.85 f_c \beta_1 b_f}$ $\quad c = 1.90 \text{ in}$

$a = c \cdot \beta_1$ $\quad a = 1.52 \text{ in}$

$d_s = d_{s\_neg}$ $\quad d_s = 81.42 \text{ in}$

$M_n = A_s f_y \left(d_s - \frac{a}{2}\right)$ $\quad M_n = 3774.9 \text{ kip} \cdot \text{ft}$

$A_{ps} = 0 \text{ in}^2$

$d_e = \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y}$ $\quad d_e = 81.42 \text{ in}$ $\quad$ *(AASHTO LRFD Eq. 5.7.2.8-2)*

$d_v = \text{maximum of:}$

$\frac{M_n}{A_s f_y + A_{ps} f_{ps}} = 80.66 \text{ in}$

$0.9 d_e = 73.28 \text{ in}$

$0.72h = 21.24 \text{ in}$

$d_v = 80.66 \text{ in}$

The method for calculating θ and β used in this design example are from AASHTO LRFD Appendix B5. The method from AASHTO LRFD 5.7.3.4.2 may be used instead. The method from 5.7.3.4.2 is based on the method from Appendix B5; however, it is less accurate and more conservative (often excessively conservative). The method from Appendix B5 is preferred because it is more accurate, but it requires iterating to a solution.

Determine θ and β:

$\Phi_V = 0.90$

(AASHTO LRFD Eq. 5.5.4.2)

$v_u = \frac{|V_u - (\Phi_V \cdot V_p)|}{\Phi_V \cdot b_v \cdot d_v}$ $v_u = 0.16$ ksi

Shear Stress on the Concrete (AASHTO LRFD Eq. 5.7.2.8-1)

$\frac{v_u}{f_c} = 0.03$

Using Table B5.2-1 with $\frac{v_u}{f_c} = 0.03$ and $\varepsilon_x = 0.001$

$\theta = 36.4$ deg and $\beta = 2.23$

Determining θ and β is an iterative process, therefore, assume initial shear strain value ε_x of 0.001 per LRFD B5.2 and then verify that the assumption was valid.

$$\varepsilon_x = \frac{\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})}$$

where $|M_u| = 334.5\text{kip} \cdot \text{ft}$ must be > $|V_u - V_p|d_v = 3012.12\text{kip} \cdot \text{ft}$

$\varepsilon_x = 1.38 \times 10^{-3} > 1.00 \times 10^{-3}$

use $\varepsilon_x = 1.00 \times 10^{-3}$.

Strain halfway between the compressive and tensile resultants (AASHTO LRFD Eq. B5.2-3) If ε_x< 0, then use equation B5.2-5 and re-solve for ε_x.

For values of ε_x greater than 0.001, the tensile strain in the reinforcing, $\varepsilon_t$is greater than 0.002. (ε_t= 2ε_x - ε_c, where ε_c is < 0) Grade 60 steel yields at a strain of 60 ksi / 29,000 ksi = 0.002. By limiting the tensile strain in the steel to the yield strain and using the Modulus of Elasticity of the steel prior to yield, this limits the tensile stress of the steel to the yield stress.

$V_p = 0$ kip

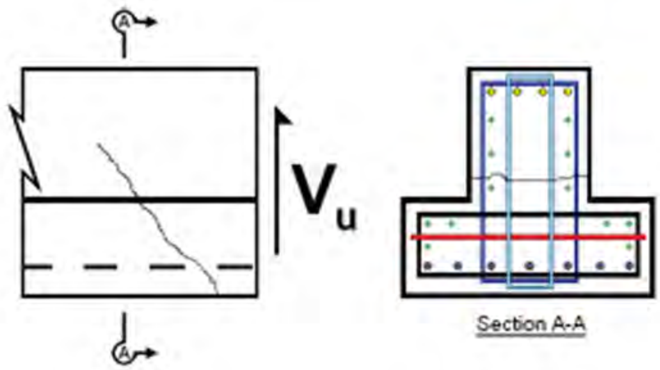
"V_p" is zero as there is no prestressing.

$A_c = b_{stem} \cdot \frac{h_{cap}}{2}$ $A_c = 1657.5\ \text{in}^2$

$s = s_{bar\_S}$ $s = 7.80$ in

(AASHTO LRFD B5.2) "A_c" is the area of concrete on the flexural tension side of the cap, from the extreme tension fiber to one half the cap depth.

"A_c" is needed if AASHTO LRFD Eq. B5.2-3 is negative.



The transverse reinforcement, "A_v", is double closed stirrups. The failure surface intersects four stirrup legs, therefore the area of the shear steel is four times the stirrup bar's area (0.44in2). See the sketch of the failure plane to the left.

Figure 4.24 Failure Surface of 0 Degree Skewed ITBC for Combined Shear and Torsion

$A_v = 2\text{legs} \cdot 2\text{stirrups} \cdot A_{bar\_S}$ $\qquad A_v = 1.76\ \text{in}^2$

$A_t = 1\text{leg} \cdot A_{bar\_S}$ $\qquad A_t = 0.44\ \text{in}^2$

$A_{oh} = (d_{stem}) \cdot (b_{stem} - 2\text{cover}) + (d_{ledge} - 2\text{cover}) \cdot (b_f - 2\text{cover})$

$A_{oh} = 3496\ \text{in}^2$

$A_0 = 0.85A_{oh}$ $\qquad A_0 = 2971.6\text{in}^2$

$p_h = (b_{stem} - 2\text{cover}) + 2(b_{ledge}) + (b_f - 2\text{cover}) + 2(h_{cap} - 2\text{cover})$

$p_h = 324\ \text{in}$

Equivalent Shear Force

$V_{u\_Eq} = \sqrt{V_u^2 + \left(\frac{0.9p_hT_u}{2A_0}\right)^2}$ $\qquad V_{u\_Eq} = 592.6\ \text{kip}$ *(AASHTO LRFD Eq. B.5.2-1)*

Shear Steel Required

$V_n =$ the lesser of:

$V_c + V_s + V_p$ *(AASHTO LRFD Eq. 5.7.3.3-1)*

$0.25 \cdot f_c \cdot b_v \cdot d_v + V_p$ *(AASHTO LRFD Eq. 5.7.3.3-2)*

Check maximum ΦV_n for section:

$\Phi V_{n\_max} = \Phi \cdot (0.25 \cdot f_c \cdot b_v \cdot d_v + V_p)$

$\Phi V_{n\_max} = 3539\ \text{kip}$

$V_u = 447.4\ \text{kip} < \Phi V_{n\_max}$ $\qquad$ MaxShearCheck = “OK!”

Calculate required shear steel:

$V_u < \Phi V_n$ *(AASHTO LRFD Eq. 1.3.2.1-1)*

$V_c = 0.0316 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v$ $V_c = 496\ \text{kip}$ *(AASHTO LRFD Eq. 5.7.3.3-3)*

$V_u < \Phi_V \cdot (V_c + V_s + V_p)$

$V_s = \frac{A_v \cdot f_y \cdot d_v \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha}{s_{req}}$ *(AASHTO LRFD Eq. 5.7.3.3-4)*

$a_{v\_req} = \frac{\frac{V_u}{\Phi_V} - V_c - V_p}{f_y \cdot d_v \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha}$ $a_{v\_req} = 0.002 \frac{in^2}{ft}$

Torsional Steel Required

$\Phi_T = 0.9$ *(AASHTO LRFD 5.5.4.2)*

$T_u \le \Phi_T T_n$ *(AASHTO LRFD Eq. 1.3.2.1-1)*

$T_n = \frac{2A_o A_t f_y \cot\theta}{s_{bar\_S}}$ *(AASHTO LRFD Eq. 5.7.3.6.2-1)*

$a_{t\_req} = \frac{T_u}{\Phi_T 2 A_o f_y \cot\theta}$ $a_{t\_req} = 0.22 \frac{in^2}{ft}$

Total Required Transverse Steel

$a_{req} = a_{v\_req} + 2sides \cdot a_{t\_req}$ $a_{req} = 0.44 \frac{in^2}{ft}$

$a_{prov} = \frac{A_v}{s_{bar\_S}}$ $a_{prov} = 2.71 \frac{in^2}{ft}$

The transverse reinforcement is designed for the side of the section where the effects of shear and torsion are additive. (AASHTO LRFD C5.7.3.6.1)

$a_{prov} > a_{req}$ TransverseSteelCheck = “OK!”

Longitudinal Reinforcement

$$A_{ps} f_{ps} + A_s f_y \ge \frac{|M_u|}{\Phi d_v} + \frac{0.5 N_u}{\Phi} + \cdots$$ *(AASHTO LRFD Eq. 5.7.3.6.3-1)*

$$\cot\theta \sqrt{\left(\left|\frac{V_u}{\Phi} - V_p\right| - 0.5V_s\right)^2 + \left(\frac{0.45\ \ _hT_u}{2A_0\Phi}\right)^2}$$

$V_s = a_{t\_req} \cdot f_y \cdot d_v \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha$ *(AASHTO LRFD Eq. 5.7.3.3-4)*

Bounded By: $V_s < \frac{V_u}{\Phi_V}$ $V_s = 497.1\ \text{kip}$ *(AASHTO LRFD Eq. 5.7.3.5-1)*

$$\frac{|M_u|}{\Phi_f d_v} + \frac{0.5N_u}{\Phi_c} + \cot\theta \sqrt{\left(\left|\frac{V_u}{\Phi_V} - V_p\right| - 0.5V_s\right)^2 + \left(\frac{0.45\ \ _hT_u}{2A_0\Phi_T}\right)^2} = 502\ \text{kip}$$

Provided Force:

$A_s f_y = 561.6\ \text{kip} > 502\ \text{kip}$ LongitudinalReinfChk = “OK!”

4.2.12.5 Maximum Spacing of Transverse Reinforcement *(AASHTO LRFD 5.7.2.6)*

Shear Stress

$v_u = \frac{|V_u - \Phi_V V_p|}{\Phi_V b_v d_v}$ $v_u = 0.158$ ksi *(AASHTO LRFD Eq. 5.7.2.8-1)*

$0.125 \cdot f_c = 0.625$ ksi

If $v_u < 0.125 \cdot f_c$ *(AASHTO LRFD Eq. 5.7.2.6-1)*

$s_{max} = \min(0.8d_v, 24in)$

If $v_u \geq 0.125 \cdot f_c$ *(AASHTO LRFD Eq. 5.7.2.6-2)*

$s_{max} = \min(0.4d_v, 12in)$

Since $v_u < 0.125 \cdot f_c$ $s_{max} = 24.00$ in

TxDOT limits the maximum transverse reinforcement spacing to 12". *(BDM-LRFD, Ch. 4, Sect. 5, Detailing)*

$s_{max} = 12.00$ in

$s_{bar\_S} = 7.80$ in $< s_{max}$ SpacingCheck= "OK!"

Hanger Reinforcement Summary:

Use double # 6 stirrups @ 7.80" maximum spacing

4.2.13 End Reinforcements (Bars U1, U2, U3, and G)

Extra vertical, horizontal, and diagonal reinforcing at the end surfaces is provided to reduce the maximum crack widths. According to the parametric analysis, it is recommended to place #6 U1 Bars, U2 Bars, and U3 Bars at the end faces and #7 G Bars at approximately 6in. spacing at the first 30” to 35” of the end of bent cap. U1 Bars are the vertical end reinforcements, U2 Bars and U3 Bars are the horizontal end reinforcements at the stem and the ledge, respectively. G Bars are the diagonal end reinforcement.

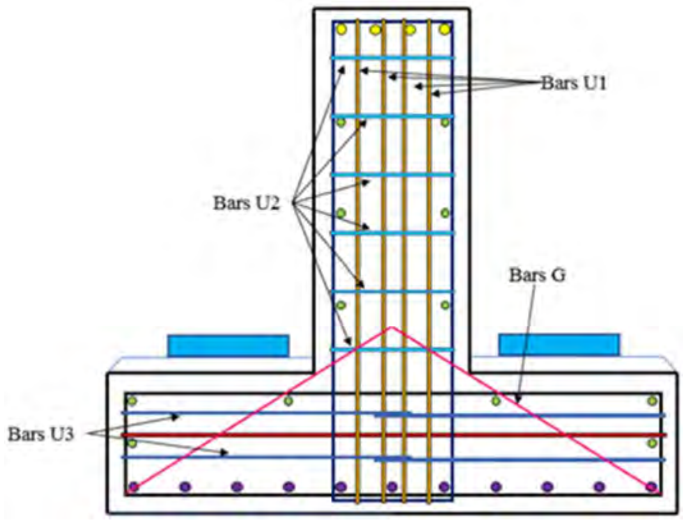


Figure 4.25 End Face Section View of 0 Degree ITBC

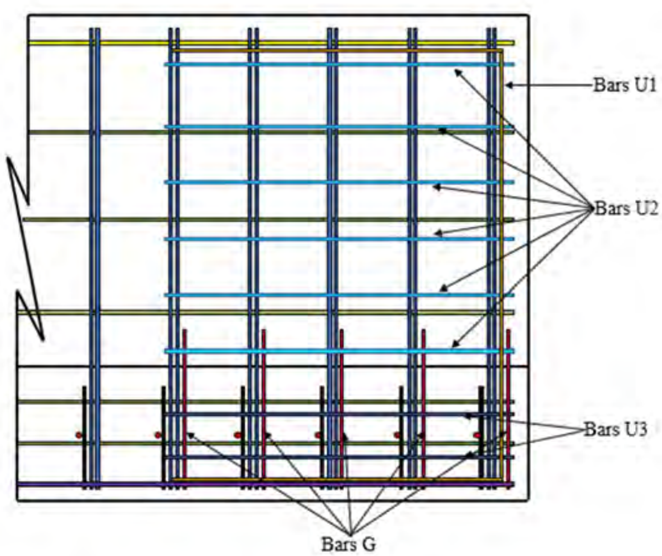


Figure 4.26 End Face Elevation View of 0 Degree ITBC

4.2.14 Skin Reinforcement (Bars T)

Try 7 ~ # 6 bars in Stem and 3 ~ # 6 bars in Ledge on each side

$A_{bar\_T} = 0.44\ in^2$

$NoTBarsStem = 7$

$NoTBarsLedge = 3$

"a" must be within $\frac{2}{3}d_e$.

(AASHTO LRFD 5.13.2.4.1)

$\frac{2}{3}d_e = 17.00\ in$

TxDOT typically uses: $a = 6\ in$

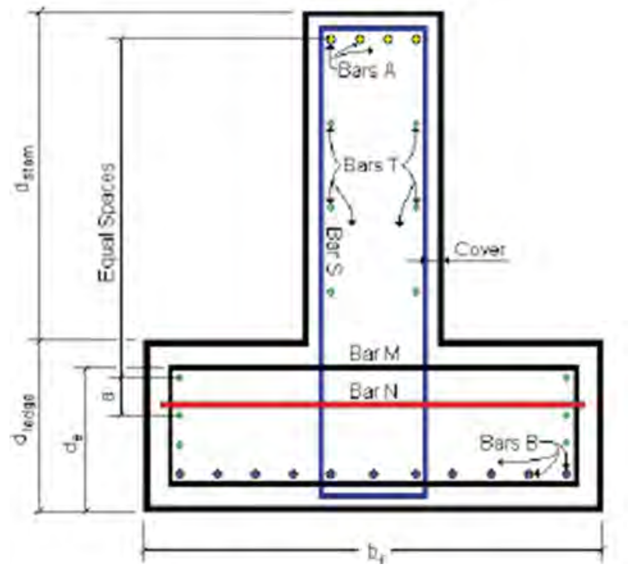


Figure 4.27 Section View for T Bars of 0 Degree Skewed ITBC

4.2.14.1 Required Area of Skin Reinforcement

(AASHTO LRFD 5.6.7)

(AASHTO LRFD Eq. 5.6.7-3)

$A_{sk\_Req} = 0.012 \cdot (d - 30)$ $\qquad A_{sk\_Req} = 0.62 \frac{in^2}{ft}$

A_{sk} need not be greater than one quarter of the main reinforcing ($A_s/4$)per side face within $d/2$ of the main reinforcing. (AASHTO LRFD 5.6.7)

"d" is the distance from the extreme compression fiber to the centroid of the extreme tension steel element. In this example design, $d = d_{s\_pos} = d_{s\_neg} = 81.42$ in.

$$A_{sk\_max} = \max\left(\frac{\frac{A_{bar\_A} \cdot BarANo}{4}}{\frac{d_{s\_neg}}{2}}, \frac{\frac{A_{bar\_B} \cdot BarBNo}{4}}{\frac{d_{s\_pos}}{2}}\right)$$

$$A_{sk\_max} = 1.26 \frac{in^2}{ft}$$

$$A_{skReq} = \min(A_{sk\_Req}, A_{sk\_max})$$

$$A_{skReq} = 0.62 \frac{in^2}{ft}$$

4.2.14.2 Required Spacing of Skin Reinforcement

(AASHTO LRFD 5.6.7)

s_{req} = minimum of:

$$\frac{A_{bar\_T}}{A_{skReq}} = 8.52\ in$$

$\frac{d_{s\_neg}}{6} = 13.57 \text{ in}$

$\frac{d_{s\_pos}}{6} = 13.57 \text{ in}$

$\& \ 12 \text{ in}$

$s_{req} = 8.52 \text{ in}$

4.2.14.3 Actual Spacing of Skin Reinforcement

Check T Bars spacing in Stem:

$$h_{top} = d_{stem} - \left(cover + \frac{d_{bar\_S}}{2} + \frac{d_{bar\_A}}{2}\right) + \left(cover + \frac{d_{bar\_M}}{2} + \frac{d_{bar\_T}}{2}\right)$$

$h_{top} = 56.67 \text{ in}$

$$s_{skStem} = \frac{h_{top}}{NoTBarsStem+1}$$

$s_{skStem} = 7.08 \text{ in}$

$s_{skStem} < s_{req}$ SkinSpacing = "OK!"

Check T Bars spacing in Ledge:

$$h_{bot} = d_{ledge} - \left(cover + \frac{d_{bar\_M}}{2} + \frac{d_{bar\_T}}{2}\right) - \left(cover + \frac{d_{bar\_S}}{2} + \frac{d_{bar\_B}}{2}\right)$$

$h_{bot} = 21.17 \text{ in}$

$$s_{skLedge} = \frac{h_{bot} - a}{NoTBarsLedge-}$$

$s_{skLedge} = 7.59 \text{ in}$

$s_{skLedge} < s_{req}$ SkinSpacing = "OK!"

Check if "a" is less than s_{req}

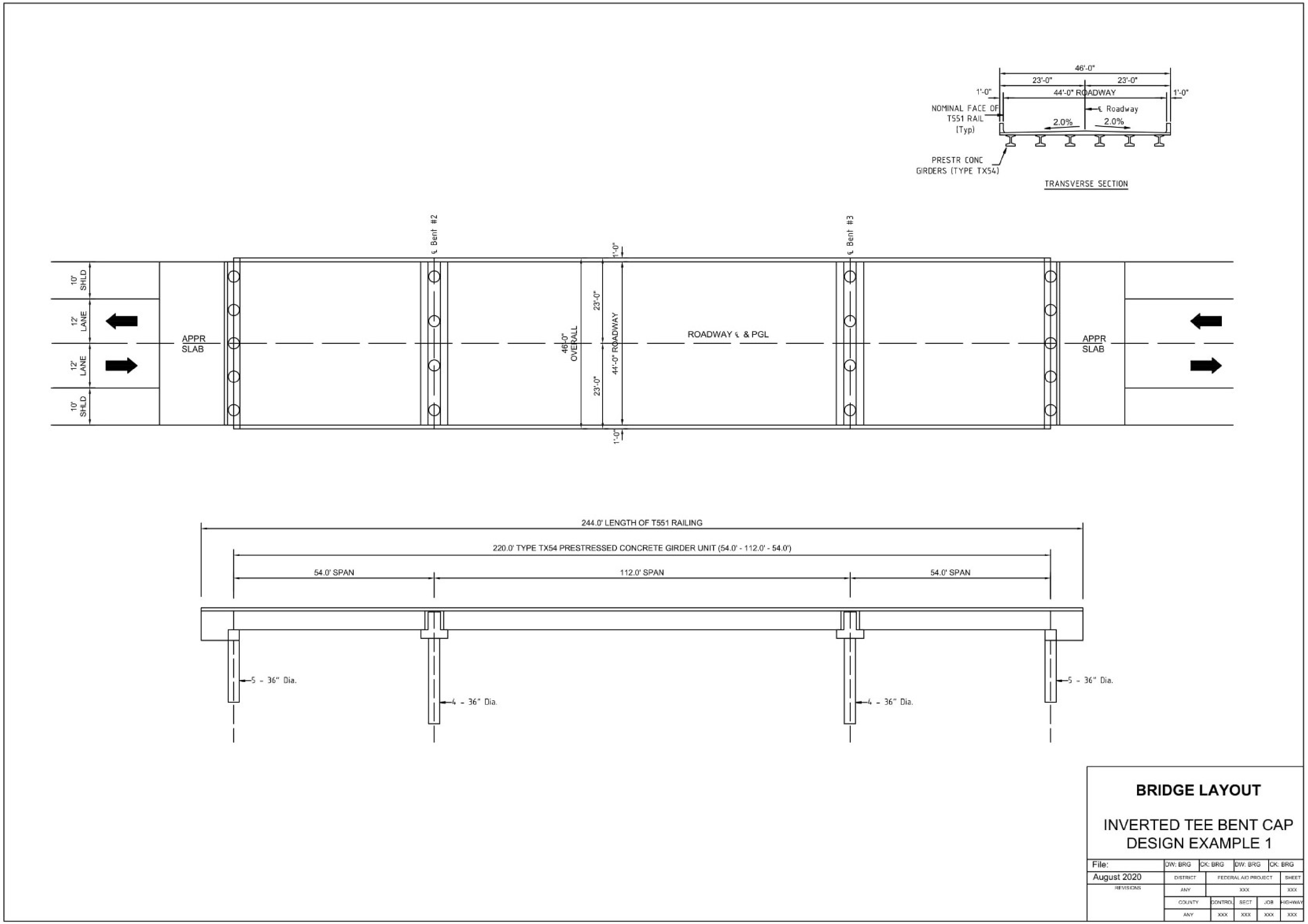
$a = 6 \text{ in} < s_{req}$ SkinSpacing = "OK!"

Skin Reinforcement Summary:

Use 7 ~ # 6 bars in Stem and 3 ~ # 6 bars in Ledge on each side

4.2.15 Design Details and Drawings

4.2.15.1 Bridge Layout



4.2.15.2 CAP 18 Input File

```
$File                         Proj              User  Date (Today
$ Num     County      Highway  Num      CSJ      Init     if Blank) Comment
$XXXX  XXXXXXXXXXXXX  XXXXXX  XXXX  XXXX-XX-XXX  XXX  XXXXXXXXXXXX XXXXXXX
00001  ___County____  Highwy  Pro#  0000-00-000  BRG               Comment
$Header Card 2 ---------------------------------------------------------------
$XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
CAP18 Version 6.00 ITBC Design Example 1, Skew = 0.00
$Problem Card ----------------------------------------------------------------
$Prob E   XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
    1 E 0 (Spans L=54'-112'-54',  Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay)
$TABLE 1 - CONTROL DATA ------------------------------------------------------
$                Enter 1 to keep:       Number cards   Options:
$                 Env Tab2 Tab3 Tab4     on Table 4  Envelope  Print   Skew Angle
$                  X    X    X    X             XX     X         XX     XXXXXXXXXX
                                                16                             0.0
$TABLE 2 - CONSTANTS ---------------------------------------------------------
$   TABLE 2a                                                Anly Opt (1=Working,
$                                  |-Movable Load Data--|  2=Load Factor,3=Both)
$                Num  Increment    |Num  Start Stop Step|Anly|  Load Factors:
$                Inc   Length      |Inc  Sta   Sta  Size| Opt| Dead     Live
$                 XX XXXXXXXXX       XXX  XXX  XXX    X    X XXXXXXXX XXXXXXXX
                 92      0.5          20    2   70    1    3    1.25     1.75
$   TABLE 2b
$    Overlay    Max #|-----------Live Load Reduction Factors---------|
$  Load Factor  Lanes| 1 lane   2 lanes   3 lanes   4 lanes   5 lanes
$    XXXXX          X  XXXX      XXXX      XXXX      XXXX      XXXX
     1.50           3  1.2       1.0       0.85      0.65      0.65
$TABLE 3 - LIST OF STATIONS --------------------------------------------------
$     Number of input values for              Str - Stringers, Sup - Supports
$                Lane Str  Sup  MCP  VCP        MCP - Moment Control Points
$                 XX   XX   XX   XX   XX        VCP - Shear Control Points
  (Num Inputs)     3    6    4   11    8
$     Left Lane Boundary Stations
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Lane Left)      2   32   60
$     Right Lane Boundary Stations
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Lane Right)    32   60   90
$     Station of Stringers (two rows max, may be at tenths of stations, XX.X)
$               XXXX XXXX XXXX XXXX XXXX XXXX XXXX XXXX XXXX XXXX
  (Stringers)      6   22   38   54   70   86
$     Station of Supports (two rows max)
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Supports)      10   34   58   82
$     Moment Control Point Stations (two rows max)
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Mom CP)         6   10   22   34   38   46   54   58   70   82
  (Mom CP)        86
$     Shear Control Point Stations (two rows max)
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Shear CP)       8   12   32   36   56   60   80   84
$TABLE 4 - STIFFNESS AND LOAD DATA -------------------------------------------
$                              Bending  Sidewalk,  Cap &
$                Station  1 if  Stiffness  Slab     Stringer  Moving    Overlay
$Comments       From  To Cont'd  of Cap    Loads     Loads     Loads    Loads,DW
$XXXXXXXXXXXXXX  XXX  XXX    X XXXXXXXXX XXXXXXXXX XXXXXXXXX XXXXXXXXX XXXXXXXXX
(CAP EI & DL)      2   90       8.09E+07            -2.427
(DL Span1, Bm1)    6    6                           -50.17              -5.04
(DL Span1, Bm2)   22   22                           -50.17              -5.04
(DL Span1, Bm3)   38   38                           -50.17              -5.04
(DL Span1, Bm4)   54   54                           -50.17              -5.04
(DL Span1, Bm5)   70   70                           -50.17              -5.04
(DL Span1, Bm6)   86   86                           -50.17              -5.04
(DL Span2, Bm1)    6    6                           -104.1              -10.5
(DL Span2, Bm2)   22   22                           -104.1              -10.5
(DL Span2, Bm3)   38   38                           -104.1              -10.5
(DL Span2, Bm4)   54   54                           -104.1              -10.5
(DL Span2, Bm5)   70   70                           -104.1              -10.5
(DL Span2, Bm6)   86   86                           -104.1              -10.5
(Dist. Lane Ld)    0   20                                     -4.92
(Conc. Lane Ld)    4    4                                     -21.3
(Conc. Lane Ld)   16   16                                     -21.3
```

4.2.15.3 CAP 18 Output File

```
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CAP18                    BENT CAP ANALYSIS       Ver.  6.2  (Jul, 2011)

    PSF             HIGHWAY  PD-    CONTROL-   CODED
     NO     COUNTY        NO   IPE  SECTION-JOB   BY    DATE
   00001  ___County____  Highwy  Pro#  0000-00-000  BRG  AUG 06, 2020 Comment

CAP18 Version 6.00 ITBC Design Example 1, Skew = 0.00
PROB     1   (Spans L=54'-112'-54',  Type TX54 Girder @ 8.0', 8" Slab, 2" O'la


ENGLISH SYSTEM UNITS

TABLE 1.  CONTROL DATA

    OPTION TO PRINT TABLE SRS (1=YES)                              0

                                    ENVELOPES    TABLE NUMBER
                                    OF MAXIMUMS   2    3    4
    KEEP FROM PRECEDING PROBLEM (1=YES)              0       0    0    0
    CARDS INPUT THIS PROBLEM                              16

    OPTION TO CLEAR ENVELOPES BEFORE LANE LOADINGS (1=YES)             0

    OPTION TO OMIT PRINT FOR TABLES (TABLE DESIGNATIONS IN PARENTHESES)
      -1(4A), -2(5) -3(4A,5), -4(4A,5,6), -5(4A,5,6,7):               0

    SKEW ANGLE, DEGREES                                 0.000


TABLE 2.  CONSTANTS

    NUMBER OF INCREMENTS FOR SLAB AND CAP                          92
    INCREMENT LENGTH, FT                                0.500
    NUMBER OF INCREMENTS FOR MOVABLE LOAD                            20
    START POSITION OF MOVABLE-LOAD STA ZERO                         2
    STOP POSITION OF MOVABLE-LOAD STA ZERO                         70
    NUMBER OF INCREMENTS BETWEEN EACH POSITION OF MOVABLE LOAD          1

    ANALYSIS OPTION (1=WORKING STRESS, 2=LOAD FACTOR, 3=BOTH)          3

    LOAD FACTOR FOR DEAD LOAD                            1.25
    LOAD FACTOR FOR OVERLAY LOAD                           1.50
    LOAD FACTOR FOR LIVE LOAD                            1.75

    MAXIMUM NUMBER OF LANES TO BE LOADED SIMULTANEOUSLY              3

    LIST OF LOAD COEFFICIENTS CORRESPONDING TO NUMBER OF LANES LOADED
        1            2            3            4            5
      1.200        1.000        0.850
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 3. LISTS OF STATIONS

	NUM OF LANES	NUM OF STRINGERS	NUM OF SUPPORTS	NUM MOM CONTR PTS	NUM SHEAR CONTR PTS
TOTAL	3	6	4	11	8

LANE LEFT 2 32 60

LANE RIGHT 32 60 90
STRINGERS 6.0 22.0 38.0 54.0 70.0 86.0
SUPPORTS 10 34 58 82
MOM CONTR 6 10 22 34 38 46 54 58 70 82
86
SHEAR CONTR 8 12 32 36 56 60 80 84

TABLE 4. STIFFNESS AND LOAD DATA

STA FROM	STA TO	CONTD IF=1	CAP BENDING STIFFNESS (K-FT*FT)	SIDEWALK, SLAB LOADS (K)	STRINGER, CAP LOADS (K)	OVERLAY LOADS (K)	MOVABLE-POSITION SLAB LOADS (K)
2	90	0	80900000.000	0.000	-2.427	0.000	0.000
6	6	0	0.000	0.000	-50.170	-5.040	0.000
22	22	0	0.000	0.000	-50.170	-5.040	0.000
38	38	0	0.000	0.000	-50.170	-5.040	0.000
54	54	0	0.000	0.000	-50.170	-5.040	0.000
70	70	0	0.000	0.000	-50.170	-5.040	0.000
86	86	0	0.000	0.000	-50.170	-5.040	0.000
6	6	0	0.000	0.000	-104.100	-10.500	0.000
22	22	0	0.000	0.000	-104.100	-10.500	0.000
38	38	0	0.000	0.000	-104.100	-10.500	0.000
54	54	0	0.000	0.000	-104.100	-10.500	0.000
70	70	0	0.000	0.000	-104.100	-10.500	0.000
86	86	0	0.000	0.000	-104.100	-10.500	0.000
0	20	0	0.000	0.000	0.000	0.000	-4.920
4	4	0	0.000	0.000	0.000	0.000	-21.300
16	16	0	0.000	0.000	0.000	0.000	-21.300

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 4A. DEAD LOAD RESULTS (WORKING STRESS)

STA	DIST X (FT)	DEFLECTION (FT)	MOMENT (K-FT)	SHEAR (K)
-1	-0.50	0.000000	0.0	0.0
0	0.00	0.000000	0.0	0.0
1	0.50	-0.000034	0.0	0.0
2	1.00	-0.000029	0.0	-0.6
3	1.50	-0.000025	-0.6	-2.4
4	2.00	-0.000021	-2.4	-4.9
5	2.50	-0.000017	-5.5	-7.3
6	3.00	-0.000013	-9.7	-94.6
7	3.50	-0.000009	-100.1	-181.9
8	4.00	-0.000005	-191.7	-184.4
9	4.50	-0.000002	-284.4	-186.8
10	5.00	0.000000	-378.5	-35.0
11	5.50	0.000001	-319.5	116.7
12	6.00	0.000001	-261.7	114.3
13	6.50	0.000000	-205.2	111.9
14	7.00	-0.000001	-149.8	109.5
15	7.50	-0.000003	-95.7	107.0
16	8.00	-0.000005	-42.8	104.6
17	8.50	-0.000007	8.9	102.2
18	9.00	-0.000009	59.4	99.8
19	9.50	-0.000011	108.7	97.3
20	10.00	-0.000013	156.7	94.9
21	10.50	-0.000014	203.6	92.5
22	11.00	-0.000015	249.2	5.1
23	11.50	-0.000015	208.7	-82.2
24	12.00	-0.000014	167.0	-84.6
25	12.50	-0.000012	124.1	-87.0
26	13.00	-0.000011	80.0	-89.5
27	13.50	-0.000009	34.6	-91.9
28	14.00	-0.000006	-11.9	-94.3
29	14.50	-0.000004	-59.7	-96.8
30	15.00	-0.000003	-108.7	-99.2
31	15.50	-0.000001	-158.9	-101.6
32	16.00	0.000000	-210.3	-104.0
33	16.50	0.000000	-262.9	-106.5
34	17.00	0.000000	-316.7	45.0
35	17.50	-0.000001	-217.9	196.5
36	18.00	-0.000003	-120.2	194.1
37	18.50	-0.000006	-23.8	191.7
38	19.00	-0.000008	71.4	104.3
39	19.50	-0.000011	80.5	17.0
40	20.00	-0.000013	88.4	14.6
41	20.50	-0.000015	95.1	12.1
42	21.00	-0.000016	100.5	9.7
43	21.50	-0.000017	104.8	7.3

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 4A. DEAD LOAD RESULTS (WORKING STRESS)

STA	DIST X (FT)	DEFLECTION (FT)	MOMENT (K-FT)	SHEAR (K)
44	22.00	-0.000018	107.8	4.9
45	22.50	-0.000019	109.6	2.4
46	23.00	-0.000019	110.2	0.0
47	23.50	-0.000019	109.6	-2.4
48	24.00	-0.000018	107.8	-4.9
49	24.50	-0.000017	104.8	-7.3
50	25.00	-0.000016	100.5	-9.7
51	25.50	-0.000015	95.1	-12.1
52	26.00	-0.000013	88.4	-14.6
53	26.50	-0.000011	80.5	-17.0
54	27.00	-0.000008	71.4	-104.3
55	27.50	-0.000006	-23.8	-191.7
56	28.00	-0.000003	-120.2	-194.1
57	28.50	-0.000001	-217.9	-196.5
58	29.00	0.000000	-316.7	-45.0
59	29.50	0.000000	-262.9	106.5
60	30.00	0.000000	-210.3	104.0
61	30.50	-0.000001	-158.9	101.6
62	31.00	-0.000003	-108.7	99.2
63	31.50	-0.000004	-59.7	96.8
64	32.00	-0.000006	-11.9	94.3
65	32.50	-0.000009	34.6	91.9
66	33.00	-0.000011	80.0	89.5
67	33.50	-0.000012	124.1	87.0
68	34.00	-0.000014	167.0	84.6
69	34.50	-0.000015	208.7	82.2
70	35.00	-0.000015	249.2	-5.1
71	35.50	-0.000014	203.6	-92.5
72	36.00	-0.000013	156.7	-94.9
73	36.50	-0.000011	108.7	-97.3
74	37.00	-0.000009	59.4	-99.8
75	37.50	-0.000007	8.9	-102.2
76	38.00	-0.000005	-42.8	-104.6
77	38.50	-0.000003	-95.7	-107.0
78	39.00	-0.000001	-149.8	-109.5
79	39.50	0.000000	-205.2	-111.9
80	40.00	0.000001	-261.7	-114.3
81	40.50	0.000001	-319.5	-116.7
82	41.00	0.000000	-378.5	35.0
83	41.50	-0.000002	-284.4	186.8
84	42.00	-0.000005	-191.7	184.4
85	42.50	-0.000009	-100.1	181.9
86	43.00	-0.000013	-9.7	94.6
87	43.50	-0.000017	-5.5	7.3
88	44.00	-0.000021	-2.4	4.9
89	44.50	-0.000025	-0.6	2.4
90	45.00	-0.000029	0.0	0.6

```
  91      45.50      -0.000034       0.0        0.0
  92      46.00       0.000000       0.0        0.0
  93      46.50       0.000000       0.0        0.0

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PROB    1  (Spans L=54'-112'-54',  Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)


TABLE 5.  MULTI-LANE LOADING SUMMARY    ( WORKING STRESS )
        ( *--CRITICAL NUMBER OF LANE LOADS)

    MOMENT ( FT-K )
    ------------------------------------------------------------------------
    AT    DEAD LD   LANE   POSITIVE   LOAD AT    LANE   NEGATIVE   LOAD AT
    STA    EFFECT   ORDER  MAXIMUM    LANE STA   ORDER  MAXIMUM    LANE STA
    ------------------------------------------------------------------------

     6     -9.7
                0     0.0            0     0.0
                1     0.0            1     0.0
                2     0.0            2     0.0
                3     0.0            3     0.0
                0*                   0*

    10   -378.5
                0     0.0            0   -176.2    1   2
                1     0.0            1   -176.2    1   2
                2     0.0            2     0.0
                3     0.0            3     0.0
                0*                   0*

    22    249.2
                0   202.0    0  13   0    -33.4    2  36
                1   201.2    1  12   1    -33.4    2  36
                2     9.3    3  62   2     0.0
                3     0.0            3     0.0
                0*                   0*

    34   -316.7
                0    18.7    3  62   0   -136.3    0  18
                1    18.7    3  62   1   -116.6    1  12
                2     0.0            2    -84.7    2  32
                3     0.0            3     0.0
                0*                   2*

    38     71.4
                0    83.6    2  32   0    -58.8    1   9
                1    83.6    2  32   1    -58.8    1   9
                2     3.2    3  62   2     0.0
                3     0.0            3     0.0
                0*                   0*

    46    110.2
                0    69.4    2  36   0    -27.8    1   9
                1    69.4    2  36   1    -27.8    1   9
                2     0.0            2    -27.8    3  63
                3     0.0            3     0.0
                0*                   2*

    54     71.4
                0    83.6    2  40   0    -58.8    3  63
                1    83.6    2  40   1    -58.8    3  63
                2     3.2    1  10   2     0.0
                3     0.0            3     0.0
                0*                   0*

    58   -316.7
                0    18.7    1   9   0   -136.3    0  54
                1    18.7    1   9   1   -116.6    3  60
                2     0.0            2    -84.7    2  40
                3     0.0            3     0.0
                0*                   2*
```

```
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PROB     1  (Spans L=54'-112'-54',  Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

   MOMENT ( FT-K )
   ----------------------------------------------------------------------
   AT    DEAD LD   LANE   POSITIVE   LOAD AT    LANE   NEGATIVE   LOAD AT
   STA    EFFECT   ORDER  MAXIMUM    LANE STA   ORDER  MAXIMUM    LANE STA
   ----------------------------------------------------------------------

    70    249.2
              0    202.0    0  59    0     -33.4    2  36
              1    201.2    3  60    1     -33.4    2  36
              2      9.3    1   9    2       0.0
              3      0.0             3       0.0
              0*                     0*

    82   -378.5
              0      0.0             0   -176.3    3  70
              1      0.0             1   -176.3    3  70
              2      0.0             2      0.0
              3      0.0             3      0.0
              0*                     0*

    86     -9.7
              0      0.0             0      0.0
              1      0.0             1      0.0
              2      0.0             2      0.0
              3      0.0             3      0.0
              0*                     0*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

```
SHEAR ( K )
-------------------------------------------------------------------------
AT   DEAD LD  LANE   POSITIVE   LOAD AT   LANE   NEGATIVE   LOAD AT
STA   EFFECT  ORDER  MAXIMUM    LANE STA  ORDER  MAXIMUM    LANE STA
-------------------------------------------------------------------------

 8   -184.4
         0     0.0              0    -88.1     1   2
         1     0.0              1    -88.1     1   2
         2     0.0              2      0.0
         3     0.0              3      0.0
         0*                     0*

12    114.3
         0    44.8     1   6    0     -5.6     2  36
         1    44.8     1   6    1     -5.6     2  36
         2     1.6     3  62    2      0.0
         3     0.0              3      0.0
         0*                     0*

32   -104.0
         0     1.6     3  62    0    -54.6     0  15
         1     1.6     3  62    1    -53.0     1  12
         2     0.0              2    -11.2     2  32
         3     0.0              3      0.0
         0*                     0*

36    194.1
         0    87.6     0  28    0     -7.8     3  63
         1    84.1     2  32    1     -7.8     3  63
         2    30.7     1  12    2      0.0
         3     0.0              3      0.0
         2*                     0*

56   -194.1
         0     7.8     1   9    0    -87.6     0  44
         1     7.8     1   9    1    -84.1     2  40
         2     0.0              2    -30.7     3  60
         3     0.0              3      0.0
         0*                     2*

60    104.0
         0    54.6     0  57    0     -1.6     1   9
         1    53.0     3  60    1     -1.6     1   9
         2    11.2     2  40    2      0.0
         3     0.0              3      0.0
         0*                     0*

80   -114.3
         0     5.6     2  36    0    -44.8     3  66
         1     5.6     2  36    1    -44.8     3  66
         2     0.0              2     -1.6     1   9
         3     0.0              3      0.0
         0*                     0*

84    184.4
         0    88.1     3  70    0      0.0
         1    88.1     3  70    1      0.0
         2     0.0              2      0.0
         3     0.0              3      0.0
         0*                     0*
```

```
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PROB     1  (Spans L=54'-112'-54',  Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)


   REACTION ( K )
   --------------------------------------------------------------------
   AT    DEAD LD   LANE   POSITIVE   LOAD AT    LANE   NEGATIVE   LOAD AT
   STA    EFFECT   ORDER  MAXIMUM    LANE STA   ORDER  MAXIMUM    LANE STA
   --------------------------------------------------------------------

    10    308.4
             0     127.9    1   2   0     -5.6    2  36
             1     127.9    1   2   1     -5.6    2  36
             2       1.6    3  62   2      0.0
             3       0.0            3      0.0
             0*                     0*

    34    307.8
             0     117.1    0  22   0     -9.3    3  63
             1      95.3    2  32   1     -9.3    3  63
             2      83.6    1  12   2      0.0
             3       0.0            3      0.0
             2*                     0*

    58    307.8
             0     117.1    0  50   0     -9.3    1   9
             1      95.3    2  40   1     -9.3    1   9
             2      83.6    3  60   2      0.0
             3       0.0            3      0.0
             2*                     0*

    82    308.4
             0     127.9    3  70   0     -5.6    2  36
             1     127.9    3  70   1     -5.6    2  36
             2       1.6    1   9   2      0.0
             3       0.0            3      0.0
             0*                     0*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
-1	-0.50	0.0	0.0	0.0	0.0
0	0.00	0.0	0.0	0.0	0.0
1	0.50	0.0	0.0	0.0	0.0
2	1.00	0.0	0.0	-0.6	-0.6
3	1.50	-0.6	-0.6	-2.4	-2.4
4	2.00	-2.4	-2.4	-4.9	-4.9
5	2.50	-5.5	-5.5	-7.3	-7.3
6	3.00	-9.7	-9.7	-94.6	-147.5
7	3.50	-100.1	-152.9	-181.9	-287.7
8	4.00	-191.7	-297.4	-184.4	-290.1
9	4.50	-284.4	-443.1	-186.8	-292.5
10	5.00	-378.5	-590.0	-18.1	-64.1
11	5.50	-306.4	-507.1	170.5	110.1
12	6.00	-230.8	-425.5	168.1	107.6
13	6.50	-155.9	-345.1	165.7	105.2
14	7.00	-82.2	-265.9	163.3	102.8
15	7.50	-9.3	-187.9	160.8	100.4
16	8.00	63.6	-111.2	158.4	97.9
17	8.50	136.3	-35.6	156.0	95.5
18	9.00	208.8	32.7	153.5	93.1
19	9.50	280.5	78.6	151.1	90.7
20	10.00	351.7	123.4	148.7	88.2
21	10.50	422.0	166.9	146.3	85.8
22	11.00	491.6	209.2	21.1	-8.0
23	11.50	418.8	165.0	-80.3	-147.7
24	12.00	344.9	119.4	-82.7	-150.2
25	12.50	270.2	72.4	-85.2	-152.6
26	13.00	194.5	24.0	-87.6	-155.0
27	13.50	118.3	-26.0	-90.0	-157.5
28	14.00	47.3	-77.1	-92.5	-159.9
29	14.50	-23.4	-129.5	-94.9	-162.3
30	15.00	-90.0	-183.5	-97.3	-164.7
31	15.50	-139.3	-264.0	-99.7	-167.2
32	16.00	-189.7	-347.4	-102.2	-169.6
33	16.50	-241.4	-432.1	-104.6	-172.0
34	17.00	-294.3	-518.0	88.8	27.4
35	17.50	-200.1	-361.7	311.3	187.2
36	18.00	-107.1	-224.4	308.9	184.8
37	18.50	26.9	-108.8	306.5	182.4
38	19.00	171.7	0.8	162.8	95.0
39	19.50	177.8	14.6	26.3	7.7
40	20.00	183.1	27.1	23.9	5.3
41	20.50	187.3	38.4	21.4	2.8
42	21.00	190.7	44.9	19.0	0.4

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
43	21.50	192.9	49.1	16.6	-2.0
44	22.00	193.8	52.2	14.2	-4.4
45	22.50	193.6	54.0	11.7	-6.9
46	23.00	193.5	54.6	9.3	-9.3
47	23.50	193.6	54.0	6.9	-11.7
48	24.00	193.8	52.2	4.4	-14.2
49	24.50	192.9	49.1	2.0	-16.6
50	25.00	190.7	44.9	-0.4	-19.0
51	25.50	187.3	38.4	-2.8	-21.4
52	26.00	183.1	27.1	-5.3	-23.9
53	26.50	177.8	14.6	-7.7	-26.3
54	27.00	171.7	0.8	-95.0	-162.8
55	27.50	26.9	-108.8	-182.4	-306.5
56	28.00	-107.1	-224.4	-184.8	-308.9
57	28.50	-200.1	-361.7	-187.2	-311.3
58	29.00	-294.3	-518.0	-27.4	-88.8
59	29.50	-241.4	-432.1	172.0	104.6
60	30.00	-189.7	-347.4	169.6	102.2
61	30.50	-139.3	-264.0	167.2	99.7
62	31.00	-90.0	-183.5	164.7	97.3
63	31.50	-23.4	-129.5	162.3	94.9
64	32.00	47.3	-77.1	159.9	92.5
65	32.50	118.3	-26.0	157.5	90.0
66	33.00	194.5	24.0	155.0	87.6
67	33.50	270.2	72.4	152.6	85.2
68	34.00	344.9	119.4	150.2	82.7
69	34.50	418.8	165.0	147.7	80.3
70	35.00	491.6	209.2	8.0	-21.1
71	35.50	422.0	166.9	-85.8	-146.3
72	36.00	351.7	123.4	-88.2	-148.7
73	36.50	280.5	78.6	-90.7	-151.1
74	37.00	208.8	32.7	-93.1	-153.5
75	37.50	136.3	-35.6	-95.5	-156.0
76	38.00	63.6	-111.2	-97.9	-158.4
77	38.50	-9.3	-187.9	-100.4	-160.8
78	39.00	-82.2	-265.9	-102.8	-163.3
79	39.50	-155.9	-345.1	-105.2	-165.7
80	40.00	-230.8	-425.5	-107.6	-168.1
81	40.50	-306.4	-507.1	-110.1	-170.5
82	41.00	-378.5	-590.0	64.1	18.1
83	41.50	-284.4	-443.1	292.5	186.8
84	42.00	-191.7	-297.4	290.1	184.4
85	42.50	-100.1	-152.9	287.7	181.9
86	43.00	-9.7	-9.7	147.5	94.6

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
87	43.50	-5.5	-5.5	7.3	7.3
88	44.00	-2.4	-2.4	4.9	4.9
89	44.50	-0.6	-0.6	2.4	2.4
90	45.00	0.0	0.0	0.6	0.6
91	45.50	0.0	0.0	0.0	0.0
92	46.00	0.0	0.0	0.0	0.0
93	46.50	0.0	0.0	0.0	0.0

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 7. MAXIMUM SUPPORT REACTIONS (WORKING STRESS)

STA	DIST X (FT)	MAX + REACT (K)	MAX - REACT (K)
10	5.00	461.8	301.7
34	17.00	486.7	296.7
58	29.00	486.7	296.7
82	41.00	461.8	301.7

```
PROB    1  (Spans L=54'-112'-54',  Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)


TABLE 5.  MULTI-LANE LOADING SUMMARY    ( LOAD FACTOR)
       ( *--CRITICAL NUMBER OF LANE LOADS)

    MOMENT ( FT-K )
    ------------------------------------------------------------------------
    AT    DEAD LD   LANE   POSITIVE   LOAD AT    LANE   NEGATIVE   LOAD AT
    STA    EFFECT   ORDER  MAXIMUM    LANE STA   ORDER  MAXIMUM    LANE STA
    ------------------------------------------------------------------------

     6    -12.1
               0     0.0               0     0.0
               1     0.0               1     0.0
               2     0.0               2     0.0
               3     0.0               3     0.0
               0*                      0*

    10    -480.8
               0     0.0               0   -308.4    1   2
               1     0.0               1   -308.4    1   2
               2     0.0               2     0.0
               3     0.0               3     0.0
               0*                      0*

    22     316.4
               0   353.5    0  13    0    -58.4    2  36
               1   352.1    1  12    1    -58.4    2  36
               2    16.3    3  62    2      0.0
               3     0.0             3     0.0
               0*                    0*

    34    -401.8
               0    32.7    3  62    0   -238.5    0  18
               1    32.7    3  62    1   -204.0    1  12
               2     0.0             2   -148.2    2  32
               3     0.0             3     0.0
               0*                    2*

    38     91.2
               0   146.3    2  32    0   -102.9    1   9
               1   146.3    2  32    1   -102.9    1   9
               2     5.6    3  62    2      0.0
               3     0.0             3     0.0
               0*                    0*

    46    139.7
               0   121.4    2  36    0    -48.7    1   9
               1   121.4    2  36    1    -48.7    1   9
               2     0.0             2    -48.7    3  63
               3     0.0             3     0.0
               0*                    2*

    54     91.2
               0   146.3    2  40    0   -102.9    3  63
               1   146.3    2  40    1   -102.9    3  63
               2     5.6    1  10    2      0.0
               3     0.0             3     0.0
               0*                    0*

    58    -401.8
               0    32.7    1   9    0   -238.5    0  54
               1    32.7    1   9    1   -204.0    3  60
               2     0.0             2   -148.2    2  40
               3     0.0             3     0.0
               0*                    2*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

```
MOMENT ( FT-K )
------------------------------------------------------------------------
AT   DEAD LD  LANE  POSITIVE  LOAD AT   LANE  NEGATIVE  LOAD AT
STA   EFFECT  ORDER  MAXIMUM   LANE STA  ORDER  MAXIMUM   LANE STA
------------------------------------------------------------------------

70   316.4
       0    353.5    0  59    0    -58.4    2  36
       1    352.1    3  60    1    -58.4    2  36
       2     16.3    1   9    2      0.0
       3      0.0             3      0.0
       0*                     0*

82  -480.8
       0      0.0             0   -308.4    3  70
       1      0.0             1   -308.4    3  70
       2      0.0             2      0.0
       3      0.0             3      0.0
       0*                     0*

86   -12.1
       0      0.0             0      0.0
       1      0.0             1      0.0
       2      0.0             2      0.0
       3      0.0             3      0.0
       0*                     0*
```

```
PROB    1  (Spans L=54'-112'-54',  Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

   SHEAR ( K )
   ------------------------------------------------------------------------
   AT   DEAD LD  LANE  POSITIVE  LOAD AT   LANE  NEGATIVE  LOAD AT
   STA   EFFECT  ORDER MAXIMUM   LANE STA  ORDER MAXIMUM   LANE STA
   ------------------------------------------------------------------------

    8   -234.3
            0     0.0           0   -154.2    1   2
            1     0.0           1   -154.2    1   2
            2     0.0           2     0.0
            3     0.0           3     0.0
            0*                  0*

   12    145.0
            0    78.4   1   6   0     -9.7    2  36
            1    78.4   1   6   1     -9.7    2  36
            2     2.7   3  62   2      0.0
            3     0.0           3     0.0
            0*                  0*

   32   -131.8
            0     2.7   3  62   0    -95.6    0  15
            1     2.7   3  62   1    -92.7    1  12
            2     0.0           2   -19.5   2  32
            3     0.0           3     0.0
            0*                  0*

   36    246.5
            0   153.2   0  28   0    -13.6    3  63
            1   147.2   2  32   1    -13.6    3  63
            2    53.7   1  12   2      0.0
            3     0.0           3     0.0
            2*                  0*

   56   -246.5
            0    13.6   1   9   0   -153.2    0  44
            1    13.6   1   9   1   -147.2    2  40
            2     0.0           2   -53.7   3  60
            3     0.0           3     0.0
            0*                  2*

   60    131.8
            0    95.6   0  57   0     -2.7    1   9
            1    92.7   3  60   1     -2.7    1   9
            2    19.5   2  40   2      0.0
            3     0.0           3     0.0
            0*                  0*

   80   -145.0
            0     9.7   2  36   0    -78.4    3  66
            1     9.7   2  36   1    -78.4    3  66
            2     0.0           2    -2.7   1   9
            3     0.0           3     0.0
            0*                  0*

   84    234.3
            0   154.2   3  70   0      0.0
            1   154.2   3  70   1      0.0
            2     0.0           2     0.0
            3     0.0           3     0.0
            0*                  0*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

```
REACTION ( K )
-----------------------------------------------------------------------
AT    DEAD LD  LANE   POSITIVE   LOAD AT    LANE   NEGATIVE   LOAD AT
STA   EFFECT   ORDER  MAXIMUM    LANE STA   ORDER  MAXIMUM    LANE STA
-----------------------------------------------------------------------

10    391.5
         0    223.8    1  2    0     -9.7    2  36
         1    223.8    1  2    1     -9.7    2  36
         2      2.7    3 62    2      0.0
         3      0.0            3      0.0
         0*                    0*

34    390.4
         0    205.0    0 22    0    -16.3    3  63
         1    166.8    2 32    1    -16.3    3  63
         2    146.3    1 12    2      0.0
         3      0.0            3      0.0
         2*                    0*

58    390.4
         0    205.0    0 50    0    -16.3    1   9
         1    166.8    2 40    1    -16.3    1   9
         2    146.3    3 60    2      0.0
         3      0.0            3      0.0
         2*                    0*

82    391.5
         0    223.8    3 70    0     -9.7    2  36
         1    223.8    3 70    1     -9.7    2  36
         2      2.7    1  9    2      0.0
         3      0.0            3      0.0
         0*                    0*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
-1	-0.50	0.0	0.0	0.0	0.0
0	0.00	0.0	0.0	0.0	0.0
1	0.50	0.0	0.0	0.0	0.0
2	1.00	0.0	0.0	-0.8	-0.8
3	1.50	-0.8	-0.8	-3.0	-3.0
4	2.00	-3.0	-3.0	-6.1	-6.1
5	2.50	-6.8	-6.8	-9.1	-9.1
6	3.00	-12.1	-12.1	-120.2	-212.7
7	3.50	-127.0	-219.6	-231.3	-416.4
8	4.00	-243.5	-428.5	-234.3	-419.4
9	4.50	-361.4	-639.0	-237.4	-422.4
10	5.00	-480.8	-851.0	-15.1	-95.5
11	5.50	-383.1	-734.4	242.2	136.4
12	6.00	-278.8	-619.5	239.1	133.3
13	6.50	-174.8	-506.0	236.1	130.3
14	7.00	-72.4	-394.0	233.1	127.3
15	7.50	29.0	-283.6	230.0	124.2
16	8.00	131.2	-174.6	227.0	121.2
17	8.50	233.6	-67.2	224.0	118.2
18	9.00	336.3	28.2	220.9	115.1
19	9.50	438.2	85.0	217.9	112.1
20	10.00	539.8	140.3	214.9	109.1
21	10.50	640.4	194.0	211.8	106.0
22	11.00	740.6	246.3	34.5	-16.4
23	11.50	632.4	188.3	-101.2	-219.2
24	12.00	523.1	128.6	-104.3	-222.3
25	12.50	413.0	66.9	-107.3	-225.3
26	13.00	301.6	3.3	-110.3	-228.3
27	13.50	190.1	-62.3	-113.4	-231.4
28	14.00	88.3	-129.5	-116.4	-234.4
29	14.50	-12.5	-198.1	-119.5	-237.4
30	15.00	-105.4	-269.1	-122.5	-240.5
31	15.50	-167.4	-385.7	-125.5	-243.5
32	16.00	-230.9	-506.9	-128.6	-246.5
33	16.50	-296.0	-629.7	-131.6	-249.6
34	17.00	-362.5	-754.0	134.0	26.6
35	17.50	-245.1	-528.0	450.4	233.2
36	18.00	-129.3	-334.5	447.4	230.2
37	18.50	59.0	-178.5	444.4	227.2
38	19.00	266.8	-32.3	234.6	116.1
39	19.50	272.9	-12.8	37.5	5.0
40	20.00	278.1	5.2	34.5	1.9
41	20.50	282.2	21.7	31.4	-1.1
42	21.00	285.4	30.2	28.4	-4.1
43	21.50	287.1	35.5	25.4	-7.2
44	22.00	287.2	39.3	22.3	-10.2

45	22.50	285.9	41.6	19.3	-13.2
46	23.00	285.4	42.4	16.3	-16.3
47	23.50	285.9	41.6	13.2	-19.3
48	24.00	287.2	39.3	10.2	-22.3

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
49	24.50	287.1	35.5	7.2	-25.4
50	25.00	285.4	30.2	4.1	-28.4
51	25.50	282.2	21.7	1.1	-31.4
52	26.00	278.1	5.2	-1.9	-34.5
53	26.50	272.9	-12.8	-5.0	-37.5
54	27.00	266.8	-32.3	-116.1	-234.6
55	27.50	59.0	-178.5	-227.2	-444.4
56	28.00	-129.3	-334.5	-230.2	-447.4
57	28.50	-245.1	-528.0	-233.2	-450.4
58	29.00	-362.5	-754.0	-26.6	-134.0
59	29.50	-296.0	-629.7	249.6	131.6
60	30.00	-230.9	-506.9	246.5	128.6
61	30.50	-167.4	-385.7	243.5	125.5
62	31.00	-105.4	-269.1	240.5	122.5
63	31.50	-12.5	-198.1	237.4	119.5
64	32.00	88.3	-129.5	234.4	116.4
65	32.50	190.1	-62.3	231.4	113.4
66	33.00	301.6	3.3	228.3	110.3
67	33.50	413.0	66.9	225.3	107.3
68	34.00	523.1	128.6	222.3	104.3
69	34.50	632.4	188.3	219.2	101.2
70	35.00	740.6	246.3	16.4	-34.5
71	35.50	640.4	194.0	-106.0	-211.8
72	36.00	539.8	140.3	-109.1	-214.9
73	36.50	438.2	85.0	-112.1	-217.9
74	37.00	336.3	28.2	-115.1	-220.9
75	37.50	233.6	-67.2	-118.2	-224.0
76	38.00	131.2	-174.6	-121.2	-227.0
77	38.50	29.0	-283.6	-124.2	-230.0
78	39.00	-72.4	-394.0	-127.3	-233.1
79	39.50	-174.8	-506.0	-130.3	-236.1
80	40.00	-278.8	-619.5	-133.3	-239.1
81	40.50	-383.1	-734.4	-136.4	-242.2
82	41.00	-480.8	-851.0	95.5	15.1
83	41.50	-361.4	-639.0	422.4	237.4
84	42.00	-243.5	-428.5	419.4	234.3
85	42.50	-127.0	-219.6	416.4	231.3
86	43.00	-12.1	-12.1	212.7	120.2
87	43.50	-6.8	-6.8	9.1	9.1
88	44.00	-3.0	-3.0	6.1	6.1
89	44.50	-0.8	-0.8	3.0	3.0

90	45.00	0.0	0.0	0.8	0.8
91	45.50	0.0	0.0	0.0	0.0
92	46.00	0.0	0.0	0.0	0.0

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
93	46.50	0.0	0.0	0.0	0.0

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 7. MAXIMUM SUPPORT REACTIONS (LOAD FACTOR)

STA	DIST X (FT)	MAX + REACT (K)	MAX - REACT (K)
10	5.00	660.0	379.8
34	17.00	703.5	370.9
58	29.00	703.5	370.9
82	41.00	660.0	379.8

4.2.15.4 Live Load Distribution Factor Spreadsheet

4.2.15.4.1 *Spans 1 & 3*

TXDOT BRIDGE DIVISION	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:		Rev. 10/18 - (No Interim)
	Descrip:	ITBC Design Example 1, Span 1 & 3			File:	Ex1_Span1_distribution_factors.xl		Sheet:	1 of 8

LRFD Live Load Distribution Factors*

Live Load Distribution Factors are calculated according to AASHTO LRFD Bridge Design Specifications, 8th Edition (2017 with no interim revisions) as prescribed by TxDOT policies (LRFD Design Manual July 2018) and practices. The Lever Rule is used when outside the Range of Applicability. The Range of Applicability for the Skew Correction Factors is ignored.

INPUT:

Beam Type =	Tx54	
No. Beams, N_b =	6	
CL_{brg} to CL_{brg}, L =	50.38	ft
Beam Spacing, S =	8.00	ft
Avg. Skew Angle, θ =	0.00	deg
Slab Thickness, t_s =	8.00	in
Slab Overhang, OH =	3	ft
Rail Width, RW =	1	ft
Roadway Width, W =	44	ft
Number of Lanes, N_L =	3	

Deck Slab		
Conc wt =	0.145	k/ft^3
f'_c =	4.0	ksi
E_{slab} =	3644	ksi

Beam		
weight =	0.145	k/ft^3
f'_c =	8.5	ksi
E_{beam} =	5312	ksi
y_t =	30.49	in
A =	817.0	in^2
I =	299740	in^4

Longitudinal Stiffness Parameter: (4.6.2.2.1-1)

e_g (in) = 34.49 (dist. b/w cog of bm & deck)

n = 1.000

$K_g = n(I+Ae_g^2)$ = 1271611 in^4

For typical cross sections (a,e,i,j & k), Table 4.6.2.2.1-1

RESULTS:

	Final LLDF
Interior Shear LLDF, $gV_{interior}$	**0.814**
Interior Moment LLDF, $gM_{interior}$	**0.794**
Exterior Shear LLDF, $gV_{exterior}$	**0.814**
Exterior Moment LLDF, $gM_{exterior}$	**0.794**

The Final LLDF may be modified according to the following TxDOT policies:

- Exterior beams use the interior LLDF when OH ≤ S/2.
- When OH > S/2 the exterior beam LLDF is determined by the lever rule for a single lane with a multiple presence factor of 1.0.
- In no case shall the LLDF for the exterior beams be less than the LLDFs for the interior beams.
- When the Roadway width is less than 20ft, all beams are designed for one lane loaded only.
- In no case shall the LLDF be less than $m \cdot N_L \div N_b$

CALCULATIONS:

Shear LLDF Correction for Skew (Table 4.6.2.2.3c-1)

$$\text{Corr.} = 1.0 + 0.20\left(\frac{12.0Lt_s^{\,3}}{K_g}\right)^{0.3}\tan\theta$$

= 1.0 + 0.20 * [(12.0*50.4*8^3)/(1,271,611)]^0.3 * tan(0)

Corr. = 1.000

Check θ:	0° ≤ 0° ≤ 60°	OK
Check S:	3.5' ≤ 8.0' ≤ 16.0'	OK
Check L:	20' ≤ 50.4' ≤ 240'	OK
Check N_b:	6 ≥ 4	OK

Moment LLDF Correction for Skew (Table 4.6.2.2.2e-1)

Check θ: 0° < 30° Set θ = 0°

Corr. = 1 - c_1(tan θ)^1.5

= 1 - 0(tan 0)^1.5

Corr. = 1.000

where: $$c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$$

c_1 = 0.000 because Θ < 30°

INTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3a-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 1.000

mg = 0.750 * 1.000 = 0.750

Equation

$$g = 0.36 + \left(\frac{S}{25}\right)$$

g = 0.36 + (8 / 25) = 0.680

Modify for Skew:

skew correction = 1.000

g = 0.680 * 1.000 = 0.680

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 50.4' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int1} = 0.680

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 1.000

mg = 0.875 * 1.000 = 0.875

Equation

$$g = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0}$$

g = 0.2 + (8 / 12) - (8 / 35)^2.0 = 0.814

Modify for Skew:

skew correction = 1.000

g = 0.814 * 1.000 = 0.814

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int2+} = 0.814

TxDOT Policy states $gV_{interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gV_{interior}$ is the Maximum of: gV_{int1} and $m \cdot N_L \div N_b$.

>> TxDOT Policy states that if W ≥ 20ft, $gV_{interior}$ is the Maximum of: gV_{int1}, gV_{int2+}, $m \cdot N_L \div N_b$.

$gV_{interior}$ = 0.814

TXDOT BRIDGE DIVISION	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
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	Descrip:	ITBC Design Example 1, Span 1 & 3			File:	Ex1_Span1_distribution_factors.xl			Sheet: 3 of 8

INTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 1.000

mg = 0.750 * 1.000 = 0.750

Equation

$$g = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^{\,3}}\right)^{0.1}$$

g = 0.06 + (8/14)^0.4 * (8/50.4)^0.3 * (1,271,611/(12*50.4*8^3))^0.1 = 0.590

Modify for Skew:

skew correction = 1.000

g = 0.590 * 1.000 = 0.590

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 50.4' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Check K_g: 10,000 ≤ 1,271,611 ≤ 7,000,000 OK

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int1} = 0.590

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 1.000

mg = 0.875 * 1.000 = 0.875

Equation

$$g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^{\,3}}\right)^{0.1}$$

g = 0.075 + (8/9.5)^0.6 * (8/50.4)^0.2 * (1,271,611/(12*50.4*8^3))^0.1 = 0.794

Modify for Skew:

skew correction = 1.000

g = 0.794 * 1.000 = 0.794

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int2+} = 0.794

TxDOT Policy states $gM_{Interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gM_{Interior}$ is the Maximum of: gM_{int1} and $m \cdot N_L \div N_b$.

>> TxDOT Policy states that if W ≥ 20ft, $gM_{Interior}$ is the Maximum of: gM_{int1}, gM_{int2+}, $m \cdot N_L \div N_b$.

$gM_{Interior}$ = 0.794

EXTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.0 = 0.625 — TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 1.000

mg = 0.625 * 1.000 = 0.625

Use Lever Rule, as per AASHTO LRFD Table 4.6.2.2.3b-1.

gV_{ext1} = 0.625

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 1.000

mg = 0.625 * 1.000 = 0.625

Equation

d_e = dist. b/w CL web to curb

d_e = OH - Rail Width

d_e = 3ft - 1ft = 2.0 ft

$$e = 0.6 + \left(\frac{d_e}{10}\right)$$

e = 0.6 + (2.0/10) = 0.800

$g = e^{*}gV_{int2+Eq}$

g = 0.800 * 0.814 = 0.651

Skew Correction is included in gV(interior).

Range of Applicability (ROA) Checks — Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: -1.0' ≤ 2.0' ≤ 5.5' OK

Check N_b: 6 ≠ 3 OK

Use Equation from Table 4.6.2.2.3b-1 because all criteria is OK.

gV_{ext2+} = 0.651

TxDOT Policy states $gV_{Exterior}$ must be ≥ $gV_{interior}$

$gV_{interior}$ = 0.814

TxDOT Policy states $gV_{Exterior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

>> TxDOT Policy states that if OH ≤ S/2, $gV_{Exterior}$ is $gV_{interior}$.

TxDOT Policy states that if OH > S/2 and W < 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, $gV_{interior}$, and $m \cdot N_L \div N_b$.

TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, gV_{ext2+}, $gV_{interior}$, and $m \cdot N_L \div N_b$.

$gV_{exterior}$ = 0.814

EXTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2d-1):

One Lane Loaded

Lever Rule

mg = 0.625 * 1.0 = 0.625 — TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 1.000

mg = 0.625 * 1.000 = 0.625

Use Lever Rule as per AASHTO LRFD Table 4.6.2.2.2d-1.

gM_{ext1} = 0.625

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 1.000

mg = 0.625 * 1.000 = 0.625

Equation

$$e = 0.77 + \left(\frac{d_e}{9.1}\right)$$

e = 0.77 + (2.0/9.1) = 0.990

$g = e^{*}gM_{int2+Eq}$

g = 0.99 * 0.794 = 0.786

Skew Correction included in gM(interior).

Range of Applicability (ROA) Checks — Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: $-1.0' \le 2.0' \le 5.5'$ OK

Check N_b: $6 \neq 3$ OK

Use Equation from Table 4.6.2.2.2d-1 because all criteria is OK.

gM_{ext2+} = 0.786

TxDOT Policy states $gM_{Exterior}$ must be $\ge gM_{Interior}$

$gM_{interior}$ = 0.794

TxDOT Policy states $gM_{Exterior}$ must be $\ge m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

\>> TxDOT Policy states that if OH ≤ S/2, $gM_{Exterior}$ is $gM_{Interior}$.

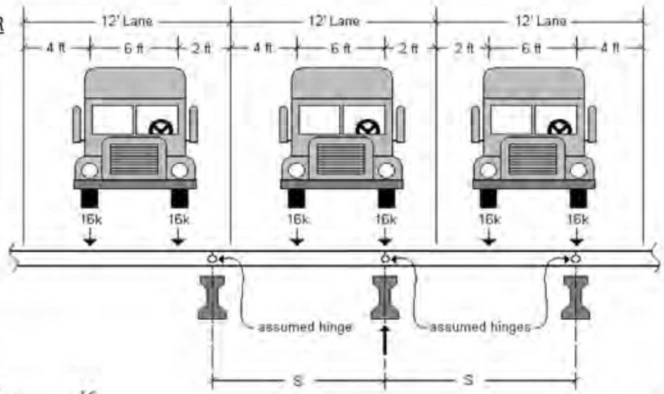
TxDOT Policy states that if OH > S/2 and W < 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, $gM_{interior}$, and $m \cdot N_L \div N_b$.

TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, gM_{ext2+}, $gM_{interior}$, and $m \cdot N_L \div N_b$.

$gM_{exterior}$ = 0.794

LEVER RULE S = 8.0 ft

INTERIOR



For S < 4:

One Lane = $\frac{16}{32}$ = 0.500

For 4 ≤ S < 6:

One Lane = $\frac{16}{32}$ = 0.500

Two Lanes = $\frac{16}{32}\left(1+\frac{S-4}{S}\right)$ = 0.750

>: **For 6 ≤ S < 10:**

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}\right)$ = 0.875

For 10 ≤ S < 12:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

For 12 ≤ S < 16:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

For 16 ≤ S < 18:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-16}{S}\right)$ = 0.000

LEVER RULE S = 8.0 ft

INTERIOR (con't)

For 18 ≤ S < 22:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}\right)$ = -0.625

For 22 ≤ S ≤ 24:

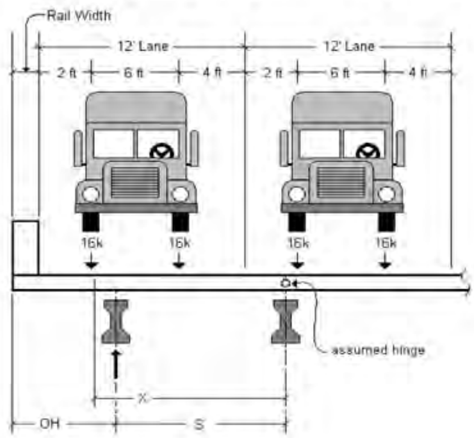
One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}+\frac{S-22}{S}\right)$ = -1.500

EXTERIOR



S = 8.0 ft
OH = 3.0 ft
Rail Width = RW = 1.0 ft
X = S+OH-RW-2ft = 8.0 ft

For X < 6:

One Lane = $\frac{16}{32}\left(\frac{X}{S}\right)$ = 0.500

>: **For 6 ≤ X < 12:**

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

For 12 ≤ X < 18:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}\right)$ = 0.375

LEVER RULE

EXTERIOR (con't) S = 8.0 ft OH = 3.0 ft
RW = 1.0 ft X = S+OH-RW-2ft = 8.0 ft

For 18 ≤ X < 24:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

For 24 ≤ X < 30:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}\right)$ = -1.250

For 30 ≤ X < 36:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

For 36 ≤ X < 42:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}\right)$ = -4.375

For 42 ≤ X ≤ 48:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}+\frac{X-42}{S}\right)$ = -6.500

INTERIOR			EXTERIOR		
One Lane Loaded	=	0.625	One Lane Loaded	=	0.625
Two Lanes Loaded	=	0.875	Two Lanes Loaded	=	0.625
Three Lanes Loaded	=	0.875	Three Lanes Loaded	=	0.625
Four Lanes Loaded	=	0.875	Four Lanes Loaded	=	0.625

4.2.15.4.2 *Span 2*

TXDOT BRIDGE DIVISION	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/14/20	2017 LRFD Specs
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	Descrip:	ITBC Design Example 1, Span 2			File:	Ex1_Span2_distribution_factors.xl		Sheet:	1 of 8

LRFD Live Load Distribution Factors*

Live Load Distribution Factors are calculated according to AASHTO LRFD Bridge Design Specifications, 8th Edition (2017 with no interim revisions) as prescribed by TxDOT policies (LRFD Design Manual July 2018) and practices. The Lever Rule is used when outside the Range of Applicability. The Range of Applicability for the Skew Correction Factors is ignored.

INPUT:

Beam Type =	Tx54	
No. Beams, N_b =	6	
CL_{brg} to CL_{brg}, L =	106.75	ft
Beam Spacing, S =	8.00	ft
Avg. Skew Angle, θ =	0.00	deg
Slab Thickness, t_s =	8.00	in
Slab Overhang, OH =	3	ft
Rail Width, RW =	1	ft
Roadway Width, W =	44	ft
Number of Lanes, N_L =	3	

Deck Slab			Beam		
Conc wt =	0.145	k/ft^3	weight =	0.145	k/ft^3
f'_c =	4.0	ksi	f'_c =	8.5	ksi
E_{slab} =	3644	ksi	E_{beam} =	5312	ksi
			y_t =	30.49	in
			A =	817.0	in^2
			I =	299740	in^4

Longitudinal Stiffness Parameter: (4.6.2.2.1-1)

e_g (in) = 34.49 (dist. b/w cog of bm & deck)

n = 1.000

$K_g = n(I + Ae_g^2)$ = 1271611 in^4

**For typical cross sections (a,e,i,j & k). Table 4.6.2.2.1-1*

RESULTS:

	Final LLDF
Interior Shear LLDF, $gV_{interior}$	**0.814**
Interior Moment LLDF, $gM_{interior}$	**0.649**
Exterior Shear LLDF, $gV_{exterior}$	**0.814**
Exterior Moment LLDF, $gM_{exterior}$	**0.649**

The Final LLDF may be modified according to the following TxDOT policies:

- ♦ Exterior beams use the interior LLDF when OH ≤ S/2.
- ♦ When OH > S/2 the exterior beam LLDF is determined by the lever rule for a single lane with a multiple presence factor of 1.0.
- ♦ In no case shall the LLDF for the exterior beams be less than the LLDFs for the interior beams.
- ♦ When the Roadway width is less than 20ft, all beams are designed for one lane loaded only.
- ♦ In no case shall the LLDF be less than $m \cdot N_L \div N_b$.

CALCULATIONS:

Shear LLDF Correction for Skew (Table 4.6.2.2.3c-1)

$$\text{Corr.} = 1.0 + 0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}\tan\theta$$

= 1.0 + 0.20 * [(12.0*106.8*8^3)/(1,271,611)]^0.3 * tan(0)

Corr. = 1.000

Check θ:	0° ≤ 0° ≤ 60°	OK
Check S:	3.5' ≤ 8.0' ≤ 16.0'	OK
Check L:	20' ≤ 106.8' ≤ 240'	OK
Check N_b:	6 ≥ 4	OK

Moment LLDF Correction for Skew (Table 4.6.2.2.2e-1)

Check θ: 0° < 30° Set θ = 0°

Corr. = 1 - c_1(tan θ)^1.5

= 1 - 0(tan 0)^1.5

Corr. = 1.000

where: $$c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$$

c_1 = 0.000 because Θ < 30°

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/14/20	2017 LRFD Specs
BRIDGE	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:		Rev. 10/18 - (No Interim)
DIVISION	Descrip:	ITBC Design Example 1, Span 2			File:	Ex1_Span2_distribution_factors.xl			Sheet: 2 of 8

INTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3a-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 1.000

mg = 0.750 * 1.000 = 0.750

Equation

$$g = 0.36 + \left(\frac{S}{25}\right)$$

g = 0.36 + (8 / 25) = 0.680

Modify for Skew:

skew correction = 1.000

g = 0.680 * 1.000 = 0.680

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 106.8' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int1} = 0.680

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 1.000

mg = 0.875 * 1.000 = 0.875

Equation

$$g = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0}$$

g = 0.2 + (8 / 12) - (8 / 35)^2.0 = 0.814

Modify for Skew:

skew correction = 1.000

g = 0.814 * 1.000 = 0.814

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int2+} = 0.814

TxDOT Policy states $gV_{interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gV_{interior}$ is the Maximum of: gV_{int1} and $m \cdot N_L \div N_b$.

>> TxDOT Policy states that if W ≥ 20ft, $gV_{interior}$ is the Maximum of: gV_{int1}, gV_{int2+}, $m \cdot N_L \div N_b$.

$gV_{interior}$ = 0.814

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/14/20	2017 LRFD Specs
BRIDGE	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:		Rev. 10/18 - (No Interim)
DIVISION	Descrip:	ITBC Design Example 1, Span 2			File:	Ex1_Span2_distribution_factors.xl			Sheet: 3 of 8

INTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 1.000

mg = 0.750 * 1.000 = 0.750

Equation

$$g = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^{\,3}}\right)^{0.1}$$

g = 0.06 + (8/14)^0.4 * (8/106.8)^0.3 * (1,271,611/(12*106.8*8^3))^0.1 = 0.453

Modify for Skew:

skew correction = 1.000

g = 0.453 * 1.000 = 0.453

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 106.8' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Check K_g: 10,000 ≤ 1,271,611 ≤ 7,000,000 OK

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int1} = 0.453

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 1.000

mg = 0.875 * 1.000 = 0.875

Equation

$$g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^{\,3}}\right)^{0.1}$$

g = 0.075 + (8/9.5)^0.6 * (8/106.8)^0.2 * (1,271,611/(12*106.8*8^3))^0.1 = 0.649

Modify for Skew:

skew correction = 1.000

g = 0.649 * 1.000 = 0.649

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int2+} = 0.649

TxDOT Policy states $gM_{interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gM_{interior}$ is the Maximum of: gM_{int1} and $m \cdot N_L \div N_b$.

>> TxDOT Policy states that if W ≥ 20ft, $gM_{interior}$ is the Maximum of: gM_{int1}, gM_{int2+}, $m \cdot N_L \div N_b$.

$gM_{interior}$ = 0.649

EXTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.0 = 0.625 — TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 1.000

mg = 0.625 * 1.000 = 0.625

Use Lever Rule, as per AASHTO LRFD Table 4.6.2.2.3b-1.

gV_{ext1} = 0.625

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 1.000

mg = 0.625 * 1.000 = 0.625

Equation

d_e = dist. b/w CL web to curb

d_e = OH - Rail Width

d_e = 3ft - 1ft = 2.0 ft

$$e = 0.6 + \left(\frac{d_e}{10}\right)$$

e = 0.6 + (2.0/10) = 0.800

$g = e^*gV_{int2+Eq}$

g = 0.800 * 0.814 = 0.651

Skew Correction is included in gV(interior).

Range of Applicability (ROA) Checks — Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: -1.0' ≤ 2.0' ≤ 5.5' OK

Check N_b: 6 ≠ 3 OK

Use Equation from Table 4.6.2.2.3b-1 because all criteria is OK.

gV_{ext2+} = 0.651

TxDOT Policy states $gV_{Exterior}$ must be ≥ $gV_{interior}$

$gV_{interior}$ = 0.814

TxDOT Policy states $gV_{Exterior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

>> TxDOT Policy states that if OH ≤ S/2, $gV_{Exterior}$ is $gV_{interior}$.

TxDOT Policy states that if OH > S/2 and W < 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, $gV_{interior}$, and $m \cdot N_L \div N_b$.

TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, gV_{ext2+}, $gV_{interior}$, and $m \cdot N_L \div N_b$.

$gV_{exterior}$ = 0.814

EXTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2d-1):

One Lane Loaded

Lever Rule

mg = 0.625 * 1.0 = 0.625 — TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 1.000

mg = 0.625 * 1.000 = 0.625

Use Lever Rule as per AASHTO LRFD Table 4.6.2.2.2d-1.

gM_{ext1} = 0.625

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 1.000

mg = 0.625 * 1.000 = 0.625

Equation

$$e = 0.77 + \left(\frac{d_e}{9.1}\right)$$

e = 0.77 + (2.0/9.1) = 0.990

$g = e^*gM_{int2+Eq}$

g = 0.99 * 0.649 = 0.643

Skew Correction included in gM(interior).

Range of Applicability (ROA) Checks — Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: $-1.0' \le 2.0' \le 5.5'$ OK

Check N_b: $6 \neq 3$ OK

Use Equation from Table 4.6.2.2.2d-1 because all criteria is OK.

gM_{ext2+} = 0.643

TxDOT Policy states $gM_{Exterior}$ must be $\ge gM_{Interior}$

$gM_{Interior}$ = 0.649

TxDOT Policy states $gM_{Exterior}$ must be $\ge m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

>> TxDOT Policy states that if OH ≤ S/2, $gM_{Exterior}$ is $gM_{Interior}$.

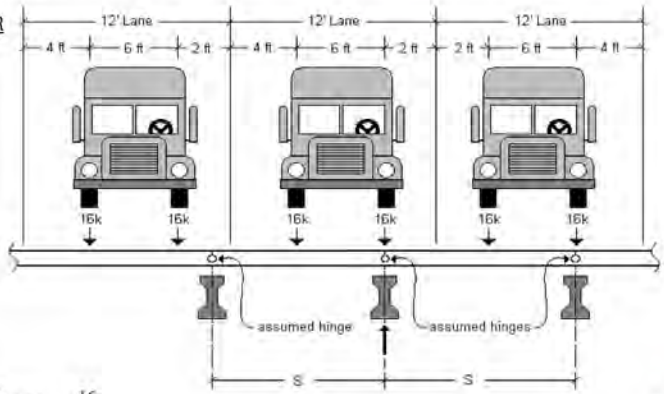
TxDOT Policy states that if OH > S/2 and W < 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, $gM_{Interior}$, and $m \cdot N_L \div N_b$.

TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, gM_{ext2+}, $gM_{Interior}$, and $m \cdot N_L \div N_b$.

$gM_{exterior}$ = 0.649

LEVER RULE S = 8.0 ft

INTERIOR



For S < 4:

One Lane = $\frac{16}{32}$ = 0.500

For 4 ≤ S < 6:

One Lane = $\frac{16}{32}$ = 0.500

Two Lanes = $\frac{16}{32}\left(1+\frac{S-4}{S}\right)$ = 0.750

>: **For 6 ≤ S < 10:**

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}\right)$ = 0.875

For 10 ≤ S < 12:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

For 12 ≤ S < 16:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

For 16 ≤ S < 18:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-16}{S}\right)$ = 0.000

LEVER RULE S = 8.0 ft

INTERIOR (con't)

For 18 ≤ S < 22:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}\right)$ = -0.625

For 22 ≤ S ≤ 24:

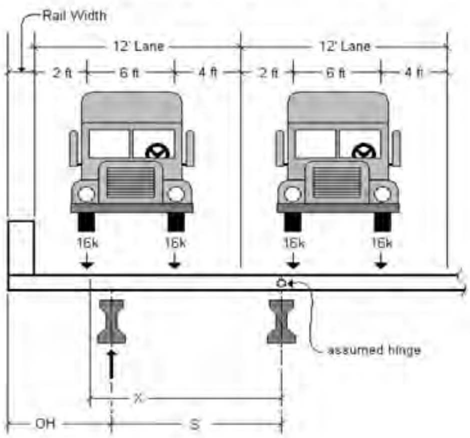
One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}+\frac{S-22}{S}\right)$ = -1.500

EXTERIOR



S = 8.0 ft
OH = 3.0 ft
Rail Width = RW = 1.0 ft
X = S+OH-RW-2ft = 8.0 ft

For X < 6:

One Lane = $\frac{16}{32}\left(\frac{X}{S}\right)$ = 0.500

>: **For 6 ≤ X < 12:**

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

For 12 ≤ X < 18:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}\right)$ = 0.375

LEVER RULE

EXTERIOR (con't) S = 8.0 ft OH = 3.0 ft
RW = 1.0 ft X = S+OH-RW-2ft = 8.0 ft

For 18 ≤ X < 24:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

For 24 ≤ X < 30:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}\right)$ = -1.250

For 30 ≤ X < 36:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

For 36 ≤ X < 42:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}\right)$ = -4.375

For 42 ≤ X ≤ 48:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}+\frac{X-42}{S}\right)$ = -6.500

INTERIOR			EXTERIOR		
One Lane Loaded	=	0.625	One Lane Loaded	=	0.625
Two Lanes Loaded	=	0.875	Two Lanes Loaded	=	0.625
Three Lanes Loaded	=	0.875	Three Lanes Loaded	=	0.625
Four Lanes Loaded	=	0.875	Four Lanes Loaded	=	0.625

4.2.15.5 Concrete Section Shear Capacity Spreadsheet

Texas Department of Transportation

County:	ANY	Descrip:	ITBC Design Example 1 - Bent 2
Highway:	ANY		
C-S-J:	XXXXXX	Design:	BRG
		Ck Dsn:	BRG
Bridge Division	Rev: 09/26/08	Date:	Aug-20

CONCRETE SECTION SHEAR CAPACITY BY AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, FOURTH EDITION, 2007

Resistance Factors:

ϕ_V =	0.9
ϕ_M =	0.9
ϕ_N =	0.75

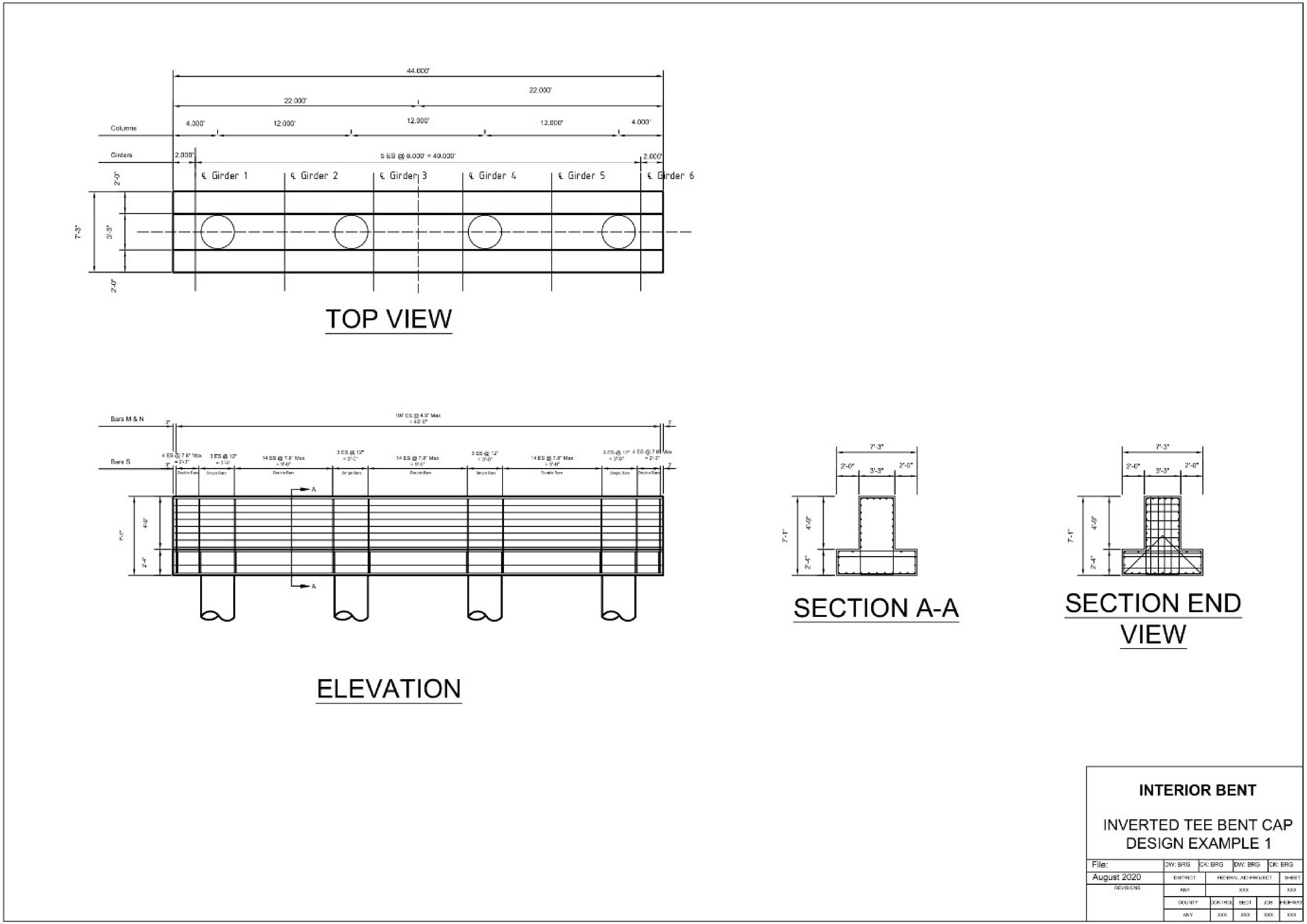
Units: US

Concrete:			**Mild Steel:**			**Prestressed Steel:**		
f'c =	5	ksi	fy =	60	ksi	fpu =	270	ksi
Ec =	4070	ksi	Es =	29000	ksi	Ep =	28500	ksi

	Units	SECTIONS 8	12	32	36	56	60	80	84
Input Data									
Bending moment, **Mu**	kip-ft	428.5	619.5	506.9	334.5	334.5	506.9	619.5	429
Shear force, **Vu**	kip	234.3	239.1	128.6	447.4	230.2	246.5	133.3	419.4
Axial force, **Nu (+ if tensile)**	kip	0	0	0	0	0	0	0	0
Web width, **bv**	in	39.00	39.00	39.00	39.00	39.00	39.00	39.00	39.00
Shear depth, **dv**	in	80.79	80.79	80.79	80.79	80.79	80.79	80.79	80.79
Mild steel reinf. area, **As**	in^2	9.36	9.36	9.36	9.36	9.36	9.36	9.36	9.36
Conc area on tension side, **Ac**	in^2	1657.5	1657.5	1657.5	1657.5	1657.5	1657.5	1657.5	1657.5
Area of stirrups, **Av**	in^2	1.76	1.76	1.76	1.76	1.76	1.76	1.76	1.76
Stirrup spacing, **s**	in	7.8	7.8	7.8	7.8	7.8	7.8	7.8	7.8
Prestressed steel area, **Aps**	in^2	0	0	0	0	0	0	0	0
Prestress shear, **Vp**	kip	0	0	0	0	0	0	0	0
Average prestress, **fps**	ksi	0	0	0	0	0	0	0	0
Torsional moment, **Tu**	kip-ft	660	330	330	660	660	330	330	660
Shear flow area, **Ao**	in^2	2971.6	2971.6	2971.6	2971.6	2971.6	2971.6	2971.6	2971.6
Area of one leg of stirrup, **At**	in^2	0.44	0.44	0.44	0.44	0.44	0.44	0.44	0.44
Perimeter of stirrup, **Ph**	in	324	324	324	324	324	324	324	324
Calculated Values									
Vc	kip	529.9	527.6	594.4	496.5	532.1	525.4	590.0	496.5
Vs	kip	1517.9	1567.9	1865.6	1363.9	1526.6	1555.7	1842.3	1363.9
ϕVn	kip	**1843**	**1886**	**2214**	**1674**	**1853**	**1873**	**2189**	**1674**
ε_x		7.55E-04	7.68E-04	4.45E-04	1.00E-03	7.43E-04	7.89E-04	4.59E-04	1.00E-03
θ	deg	33.74	33.90	29.60	36.40	33.60	34.10	29.90	36.40
β		2.380	2.370	2.670	2.230	2.390	2.360	2.650	2.230
Req'd Shear reinf. **Av/S**	in^2/in	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Req'd Torsion reinf. **At/S**	in^2/in	0.016	0.008	0.007	0.018	0.016	0.008	0.007	0.018
Maximum stirrup spacing, **Smax**	in	24.0	24.0	24.0	24.0	24.0	24.0	24.0	24.0
Conclusion									
Shear Reinforcing		OK	OK	OK	OK	OK	OK	OK	OK
Longitudinal Reinforcing		OK	OK	OK	OK	OK	OK	OK	OK

Note: Longitudinal Reinforcing check can be ignored for typical multi-column bent caps. For straddle bents with no overhangs, this check must be considered. Refer to LRFD 5.8.3.5 for further information.
If torsion is not being considered, leave last five rows of input data blank.

4.2.15.6 Bent Cap Details



4.3 INVERTED-T BENT CAP DESIGN EXAMPLE 2 (30° SKEW ANGLE)

Design example is in accordance with the AASHTO LRFD Bridge Design Specifications, 8th Ed. (2017) as prescribed by TxDOT Bridge Manual - LRFD (January 2020).

4.3.1 Design Parameters

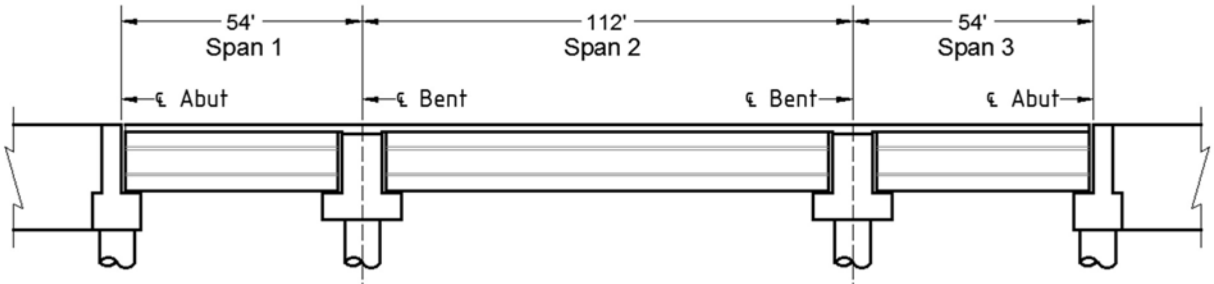


Figure 4.28 Spans of the Bridge with 30 Degree Skewed ITBC

Span 1

54’ Type TX54 Girders (0.851 k/ft)

6 Girders Spaced @ 9.24’ along the axis of bent with 3’ overhangs

2” Haunch

Span 2

112’ Type TX54 Girders (0.851 k/ft)

6 Girders Spaced @ 9.24’ along the axis of bent with 3’ overhangs

3.75” Haunch

Span 3

54’ Type TX54 Girders (0.851 k/ft)

6 Girders Spaced @ 9.24’ along the axis of bent with 3’ overhangs

2” Haunch

All Spans

Deck is 46 ft wide

Type T551 Rail (0.382 k/ft)

8” Thick Slab (0.100 ksf)

Assume 2” Overlay @ 140 pcf (0.023 ksf)

Use Class “C” Concrete

$f'_c = 5$ ksi

$w_c = 150$ pcf (for weight)

$w_c = 145$ pcf (for Modulus of Elasticity calculation)

“AASHTO LRFD” refers to the ASSHTO LRFD Bridge Design Specification, 8th Ed. (2017)..

“BDM-LRFD” refers to the TxDOT Bridge Design Manual - LRFD (January 2020).

“TxSP” refers to TxDOT guidance, recommendations, and standard practice.

"Furlong & Mirza" refers to "Strength and Serviceability of Inverted T-Beam Bent Caps Subject to Combined Flexure, Shear, and Torsion", Center for Highway Research Research Report No. 153-1F, The University of Texas at Austin, August 1974.

The basic bridge geometry can be found on the Bridge Layout located in the Appendices.

(TxSP)

(BDM-LRFD, Ch. 4, Sect. 5, Materials)

Grade 60 Reinforcing — *(BDM-LRFD, Ch. 4, Sect. 5, Materials)*

$f_y = 60$ ksi

Bents

Use 36" Diameter Columns (Typical for Type TX54 Girders)

Define Variables

Back Span	**Forward Span**	
$Span1 = 54ft$	$Span2 = 112ft$	*Span Length*
$GdrSpa1 = 8ft$	$GdrSpa2 = 8ft$	*Girder Spacing (Normalized values)*
$GdrNo1 = 6$	$GdrNo2 = 6$	*Number of Girders in Span*
$GdrWt1 = 0.851klf$	$GdrWt2 = 0.851klf$	*Weight of Girder*
$Haunch1 = 2in$	$Haunch2 = 3.75in$	*Size of Haunch*

Bridge

$Skew = 30deg$ — *Skew of Bents*

$BridgeW = 46ft$ — *Width of Bridge Deck*

$RdwyW = 44ft$ — *Width of Roadway*

$GirderD = 54in$ — *Depth of Type TX54 Girder*

$BrgSeat = 1.5in$ — *Bearing Seat Buildup*

$BrgPad = 2.75in$ — *Bearing Pad Thickness*

$SlabThk = 8in$ — *Thickness of Bridge Slab*

$OverlayThk = 2in$ — *Thickness of Overlay*

$RailWt = 0.372klf$ — *Weight of Rail*

$w_c = 0.150kcf$ — *Unit Weight of Concrete for Loads*

$w_{Olay} = 0.140kcf$ — *Unit Weigh of Overlay*

Bents

$f_c = 5ksi$ — *Concrete Strength*

$w_{cE} = 0.145kcf$ — *Unit Weight of Concrete for E_c*

$E_c = 33000 \cdot w_{cE}^{1.5} \cdot \sqrt{f_c}$ $\quad E_c = 4074$ ksi — *Modulus of Elasticity of Concrete (AASHTO LRFD Eq. C5.4.2.4-2)*

$f_y = 60ksi$ — *Yield Strength of Reinforcement*

$E_s = 29000ksi$ — *Modulus of Elasticity of Steel*

$D_{column} = 36in$ — *Diameter of Columns*

Dynamic Load Allowance
(AASHTO LRFD Table 3.6.2.1-1)

Other Variables

$\text{IM} = 33\%$

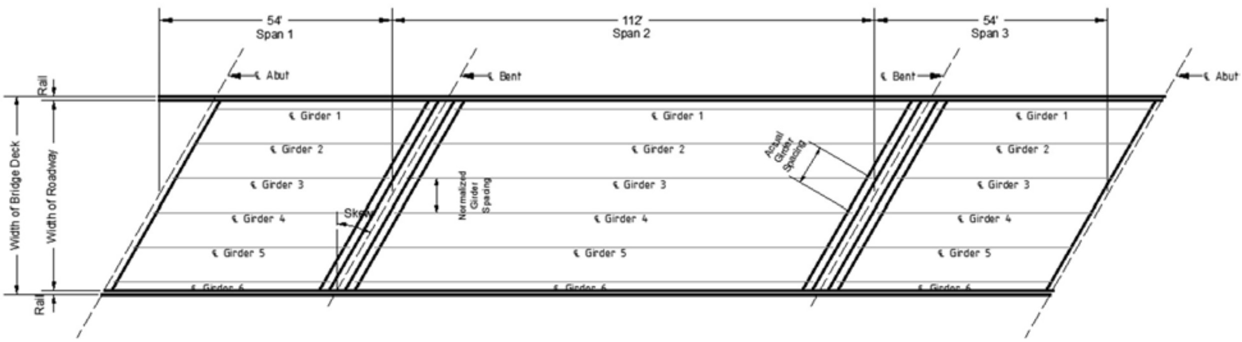


Figure 4.29 Top View of the 30 Degrees Skewed ITBC with Spans and Girders

4.3.2 Determine Cap Dimensions

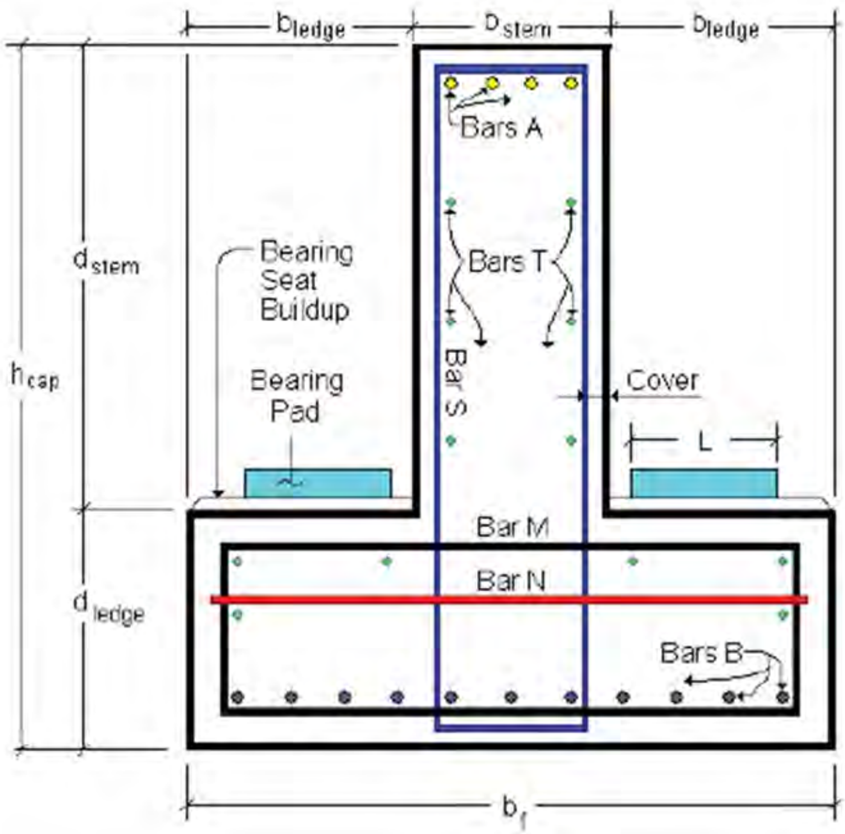


Figure 4.30 Section View of 30 Degrees Skewed ITBC

4.3.2.1 Stem Width

The stem is typically at least 3" wider than the Diameter of the Column (36") to allow for the extension of the column reinforcement into the Cap. (TxSP)

$b_{stem} = D_{column} + 3in$ $\qquad$ $b_{stem} = 39\ in$

4.3.2.2 Stem Height

Haunch2 is the larger of the two haunches.

Distance from Top of Slab to Top of Ledge:

$$D_{Slab\_to\_Ledge} = SlabThk + Haunch2 + GirderD + BrgPad + BrgSeat$$

$D_{Slab\_to\_Ledge} = 70.00\ in$

StemHaunch $= 3.75\ in$

The top of the stem must be 2.5" below the bottom of the slab. (BDM-LRFD, Ch. 4, Sect. 5, Geometric Constraints)

Accounting for the 1/2" of bituminous fiber, the top of the stem must have at least 2" of haunch on it, but the haunch should not be less than either of the haunches of the adjacent spans.

$d_{stem} = D_{Slab\_to\_Ledge} - SlabThk - StemHaunch - 0.5in$

The stem must accommodate ½" of bituminous fiber.

$d_{stem} = 57.75\ in$

Use: $d_{stem} = 57\ in$

Round the Stem Height down to the nearest 1". (TxSP)

4.3.2.3 Ledge Width

The Ledge Width must be adequate for Bar M to develop fully.

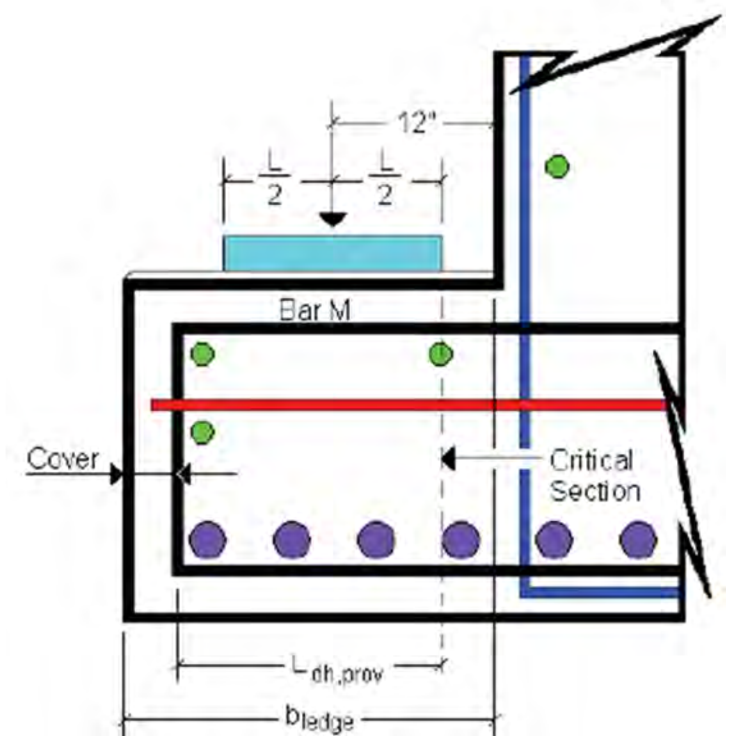


Figure 4.31 Ledge Section of 30 Degrees ITBC

"$L_{dh,prov}$" must be greater than or equal to "$L_{dh,req}$" for Bar M.

$cover = 2.5\ in$

"cover" is measured from the center of the transverse bars.

$L = 8\ in$

"L" is the length of the Bearing Pad along the girder. A typical type TX54 bearing pad is 8" × 21" *as shown in the IGEB standard.*

Determine the Required Development Length of Bar M:

Try # 6 Bar for Bar M.

$d_{bar\_M} = 0.750\ in$

$A_{bar\_M} = 0.44\ in^2$

Basic Development Length

$L_{dh} = \frac{38.0 \cdot d_{bar\_M}}{60} \cdot \left(\frac{f_y}{\sqrt{f_c}}\right)$ $L_{dh} = 12.75\ in$ *(AASHTO LRFD Eq. 5.10.8.2.4a-2)*

Modification Factors for L_{dh}: *(AASHTO LRFD 5.10.8.2.4b)*

Is Top Cover greater than or equal to 2.5", and Side Cover greater than or equal to 2"?

$$\text{SideCover} = \text{cover} - \frac{d_{bar\_M}}{2} = 2.13 \text{ in}$$

$$\text{TopCover} = \text{cover} - \frac{d_{bar_M}}{2} = 2.13 \text{ in}$$

"Side Cover" and "Top Cover" are the clear cover on the side and top of the hook respectively. The dimension "cover" is measured from the center of Bar M.

No. Reinforcement Confinement Factor, $\lambda_{rc} = 1.0$

Coating Factor, $\lambda_{cw} = 1.0$

Excess Reinforcement Factor, $\lambda_{er} = 1.0$

Concrete Density Modification Factor, $\lambda = 1.0$ *(AASHTO LRFD 5.4.2.8)*

The Required Development Length: *(AASHTO LRFD 5.10.8.2.4a)*

$$L_{dh\_req} = \max\left(L_{dh} \cdot \left(\frac{\lambda_{rc} \cdot \lambda_{cw} \cdot \lambda_{er}}{\lambda}\right), 8 \cdot d_{bar\_M}, 6\text{in.}\right)$$

Therefore,

$$L_{dh\_req} = 12.75 \text{ in}$$

$$b_{ledge\_min} = L_{dh\_req} + \text{cover} + 12\text{in} - \frac{L}{2} \qquad b_{ledge\_min} = 23.25 \text{ in}$$

The distance from the face of the stem to the center of bearing is 12" for TxGirders (IGEB).

Use:

$$b_{ledge} = 24 \text{ in}$$

Width of Bottom Flange:

$$b_f = 2 \cdot b_{ledge} + b_{stem} \qquad b_f = 87 \text{ in}$$

4.3.2.4 Ledge Depth

As a general rule of thumb, Ledge Depth is greater than or equal to 2'-3". This is the depth at which a bent from a typical bridge will pass the punching shear check.

Use a Ledge Depth of 28".

$$d_{ledge} = 28 \text{ in}$$

Total Depth of Cap:

$$h_{cap} = d_{stem} + d_{ledge} \qquad h_{cap} = 85 \text{ in}$$

4.3.2.5 Summary of Cross Sectional Dimensions

$b_{stem} = 39 \text{ in}$

$d_{stem} = 57 \text{ in}$

$b_{ledge} = 24 \text{ in}$

$d_{ledge} = 28 \text{ in}$

$h_{cap} = 85 \text{ in}$

4.3.2.6 Length of Cap

First define Girder Spacing and End Distance:

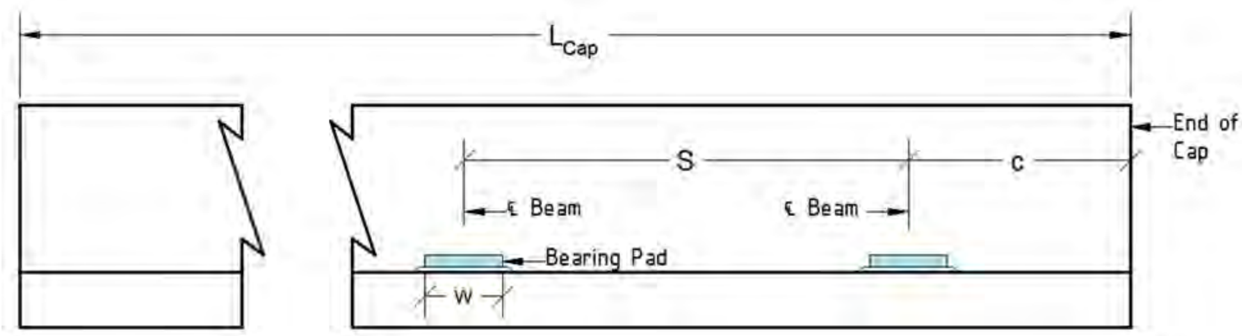


Figure 4.32 Elevation View of 30 Degrees Skewed ITBC

$S = 8\ \text{ft}$ *Girder Spacing*

$c = 2\ \text{ft}$ *"c" is the distance from the Center Line of the Exterior Girder to the Edge of the Cap measured along the Cap.*

$L_{Cap} = S \cdot (GdrNo1 - 1) + 2c$ $L_{Cap} = 44\ \text{ft}$ *Length of Cap*

TxDOT policy is as follows, "The edge distance between the exterior bearing pad and the end of the inverted T-beam shall not be less than 12in." (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria) replacing the statement in AASHTO LRFD 5.13.2.5.5 stating it shall not be less than d_f. Preferably, the stem should extend at least 3" beyond the edge of the bearing seat.

Bearing Pad Dimensions: *(IGEB standard)*

$L = 8\ \text{in}$ *Length of Bearing Pad*

$W = 21\ \text{in}$ *Width of Bearing Pad*

4.3.3 Cross Sectional Properties of Cap

$A_g = d_{ledge} \cdot b_f + d_{stem} \cdot b_{stem}$ $A_g = 4659\text{in}^2$

$ybar = \frac{d_{ledge} \cdot b_f \cdot \left(\frac{1}{2} d_{ledge}\right) + d_{stem} \cdot b_{stem} \cdot \left(d_{ledge} + \frac{1}{2} d_{stem}\right)}{A_g}$ $ybar = 34.3\ \text{in}$ *Distance from bottom of the cap to the center of gravity of the cap*

$$I_g = \frac{b_f \cdot d_{ledge}^3}{12} + b_f \cdot d_{ledge} \cdot \left(ybar - \frac{1}{2} d_{ledge}\right)^2 + \frac{b_{stem} \cdot d_{stem}^3}{12} + \cdots$$

$$b_{stem} \cdot d_{stem} \cdot \left[ybar - \left(d_{ledge} + \frac{1}{2} d_{stem}\right)\right]^2$$ $I_g = 2.86 \times 10^6\ \text{in}^4$

4.3.4 Cap Analysis

4.3.4.1 Cap Model

Assume:

4 Columns Spaced @ 12'-0"

The cap will be modeled as a continuous beam with simple supports using TxDOT's CAP18 program.

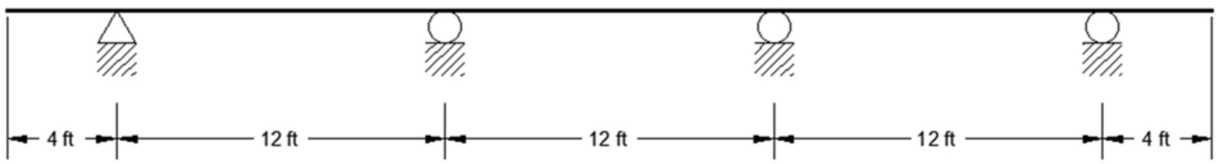


Figure 4.33 Continuous Beam Model for 30 Degrees Skewed ITBC

TxDOT does not consider frame action for typical multi-column bents (BDM-LRFD, Ch. 4, Sect. 5, Structural Analysis).

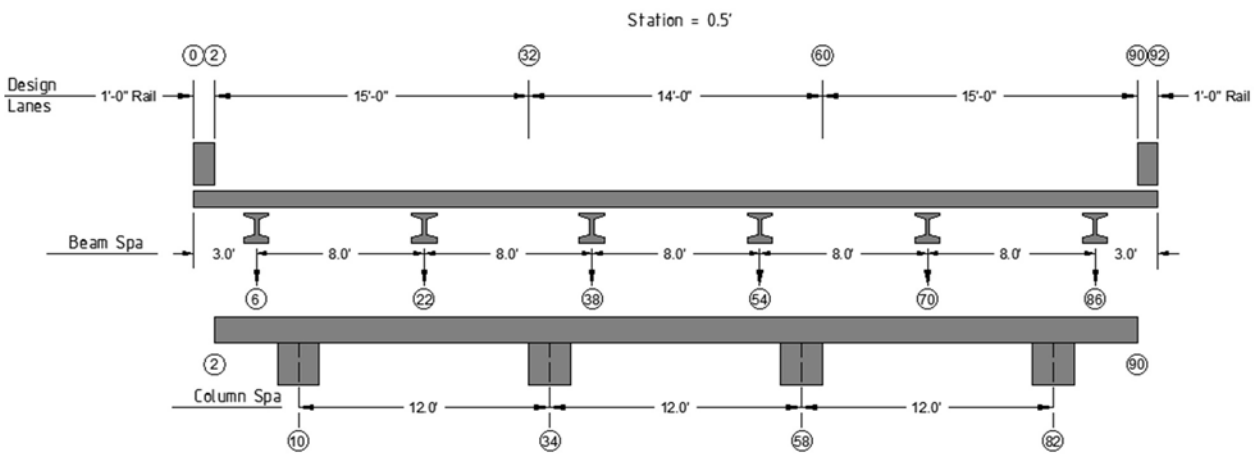


Figure 4.34 Cap 18 Model of 30 Degrees Skewed ITBC

The circled numbers in Figure 4.34 are the stations that will be used in the CAP 18 input file. One station is 0.5 ft in the direction perpendicular to the pgl, not parallel to the bent.

$$\text{station} = 0.5\ \text{ft}$$ *Station increment for CAP 18*

Recall:

$$E_c = 4074\ \text{ksi} \qquad I_g = 2.86 \times 10^6\ \text{in}^4$$

$$E_c I_g = 1.165 \times 10^{10}\ \text{kip} \cdot \text{in}^2 / \left(12\frac{\text{in}}{\text{ft}}\right)^2 \qquad E_c I_g = 8.09 \times 10^7 \text{kip} \cdot \text{ft}^2$$

4.3.4.1.1 *Dead Load*

Values used in the following equations can be found on "4.3.1 Design Parameters"

SPAN 1

$$\text{Rail1} = \frac{2\cdot \text{RailWt}\cdot \frac{\text{Span1}}{2}}{\min(\text{GdrNo1},6)} \qquad \text{Rail1} = 3.44\,\frac{\text{kip}}{\text{girder}}$$

Rail Weight is distributed evenly among stringers, up to 3 stringers per rail (TxSP).

$$\text{Slab1} = w_c \cdot \text{GdrSpa1} \cdot \text{SlabThk} \cdot \frac{\text{Span1}}{2} \cdot 1.10 \qquad \text{Slab1} = 23.76\,\frac{\text{kip}}{\text{girder}}$$

Increase slab DL by 10% to account for haunch and thickened slab ends.

$$\text{Girder1} = \text{GdrWt1} \cdot \frac{\text{Span1}}{2} \qquad \text{Girder1} = 22.98\,\frac{\text{kip}}{\text{girder}}$$

$$\text{DLRxn1} = (\text{Rail1} + \text{Slab1} + \text{Girder1}) \qquad \text{DLRxn1} = 50.17\,\frac{\text{kip}}{\text{girder}}$$

Overlay is calculated separetely, because it has different load factor than the rest of the dead loads.

$$\text{Overlay1} = w_{\text{Olay}} \cdot \text{GdrSpa1} \cdot \text{OverlayThk} \cdot \frac{\text{Span1}}{2} \qquad \text{Overlay1} = 5.04\,\frac{\text{kip}}{\text{girder}}$$

Design for future overlay.

SPAN 2

$$\text{Rail2} = \frac{2\cdot \text{RailWt}\cdot \frac{\text{Span2}}{2}}{\min(\text{GdrNo2},6)} \qquad \text{Rail2} = 7.13\,\frac{\text{kip}}{\text{girder}}$$

$$\text{Slab2} = w_c \cdot \text{GdrSpa2} \cdot \text{SlabThk} \cdot \frac{\text{Span2}}{2} \cdot 1.10 \qquad \text{Slab2} = 49.28\,\frac{\text{kip}}{\text{girder}}$$

$$\text{Girder2} = \text{GdrWt1} \cdot \frac{\text{Span2}}{2} \qquad \text{Girder2} = 47.66\,\frac{\text{kip}}{\text{girder}}$$

$$\text{DLRxn2} = (\text{Rail2} + \text{Slab2} + \text{Girder2}) \qquad \text{DLRxn2} = 104.07\,\frac{\text{kip}}{\text{girder}}$$

$$\text{Overlay2} = w_{\text{Olay}} \cdot \text{GdrSpa2} \cdot \text{OverlayThk} \cdot \frac{\text{Span2}}{2} \qquad \text{Overlay2} = 10.45\,\frac{\text{kip}}{\text{girder}}$$

CAP

$$\text{Cap} = w_c \cdot A_g = 4.853\,\frac{\text{kip}}{\text{ft}} \cdot \frac{0.5\text{ft}}{\text{station}} \qquad \text{Cap} = 2.427\,\frac{\text{kip}}{\text{station}}$$

4.3.4.1.2 *Live Load*

AASHTO LRFD 3.6.1.2.2 and 3.6.1.2.4)

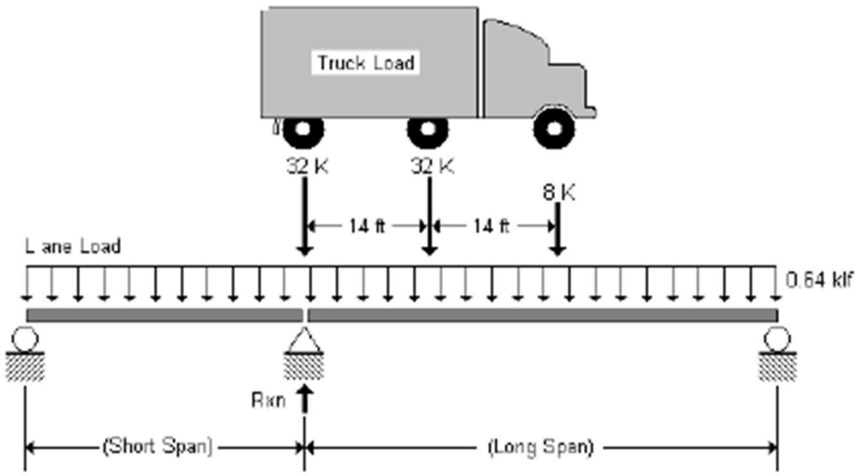


Figure 4.35 Live Load Model of 30 Degrees Skewed ITBC

$\text{LongSpan} = \max(\text{Span1}, \text{Span2})$ $\text{LongSpan} = 112\ \text{ft}$

$\text{ShortSpan} = \min(\text{Span1}, \text{Span2})$ $\text{ShortSpan} = 54\ \text{ft}$

$\text{IM} = 0.33$

$$\text{Lane} = 0.64\text{klf} \cdot \left(\frac{\text{LongSpan}+\text{ShortSpan}}{2}\right)$$

$$\text{Lane} = 53.12\,\frac{\text{kip}}{\text{lane}}$$

$$\text{Truck} = 32\text{kip} + 32\text{kip} \cdot \left(\frac{\text{LongSpan}-14\text{ft}}{\text{LongSpan}}\right) + 8\text{kip} \cdot \left(\frac{\text{LongSpan}-28\text{ft}}{\text{LongSpan}}\right)$$

$$\text{Truck} = 66.00\,\frac{\text{kip}}{\text{lane}}$$

Use HL-93 Live Load. For maximum reaction at interior bents, "Design Truck" will always govern over "Design Tandem". For the maximum reaction when the long span is more than twice as long as the short span, place the rear (32 kip) axle over the support and the middle (32 kip) and front (8 kip) axles on the long span. For the maximum reaction when the long span is less than twice as long as the short span, place the middle (32 kip) axle over the support, the front (8 kip) axle on the short span and the rear (32 kip) axle on the

$$\text{LLRxn} = \text{Lane} + \text{Truck} \cdot (1 + \text{IM})$$

$$\text{LLRxn} = 140.90\,\frac{\text{kip}}{\text{lane}}$$

Combine "Design Truck" and "Design Lane" loadings (AASHTO LRFD 3.6.1.3). Dynamic load allowance, IM, does not apply to "Design Lane." (AASHTO LRFD 3.6.1.2.4)

$$P = 16.0\text{kip} \cdot (1 + IM)$$

$$P = 21.28\ \text{kip}$$

$$w = \frac{LLRxn-(2 \cdot P)}{10\text{ft}}$$

$$w = 9.83\frac{\text{kip}}{\text{ft}} \cdot \frac{0.5\text{ft}}{\text{station}}$$

$$w = 4.92\frac{\text{kip}}{\text{station}}$$

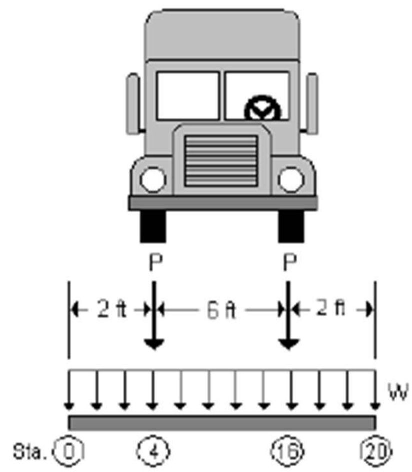


Figure 4.36 Live Load Model of 30 Degrees Skewed ITBC for CAP18

The Live Load is applied to the slab by two 16 kip wheel loads increased by the dynamic load allowance with the reminder of the live load distributed over a 10 ft (AASHTO LRFD 3.6.1.2.1) design lane width. (TxSP)

The Live Load applied to the slab is distributed to the beams assuming the slab is hinged at each beam except the outside beam. (BDM-LRFD, Ch. 4, Sect. 5, Structural Analysis)

4.3.4.1.3 *Cap 18 Data Input*

Input "Multiple Presence Factors" into CAP18 as "Load Reduction Factors".

Multiple Presence Factors, m (AASHTO LRFD Table 3.6.1.1.2-1)

No. of Lanes	Factor "m"
1	1.20
2	1.00
3	0.85
>3	0.65

Limit States (AASHTO LRFD 3.4.1)

The cap design need only consider Strength I, Service I, and Service I with DL (TxSP).

Strength I

Live Load and Dynamic Load Allowance LL+IM = 1.75

Dead Load Components DC = 1.25

Dead Load Wearing Surface (Overlay) DW = 1.50

TxDOT allows the Overlay Factor to be reduced to 1.25 (TxSP), since overlay is typically used in design only to increase the safety factor, but in this example we will use DW=1.50.

Service I

Live Load and Dynamic Load Allowance LL+IM = 1.00

Dead Load and Wearing Surface DC & DW = 1.00

Dead Load

TxDOT considers Service level Dead Load only with a limit reinforcement stress of 22 ksi to minimize cracking. (BDM-LRFD, Chapter 4, Section 5, Design Criteria)

4.3.4.1.4 *Cap 18 Output*

	Max +M	**Max -M**
Dead Load:	$M_{posDL} = 294.2 \text{ kip} \cdot \text{ft}$	$M_{negDL} = -443.9 \text{ kip} \cdot \text{ft}$
Service Load:	$M_{posServ} = 574.3 \text{ kip} \cdot \text{ft}$	$M_{negServ} = -688.2 \text{ kip} \cdot \text{ft}$
Factored Load:	$M_{posUlt} = 863.4 \text{ kip} \cdot \text{ft}$	$M_{negUlt} = -991.3 \text{ kip} \cdot \text{ft}$

4.3.4.2 Girder Reactions on Ledge

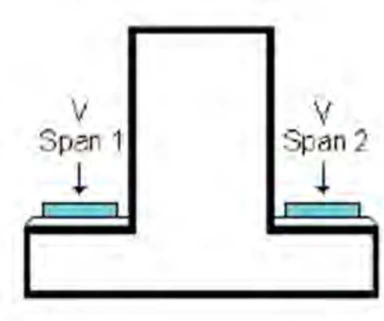


Figure 4.37 Girder Reactions on the Ledge of 30 Degrees Skewed ITBC

Dead Load

$\text{DLSpan1} = \text{Rail1} + \text{Slab1} + \text{Girder1}$ $\qquad$ $\text{DLSpan1} = 50.17 \frac{\text{kip}}{\text{girder}}$

$\text{Overlay1} = 5.04 \frac{\text{kip}}{\text{girder}}$

$\text{DLSpan2} = \text{Rail2} + \text{Slab2} + \text{Girder2}$ $\qquad$ $\text{DLSpan2} = 104.07 \frac{\text{kip}}{\text{girder}}$

$\text{Overlay2} = 10.45 \frac{\text{kip}}{\text{girder}}$

Live Load

Loads per Lane:

Use HL-93 Live Load. For maximum reaction at interior bents, "Design Truck" will always govern over "Design Tandem" for Spans greater than 26ft. For the maximum reaction, place the back (32 kips) axle over the support.

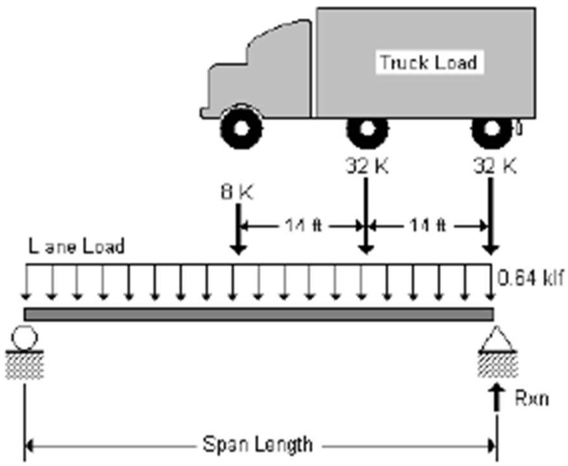


Figure 4.38 Live Load Model of 30 Degrees Skewed ITBC for Girder Reactions on Ledge

$$\mathbf{LaneSpan1 = 0.64klf \cdot \left(\frac{Span1}{2}\right)} \qquad \mathbf{LaneSpan1 = 17.28\,\frac{kip}{lane}}$$

$$\mathbf{LaneSpan2 = 0.64klf \cdot \left(\frac{Span2}{2}\right)} \qquad \mathbf{LaneSpan2 = 35.84\,\frac{kip}{lane}}$$

$$\mathbf{TruckSpan1 = 32kip + 32kip \cdot \left(\frac{Span1-14ft}{Span1}\right) + 8kip \cdot \left(\frac{Span1-28ft}{Span1}\right)}$$

$$\text{TruckSpan1} = 59.56\,\frac{\text{kip}}{\text{lane}}$$

$$\mathbf{TruckSpan2 = 32kip + 32kip \cdot \left(\frac{Span2-14ft}{Span2}\right) + 8kip \cdot \left(\frac{Span2-28ft}{Span2}\right)}$$

$$\text{TruckSpan2} = 66.00\,\frac{\text{kip}}{\text{lane}}$$

$$\text{IM} = 0.33$$

Combine "Design Truck" and "Design Lane" loadings (AASHTO LRFD 3.6.1.3).

$$\text{LLRxnSpan1} = \text{LaneSpan1} + \text{TruckSpan1} * (1 + \text{IM})$$

$$\text{LLRxnSpan1} = 96.49\,\frac{\text{kip}}{\text{lane}}$$

Dynamic load allowance, IM, does not apply to "Design Lane." (AASHTO LRFD 3.6.1.2.4).

$$\text{LLRxnSpan2} = \text{LaneSpan2} + \text{TruckSpan2} * (1 + \text{IM})$$

$$\text{LLRxnSpan2} = 123.62\,\frac{\text{kip}}{\text{girder}}$$

$$\text{gV}_{\text{Span1\_Int}} = 0.876$$

$$\text{gV}_{\text{Span1\_Ext}} = 0.876$$

$$\text{gV}_{\text{Span2\_Int}} = 0.891$$

$$\text{gV}_{\text{Span2\_Ext}} = 0.891$$

The Live Load Reactions are assumed to be the Shear Live Load Distribution Factor multiplied by the Live Load Reaction per Lane. The Shear Live Load Distribution Factor is calculated using the "LRFD Live Load Distribution Factors" Spreadsheet found in the Appendices.

The Exterior Girders must have a Live Load Distribution Factor equal to or greater than the Interior Girders. This is to

$\mathrm{LLSpan1Int} = gV_{\mathrm{Span1\_Int}} \cdot \mathrm{LLRxnSpan1}$ $\mathrm{LLSpan1Int} = 84.53 \frac{\mathrm{kip}}{\mathrm{girder}}$

$\mathrm{LLSpan1Ext} = gV_{\mathrm{Span1\_Ext}} \cdot \mathrm{LLRxnSpan1}$ $\mathrm{LLSpan1Ext} = 84.53 \frac{\mathrm{kip}}{\mathrm{girder}}$

$\mathrm{LLSpan2Int} = gV_{\mathrm{Span2\_Int}} \cdot \mathrm{LLRxnSpan2}$ $\mathrm{LLSpan2Int} = 110.15 \frac{\mathrm{kip}}{\mathrm{girder}}$

$\mathrm{LLSpan2Ext} = gV_{\mathrm{Span2\_Ext}} \cdot \mathrm{LLRxnSpan2}$ $\mathrm{LLSpan2Ext} = 110.15 \frac{\mathrm{kip}}{\mathrm{girder}}$

Span 1

Interior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{\mathrm{s\_Span1Int}} = \mathrm{DLSpan1} + \mathrm{Overlay1} + \mathrm{LLSpan1Int}$$

$$V_{\mathrm{s\_Span1Int}} = 140\ \mathrm{kip}$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{\mathrm{u\_Span1Int}} = 1.25 \cdot \mathrm{DLSpan1} + 1.5 \cdot \mathrm{Overlay1} + 1.75 \cdot \mathrm{LLSpan1Int}$$

$$V_{\mathrm{u\_Span1Int}} = 218\ \mathrm{kip}$$

Exterior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{\mathrm{s\_Span1Ext}} = \mathrm{DLSpan1} + \mathrm{Overlay1} + \mathrm{LLSpan1Ext}$$

$$V_{\mathrm{s\_Span1Ext}} = 140\ \mathrm{kip}$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{\mathrm{u\_Span1Ext}} = 1.25 \cdot \mathrm{DLSpan1} + 1.5 \cdot \mathrm{Overlay1} + 1.75 \cdot \mathrm{LLSpan1Ext}$$

$$V_{\mathrm{u\_Span1Ext}} = 218\ \mathrm{kip}$$

Span 2

Interior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{\mathrm{s\_Span2Int}} = \mathrm{DLSpan2} + \mathrm{Overlay2} + \mathrm{LLSpan2Int}$$

$$V_{\mathrm{s\_Span2Int}} = 225\ \mathrm{kip}$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{\mathrm{u\_Span2Int}} = 1.25 \cdot \mathrm{DLSpan2} + 1.5 \cdot \mathrm{Overlay2} + 1.75 \cdot \mathrm{LLSpan2Int}$$

$$V_{\mathrm{u\_Span2Int}} = 339\ \mathrm{kip}$$

Exterior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{s\_Span2Ext} = DLSpan2 + Overlay2 + LLSpan2Ext$$

$$V_{s\_Span2Ext} = 225\ kip$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{u\_Span2Ext} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot LLSpan2Ext$$

$$V_{u\_Span2Ext} = 339\ kip$$

4.3.4.3 Torsional Loads

To maximize the torsion, the live load only acts on the longer span.

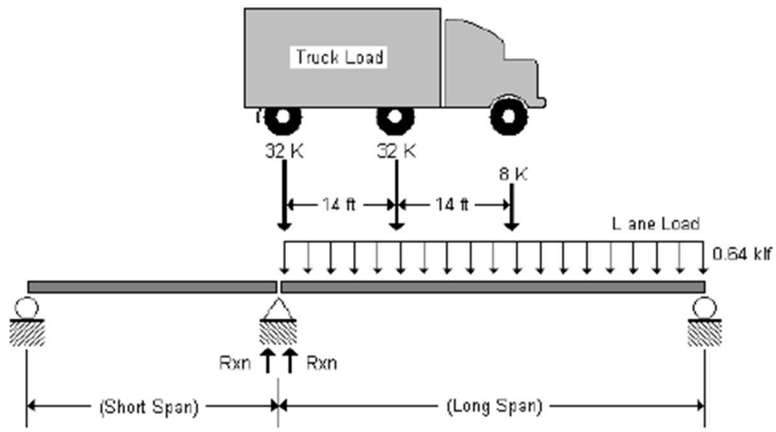


Figure 4.39 Live Load Model of 30 Degrees Skewed ITBC for Torsional Loads

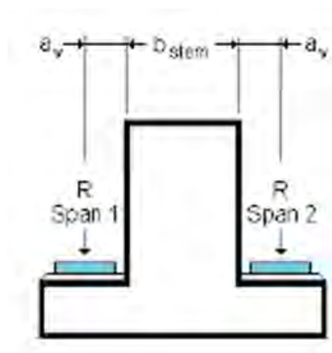


Figure 4.40 Loads on the Ledge of 30 Degrees Skewed ITBC for Torsion

$a_v = 12$ in

"a_v" is the value for the distance from the face of the stem to the center of bearing for the girders. 12" is the typical values for TxGirders on ITBC (IGEB). The lever arm is the distance from the center line of bearing to the centerline of the cap.

$b_{stem} = 39$ in

$LeverArm = a_v + \frac{1}{2} b_{stem}$ $\qquad LeverArm = 31.5$ in

Interior Girders

Girder Reactions

$$R_{u\_Span1} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1$$

$$R_{u\_Span1} = 70\ \text{kip}$$

$$R_{u\_Span2} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2\_Int} \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)]$$

$$R_{u\_Span2} = 339\ \text{kip}$$

Torsional Load

$$T_{u\_Int} = \left|R_{u\_Span1} - R_{u\_Span2}\right| \cdot LeverArm$$

$$T_{u\_Int} = 706\ \text{kip} \cdot \text{ft}$$

<u>Exterior Girders</u>

Girder Reactions

$$R_{u\_Span1} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1$$

$$R_{u\_Span1} = 70\ \text{kip}$$

$$R_{u\_Span2} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2\_Ext} \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)]$$

$$R_{u\_Span2} = 339\ \text{kip}$$

Torsional Load

$$T_{u\_Ext} = \left|R_{u\_Span1} - R_{u\_Span2}\right| \cdot LeverArm$$

$$T_{u\_Ext} = 706\ \text{kip} \cdot \text{ft}$$

Torsion on Cap

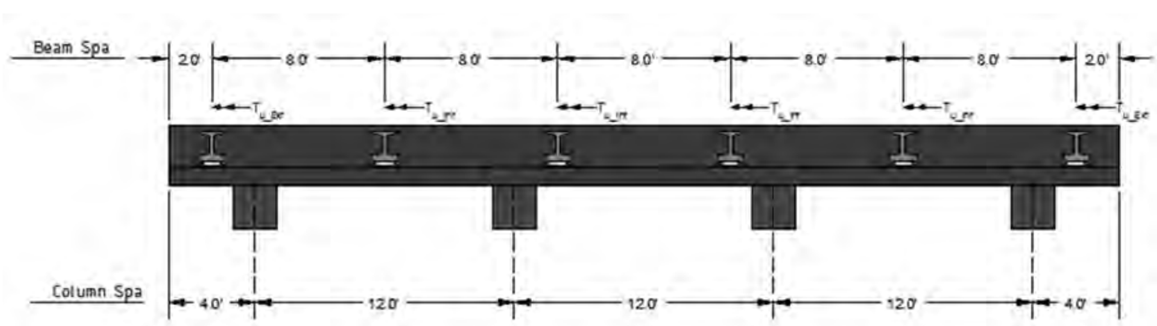


Figure 4.41 Elevation View of 30 Degrees Skewed ITBC with Torsion Loads

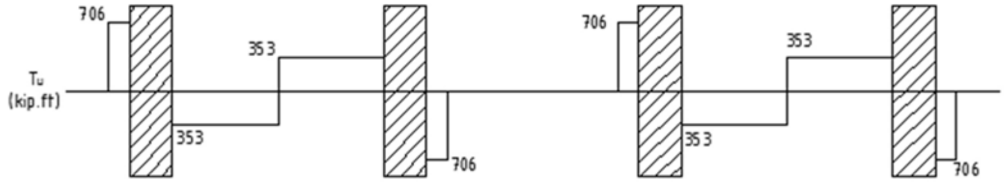


Figure 4.42 Torsion Diagram of 30 Degrees Skewed ITBC

Analyzed assuming Bents are torsionally rigid at Effective Face of Columns.

$T_u = 706\ \text{kip} \cdot \text{ft}$ *Maximum Torsion on Cap*

4.3.4.4 Load Summary

Ledge Loads

Interior Girder

Service Load

$V_{s\_Int} = \max(V_{s\_Span1Int}, V_{s\_Span2Int})$ $V_{s\_Int} = 224.67\ \text{kip}$

Factored Load

$V_{u\_Int} = \max(V_{u\_Span1Int}, V_{u\_Span2Int})$ $V_{u\_Int} = 338.53\ \text{kip}$

Exterior Girder

Service Load

$V_{s\_Ext} = \max(V_{s\_Span1Ext}, V_{s\_Span2Ext})$ $V_{s\_Ext} = 224.67\ \text{kip}$

Factored Load

$V_{u\_Ext} = \max(V_{u\_Span1Ext}, V_{u\_Span2Ext})$ $V_{u\_Ext} = 338.53\ \text{kip}$

Cap Loads

Positive Moment (From CAP18)

Dead Load: $M_{posDL} = 294.4\ \text{kip} \cdot \text{ft}$

Service Load: $M_{posServ} = 574.3\ \text{kip} \cdot \text{ft}$

Factored Load: $M_{posUlt} = 863.4\ \text{kip} \cdot \text{ft}$

Negative Moment (From CAP18)

Dead Load: $M_{negDL} = -443.9\ \text{kip} \cdot \text{ft}$

Service Load: $M_{negServ} = -688.2\ \text{kip} \cdot \text{ft}$

Factored Load: $M_{negUlt} = -991.3 \text{ kip} \cdot \text{ft}$

Maximum Torsion and Concurrent Shear and Moment (Strength I)

$T_u = 706 \text{ kip} \cdot \text{ft}$

$V_u = 452.1 \text{ kip}$

$M_u = 394.2 \text{ kip} \cdot \text{ft}$

Located two stations away from centerline of column.

V_u and M_u values are from CAP18

4.3.5 Locate and Describe Reinforcing

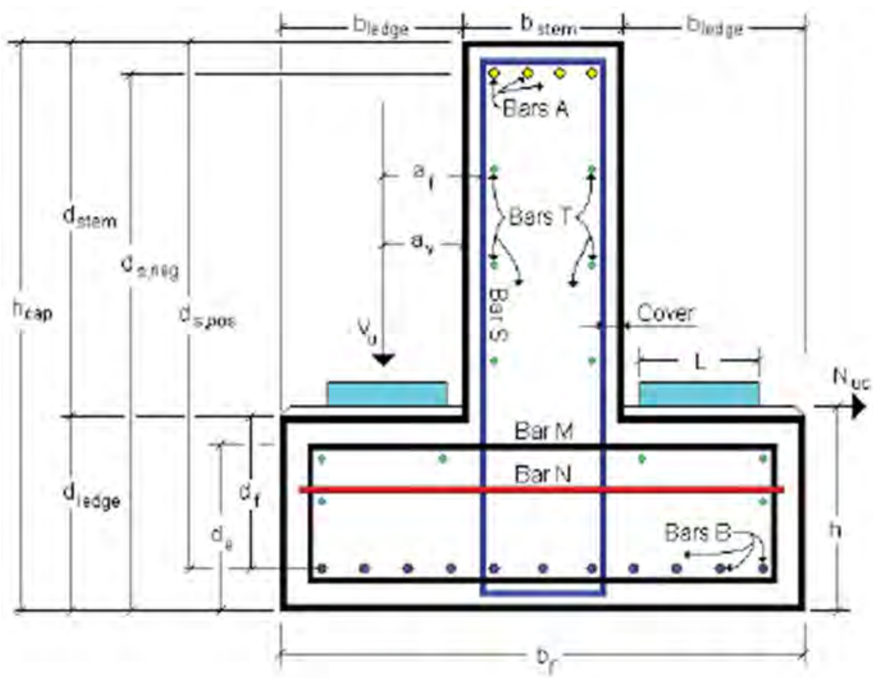


Figure 4.43 Section View of 30 Degrees Skewed ITBC

Recall:

$b_{stem} = 39 \text{ in}$

$d_{stem} = 57 \text{ in}$

$b_{ledge} = 24 \text{ in}$

$d_{ledge} = 28 \text{ in}$

$b_f = 87 \text{ in}$

$h_{cap} = 85 \text{ in}$

$\text{cover} = 2.5 \text{ in}$

4.3.5.1 Describe Reinforcing Bars

In the calculation of b_{ledge}, # 6 Bar M was considered. Bar M must be # 6 or smaller to allow it fully develop.

To prevent confusion, use the same bar size for Bar N as Bar M.

Use # 11 bars for Bar A

$A_{bar\_A} = 1.56 \text{ in}^2$ $\qquad d_{bar\_A} = 1.410 \text{ in}$

Use # 11 bars for Bar B

$A_{bar\_B} = 1.56 \text{ in}^2$ $\qquad d_{bar\_B} = 1.410 \text{ in}$

Use # 6 bars for Bar M

$A_{bar\_M} = 0.44 \text{ in}^2$ $\qquad d_{bar\_M} = 0.75 \text{ in}$

Use # 6 bars for Bar N

$A_{bar\_N} = 0.44 \text{ in}^2$ $\qquad d_{bar\_N} = 0.75 \text{ in}$

Use # 6 bars for Bar S

$A_{bar\_S} = 0.44 \text{ in}^2$ $\qquad d_{bar\_S} = 0.75 \text{ in}$

Use # 6 bars for Bar T

$A_{bar\_T} = 0.44 \text{ in}^2$ $\qquad d_{bar\_T} = 0.75 \text{ in}$

4.3.5.2 Calculate Dimensions

$d_{s\_neg} = h_{cap} - \text{cover} - \frac{1}{2} d_{bar\_S} - \frac{1}{2} d_{bar\_A}$ $\qquad d_{s\_neg} = 81.42 \text{ in}$

$d_{s\_pos} = h_{cap} - \text{cover} - \frac{1}{2} \max(d_{bar\_S}, d_{bar\_M}) - \frac{1}{2} d_{bar\_B}$ $\qquad d_{s\_pos} = 81.42 \text{ in}$

$a_v = 12 \text{ in}$

$a_f = a_v + \text{cover}$ $\qquad a_f = 14.50 \text{ in}$

$d_e = d_{ledge} - \text{cover}$ $\qquad d_e = 25.50 \text{ in}$

$d_f = d_{ledge} - \text{cover} - \frac{1}{2} d_{bar\_M} - \frac{1}{2} d_{bar\_B}$ $\qquad d_f = 24.42 \text{ in}$

$h = d_{ledge} + \text{BrgSeat}$ $\qquad h = 29.50 \text{ in}$

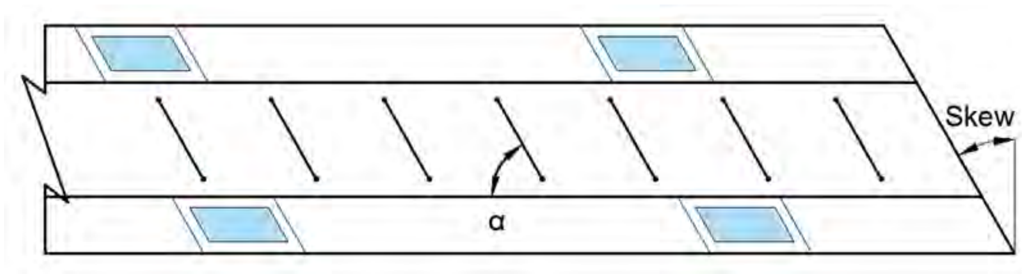


Figure 4.44 Plan View of 30 Degrees Skewed ITBC

$\alpha = 60 \text{ deg}$ *Angle of Bars S (Angle from the horizontal)*

Recall:

$L = 8 \text{ in}$ *Dimension of Bearing Pad*

$W = 21 \text{ in}$

4.3.6 Check Bearing

The load on the bearing pad propagates along a truncated pyramid whose top has the area A_1 and whose base has the area A_2. A_1 is the loaded area (the bearing pad area: L×W). A_2 is the area of the lowest rectangle contained wholly within the support (the Inverted Tee Cap). A_2 must not overlap the truncated pyramid of another load in either direction, nor can it extend beyond the edges of the cap in any direction.

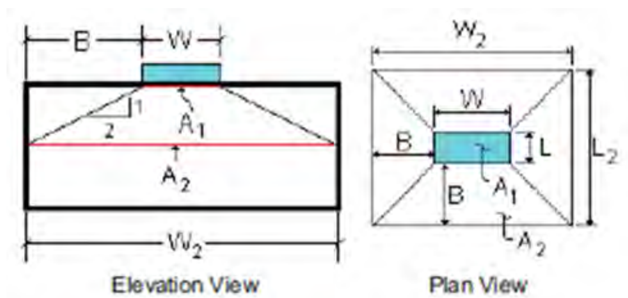


Figure 4.45. Bearing Check for 30 Degrees Skew Angle

Resistance Factor (ϕ) = 0.7 *(AASHTO LRFD 5.5.4.2)*

$A_1 = L \cdot W$ $A_1 = 168 \text{ in}^2$ *Area under Bearing Pad*

Interior Girders

$$B = \min\left[\left(b_{ledge} - a_v\right) - \frac{1}{2}L, \left(a_v + \frac{1}{2}b_{stem}\right) - \frac{1}{2}L, 2d_{ledge}, \frac{1}{2}S - \frac{1}{2}W\right]$$

"B" is the distance from perimeter of A_1 to the perimeter of A_2 as seen in the above figure

$B = 8 \text{ in.}$

$L_2 = L + 2 \cdot B$ $L_2 = 24.00 \text{ in}$

$W_2 = W + 2 \cdot B$ $W_2 = 37.00 \text{ in}$

$A_2 = L_2 \cdot W_2$ $A_2 = 888 \text{ in}^2$

Modification factor

$m = \min\left(\sqrt{\frac{A_2}{A_1}}, 2\right) = 2.29 \text{ and } 2 \quad m = 2$ *AASHTO LRFD Eq. 5.6.5-3*

$\phi V_n = \phi \;\; 0.85 \;\; f_c \;\; A_1 \;\; m$ $\phi V_n = 999.6 \text{ kips}$ *AASHTO LRFD Eqs. 5.6.5-1 and 5.6.5-2.*

$V_{u\_Int} = 338.53 < \phi V_n$ BearingChk = "OK!" *V_{u_int} from "4.3.4.4 Load Summary".*

Exterior Girders

$$B = \min\left[\left(b_{ledge} - a_v\right) - \frac{1}{2}L, \left(a_v + \frac{1}{2}b_{stem}\right) - \frac{1}{2}L, 2d_{ledge}, \frac{1}{2}S - \frac{1}{2}W, c - \frac{1}{2}W\right]$$

B= 8 in. *"B" is the distance from perimeter of A_1 to the perimeter of A_2 as seen in the above figure*

$L_2 = L + 2 \;\; B$ $L_2 = 24.00 \text{ in}$

$W_2 = W + 2 \;\; B$ $W_2 = 37.00 \text{ in}$

$A_2 = L_2 \;\; W_2$ $A_2 = 888 \text{ in}^2$

Modification factor

$m = \min\left(\sqrt{\frac{A_2}{A_1}}, 2\right) = 2.29 \text{ and } 2 \quad m = 2$ *AASHTO LRFD Eq. 5.6.5-3*

$\phi V_n = \phi \;\; 0.85 \;\; f_c \;\; A_1 \;\; m$ $\phi V_n = 999.6 \text{ kips}$ *AASHTO LRFD Eqs. 5.6.5-1 and 5.6.5-2:*

$V_{u\_ext} = 338.53 \text{ kips} < \Phi V_n$ BearingChk= "OK!" *V_{u_ext} from "4.3.4.4 Load Summary".*

4.3.7 Check Punching Shear

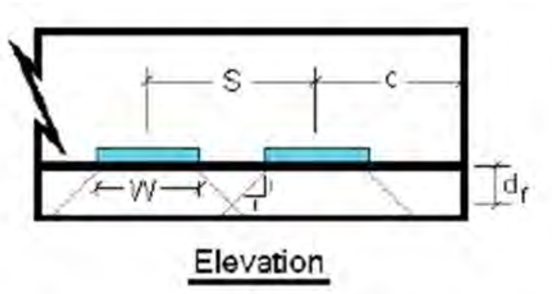


Figure 4.46 Punching Shear Check for 30 Degrees Skew Angle

AASHTO LRFD ***5.8.4.3.4****, the truncated pyramids assumed as failure surfaces for punching shear shall not overlap.*

Resistance Factor (ϕ) = 0.90 AASHTO LRFD 5.5.4.2.

Determine if the Shear Cones Intersect

Is $\frac{1}{2}S - \frac{1}{2}W \geq d_f$?

Yes. Therefore, shear cones do not intersect in the longitudinal direction of the cap.

$\frac{1}{2}S - \frac{1}{2}W = 37.5 \text{ in}$

$d_f = 24.42 \text{ in}$

TxDOT uses "d_f" instead of "d_e" for Punching Shear (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria). This is because "d_f" has traditionally been used for inverted tee bents and was sed in the Inverted Tee Research (Furiong % Mirza pg. 58).

Is $\frac{1}{2}b_{stem} + a_v - \frac{1}{2}L \geq d_f$?

Yes. Therefore, shear cones do not intersect in the transverse direction of the cap.

$\frac{1}{2}b_{stem} + a_v - \frac{1}{2}L = 27.5 \text{ in}$

$d_f = 24.42 \text{ in}$

Interior Girders

$V_n = 0.125 \,\square\, \lambda \sqrt{f'_c}\, b_o\, d_f$ $V_n = 585.91 \text{ kips}$ *AASHTO LRFD 5.8.4.3.4-3*

$b_o = W + 2L + 2d_f$ $b_o = 84.84 \text{ in}$ *AASHTO LRFD 5.8.4.3.4-4*

$\phi V_n = 527.32 \text{ kips}$

$V_{u\_Int} = 338.53 \text{ kips} < \phi V_n$ PunchingShearChk= "OK!" *V_{u_int} from "4.3.4.4 Load Summary"*

.

Exterior Girders

$V_n = \min[(0.125 \cdot \sqrt{f_c} \cdot \left(\frac{1}{2}W + L + d_f + c\right) * d_f, 0.125 \cdot \sqrt{f_c} \cdot (W + 2L + 2d_f) * d_f)]$

$V_n = 545.15$ kips

AASHTO LRFD 5.8.4.3.4-3 and 5.8.4.3.4-5

$\phi V_n = 411.09$ kips

$V_{u\_ext} = 338.53 \text{ kips} < \phi V_n$

PunchingShearChk= "OK!"

V_{u_ext} "4.3.4.4 Load Summary".

4.3.8 Check Shear Friction

Resistance Factor (ϕ) =0.90

AASHTO LRFD 5.5.4.2

Determine the Distribution Width

Interior Girders

$b_{s\_Int} = \min(W + 4a_v, S)$

"S" is the girder spacing.

= min (69 in, 96 in)

$b_{s\_Int} = 69$ in

$A_{cv} = b_{s\_Int} \cdot d_e$

$A_{cv} = 1759.5$ in2

Exterior Girders

$b_{s\_Ext} = \min(W + 4a_v, S, 2c)$

"S" is the girder spacing.

= min [69, 96, 48]

= 48 in

$A_{cv} = b_{s\_ext} \cdot d_e$

$A_{cv} = 1224$ in2

Interior Girders

$V_n = \min(0.2 \cdot f_c \cdot A_{cv}, 0.8 \cdot A_{cv})$

$V_n = 1408$ kips

AASHTO LRFD 5.8.4.2.2-1 and 5.8.4.2.2-2

= min (1759.5, 1408)

$\phi V_n = 1267$ kips

$V_{u\_Int} = 338.53 \text{ kips} < \phi V_n$

ShearFrictionChk= "OK!"

V_{u_int} from "4.3.4.4 Load Summary".

Exterior Girders

$V_n = \min(0.2 \cdot f_c \cdot A_{cv}, 0.8 \cdot A_{cv})$ = min (1224, 979.2)	V_n = 979.2 kips	*AASHTO LRFD 5.8.4.2.2-1 and 5.8.4.2.2-2*
$\phi V_n = 881$ kips		
$V_{u\_ext} = 338.53$ kips $< \phi V_n$	ShearFrictionChk= "OK!"	*V_{u_ext} from "4.3.4.4 Load Summary".*

4.3.9 Flexural Reinforcement for Negative Bending (Bars A)

$M_{dl} = |M_{negDL}|$ $\quad$ $M_{dl} = 443.9$ kip · ft

$M_s = |M_{negServ}|$ $\quad$ $M_s = 688.2$ kip · ft

$M_u = |M_{negUlt}|$ $\quad$ $M_u = 991.3$ kip · ft

4.3.9.1 Minimum Flexural Reinforcement

Factored Flexural Resistance, M_r, must be greater than or equal to the lesser of $1.2M_{cr}$ (Cracking Moment) or $1.33M_u$ (Ultimate Moment).

$I_g = 2.86 \times 10^6\ in^4$ — *Gross Moment of Inertia*

$h_{cap} = 85$ in — *Depth of Cap*

$ybar = 34.3$ in — *Distance to the Center of Gravity of the Cap from the bottom of the Cap*

$f_r = 0.24\sqrt{f_c}$ $\quad$ $f_r = 0.537$ ksi — *Modulus of Rupture (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)*

$y_t = h_{cap} - ybar$ $\quad$ $y_t = 50.70$ in — *Distance from Center of Gravity to extreme tension fiber*

$S = \frac{I_g}{y_t}$ $\quad$ $S = 5.64 \times 10^4\ in^3$ — *Section Modulus for the extreme tension fiber*

$M_{cr} = S \cdot f_r \cdot \frac{1ft}{12in}$ $\quad$ $M_{cr} = 2523.9$ kip · ft — *Cracking Moment (AASHTO LRFD Eq. 5.6.3.3-1)*

M_f = minimum of: — *Design the lesser of $1.2M_{cr}$ or $1.33M_u$ when determining mininum area of steel required.*

$1.2M_{cr} = 3028.7$ kip · ft

$1.33M_u = 1318.4$ kip · ft

Thus, M_r must be greater than $M_f = 1318.4$ kip · ft

4.3.9.2 Moment Capacity Design

Try, 7 ~ #11's Top

$\text{BarANo} = 7$		*Number of bars in tension*
$d_{bar\_A} = 1.410\ \text{in}$		*Diameter of main reinforcing bars*
$A_{bar\_A} = 1.56\ \text{in}^2$		*Area of main reinforcing bars*
$A_s = \text{BarANo} \cdot A_{bar\_A}$	$A_s = 10.92\ \text{in}^2$	*Area of steel in tension*
$d_{stirrup} = d_{bar\_S}$	$d_{stirrup} = 0.75\ \text{in}$	*Diameter of shear reinforcing bars*
$d = d_{s\_neg}$	$d = 81.42\ \text{in}$	
$b = b_f$	$b = 87\ \text{in}$	
$f_c = 5.0\ \text{ksi}$		*Compressive Strength of Concrete*
$f_y = 60\ \text{ksi}$		*Yield Strength of Rebar*
$\beta_1 = 0.85 - 0.05(f_c - 4\text{ksi})$		*(AASHTO LRFD 5.6.2.2)*
Bounded by: $0.65 \le \beta_1 \le 0.85$	$\beta_1 = 0.80$	
$c = \dfrac{A_s f_y}{0.85\ {}_c\beta_1 b}$	$c = 2.22\ \text{in}$	*Depth of Cross Section under Compression under Ultimate Load (AASHTO LRFD Eq. 5.6.3.1.2-4)*

This "c" is the distance from the extreme compression fiber to the neutral axis, not the distance from the center of bearing of the last girder to the end of the cap.

$a = c \cdot \beta_1$	$a = 1.78\ \text{in}$	*Depth of Equivalent Stress Block (AASHTO LRFD 5.6.2.2)*

Note: "a" is less than "d_{ledge}". Therefore the equivalent stress block acts over a rectangular area. If "a" was greater than "d_{ledge}", it would act over a Tee shaped area.

$M_n = A_s f_y \left(d - \frac{a}{2}\right) \cdot \frac{1\text{ft}}{12\text{in}}$	$M_n = 4397\ \text{kip} \cdot \text{ft}$	*Nominal Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.2-1)*
$\varepsilon_s = 0.003 \cdot \frac{d-c}{c}$	$\varepsilon_s = 0.107$	*Strain in Reinforcing at Ultimate*
$\varepsilon_s > 0.005$		
FlexureBehavior = "Tension Controlled"		*(AASHTO LRFD 5.6.2.1)*
$\Phi_M = 0.90$		*(AASHTO LRFD 5.5.4.2)*
$M_r = \Phi_M M_n$	$M_r = 3957.3\ \text{kip} \cdot \text{ft}$	*Factored Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.1-1)*
$M_f = 1318.4\ \text{kip} \cdot \text{ft} < M_r$	MinReinfChk = "OK!"	
$M_u = 991.3\ \text{kip} \cdot \text{ft} < M_r$	UltimateMom = "OK!"	

4.3.9.3 Check Serviceability

To find s_{max}:

For service loads, the stress on the cross-section is located as shown in Figure 4.47.

Modular Ratio:

$$n = \frac{E_s}{E_c} \qquad n = 7.12$$

Tension Reinforcement Ratio:

$$\rho = \frac{A_s}{b \cdot d} \qquad \rho = 0.0015$$

$$k = \sqrt{(2\rho n) + (\rho n)^2} - (\rho n) \qquad k = 0.136$$

$$d \cdot k = 11.07\ \text{in} \ < \ d_{ledge} = 28\ \text{in}$$

Therefore, the compression force acts over a rectangular area.

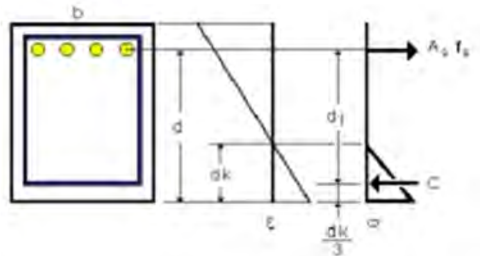


Figure 4.47 Stresses on the Cross Section for Service Loads of 30 Degrees Skewed ITBC

$$j = 1 - \frac{k}{3} \qquad j = 0.955$$

If the compression force does not act over rectangular area, j will be different.

$$f_{ss} = \frac{M_s}{A_s \cdot j \cdot d} \cdot \frac{12in}{1ft} \qquad f_{ss} = 9.73\ \text{ksi}$$

Service Load Bending Stress in outer layer of the reinforcing.

$$f_a = 0.6 f_y \qquad f_a = 36.00\ \text{ksi}$$

Allowable Bending Stress (BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

$f_{ss} < f_a$ ServiceStress = "OK!"

$$d_c = cover + \frac{1}{2} d_{stirrup} + \frac{1}{2} d_{bar\_A} \qquad d_c = 3.58\ \text{in}$$

Exposure Condition Factor:

$$\gamma_e = 1.00$$

For Class 1 Exposure Conditions. For areas where deicing chenicals are frequently used, design for Class 2 Exposure ($\gamma_e = 0.75$). (BDM-LRFD Ch. 4, Sect. 5, Design Criteria) (AASHTO LRFD Eq. 5.6.7-1)

$$\beta_s = 1 + \frac{d_c}{0.7(h_{cap} - d_c)} \qquad \beta_s = 1.06$$

$$s_{max} = \min\left(\frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c, 12in.\right) \qquad s_{max} = 12\ \text{in}$$

$$s_{Actual} = \frac{b_{stem} - 2d_c}{BarANo - 1} \qquad s_{Actual} = 5.31\ \text{in}$$

A good practice is to place a bar every 12 in along each surface of the bent. (TxSP)

$s_{actual} < s_{max}$ ServiceabilityCheck = "OK!

4.3.9.4 Check Dead Load

Check allowable M_{dl}: $f_{dl} = 22\ \text{ksi}$

TxDOT limits dead load stress to 22 ksi, which is set to limit observed cracking under dead load.

$$M_a = A_s \cdot d \cdot j \cdot f_{dl} \cdot \frac{1ft}{12in} \qquad M_a = 1556.7\ \text{kip} \cdot \text{ft}$$

Allowable Dead Load Moment

$M_{dl} = 443.9\ \text{kip} \cdot \text{ft} \ < \ M_a$ DeadLoadMom = "OK!"

4.3.10 Flexural Reinforcement for Positive Bending (Bars B)

$M_{dl} = M_{posDL}$	$M_{dl} = 294.4\ kip \cdot ft$
$M_s = M_{posServ}$	$M_s = 574.3\ kip \cdot ft$
$M_u = M_{posUlt}$	$M_u = 863.4\ kip \cdot ft$

4.3.10.1 Minimum Flexural Reinforcement

Factored Flexural Resistance, M_r, must be greater than or equal to the lesser of $1.2M_{cr}$ (Cracking Moment) or $1.33M_u$ (Ultimate Moment).

$I_g = 2.86 \times 10^6\ in^4$		*Gross Moment of Inertia*
$y_t = ybar$	$y_t = 34.3\ in$	*Distance to the Center of Gravity of the Cap from the top of the Cap*
$f_r = 0.24\sqrt{f_c}$	$f_r = 0.537\ ksi$	*Modulus of Rupture (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)*
$S = \frac{I_g}{y_t}$	$S = 8.34 \times 10^4\ in^3$	*Section Modulus for the extreme tension fiber*
$M_{cr} = S \cdot f_r \cdot \frac{1ft}{12in}$	$M_{cr} = 3732.2\ kip \cdot ft$	*Cracking Moment (AASHTO LRFD Eq. 5.6.3.3-1)*

M_f = minimum of:

$1.2M_{cr} = 4478.6\ kip \cdot ft$

$1.33M_u = 1148.3\ kip \cdot ft$

Design the lesser of $1.2M_{cr}$ or $1.33M_u$ when determining mininum area of steel required.

Thus, M_r must be greater than $M_f = 1148.3\ kip \cdot ft$

4.3.10.2 Moment Capacity Design

Try, 11 ~ #11's Bottom

Number of bars in tension

$BarBNo = 11$

Diameter of main reinforcing bars

$d_{bar\_B} = 1.41\ in$

Area of main reinforcing bars

$A_{bar\_B} = 1.56\ in^2$

Area of steel in tension

$A_s = BarBNo \cdot A_{bar\_B}$ $\quad A_s = 17.16\ in^2$

$d = d_{s\_pos}$ $\quad d = 81.42\ in$

$b = b_{stem}$ $\quad b = 39\ in$

Compressive Strength of Concrete

$f_c = 5.0\ ksi$

Yield Strength of Rebar

$f_y = 60\ ksi$

(AASHTO LRFD 5.6.2.2)

$\beta_1 = 0.85 - 0.05(f_c - 4ksi)$

Bounded by: $0.65 \leq \beta_1 \leq 0.85$ $\quad \beta_1 = 0.80$

Depth of Cross Section under Compression under Ultimate Load (AASHTO LRFD Eq. 5.6.3.1.2-4)

$c = \frac{A_s f_y}{0.85 f_c \beta_1 b}$ $\quad c = 7.76\ in$

This "c" is the distance from the extreme compression fiber to the neutral axis, not the distance from the center of bearing of the last girder to the end of the cap.

Depth of Equivalent Stress Block (AASHTO LRFD 5.6.2.2)

$a = c \cdot \beta_1$ $\quad a = 6.21\ in$

Note: "a" is less than "d_{stem}". Therefore the equivalent stress block acts over a rectangular area. If "a" was greater than "d_{stem}", it would act over a Tee shaped area.

Nominal Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.2-1)

$M_n = A_s f_y \left(d - \frac{a}{2}\right) \cdot \frac{1ft}{12in}$ $\quad M_n = 6719.4\ kip \cdot ft$

Strain in Reinforcing at Ultimate

$\varepsilon_s = 0.003 \cdot \frac{d-c}{c}$ $\quad \varepsilon_s = 0.028$

$\varepsilon_s > 0.005$

FlexureBehavior = "Tension Controlled" *(AASHTO LRFD 5.6.2.1)*

$\Phi_M = 0.90$ *(AASHTO LRFD 5.5.4.2)*

Factored Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.1-1)

$M_r = \Phi_M \cdot M_n$ $\quad M_r = 6047.5\ kip \cdot ft$

$M_f = 1148.3\ kip \cdot ft < M_r$ MinReinfChk = "OK!"

$M_u = 863.4\ kip \cdot ft < M_r$ UltimateMom = "OK!"

4.3.10.3 Check Serviceability

To find s_{max}:

For service loads, the stress on the cross-section is located as shown in Figure 4.48.

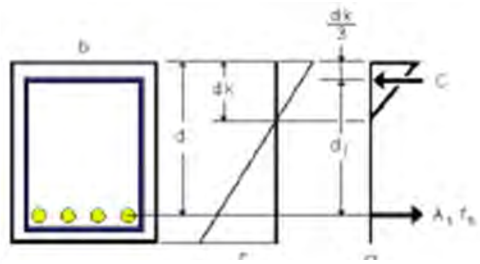


Figure 4.48 Stresses on the Cross Section for Bars B for Service Loads of 30 Degrees Skewed ITBC

Modular Ratio:

$$n = \frac{E_s}{E_c} \qquad n = 7.12$$

Tension Reinforcement Ratio:

$$\rho = \frac{A_s}{b \cdot d} \qquad \rho = 0.0054$$

$$k = \sqrt{(2\rho n) + (\rho n)^2} - (\rho n) \qquad k = 0.242$$

$$d \cdot k = 19.70\ \text{in} \ < \ d_{stem} = 57.00\ \text{in}$$

Therefore, the compression force acts over a rectangular area

$$j = 1 - \frac{k}{3} \qquad j = 0.919$$

If the compression force does not act over rectangular area, j will be different.

$$f_{ss} = \frac{M_s}{A_s \cdot j \cdot d} \cdot \frac{12\text{in}}{1\text{ft}} \qquad f_{ss} = 5.37\ \text{ksi}$$

Service Load Bending Stress in outer layer of the reinforcing.

$$f_a = 0.6 f_y \qquad f_a = 36.00\ \text{ksi}$$

Allowable Bending Stress (BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

$f_{ss} < f_a$ ServiceStress = “OK!”

$$d_c = \text{cover} + \frac{1}{2} d_{stirrup} + \frac{1}{2} d_{bar\_B} \qquad d_c = 3.58\ \text{in}$$

Exposure Condition Factor:

$$\gamma_e = 1.00$$

For Class 1 Exposure Conditions. For areas where deicing chenicals are frequently used, design for Class 2 Exposure ($\gamma_e = 0.75$). (BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

$$\beta_s = 1 + \frac{d_c}{0.7(h_{cap} - d_c)} \qquad \beta_s = 1.06$$

$$s_{max} = \min\left(\frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c, 12\text{in.}\right) \qquad s_{max} = 12\ \text{in}$$

(AASHTO LRFD Eq. 5.6.7-1)

A good practice is to place a bar every 12 in along each surface of the bent. (TxSP)

Bars Inside Stirrup Bar S

Number of Bars B that are inside Stirrup Bar S.

Try: BarBInsideSNo = 5

$$s_{Actual} = \frac{b_{stem} - 2\left(\text{cover} + \frac{1}{2} d_{bar\_S} + \frac{1}{2} d_{bar\_B}\right)}{\text{BarBInsideSNo} - 1} \qquad s_{Actual} = 7.96\ \text{in}$$

$s_{actual} < s_{max}$ ServiceabilityCheck = “OK

Bars Outside Stirrup Bar S

$BarBOutsideSNo = 11 - BarBInsideSNo$

Number of Bars B that are inside Stirrup Bar S.

$BarBOutsideSNo = 6$

$$s_{Actual} = \frac{2b_{ledge} + 2\left(cover + \frac{1}{2}d_{bar\_S} + \frac{1}{2}d_{bar\_B} - cove \quad \frac{1}{2}d_{bar\_M} - \frac{1}{2}d_{bar\_B}\right)}{BarBOutsideSNo}$$

$s_{actual} = 8.00\ in\ < s_{max}$ ServiceabilityCheck = "OK

4.3.10.4 Check Dead Load

Check allowable M_{dl}: $f_{dl} = 22\ ksi$

TxDOT limits dead load stress to 22 ksi. This is due to observed cracking under dead load.

$M_a = A_s \cdot d \cdot j \cdot f_{dl} \cdot \frac{1ft}{12in}$ $M_a = 2354.00\ kip$ *Allowable Dead Load Moment*

$M_{dl} = 294.4\ kip \cdot ft\ < \ M_a$ DeadLoadMom = "OK!"

Flexural Steel Summary:

Use 7 ~ # 11 Bars on Top

& 11 ~ # 11 Bars on Bottom

4.3.11 Ledge Reinforcement (Bars M & N)

Try Bars M and Bars N at a 4.70" spacing.

Use trial and error to determine the spacing needed for the ledge reinforcing.

It is typical for Bars M & N to be paired together

$$s_{bar\_M} = 4.70 \text{ in}$$

$$s_{bar\_N} = 4.70 \text{ in}$$

4.3.11.1 Determine Distribution Widths

These distribution widths will be used on the following pages to determine the required ledge reinforcement per foot of cap.

Distribution Width for Shear (AASHTO LRFD 5.8.4.3.2)

Note: These are the same distribution widths used for the Shear Friction check.

"S" is the girder spacing.

Interior Girders

$$b_{s\_Int} = \min(W + 4a_v, S)$$

$$b_{s\_Int} = 69.00 \text{ in}$$

Exterior Girders

"c" is the distance from the center of bearing of the outside beam to the end of the ledge.

$$b_{s\_Ext} = \min(W + 4a_v, 2c, S)$$

$$b_{s\_Ext} = 48.00 \text{ in}$$

Distribution Width for Bending and Axial Loads (AASHTO LRFD 5.8.4.3.3)

Interior Girders

$$b_{m\_Int} = \min(W + 5a_f, S)$$

$$b_{m\_Int} = 93.50 \text{ in}$$

Exterior Girders

$$b_{m\_Ext} = \min(W + 5a_f, 2c, S)$$

$$b_{m\_Ext} = 48.00 \text{ in}$$

4.3.11.2 Reinforcing Required for Shear Friction

AASHTO LRFD 5.7.4.1

$\Phi = 0.90$

(AASHTO LRFD 5.5.4)

$\mu = 1.4 \qquad c_1 = 0\ \text{ksi} \qquad P_c = 0\ \text{kip}$

Recall: $\quad d_e = 25.50\ \text{in}$

"μ" is 1.4 for monolithically placed concrete. (AASHTO LRFD 5.7.4.4)

For clarity, the cohesion factor is labeled "c_1". This is to prevent confusion with "c", the distance from the last girder to the edge of the cap. c_1 is 0ksi for corbels and ledges. (AASHTO LRFD 5.7.4.4)

"P_c" is zero as there is no axial compression.

Minimum Reinforcing (AASHTO LRFD Eq. 5.7.4.2-1)

$$A_{vf\_min} = \frac{0.05\ \text{ksi} \cdot A_{cv}}{f_y}$$

$$A_{cv} = d_e \cdot b_s \quad \text{and} \quad a_{vf} = \frac{A_{vf}}{b_s}$$

$$a_{vf\_min} = \frac{0.05\text{ksi} \cdot d_e}{f_y} \qquad a_{vf\_min} = 0.26\frac{\text{in}^2}{\text{ft}}$$

Minimum Reinforcing required for Shear Friction

Interior Girders

$$A_{cv} = d_e \cdot b_{s\_Int} \qquad A_{cv} = 1759\ \text{in}^2$$

$$V_{u\_Int} = 338.5\ \text{kip}$$

From "4.3.4.4 Load Summary".

$$V_n = c_1 A_{cv} + \mu\left(A_{vf} f_y + P_c\right)$$

(AASHTO LRFD Eq. 5.7.4.3-3)

$$\Phi V_n \geq V_u$$

(AASHTO LRFD Eq. 5.7.4.3-1 & AASHTO LRFD Eq. 5.7.4.3-2)

$$\Phi \cdot \left[c_1 A_{cv} + \mu\left(A_{vf} f_y + P_c\right)\right] \geq V_u$$

$$A_{vf} = \frac{\dfrac{\frac{V_{u\_Int}}{\Phi} - c_1 A_{cv}}{\mu} - P_c}{f_y} \qquad A_{vf} = 4.48\ \text{in}^2$$

Required Reinforcing for Shear Friction

$$a_{vf\_Int} = \frac{A_{vf}}{b_{s\_Int}} \qquad a_{vf\_Int} = 0.78\frac{\text{in}^2}{\text{ft}}$$

Required Reinforcing for Shear Friction per foot length of cap

Exterior Girders

$A_{cv} = d_e \cdot b_{s\_Ext}$ $A_{cv} = 1224\ \text{in}^2$

$V_{u\_Ext} = 338.5\ \text{kip}$ *From "4.3.4.4 Load Summary".*

$$V_n = c_1 A_{cv} + \mu(A_{vf} f_y + P_c)$$ *(AASHTO LRFD Eq. 5.7.4.3-3)*

$$\Phi V_n \geq V_u$$ *(AASHTO LRFD Eq. 5.7.4.3-1 & AASHTO LRFD Eq. 5.7.4.3-2)*

$$\Phi \cdot [c_1 A_{cv} + \mu(A_{vf} f_y + P_c)] \geq V_u$$

$A_{vf} = \dfrac{\dfrac{\frac{V_{u\_Ext}}{\Phi} - c_1 A_{cv}}{\mu} - P_c}{f_y}$ $A_{vf} = 4.48\ \text{in}^2$ *Required Reinforcing for Shear Friction*

$a_{vf\_Ext} = \dfrac{A_{vf}}{b_{s\_Ext}}$ $a_{vf\_Ext} = 1.12\ \dfrac{\text{in}^2}{\text{ft}}$ *Required Reinforcing for Shear Friction per foot length of cap*

4.3.11.3 Reinforcing Required for Flexure

AASHTO LRFD 5.8.4.2.1

Recall: $h = 29.50\ \text{in}$ $d_e = 25.50\ \text{in}$ $a_v = 12\ \text{in}$ *From "4.3.5.2 Calculate Dimensions"*

Interior Girders

$V_{u\_Int} = 338.5\ \text{kip}$ *From "4.3.4.4 Load Summary".*

$N_{uc\_Int} = 0.2 \cdot V_{u\_Int}$ $N_{uc\_Int} = 67.7\ \text{kip}$ *(AASHTO LRFD 5.8.4.2.1)*

$M_{u\_Int} = V_{u\_Int} \cdot a_v + N_{uc\_Int}(h - d_e)$ $M_{u\_Int} = 361.1\ \text{kip} \cdot \text{ft}$ *(AASHTO LRFD Eq. 5.8.4.2.1-1)*

Use the following equations to solve for A_f:

$$\Phi M_n \geq M_{u\_Int}$$ *(AASHTO LRFD Eq. 1.3.2.1-1)*

$$M_n = A_f f_y \left(d_e - \frac{a}{2}\right)$$ *(AASHTO LRFD Eq.5.6.3.2.2-1)*

$$c = \frac{A_f f_y}{\alpha_1 f_c \beta_1 b_{m\_Int}}$$ *(AASHTO LRFD Eq. 5.6.3.1.2-4)*

$$\alpha_1 = 0.85$$

$$\beta_1 = 0.80$$ *(AASHTO LRFD 5.6.2.2)*

$$a = c\beta_1$$

$$0.75 \leq \Phi = 0.65 + 0.15\left(\frac{d_e}{c} - 1\right) \leq 0.90$$ *AASHTO LRFD 5.5.4.2*

Solve for A_f: $A_f = 3.18\ \text{in}^2$ *Required Reinforcing for Flexure*

$a_{f\_Int} = \dfrac{A_f}{b_{m\_Int}}$ $a_{f\_Int} = 0.41\dfrac{\text{in}^2}{\text{ft}}$ *Required Reinforcing for Flexure per foot length of cap*

Exterior Girders

$V_{u\_Ext} = 338.5$ kip *From "4.3.4.4 Load Summary".*

$N_{uc\_Ext} = 0.2 \cdot V_{u\_Ext}$ $N_{uc\_Ext} = 67.7$ kip *(AASHTO LRFD 5.8.4.2.1)*

$M_{u\_Ext} = V_{u\_Ext} \cdot a_v + N_{uc\_Ext}(h - d_e)$ $M_{u\_Ext} = 361.1$ kip · ft *(AASHTO LRFD Eq. 5.8.4.2.1-1)*

Use the following equations to solve for A_f:

$\Phi M_n \geq M_{u\_Ext}$ *(AASHTO LRFD Eq. 1.3.2.1-1)*

$M_n = A_f f_y \left(d_e - \frac{a}{2}\right)$ *(AASHTO LRFD Eq.5.6.3.2.2-1)*

$c = \frac{A_f f_y}{\alpha_1 f_c \beta_1 b_{m\_Ext}}$ *(AASHTO LRFD Eq. 5.6.3.1.2-4)*

$\alpha_1 = 0.85$

$\beta_1 = 0.80$ *(AASHTO LRFD 5.6.2.2)*

$a = c\beta_1$

$0.75 \leq \Phi = 0.65 + 0.15\left(\frac{d_e}{c} - 1\right) \leq 0.90$ *AASHTO LRFD 5.5.4.2*

Solve for A_f: $A_f = 3.21$ in^2 *Required Reinforcing for Flexure*

$a_{f\_Ext} = \frac{A_f}{b_{m\_Ext}}$ $a_{f\_Ext} = 0.80 \frac{in^2}{ft}$ *Required Reinforcing for Flexure per foot length of cap*

4.3.11.4 Reinforcing Required for Axial Tension

(AASHTO LRFD 5.8.4.2.2)

$\Phi = 0.90$ *AASHTO LRFD 5.5.4.2*

Interior Girders:

$N_{uc\_Int} = 0.2V_{u\_Int}$ $N_{uc\_Int} = 67.7$ kip

$A_n = \frac{N_{uc\_Int}}{\Phi f_y}$ $A_n = 1.25$ in^2 *Required Reinforcing for Axial Tension*

$a_{n\_Int} = \frac{A_n}{b_{m\_Int}}$ $a_{n\_Int} = 0.16 \frac{in^2}{ft}$ *Required Reinforcing for Axial Tension per foot length of cap*

Exterior Girders:

$N_{uc\_Ext} = 0.2V_{u\_Int}$ $N_{uc_Ext} = 67.7$kip

$A_n = \frac{N_{uc\_Ext}}{\Phi f_y}$ $A_n = 1.25$ in^2 *Required Reinforcing for Axial Tension*

$a_{n\_Ext} = \frac{A_n}{b_{m\_Ext}}$ $a_{n\_Ext} = 0.31 \frac{in^2}{ft}$ *Required Reinforcing for Axial Tension per foot length of cap*

4.3.11.5 Minimum Reinforcing *(AASHTO LRFD 5.8.4.2.1)*

$a_{s\_min} = 0.04 \frac{f_c}{f_y} d_e$ $\quad a_{s\_min} = 1.02 \frac{in^2}{ft}$ *Minimum Required Reinforcing*

4.3.11.6 Check Required Reinforcing

Actual Reinforcing:

$a_s = \frac{A_{bar\_M}}{s_{bar\_M}}$ $\quad a_s = 1.12 \frac{in^2}{ft}$ *Primary Ledge Reinforcing Provided*

$a_h = \frac{A_{bar\_N}}{s_{bar\_N}}$ $\quad a_h = 1.12 \frac{in^2}{ft}$ *Auxiliary Ledge Reinforcing Provided*

Checks: $A_s \geq A_{s\_min}$ *(AASHTO LRFD 5.8.4.2.1)*

$A_s \geq A_f + A_n$ *(AASHTO LRFD 5.8.4.2.2)*

$A_s \geq \frac{2A_{vf}}{3} + A_n$ *(AASHTO LRFD Eq. 5.8.4.2.2-5)*

$A_h \geq 0.5(A_s - A_n)$ *(AASHTO LRFD Eq. 5.8.4.2.2-6)*

Check Interior Girders:

Bar M:

Check if: $a_s \geq a_{s\_min}$ *(AASHTO LRFD 5.8.4.2.1)*

$a_s \geq a_{f\_Int} + a_{n\_Int}$ *(AASHTO LRFD 5.8.4.2.2)*

$a_s \geq \frac{2a_{vf\_Int}}{3} + a_{n\_Int}$ *(AASHTO LRFD Eq. 5.8.4.2.2-5)*

$a_s = 1.12 \frac{in^2}{ft}$

$a_{s\_min} = 1.02 \frac{in^2}{ft} \quad < \quad a_s$

$a_{f\_Int} + a_{n\_Int} = 0.57 \frac{in^2}{ft} \quad < \quad a_s$

$\frac{2a_{vf\_Int}}{3} + a_{n\_Int} = 0.68 \frac{in^2}{ft} \quad < \quad a_s$

BarMCheck = "OK!"

Bar N:

Check if: $a_h \geq 0.5 \cdot (a_s - a_{n\_Int})$ *(AASHTO LRFD Eq. 5.8.4.2.2-6)*

$a_s =$ The maximum of:

$a_{f\_Int} + a_{n\_Int}$

$\frac{2a_{vf\_Int}}{3} + a_{n\_Int}$

$a_s = 0.68 \frac{in^2}{ft}$

"a_s" in this equation is the steel required for Bar M, based on the requirements for Bar M in AASHTO LRFD 5.8.4.2.2. This is derived from the suggestion that Ah should not be less than $A_f/2$ nor less than $A_{vf}/3$ (Furlong & Mirza pg. 73 & 74)

$0.5 \cdot (a_s - a_{n\_Int}) = 0.26 \frac{in^2}{ft} < a_h$

BarNCheck = “OK!”

Check Exterior Girders:

Bar M:

Check if: $a_s \geq a_{s\_min}$ *(AASHTO LRFD 5.8.4.2.1)*

$a_s \geq a_{f\_Ext} + a_{n\_Ext}$ *(AASHTO LRFD 5.8.4.2.2)*

$a_s \geq \frac{2a_{vf\_Ext}}{3} + a_{n\_Ext}$ *(AASHTO LRFD Eq. 5.8.4.2.2-5)*

$a_s = 1.12 \frac{in^2}{ft}$

$a_{s\_min} = 1.02 \frac{in^2}{ft} < a_s$

$a_{f\_Ext} + a_{n\_Ext} = 1.11 \frac{in^2}{ft} < a_s$

$\frac{2a_{vf\_Ext}}{3} + a_{n\_Ext} = 1.06 \frac{in^2}{ft} < a_s$

BarMCheck = “OK!”

Bar N:

Check if: $a_h \geq 0.5 \cdot (a_s - a_{n\_Ext})$ *(AASHTO LRFD Eq. 5.8.4.2.2-6)*

$a_s =$ The maximum of:

$a_{f\_Ext} + a_{n\_Ext}$

$\frac{2a_{vf\_Ext}}{3} + a_{n\_Ext}$

$a_s = 1.11 \frac{in^2}{ft}$

"a_s" in this equation is the steel required for Bar M, based on the requirements for Bar M in AASHTO LRFD 5.8.4.2.2. This is derived from the suggestion that Ah should not be less than $A_f/2$ nor less than $A_{vf}/3$ (Furlong & Mirza pg. 73 & 74)

$0.5 \cdot (a_s - a_{n\_Ext}) = 0.40 \frac{in^2}{ft} < a_h$

BarNCheck = “OK!”

Ledge Reinforcement Summary:

Use # 6 primary ledge reinforcing @ 4.70” maximum spacing

& # 6 auxiliary ledge reinforcing @ 4.70” maximum spacing

4.3.12 Hanger Reinforcement (Bars S)

Try Double # 6 Stirrups at a 7.40" spacing.

Use trial and error to determine the spacing needed for the hanger reinforcing.

It is typical for Bars S to have an integer multiple of the spacing of Bars M & N for practical reasons.

$s_{bar\_S} = 7.40 \text{ in}$

$A_{hr} = 2\text{stirrups} \cdot A_{bar\_S}$ $\qquad A_{hr} = 0.88 \text{ in}^2$

$A_v = 2\text{legs} \cdot A_{hr}$ $\qquad A_v = 1.76 \text{ in}^2$

4.3.12.1 Check Minimum Transverse Reinforcement

$b_v = b_{stem}$ $\qquad b_v = 39 \text{ in}$

$A_{v\_min} = 0.0316\lambda\sqrt{f_c}\frac{b_v \cdot s_{bar\_S}}{f_y}$ *(AASHTO LRFD Eq. 5.7.2.5-1)*

$\lambda = 1.0$ for normal weight concrete *(AASHTO LRFD 5.4.2.8)*

$A_{v\_min} = 0.34 \text{ in}^2$

$A_v > A_{v\_min}$ $\qquad$ MinimumSteelCheck = "OK!"

4.3.12.2 Check Service Limit State

AASHTO LRFD 5.8.4.3.5 with notifications from BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

Interior Girders

TxDOT uses "2/3 f_y" from the original research (Furlong & Mirza Eq. 5.4) instead of "0.5 f_y" from AASHTO LRFD Eq. 5.8.4.3.5-1. (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)

V_{all} = minimum of:

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{bar\_S}} \cdot (W + 3a_v) = 228 \text{ kip}$$

Bounded by: $(W + 3a_v) \leq \min(S, 2c)$

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{bar\_S}} \cdot S = 457 \text{ kip}$$

(BDM-LRFD Ch.4, Sect. 5, Design Criteria modified to limit the distribution width to the girder spacing. This will prevent distribution widths from overlapping)

$V_{all} = 228 \text{ kip}$

$V_{s\_Int} = 225 \text{ kip} < V_{all}$ $\qquad$ ServiceCheck = "OK!"

Exterior Girders

TxDOT uses "2/3 f_y" from the original research (Furlong & Mirza Eq. 5.4) instead of "0.5 f_y" from AASHTO LRFD Eq. 5.8.4.3.5-1. (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)

V_{all} = minimum of:

V_{all} for the Interior Girder

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{bar\_S}} \cdot \left(\frac{W + 3a_v}{2} + c\right) = 228 \text{ kip}$$

Bounded by: $(W + 3a_v) \leq \min(S, 2c)$

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{bar\_S}} \cdot \left(\frac{S}{2} + c\right) = 342 \text{ kip}$$

(BDM-LRFD Ch.4, Sect. 5, Design Criteria Modified to limit the distribution width to half the girder spacing and the distance to the edge of the cap. This will prevent distribution widths from overlapping or extending over the edge of the cap.)

$V_{all} = 228$ kip

$V_{s\_Ext} = 225 \text{ kip} < V_{all}$ ServiceCheck = "OK!"

4.3.12.3 Check Strength Limit State

(AASHTO LRFD 5.8.4.3.5)

$\Phi = 0.90$ *(AASHTO LRFD Eq. 5.5.4.2)*

Interior Girders:

V_n = minimum of:

$$\frac{A_{hr} \cdot f_y}{s_{bar\_S}} \cdot S = 685 \text{ kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-2)*

$$\left(0.063\sqrt{f_c} \cdot b_f \cdot d_f\right) + \frac{A_{hr} \cdot f_y}{s_{bar\_S}} (W + 2d_f) = 798 \text{ kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-3)*

$V_n = 685$ kip

$\Phi V_n = 617$ kip

$V_{u\_Int} = 339 \text{ kip} < \Phi V_n$ UltimateCheck = "OK!"

Exterior Girders:

V_n = minimum of:

V_n for the Interior Girder

$$\frac{A_{hr} \cdot f_y}{s_{bar\_S}} \cdot \left(\frac{S}{2} + c\right) = 514 \text{ kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-2)*

$$\left(0.063\sqrt{f_c} \cdot b_f \cdot d_f\right) + \frac{A_{hr} \cdot f_y}{s_{bar\_S}} \left(\frac{W + 2d_f}{2} + c\right) = 720 \text{ kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-3)*

(These equations are modified to limit the distribution width to the edge of the cap)

$V_n = 514$ kip

$\Phi V_n = 463$ kip

$V_{u\_Ext} = 339 \text{ kip} < \Phi V_n$ UltimateCheck = "OK!"

4.3.12.4 Check Combined Shear and Torsion

The following calculations are for Station 36. All critical locations must be checked. See the Concrete Section Shear Capacity spreadsheet in the appendices for calculations at other locations. Shear and Moment were calculated using the CAP 18 program.

$M_u = 394.2 \text{ kip}\cdot\text{ft}$ $V_u = 452.1 \text{ kip}$ $N_u = 0 \text{ kip}$ $T_u = 706 \text{ kip}\cdot\text{ft}$

Recall:

$\beta_1 = 0.80$ $f_y = 60 \text{ ksi}$

$f_c = 5.0 \text{ ksi}$ $E_s = 29000 \text{ ksi}$

$b_f = 87 \text{ in}$ $h_{cap} = 85 \text{ in}$ $b_{stem} = 39 \text{ in}$ $h = 29.50 \text{ in}$

$b_v = b_{stem}$ $b_v = 39 \text{ in}$

Find d_v:

(AASHTO LRFD 5.7.2.8)

Shears are maximum near the column faces. In these regions the cap is in negative bending with tension in the top of the cap. Therefore, the calculations are based on the steel in the top of the bent cap.

$A_s = A_{bar\_A}\cdot BarANo$ $A_s = 10.92 \text{ in}^2$

$c = \frac{A_s f_y}{0.85 f_c \beta_1 b_f}$ $c = 2.21 \text{ in}$

$a = c\cdot\beta_1$ $a = 1.77 \text{ in}$

$d_s = d_{s\_neg}$ $d_s = 81.42 \text{ in}$

$M_n = A_s f_y\left(d_s - \frac{a}{2}\right)$ $M_n = 4397.2 \text{ kip}\cdot\text{ft}$

$A_{ps} = 0 \text{ in}^2$

$d_e = \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y}$ $d_e = 81.42 \text{ in}$ *(AASHTO LRFD Eq. 5.7.2.8-2)*

$d_v =$ maximum of:

$\frac{M_n}{A_s f_y + A_{ps} f_{ps}} = 80.53 \text{ in}$

$0.9 d_e = 73.28 \text{ in}$

$0.72h = 21.24 \text{ in}$

$d_v = 80.53 \text{ in}$

The method for calculating θ and β used in this design example are from AASHTO LRFD Appendix B5. The method from AASHTO LRFD 5.7.3.4.2 may be used instead. The method from 5.7.3.4.2 is based on the method from Appendix B5; however, it is less accurate and more conservative (often excessively conservative). The method from Appendix B5 is preferred because it is more accurate, but it requires iterating to a solution.

Determine θ and β:

$\Phi_V = 0.90$ *(AASHTO LRFD Eq. 5.5.4.2)*

$v_u = \frac{|V_u - (\Phi_V \cdot V_p)|}{\Phi_V \cdot b_v \cdot d_v}$ $v_u = 0.16$ ksi *Shear Stress on the Concrete (AASHTO LRFD Eq. 5.7.2.8-1)*

$\frac{v_u}{f_c} = 0.03$

Using Table B5.2-1 with $\frac{v_u}{f_c} = 0.03$ and $\varepsilon_x = 0.001$

$\theta = 36.4$ deg and $\beta = 2.23$

Determining θ and β is an iterative process, therefore, assume initial shear strain value ε_x of 0.001 per LRFD B5.2 and then verify that the assumption was valid.

$$\varepsilon_x = \frac{\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})}$$

Strain halfway between the compressive and tensile resultants (AASHTO LRFD Eq. B5.2-3) If $\varepsilon_x < 0$, then use equation B5.2-5 and re-solve for ε_x.

where $|M_u| = 394.2$ kip · ft must be > $|V_u - V_p|d_v = 3034$ kip · ft

$\varepsilon_x = 1.20 \times 10^{-3} > 1.00 \times 10^{-3}$

use $\varepsilon_x = 1.00 \times 10^{-3}$.

For values of ε_x greater than 0.001, the tensile strain in the reinforcing, ε_t is greater than 0.002. ($\varepsilon_t = 2\varepsilon_x - \varepsilon_c$, where ε_c is < 0) Grade 60 steel yields at a strain of 60 ksi / 29,000 ksi = 0.002. By limiting the tensile strain in the steel to the yield strain and using the Modulus of Elasticity of the steel prior to yield, this limits the tensile stress of the steel to the yield stress. ε_x has not changed from the assumed value, therefore no iterations are required.

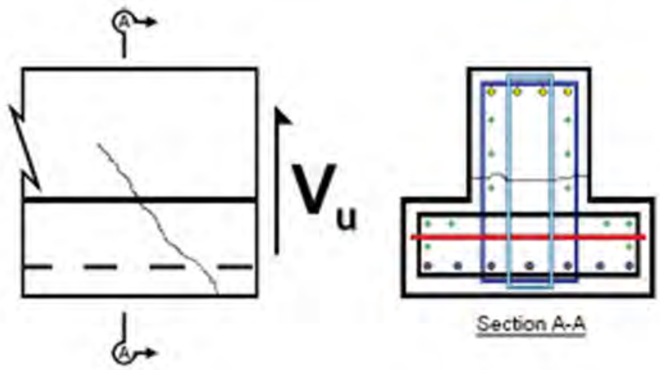
$V_p = 0$ kip *"V_p" is zero as there is no prestressing.*

$A_c = b_{stem} \cdot \frac{h_{cap}}{2}$ $A_c = 1657.5$ in^2

$s = s_{bar\_S}$ $s = 7.40$ in

(AASHTO LRFD B5.2) "A_c" is the area of concrete on the flexural tension side of the cap, from the extreme tension fiber to one half the cap depth.

"A_c" is needed if AASHTO LRFD Eq. B5.2-3 is negative.



The transverse reinforcement, "A_v", is double closed stirrups. The failure surface intersects four stirrup legs, therefore the area of the shear steel is four times the stirrup bar's area (0.44in2). See the sketch of the failure plane to the left.

Figure 4.49 Failure Surface of 30 Degrees Skewed ITBC for Combined Shear and Torsion

$A_v = 2legs \cdot 2stirrups \cdot A_{bar\_S}$ $\quad A_v = 1.76\ in^2$

$A_t = 1leg \cdot A_{bar\_S}$ $\quad A_t = 0.44\ in^2$

$A_{oh} = (d_{stem}) \cdot (b_{stem} - 2cover) + (d_{ledge} - 2cover) \cdot (b_f - 2cover)$

$A_{oh} = 3496\ in^2$

$A_0 = 0.85A_{oh}$ $\quad A_0 = 2971.6in^2$

$p_h = (b_{stem} - 2cover) + 2(b_{ledge}) + (b_f - 2cover) + 2(h_{cap} - 2cover)$

$p_h = 324\ in$

<u>Equivalent Shear Force</u>

$V_{u\_Eq} = \sqrt{V_u^2 + \left(\frac{0.9p_hT_u}{2A_0}\right)^2}$ $\quad V_{u\_Eq} = 614.2\ kip$ *(AASHTO LRFD Eq. B.5.2-1)*

<u>Shear Steel Required</u>

$V_n =$ the lesser of:

$V_c + V_s + V_p$ *(AASHTO LRFD Eq. 5.7.3.3-1)*

$0.25 \cdot f_c \cdot b_v \cdot d_v + V_p$ *(AASHTO LRFD Eq. 5.7.3.3-2)*

Check maximum ΦV_n for section:

$\Phi V_{n\_max} = \Phi \cdot (0.25 \cdot f_c \cdot b_v \cdot d_v + V_p)$

$\Phi V_{n\_max} = 3533\ kip$

$V_u = 452.1\ kip\ <\ \Phi V_{n\_max}$ $\quad$ MaxShearCheck = "OK!"

Calculate required shear steel:

$V_u < \Phi V_n$ *(AASHTO LRFD Eq. 1.3.2.1-1)*

$V_c = 0.0316 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v$ $\quad V_c = 495\ \text{kip}$ *(AASHTO LRFD Eq. 5.7.3.3-3)*

$V_u < \Phi_V \cdot (V_c + V_s + V_p)$

$V_s = \frac{A_v \cdot f_y \cdot d_v \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha}{s_{req}}$ *(AASHTO LRFD Eq. 5.7.3.3-4)*

$a_{v\_req} = \frac{\frac{V_u}{\Phi_V} - V_c - V_p}{f_y \cdot d_v \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha}$ $\quad a_{v\_req} = 0.011 \frac{in^2}{ft}$

Torsional Steel Required

$\Phi_T = 0.9$ *(AASHTO LRFD 5.5.4.2)*

$T_u \le \Phi_T T_n$ *(AASHTO LRFD Eq. 1.3.2.1-1)*

$T_n = \frac{2A_o A_t f_y \cot\theta}{s_{bar\_S}}$ *(AASHTO LRFD Eq. 5.7.3.6.2-1)*

$a_{t\_req} = \frac{T_u}{\Phi_T 2A_o f_y \cot\theta}$ $\quad a_{t\_req} = 0.23 \frac{in^2}{ft}$

Total Required Transverse Steel

The transverse reinforcement is designed for the side of the section where the effects of shear and torsion are additive. (AASHTO LRFD C5.7.3.6.1)

$a_{req} = a_{v\_req} + 2sides \cdot a_{t\_req}$ $\quad a_{req} = 0.47 \frac{in^2}{ft}$

$a_{prov} = \frac{A_v}{s_{bar\_S}}$ $\quad a_{prov} = 2.85 \frac{in^2}{ft}$

$a_{prov} > a_{req}$ TransverseSteelCheck = "OK!"

Longitudinal Reinforcement

$$A_{ps} f_{ps} + A_s f_y \ge \frac{|M_u|}{\Phi d_v} + \frac{0.5N_u}{\Phi} + \cdots$$ *(AASHTO LRFD Eq. 5.7.3.6.3-1)*

$$\cot\theta \sqrt{\left(\left|\frac{V_u}{\Phi} - V_p\right| - 0.5V_s\right)^2 + \left(\frac{0.45 p_h T_u}{2A_o \Phi}\right)^2}$$

$V_s = a_{t\_req} \cdot f_y \cdot d_v \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha$ *(AASHTO LRFD Eq. 5.7.3.3-4)*

Bounded By: $V_s < \frac{V_u}{\Phi_V}$ $\quad V_s = 502.3\ \text{kip}$ *(AASHTO LRFD Eq. 5.7.3.5-1)*

$$\frac{|M_u|}{\Phi_f d_v} + \frac{0.5N_u}{\Phi_c} + \cot\theta \sqrt{\left(\left|\frac{V_u}{\Phi_V} - V_p\right| - 0.5V_s\right)^2 + \left(\frac{0.45 p_h T_u}{2A_o \Phi_T}\right)^2} = 528\ \text{kip}$$

Provided Force:

$A_s f_y = 655.2\ \text{kip} > 528\ \text{kip}$ LongitudinalReinfChk = "OK!"

4.3.12.5 Maximum Spacing of Transverse Reinforcement *(AASHTO LRFD 5.7.2.6)*

Shear Stress

$v_u = \frac{|V_u - \Phi_V V_p|}{\Phi_V b_v d_v}$ $v_u = 0.16$ ksi *(AASHTO LRFD Eq. 5.7.2.8-1)*

$0.125 \cdot f_c = 0.625$ ksi

If $v_u < 0.125 \cdot f_c$ *(AASHTO LRFD Eq. 5.7.2.6-1)*

$s_{max} = \min(0.8d_v, 24\text{in})$

If $v_u \geq 0.125 \cdot f_c$ *(AASHTO LRFD Eq. 5.7.2.6-2)*

$s_{max} = \min(0.4d_v, 12\text{in})$

Since $v_u < 0.125 \cdot f_c$ $s_{max} = 24.00$ in

TxDOT limits the maximum transverse reinforcement spacing to 12". *(BDM-LRFD, Ch. 4, Sect. 5, Detailing)*

$s_{max} = 12.00$ in

$s_{bar\_S} = 7.40$ in $< s_{max}$ SpacingCheck= "OK!"

Hanger Reinforcement Summary:

Use double # 6 stirrups @ 7.40" maximum spacing

4.3.13 End Reinforcements (Bars U1, U2, U3, and G)

Extra vertical, horizontal, and diagonal reinforcing at the end surfaces is provided to reduce the maximum crack widths. According to the parametric analysis, it is recommended to place #6 U1 Bars, U2 Bars, and U3 Bars at the end faces and #7 G Bars at approximately 6in. spacing at the first 30” to 35” of the end of bent cap. U1 Bars are the vertical end reinforcements, U2 Bars and U3 Bars are the horizontal end reinforcements at the stem and the ledge, respectively. G Bars are the diagonal end reinforcement.

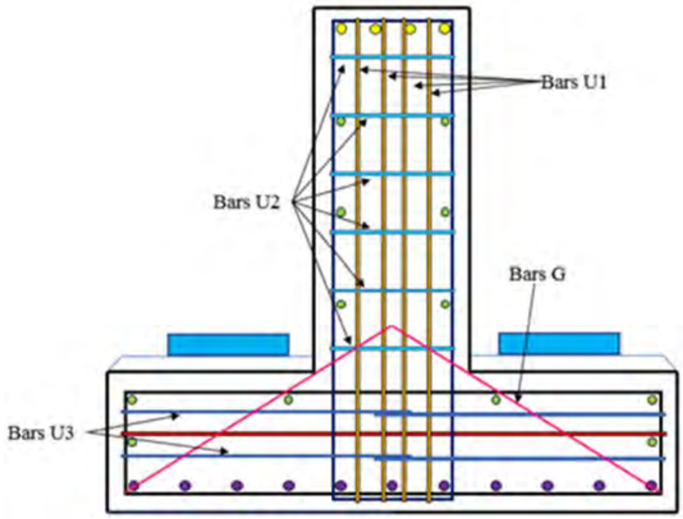


Figure 4.50 End Face Section View of 30 Degrees Skewed ITBC

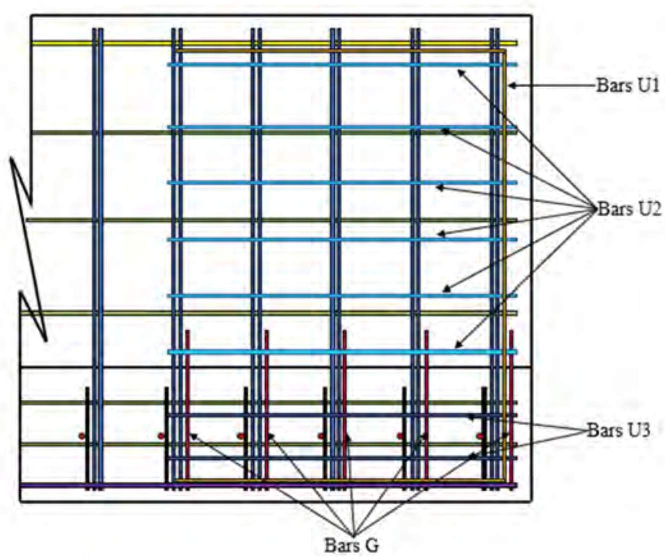


Figure 4.51 End Face Elevation View of 30 Degrees Skewed ITBC

4.3.14 Skin Reinforcement (Bars T)

Try 7 ~ # 6 bars in Stem and 3 ~ # 6 bars in Ledge on each side

$A_{bar\_T} = 0.44\ in^2$

$NoTBarsStem = 7$

$NoTBarsLedge = 3$

"a" must be within $\frac{2}{3}d_e$.

(AASHTO LRFD 5.13.2.4.1)

$\frac{2}{3}d_e = 17.00\ in$

TxDOT typically uses: $a = 6\ in$

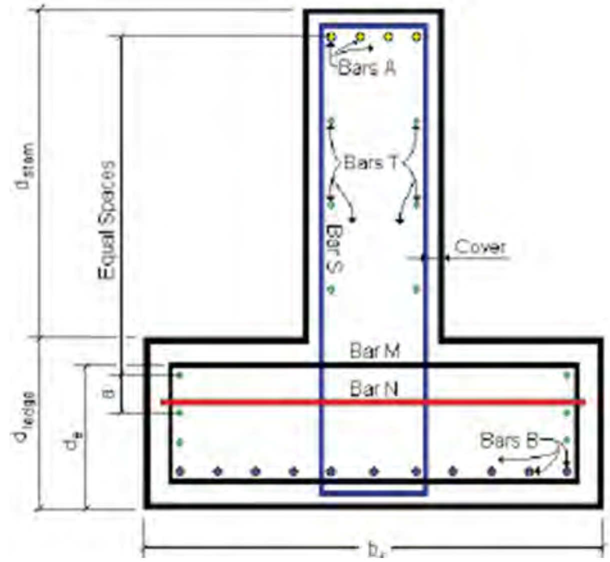


Figure 4.52 Section View for T Bars of 30 Degrees Skewed ITBC

4.3.14.1 Required Area of Skin Reinforcement

(AASHTO LRFD 5.6.7)

(AASHTO LRFD Eq. 5.6.7-3)

$A_{sk\_Req} = 0.012 \cdot (d - 30)$ $\qquad A_{sk\_Req} = 0.62\frac{in^2}{ft}$

A_{sk} need not be greater than one quarter of the main reinforcing ($A_s/4$)per side face within $d/2$ of the main reinforcing. (AASHTO LRFD 5.6.7)

"d" is the distance from the extreme compression fiber to the centroid of the extreme tension steel element. In this example design, $d = d_{s\_pos} = d_{s\_neg} = 81.42$ in.

$$A_{sk\_max} = \max\left(\frac{\frac{A_{bar\_A} \cdot BarANo}{4}}{\frac{d_{s\_neg}}{2}}, \frac{\frac{A_{bar\_B} \cdot BarBNo}{4}}{\frac{d_{s\_pos}}{2}}\right)$$

$$A_{sk\_max} = 1.26\frac{in^2}{ft}$$

$$A_{skReq} = \min(A_{sk\_Req}, A_{sk\_max})$$

$$A_{skReq} = 0.62\frac{in^2}{ft}$$

4.3.14.2 Required Spacing of Skin Reinforcement

(AASHTO LRFD 5.6.7)

s_{req} = minimum of:

$\frac{A_{bar\_T}}{A_{skReq}} = 8.52\ in$

$\frac{d_{s\_neg}}{6} = 13.57\ in$

$\frac{d_{s\_pos}}{6} = 13.57\ \text{in}$

& 12 in

$s_{req} = 8.52\ \text{in}$

4.3.14.3 Actual Spacing of Skin Reinforcement

Check T Bars spacing in Stem:

$$h_{top} = d_{stem} - \left(cover + \frac{d_{bar\_S}}{2} + \frac{d_{bar\_A}}{2}\right) + \left(cover + \frac{d_{bar\_M}}{2} + \frac{d_{bar\_T}}{2}\right)$$

$h_{top} = 56.67\ \text{in}$

$$s_{skStem} = \frac{h_{top}}{NoTBarsSte}$$

$s_{skStem} = 7.08\ \text{in}$

$s_{skStem} < s_{req}$ SkinSpacing = "OK!"

Check T Bars spacing in Ledge:

$$h_{bot} = d_{ledge} - \left(cover + \frac{d_{bar\_M}}{2} + \frac{d_{bar\_T}}{2}\right) - \left(cover + \frac{d_{bar\_S}}{2} + \frac{d_{bar\_B}}{2}\right)$$

$h_{bot} = 21.17\ \text{in}$

$$s_{skLedge} = \frac{h_{bot} - a}{NoTBarsLedge}$$

$s_{skLedge} = 7.59\ \text{in}$

$s_{skLedge} < s_{req}$ SkinSpacing = "OK!"

Check if "a" is less than s_{req}

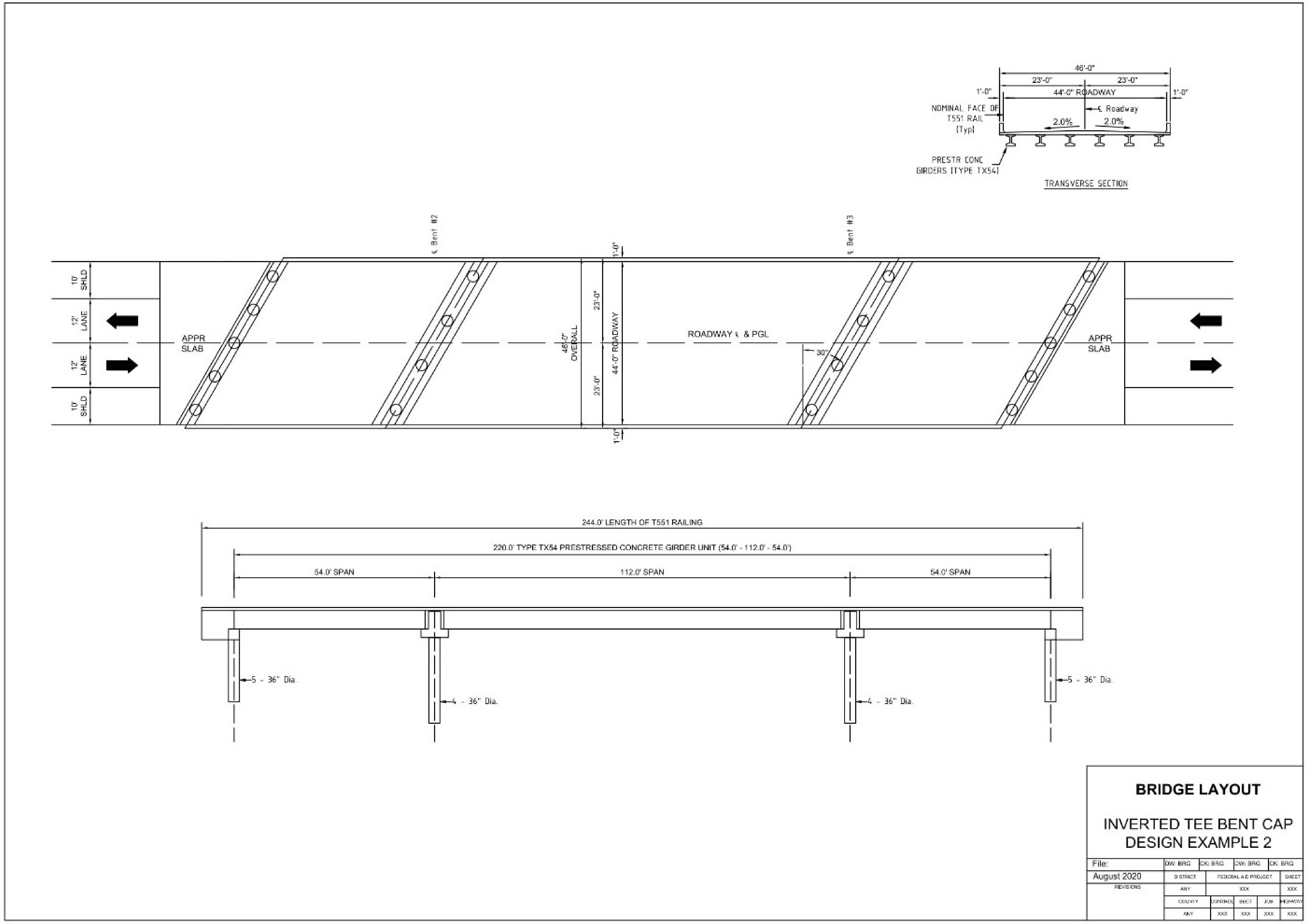
$a = 6\ \text{in} < s_{req}$ SkinSpacing = "OK!"

Skin Reinforcement Summary:

Use 7 ~ # 6 bars in Stem and 3 ~ # 6 bars in Ledge on each side

4.3.15 Design Details and Drawings

4.3.15.1 Bridge Layout



4.3.15.2 CAP 18 Input File

```
$File                          Proj                User  Date (Today
$ Num     County      Highway  Num      CSJ        Init     if Blank) Comment
$XXXX  XXXXXXXXXXXXX  XXXXXX  XXXX  XXXX-XX-XXX  XXX  XXXXXXXXXXXX XXXXXXX
00001  ___County____  Highwy  Pro#  0000-00-000  BRG               Comment
$Header Card 2 ----------------------------------------------------------------
$XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
CAP18 Version 6.00 ITBC Design Example 2, Skew = 30.00
$Problem Card -----------------------------------------------------------------
$Prob E   XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
    1 E 0 (Spans L=54'-112'-54',  Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay)
$TABLE 1 - CONTROL DATA -------------------------------------------------------
$                Enter 1 to keep:       Number cards   Options:
$                 Env Tab2 Tab3 Tab4     on Table 4  Envelope  Print    Skew Angle
$                  X    X    X    X             XX     X         XX     XXXXXXXXXX
                                                16                           30.0
$TABLE 2 - CONSTANTS ----------------------------------------------------------
$   TABLE 2a                                            Anly Opt (1=Working,
$                                   |-Movable Load Data--|  2=Load Factor,3=Both)
$                Num  Increment     |Num  Start Stop Step|Anly|  Load Factors:
$                Inc   Length       |Inc  Sta   Sta  Size| Opt| Dead      Live
$                 XX XXXXXXXXX        XXX  XXX  XXX    X    X XXXXXXXX XXXXXXXX
                  92     0.5           20    2   70    1    3    1.25      1.75
$   TABLE 2b
$    Overlay    Max #|-----------Live Load Reduction Factors---------|
$  Load Factor  Lanes| 1 lane   2 lanes    3 lanes    4 lanes    5 lanes
$    XXXXX          X  XXXX       XXXX       XXXX       XXXX       XXXX
     1.50           3  1.2        1.0        0.85       0.65       0.65
$TABLE 3 - LIST OF STATIONS ---------------------------------------------------
$     Number of input values for                Str - Stringers, Sup - Supports
$                Lane Str  Sup  MCP  VCP        MCP - Moment Control Points
$                 XX   XX   XX   XX   XX        VCP - Shear Control Points
  (Num Inputs)     3    6    4   11    8
$     Left Lane Boundary Stations
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Lane Left)      2   32   60
$     Right Lane Boundary Stations
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Lane Right)    32   60   90
$     Station of Stringers (two rows max, may be at tenths of stations, XX.X)
$               XXXX XXXX XXXX XXXX XXXX XXXX XXXX XXXX XXXX XXXX
  (Stringers)      6   22   38   54   70   86
$     Station of Supports (two rows max)
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Supports)      10   34   58   82
$     Moment Control Point Stations (two rows max)
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Mom CP)         6   10   22   34   38   46   54   58   70   82
  (Mom CP)        86
$     Shear Control Point Stations (two rows max)
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Shear CP)       8   12   32   36   56   60   80   84
$TABLE 4 - STIFFNESS AND LOAD DATA --------------------------------------------
$                             Bending   Sidewalk,  Cap &
$                Station  1 if  Stiffness  Slab      Stringer  Moving     Overlay
$Comments        From  To Cont'd  of Cap    Loads     Loads     Loads     Loads,DW
$XXXXXXXXXXXXXX  XXX  XXX    X XXXXXXXXX XXXXXXXXX XXXXXXXXX XXXXXXXXX XXXXXXXXX
(CAP EI & DL)      2   90      8.09E+07            -2.427
(DL Span1, Bm1)    6    6                          -50.17              -5.04
(DL Span1, Bm2)   22   22                          -50.17              -5.04
(DL Span1, Bm3)   38   38                          -50.17              -5.04
(DL Span1, Bm4)   54   54                          -50.17              -5.04
(DL Span1, Bm5)   70   70                          -50.17              -5.04
(DL Span1, Bm6)   86   86                          -50.17              -5.04
(DL Span2, Bm1)    6    6                          -104.1              -10.5
(DL Span2, Bm2)   22   22                          -104.1              -10.5
(DL Span2, Bm3)   38   38                          -104.1              -10.5
(DL Span2, Bm4)   54   54                          -104.1              -10.5
(DL Span2, Bm5)   70   70                          -104.1              -10.5
(DL Span2, Bm6)   86   86                          -104.1              -10.5
(Dist. Lane Ld)    0   20                                    -4.92
(Conc. Lane Ld)    4    4                                    -21.3
(Conc. Lane Ld)   16   16                                    -21.3
```

4.3.15.3 CAP 18 Output File

```
AUG 07, 2020     TEXAS DEPARTMENT OF TRANSPORTATION (TxDOT)        PAGE  1
CAP18                  BENT CAP ANALYSIS       Ver.  6.2  (Jul, 2011)

   PSF            HIGHWAY  PD-    CONTROL-   CODED
    NO     COUNTY      NO   IPE  SECTION-JOB   BY    DATE
  00001  ___County____  Highwy  Pro#  0000-00-000  BRG  AUG 07, 2020 Comment

CAP18 Version 6.00 ITBC Design Example 2, Skew = 30.00
PROB    1  (Spans L=54'-112'-54',  Type TX54 Girder @ 8.0', 8" Slab, 2" O'la


ENGLISH SYSTEM UNITS

TABLE 1.  CONTROL DATA

   OPTION TO PRINT TABLE SRS (1=YES)                          0

                                ENVELOPES    TABLE NUMBER
                                OF MAXIMUMS   2   3   4
   KEEP FROM PRECEDING PROBLEM (1=YES)            0      0   0   0
   CARDS INPUT THIS PROBLEM                          16

   OPTION TO CLEAR ENVELOPES BEFORE LANE LOADINGS (1=YES)            0

   OPTION TO OMIT PRINT FOR TABLES (TABLE DESIGNATIONS IN PARENTHESES)
     -1(4A), -2(5) -3(4A,5), -4(4A,5,6), -5(4A,5,6,7):            0

   SKEW ANGLE, DEGREES                               30.000


TABLE 2.  CONSTANTS

   NUMBER OF INCREMENTS FOR SLAB AND CAP                          92
   INCREMENT LENGTH, FT                              0.500
   NUMBER OF INCREMENTS FOR MOVABLE LOAD                           20
   START POSITION OF MOVABLE-LOAD STA ZERO                         2
   STOP POSITION OF MOVABLE-LOAD STA ZERO                         70
   NUMBER OF INCREMENTS BETWEEN EACH POSITION OF MOVABLE LOAD          1

   ANALYSIS OPTION (1=WORKING STRESS, 2=LOAD FACTOR, 3=BOTH)          3

   LOAD FACTOR FOR DEAD LOAD                          1.25
   LOAD FACTOR FOR OVERLAY LOAD                         1.50
   LOAD FACTOR FOR LIVE LOAD                          1.75

   MAXIMUM NUMBER OF LANES TO BE LOADED SIMULTANEOUSLY              3

   LIST OF LOAD COEFFICIENTS CORRESPONDING TO NUMBER OF LANES LOADED
       1          2          3          4          5
     1.200      1.000      0.850
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 3. LISTS OF STATIONS

	NUM OF LANES	NUM OF STRINGERS	NUM OF SUPPORTS	NUM MOM CONTR PTS	NUM SHEAR CONTR PTS
TOTAL	3	6	4	11	8

LANE LEFT 2 32 60

LANE RIGHT 32 60 90
STRINGERS 6.0 22.0 38.0 54.0 70.0 86.0
SUPPORTS 10 34 58 82
MOM CONTR 6 10 22 34 38 46 54 58 70 82
86
SHEAR CONTR 8 12 32 36 56 60 80 84

TABLE 4. STIFFNESS AND LOAD DATA

STA FROM	STA TO	FIXED-OR-MOVABLE CONTD IF=1	CAP BENDING STIFFNESS (K-FT*FT)	SIDEWALK, SLAB LOADS (K)	STRINGER, CAP LOADS (K)	OVERLAY LOADS (K)	MOVABLE-POSITION SLAB LOADS (K)
2	90	0	80900000.000	0.000	-2.427	0.000	0.000
6	6	0	0.000	0.000	-50.170	-5.040	0.000
22	22	0	0.000	0.000	-50.170	-5.040	0.000
38	38	0	0.000	0.000	-50.170	-5.040	0.000
54	54	0	0.000	0.000	-50.170	-5.040	0.000
70	70	0	0.000	0.000	-50.170	-5.040	0.000
86	86	0	0.000	0.000	-50.170	-5.040	0.000
6	6	0	0.000	0.000	-104.100	-10.500	0.000
22	22	0	0.000	0.000	-104.100	-10.500	0.000
38	38	0	0.000	0.000	-104.100	-10.500	0.000
54	54	0	0.000	0.000	-104.100	-10.500	0.000
70	70	0	0.000	0.000	-104.100	-10.500	0.000
86	86	0	0.000	0.000	-104.100	-10.500	0.000
0	20	0	0.000	0.000	0.000	0.000	-4.920
4	4	0	0.000	0.000	0.000	0.000	-21.300
16	16	0	0.000	0.000	0.000	0.000	-21.300

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 4A. DEAD LOAD RESULTS (WORKING STRESS)

STA	DIST X (FT)	DEFLECTION (FT)	MOMENT (K-FT)	SHEAR (K)
-1	-0.58	0.000000	0.0	0.0
0	0.00	0.000000	0.0	0.0
1	0.58	-0.000051	0.0	0.0
2	1.15	-0.000045	0.0	-0.7
3	1.73	-0.000039	-0.8	-2.8
4	2.31	-0.000032	-3.2	-5.6
5	2.89	-0.000026	-7.3	-8.4
6	3.46	-0.000020	-12.9	-96.1
7	4.04	-0.000014	-118.3	-183.8
8	4.62	-0.000008	-225.2	-186.6
9	5.20	-0.000003	-333.8	-189.4
10	5.77	0.000000	-443.9	-34.4
11	6.35	0.000002	-373.5	120.6
12	6.93	0.000002	-304.7	117.8
13	7.51	0.000000	-237.5	115.0
14	8.08	-0.000002	-172.0	112.2
15	8.66	-0.000005	-108.0	109.4
16	9.24	-0.000008	-45.7	106.6
17	9.81	-0.000012	15.0	103.8
18	10.39	-0.000015	74.1	101.0
19	10.97	-0.000018	131.6	98.2
20	11.55	-0.000021	187.5	95.4
21	12.12	-0.000023	241.7	92.6
22	12.70	-0.000024	294.4	4.8
23	13.28	-0.000024	247.3	-82.9
24	13.86	-0.000022	198.7	-85.7
25	14.43	-0.000020	148.4	-88.5
26	15.01	-0.000017	96.5	-91.3
27	15.59	-0.000014	43.0	-94.1
28	16.17	-0.000011	-12.1	-96.9
29	16.74	-0.000007	-68.8	-99.7
30	17.32	-0.000004	-127.2	-102.5
31	17.90	-0.000002	-187.2	-105.3
32	18.48	0.000000	-248.8	-108.1
33	19.05	0.000001	-312.0	-110.9
34	19.63	0.000000	-376.8	44.9
35	20.21	-0.000002	-260.2	200.6
36	20.78	-0.000005	-145.1	197.8
37	21.36	-0.000009	-31.7	195.0
38	21.94	-0.000013	80.1	107.3
39	22.52	-0.000017	92.2	19.6
40	23.09	-0.000020	102.7	16.8
41	23.67	-0.000023	111.6	14.0
42	24.25	-0.000025	118.9	11.2
43	24.83	-0.000027	124.6	8.4

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 4A. DEAD LOAD RESULTS (WORKING STRESS)

STA	DIST X (FT)	DEFLECTION (FT)	MOMENT (K-FT)	SHEAR (K)
44	25.40	-0.000028	128.6	5.6
45	25.98	-0.000029	131.0	2.8
46	26.56	-0.000029	131.8	0.0
47	27.14	-0.000029	131.0	-2.8
48	27.71	-0.000028	128.6	-5.6
49	28.29	-0.000027	124.6	-8.4
50	28.87	-0.000025	118.9	-11.2
51	29.44	-0.000023	111.6	-14.0
52	30.02	-0.000020	102.7	-16.8
53	30.60	-0.000017	92.2	-19.6
54	31.18	-0.000013	80.1	-107.3
55	31.75	-0.000009	-31.7	-195.0
56	32.33	-0.000005	-145.1	-197.8
57	32.91	-0.000002	-260.2	-200.6
58	33.49	0.000000	-376.8	-44.9
59	34.06	0.000001	-312.0	110.9
60	34.64	0.000000	-248.8	108.1
61	35.22	-0.000002	-187.2	105.3
62	35.80	-0.000004	-127.2	102.5
63	36.37	-0.000007	-68.8	99.7
64	36.95	-0.000011	-12.1	96.9
65	37.53	-0.000014	43.0	94.1
66	38.11	-0.000017	96.5	91.3
67	38.68	-0.000020	148.4	88.5
68	39.26	-0.000022	198.7	85.7
69	39.84	-0.000024	247.3	82.9
70	40.41	-0.000024	294.4	-4.8
71	40.99	-0.000023	241.7	-92.6
72	41.57	-0.000021	187.5	-95.4
73	42.15	-0.000018	131.6	-98.2
74	42.72	-0.000015	74.1	-101.0
75	43.30	-0.000012	15.0	-103.8
76	43.88	-0.000008	-45.7	-106.6
77	44.46	-0.000005	-108.0	-109.4
78	45.03	-0.000002	-172.0	-112.2
79	45.61	0.000000	-237.5	-115.0
80	46.19	0.000002	-304.7	-117.8
81	46.77	0.000002	-373.5	-120.6
82	47.34	0.000000	-443.9	34.4
83	47.92	-0.000003	-333.8	189.4
84	48.50	-0.000008	-225.2	186.6
85	49.07	-0.000014	-118.3	183.8
86	49.65	-0.000020	-12.9	96.1
87	50.23	-0.000026	-7.3	8.4
88	50.81	-0.000032	-3.2	5.6
89	51.38	-0.000039	-0.8	2.8
90	51.96	-0.000045	0.0	0.7

```
  91     52.54     -0.000051      0.0       0.0
  92     53.12      0.000000      0.0       0.0
  93     53.69      0.000000      0.0       0.0

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PROB     1  (Spans L=54'-112'-54',  Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)


TABLE 5.  MULTI-LANE LOADING SUMMARY    ( WORKING STRESS )
       ( *--CRITICAL NUMBER OF LANE LOADS)

   MOMENT ( FT-K )
   -----------------------------------------------------------------------
   AT    DEAD LD   LANE   POSITIVE   LOAD AT    LANE   NEGATIVE   LOAD AT
   STA    EFFECT  ORDER  MAXIMUM    LANE STA  ORDER  MAXIMUM    LANE STA
   -----------------------------------------------------------------------

    6    -12.9
            0     0.0            0     0.0
            1     0.0            1     0.0
            2     0.0            2     0.0
            3     0.0            3     0.0
            0*                   0*

   10   -443.9
            0     0.0            0    -203.5     1   2
            1     0.0            1    -203.5     1   2
            2     0.0            2      0.0
            3     0.0            3      0.0
            0*                   0*

   22    294.4
            0   233.3    0  13   0     -38.5     2  36
            1   232.3    1  12   1     -38.5     2  36
            2    10.8    3  62   2       0.0
            3     0.0            3     0.0
            0*                   0*

   34   -376.8
            0    21.6    3  62   0    -157.4     0  18
            1    21.6    3  62   1    -134.6     1  12
            2     0.0            2     -97.8    2  32
            3     0.0            3     0.0
            0*                   2*

   38     80.1
            0    96.5    2  32   0     -67.9     1   9
            1    96.5    2  32   1     -67.9     1   9
            2     3.7    3  62   2       0.0
            3     0.0            3     0.0
            0*                   0*

   46    131.8
            0    80.1    2  36   0     -32.1     1   9
            1    80.1    2  36   1     -32.1     1   9
            2     0.0            2     -32.1    3  63
            3     0.0            3     0.0
            0*                   2*

   54     80.1
            0    96.5    2  40   0     -67.9     3  63
            1    96.5    2  40   1     -67.9     3  63
            2     3.7    1  10   2       0.0
            3     0.0            3     0.0
            0*                   0*

   58   -376.8
            0    21.6    1   9   0    -157.4     0  54
            1    21.6    1   9   1    -134.6     3  60
            2     0.0            2     -97.8    2  40
            3     0.0            3     0.0
            0*                   2*
```

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PROB     1   (Spans L=54'-112'-54',  Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

    MOMENT ( FT-K )
    ----------------------------------------------------------------------
    AT    DEAD LD   LANE   POSITIVE   LOAD AT    LANE   NEGATIVE   LOAD AT
    STA    EFFECT   ORDER  MAXIMUM    LANE STA   ORDER  MAXIMUM    LANE STA
    ----------------------------------------------------------------------


    70    294.4
             0     233.3    0  59    0     -38.5     2  36
             1     232.3    3  60    1     -38.5     2  36
             2      10.8    1   9    2       0.0
             3       0.0             3       0.0
             0*                      0*

    82   -443.9
             0       0.0             0    -203.5     3  70
             1       0.0             1    -203.5     3  70
             2       0.0             2       0.0
             3       0.0             3       0.0
             0*                      0*

    86    -12.9
             0       0.0             0       0.0
             1       0.0             1       0.0
             2       0.0             2       0.0
             3       0.0             3       0.0
             0*                      0*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

SHEAR (K)

AT STA	DEAD LD EFFECT	LANE ORDER	POSITIVE MAXIMUM	LOAD AT LANE STA	LANE ORDER	NEGATIVE MAXIMUM	LOAD AT LANE STA
8	-186.6						
		0	0.0		0	-88.1	1 2
		1	0.0		1	-88.1	1 2
		2	0.0		2	0.0	
		3	0.0		3	0.0	
		0*			0*		
12	117.8						
		0	44.8	1 6	0	-5.6	2 36
		1	44.8	1 6	1	-5.6	2 36
		2	1.6	3 62	2	0.0	
		3	0.0		3	0.0	
		0*			0*		
32	-108.1						
		0	1.6	3 62	0	-54.6	0 15
		1	1.6	3 62	1	-53.0	1 12
		2	0.0		2	-11.2	2 32
		3	0.0		3	0.0	
		0*			0*		
36	197.8						
		0	87.6	0 28	0	-7.8	3 63
		1	84.1	2 32	1	-7.8	3 63
		2	30.7	1 12	2	0.0	
		3	0.0		3	0.0	
		2*			0*		
56	-197.8						
		0	7.8	1 9	0	-87.6	0 44
		1	7.8	1 9	1	-84.1	2 40
		2	0.0		2	-30.7	3 60
		3	0.0		3	0.0	
		0*			2*		
60	108.1						
		0	54.6	0 57	0	-1.6	1 9
		1	53.0	3 60	1	-1.6	1 9
		2	11.2	2 40	2	0.0	
		3	0.0		3	0.0	
		0*			0*		
80	-117.8						
		0	5.6	2 36	0	-44.8	3 66
		1	5.6	2 36	1	-44.8	3 66
		2	0.0		2	-1.6	1 9
		3	0.0		3	0.0	
		0*			0*		
84	186.6						
		0	88.1	3 70	0	0.0	
		1	88.1	3 70	1	0.0	
		2	0.0		2	0.0	
		3	0.0		3	0.0	
		0*			0*		

```
PROB     1   (Spans L=54'-112'-54',  Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

   REACTION ( K )
   ---------------------------------------------------------------------
   AT    DEAD LD   LANE   POSITIVE   LOAD AT    LANE   NEGATIVE   LOAD AT
   STA    EFFECT   ORDER  MAXIMUM    LANE STA   ORDER  MAXIMUM    LANE STA
   ---------------------------------------------------------------------

   10    315.6
             0     127.9     1   2    0     -5.6     2  36
             1     127.9     1   2    1     -5.6     2  36
             2       1.6     3  62    2      0.0
             3       0.0              3      0.0
             0*                       0*

   34    317.1
             0     117.1     0  22    0     -9.3     3  63
             1      95.3     2  32    1     -9.3     3  63
             2      83.6     1  12    2      0.0
             3       0.0              3      0.0
             2*                       0*

   58    317.1
             0     117.1     0  50    0     -9.3     1   9
             1      95.3     2  40    1     -9.3     1   9
             2      83.6     3  60    2      0.0
             3       0.0              3      0.0
             2*                       0*

   82    315.6
             0     127.9     3  70    0     -5.6     2  36
             1     127.9     3  70    1     -5.6     2  36
             2       1.6     1   9    2      0.0
             3       0.0              3      0.0
             0*                       0*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
-1	-0.58	0.0	0.0	0.0	0.0
0	0.00	0.0	0.0	0.0	0.0
1	0.58	0.0	0.0	0.0	0.0
2	1.15	0.0	0.0	-0.7	-0.7
3	1.73	-0.8	-0.8	-2.8	-2.8
4	2.31	-3.2	-3.2	-5.6	-5.6
5	2.89	-7.3	-7.3	-8.4	-8.4
6	3.46	-12.9	-12.9	-96.1	-149.0
7	4.04	-118.3	-179.3	-183.8	-289.6
8	4.62	-225.2	-347.3	-186.6	-292.4
9	5.20	-333.8	-516.9	-189.4	-295.2
10	5.77	-443.9	-688.2	-17.5	-63.5
11	6.35	-358.4	-590.2	174.4	113.9
12	6.93	-269.0	-493.8	171.6	111.1
13	7.51	-180.6	-399.1	168.8	108.3
14	8.08	-93.8	-306.0	166.0	105.5
15	8.66	-8.3	-214.5	163.2	102.7
16	9.24	77.1	-124.7	160.4	99.9
17	9.81	162.1	-36.4	157.6	97.1
18	10.39	246.6	43.3	154.7	94.3
19	10.97	330.0	97.0	151.9	91.5
20	11.55	412.6	149.0	149.1	88.7
21	12.12	493.9	199.4	146.3	85.9
22	12.70	574.3	248.1	20.8	-8.3
23	13.28	489.9	196.8	-81.0	-148.4
24	13.86	404.1	143.7	-83.8	-151.2
25	14.43	317.1	88.7	-86.6	-154.0
26	15.01	228.7	31.9	-89.4	-156.8
27	15.59	139.6	-26.9	-92.2	-159.6
28	16.17	56.3	-87.4	-95.0	-162.4
29	16.74	-27.0	-149.4	-97.8	-165.2
30	17.32	-105.6	-213.6	-100.6	-168.0
31	17.90	-164.5	-308.5	-103.4	-170.8
32	18.48	-225.0	-407.1	-106.2	-173.6
33	19.05	-287.2	-507.4	-109.0	-176.4
34	19.63	-350.9	-609.2	88.7	27.3
35	20.21	-239.6	-426.3	315.4	191.3
36	20.78	-130.0	-265.4	312.6	188.5
37	21.36	26.8	-129.9	309.8	185.7
38	21.94	195.9	-1.5	165.8	98.0
39	22.52	204.6	16.1	28.9	10.3
40	23.09	212.0	31.9	26.1	7.5
41	23.67	218.1	46.2	23.3	4.7
42	24.25	223.0	54.6	20.5	1.9

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
43	24.83	226.3	60.3	17.7	-0.9
44	25.40	227.9	64.3	14.9	-3.7
45	25.98	227.9	66.8	12.1	-6.5
46	26.56	227.9	67.6	9.3	-9.3
47	27.14	227.9	66.8	6.5	-12.1
48	27.71	227.9	64.3	3.7	-14.9
49	28.29	226.3	60.3	0.9	-17.7
50	28.87	223.0	54.6	-1.9	-20.5
51	29.44	218.1	46.2	-4.7	-23.3
52	30.02	212.0	31.9	-7.5	-26.1
53	30.60	204.6	16.1	-10.3	-28.9
54	31.18	195.9	-1.5	-98.0	-165.8
55	31.75	26.8	-129.9	-185.7	-309.8
56	32.33	-130.0	-265.4	-188.5	-312.6
57	32.91	-239.6	-426.3	-191.3	-315.4
58	33.49	-350.9	-609.2	-27.3	-88.7
59	34.06	-287.2	-507.4	176.4	109.0
60	34.64	-225.0	-407.1	173.6	106.2
61	35.22	-164.5	-308.5	170.8	103.4
62	35.80	-105.6	-213.6	168.0	100.6
63	36.37	-27.0	-149.4	165.2	97.8
64	36.95	56.3	-87.4	162.4	95.0
65	37.53	139.6	-26.9	159.6	92.2
66	38.11	228.7	31.9	156.8	89.4
67	38.68	317.1	88.7	154.0	86.6
68	39.26	404.1	143.7	151.2	83.8
69	39.84	489.9	196.8	148.4	81.0
70	40.41	574.3	248.1	8.3	-20.8
71	40.99	493.9	199.4	-85.9	-146.3
72	41.57	412.6	149.0	-88.7	-149.1
73	42.15	330.0	97.0	-91.5	-151.9
74	42.72	246.6	43.3	-94.3	-154.7
75	43.30	162.1	-36.4	-97.1	-157.6
76	43.88	77.1	-124.7	-99.9	-160.4
77	44.46	-8.3	-214.5	-102.7	-163.2
78	45.03	-93.8	-306.0	-105.5	-166.0
79	45.61	-180.6	-399.1	-108.3	-168.8
80	46.19	-269.0	-493.8	-111.1	-171.6
81	46.77	-358.4	-590.2	-113.9	-174.4
82	47.34	-443.9	-688.2	63.5	17.5
83	47.92	-333.8	-516.9	295.2	189.4
84	48.50	-225.2	-347.3	292.4	186.6
85	49.07	-118.3	-179.3	289.6	183.8
86	49.65	-12.9	-12.9	149.0	96.1

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
87	50.23	-7.3	-7.3	8.4	8.4
88	50.81	-3.2	-3.2	5.6	5.6
89	51.38	-0.8	-0.8	2.8	2.8
90	51.96	0.0	0.0	0.7	0.7
91	52.54	0.0	0.0	0.0	0.0
92	53.12	0.0	0.0	0.0	0.0
93	53.69	0.0	0.0	0.0	0.0

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 7. MAXIMUM SUPPORT REACTIONS (WORKING STRESS)

STA	DIST X (FT)	MAX + REACT (K)	MAX - REACT (K)
10	5.77	469.1	308.9
34	19.63	496.0	306.0
58	33.49	496.0	306.0
82	47.34	469.1	308.9

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 5. MULTI-LANE LOADING SUMMARY (LOAD FACTOR)
(*--CRITICAL NUMBER OF LANE LOADS)

```
MOMENT ( FT-K )
--------------------------------------------------------------------------
AT   DEAD LD  LANE  POSITIVE  LOAD AT   LANE  NEGATIVE  LOAD AT
STA  EFFECT  ORDER  MAXIMUM   LANE STA  ORDER  MAXIMUM   LANE STA
--------------------------------------------------------------------------

 6   -16.2
        0    0.0          0    0.0
        1    0.0          1    0.0
        2    0.0          2    0.0
        3    0.0          3    0.0
        0*                0*

10   -563.9
        0    0.0          0   -356.2   1  2
        1    0.0          1   -356.2   1  2
        2    0.0          2     0.0
        3    0.0          3     0.0
        0*                0*

22    373.6
        0  408.2   0 13   0    -67.4   2 36
        1  406.5   1 12   1    -67.4   2 36
        2   18.9   3 62   2     0.0
        3    0.0          3    0.0
        0*                0*

34   -477.8
        0   37.8   3 62   0   -275.4   0 18
        1   37.8   3 62   1   -235.5   1 12
        2    0.0          2  -171.1   2 32
        3    0.0          3    0.0
        0*                2*

38    102.3
        0  168.9   2 32   0   -118.9   1  9
        1  168.9   2 32   1   -118.9   1  9
        2    6.5  3 62    2     0.0
        3    0.0          3    0.0
        0*                0*

46    167.0
        0  140.1   2 36   0    -56.2   1  9
        1  140.1   2 36   1    -56.2   1  9
        2    0.0          2   -56.2   3 63
        3    0.0          3    0.0
        0*                2*

54    102.3
        0  168.9   2 40   0   -118.9   3 63
        1  168.9   2 40   1   -118.9   3 63
        2    6.5  1 10    2     0.0
        3    0.0          3    0.0
        0*                0*

58   -477.8
        0   37.8   1  9   0   -275.4   0 54
        1   37.8   1  9   1   -235.5   3 60
        2    0.0          2  -171.1   2 40
        3    0.0          3    0.0
        0*                2*
```

PROB   1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

```
   MOMENT ( FT-K )
   ------------------------------------------------------------------------
   AT    DEAD LD   LANE   POSITIVE   LOAD AT    LANE   NEGATIVE   LOAD AT
   STA    EFFECT   ORDER  MAXIMUM    LANE STA   ORDER  MAXIMUM    LANE STA
   ------------------------------------------------------------------------

    70    373.6
             0    408.2    0  59    0     -67.4     2  36
             1    406.5    3  60    1     -67.4     2  36
             2     18.9    1   9    2       0.0
             3      0.0             3       0.0
             0*                     0*

    82   -563.9
             0      0.0             0    -356.2     3  70
             1      0.0             1    -356.2     3  70
             2      0.0             2       0.0
             3      0.0             3       0.0
             0*                     0*

    86    -16.2
             0      0.0             0       0.0
             1      0.0             1       0.0
             2      0.0             2       0.0
             3      0.0             3       0.0
             0*                     0*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

SHEAR (K)

AT STA	DEAD LD EFFECT	LANE ORDER	POSITIVE MAXIMUM	LOAD AT LANE	STA	LANE ORDER	NEGATIVE MAXIMUM	LOAD AT LANE	STA
8	-237.2								
		0	0.0			0	-154.2	1	2
		1	0.0			1	-154.2	1	2
		2	0.0			2	0.0		
		3	0.0			3	0.0		
		0*				0*			
12	149.3								
		0	78.4	1	6	0	-9.7	2	36
		1	78.4	1	6	1	-9.7	2	36
		2	2.7	3	62	2	0.0		
		3	0.0			3	0.0		
		0*				0*			
32	-136.9								
		0	2.7	3	62	0	-95.6	0	15
		1	2.7	3	62	1	-92.7	1	12
		2	0.0			2	-19.5	2	32
		3	0.0			3	0.0		
		0*				0*			
36	251.2								
		0	153.2	0	28	0	-13.6	3	63
		1	147.2	2	32	1	-13.6	3	63
		2	53.7	1	12	2	0.0		
		3	0.0			3	0.0		
		2*				0*			
56	-251.2								
		0	13.6	1	9	0	-153.2	0	44
		1	13.6	1	9	1	-147.2	2	40
		2	0.0			2	-53.7	3	60
		3	0.0			3	0.0		
		0*				2*			
60	136.9								
		0	95.6	0	57	0	-2.7	1	9
		1	92.7	3	60	1	-2.7	1	9
		2	19.5	2	40	2	0.0		
		3	0.0			3	0.0		
		0*				0*			
80	-149.3								
		0	9.7	2	36	0	-78.4	3	66
		1	9.7	2	36	1	-78.4	3	66
		2	0.0			2	-2.7	1	9
		3	0.0			3	0.0		
		0*				0*			
84	237.2								
		0	154.2	3	70	0	0.0		
		1	154.2	3	70	1	0.0		
		2	0.0			2	0.0		
		3	0.0			3	0.0		
		0*				0*			

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

```
REACTION ( K )
----------------------------------------------------------------------
AT    DEAD LD  LANE  POSITIVE  LOAD AT   LANE  NEGATIVE  LOAD AT
STA    EFFECT  ORDER MAXIMUM   LANE STA  ORDER MAXIMUM   LANE STA
----------------------------------------------------------------------

 10   400.5
         0    223.8    1  2    0     -9.7    2  36
         1    223.8    1  2    1     -9.7    2  36
         2      2.7  3  62   2      0.0
         3      0.0          3     0.0
         0*                  0*

 34   402.1
         0    205.0    0  22   0     -16.3    3  63
         1    166.8    2  32   1     -16.3    3  63
         2    146.3    1  12   2       0.0
         3      0.0          3     0.0
         2*                  0*

 58   402.1
         0    205.0    0  50   0     -16.3    1   9
         1    166.8    2  40   1     -16.3    1   9
         2    146.3    3  60   2       0.0
         3      0.0          3     0.0
         2*                  0*

 82   400.5
         0    223.8    3  70   0      -9.7   2  36
         1    223.8    3  70   1      -9.7   2  36
         2      2.7   1  9   2      0.0
         3      0.0          3     0.0
         0*                  0*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
-1	-0.58	0.0	0.0	0.0	0.0
0	0.00	0.0	0.0	0.0	0.0
1	0.58	0.0	0.0	0.0	0.0
2	1.15	0.0	0.0	-0.9	-0.9
3	1.73	-1.0	-1.0	-3.5	-3.5
4	2.31	-4.0	-4.0	-7.0	-7.0
5	2.89	-9.1	-9.1	-10.5	-10.5
6	3.46	-16.2	-16.2	-122.1	-214.6
7	4.04	-150.1	-256.9	-233.7	-418.7
8	4.62	-286.0	-499.7	-237.2	-422.2
9	5.20	-423.9	-744.5	-240.7	-425.7
10	5.77	-563.9	-991.3	-14.3	-94.7
11	6.35	-448.1	-853.8	247.0	141.1
12	6.93	-325.0	-718.4	243.5	137.6
13	7.51	-202.7	-585.0	240.0	134.1
14	8.08	-82.3	-453.7	236.4	130.6
15	8.66	36.6	-324.3	232.9	127.1
16	9.24	156.1	-197.0	229.4	123.6
17	9.81	275.7	-71.7	225.9	120.1
18	10.39	395.2	39.5	222.4	116.6
19	10.97	513.7	105.8	218.9	113.1
20	11.55	631.5	170.1	215.4	109.6
21	12.12	747.8	232.4	211.9	106.1
22	12.70	863.4	292.7	34.2	-16.8
23	13.28	738.2	225.3	-102.1	-220.1
24	13.86	611.3	155.8	-105.6	-223.6
25	14.43	483.2	83.6	-109.1	-227.1
26	15.01	353.5	9.0	-112.6	-230.6
27	15.59	223.3	-68.2	-116.1	-234.1
28	16.17	104.0	-147.4	-119.6	-237.6
29	16.74	-14.4	-228.7	-123.1	-241.1
30	17.32	-123.9	-312.9	-126.6	-244.6
31	17.90	-198.0	-450.0	-130.1	-248.1
32	18.48	-274.1	-592.8	-133.6	-251.6
33	19.05	-352.3	-737.6	-137.1	-255.1
34	19.63	-432.5	-884.4	133.8	26.4
35	20.21	-293.8	-620.4	455.6	238.4
36	20.78	-157.1	-394.2	452.1	234.9
37	21.36	62.8	-211.4	448.6	231.4
38	21.94	305.0	-40.3	238.4	119.8
39	22.52	314.2	-15.8	40.8	8.2
40	23.09	321.9	6.8	37.3	4.7
41	23.67	328.2	27.3	33.8	1.2
42	24.25	333.1	38.4	30.3	-2.3
43	24.83	335.9	45.5	26.8	-5.8
44	25.40	336.8	50.6	23.3	-9.3

45	25.98	335.6	53.6	19.8	-12.8
46	26.56	335.2	54.6	16.3	-16.3
47	27.14	335.6	53.6	12.8	-19.8
48	27.71	336.8	50.6	9.3	-23.3

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
49	28.29	335.9	45.5	5.8	-26.8
50	28.87	333.1	38.4	2.3	-30.3
51	29.44	328.2	27.3	-1.2	-33.8
52	30.02	321.9	6.8	-4.7	-37.3
53	30.60	314.2	-15.8	-8.2	-40.8
54	31.18	305.0	-40.3	-119.8	-238.4
55	31.75	62.8	-211.4	-231.4	-448.6
56	32.33	-157.1	-394.2	-234.9	-452.1
57	32.91	-293.8	-620.4	-238.4	-455.6
58	33.49	-432.5	-884.4	-26.4	-133.8
59	34.06	-352.3	-737.6	255.1	137.1
60	34.64	-274.1	-592.8	251.6	133.6
61	35.22	-198.0	-450.0	248.1	130.1
62	35.80	-123.9	-312.9	244.6	126.6
63	36.37	-14.4	-228.7	241.1	123.1
64	36.95	104.0	-147.4	237.6	119.6
65	37.53	223.3	-68.2	234.1	116.1
66	38.11	353.5	9.0	230.6	112.6
67	38.68	483.2	83.6	227.1	109.1
68	39.26	611.3	155.8	223.6	105.6
69	39.84	738.2	225.3	220.1	102.1
70	40.41	863.4	292.7	16.8	-34.2
71	40.99	747.8	232.4	-106.1	-211.9
72	41.57	631.5	170.1	-109.6	-215.4
73	42.15	513.7	105.8	-113.1	-218.9
74	42.72	395.2	39.5	-116.6	-222.4
75	43.30	275.7	-71.7	-120.1	-225.9
76	43.88	156.1	-197.0	-123.6	-229.4
77	44.46	36.6	-324.3	-127.1	-232.9
78	45.03	-82.3	-453.7	-130.6	-236.4
79	45.61	-202.7	-585.0	-134.1	-240.0
80	46.19	-325.0	-718.4	-137.6	-243.5
81	46.77	-448.1	-853.8	-141.1	-247.0
82	47.34	-563.9	-991.3	94.7	14.3
83	47.92	-423.9	-744.5	425.7	240.7
84	48.50	-286.0	-499.7	422.2	237.2
85	49.07	-150.1	-256.9	418.7	233.7
86	49.65	-16.2	-16.2	214.6	122.1
87	50.23	-9.1	-9.1	10.5	10.5
88	50.81	-4.0	-4.0	7.0	7.0
89	51.38	-1.0	-1.0	3.5	3.5

90	51.96	0.0	0.0	0.9	0.9
91	52.54	0.0	0.0	0.0	0.0
92	53.12	0.0	0.0	0.0	0.0

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
93	53.69	0.0	0.0	0.0	0.0

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la
(CONTINUED)

TABLE 7. MAXIMUM SUPPORT REACTIONS (LOAD FACTOR)

STA	DIST X (FT)	MAX + REACT (K)	MAX - REACT (K)
10	5.77	669.0	388.8
34	19.63	715.2	382.5
58	33.49	715.2	382.5
82	47.34	669.0	388.8

4.3.15.4 Live Load Distribution Factor Spreadsheet

4.3.15.4.1 *Spans 1 & 3*

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20
BRIDGE	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:	
DIVISION	Descrip:	ITBC Design Example 2, Span 1 & 3			File:	Ex2_Span1_distribution_factors.xl	Sheet:	1 of 8

2017 LRFD Specs
Rev. 10/18 - (No Interim)

LRFD Live Load Distribution Factors*

Live Load Distribution Factors are calculated according to AASHTO LRFD Bridge Design Specifications, 8th Edition (2017 with no interim revisions) as prescribed by TxDOT policies (LRFD Design Manual July 2018) and practices. The Lever Rule is used when outside the Range of Applicability. The Range of Applicability for the Skew Correction Factors is ignored.

INPUT:

Beam Type =	Tx54	
No. Beams, N_b =	6	
CL_{brg} to CL_{brg}, L =	50.38	ft
Beam Spacing, S =	8.00	ft
Avg. Skew Angle, θ =	30.00	deg
Slab Thickness, t_s =	8.00	in
Slab Overhang, OH =	3	ft
Rail Width, RW =	1	ft
Roadway Width, W =	44	ft
Number of Lanes, N_L =	3	

Deck Slab		
Conc wt =	0.145	k/ft^3
f'_c =	4.0	ksi
E_{slab} =	3644	ksi

Beam		
weight =	0.145	k/ft^3
f'_c =	8.5	ksi
E_{beam} =	5312	ksi
y_t =	30.49	in
A =	817.0	in^2
I =	299740	in^4

Longitudinal Stiffness Parameter: (4.6.2.2.1-1)

e_g (in) = 34.49 (dist. b/w cog of bm & deck)

n = 1.000

$K_g = n(I + Ae_g^2)$ = 1271611 in^4

*For typical cross sections (a,e,i,j & k), Table 4.6.2.2.1-1

RESULTS:

	Final LLDF
Interior Shear LLDF, $gV_{interior}$	0.876
Interior Moment LLDF, $gM_{interior}$	0.745
Exterior Shear LLDF, $gV_{exterior}$	0.876
Exterior Moment LLDF, $gM_{exterior}$	0.745

The Final LLDF may be modified according to the following TxDOT policies:
- Exterior beams use the interior LLDF when OH ≤ S/2.
- When OH > S/2 the exterior beam LLDF is determined by the lever rule for a single lane with a multiple presence factor of 1.0.
- In no case shall the LLDF for the exterior beams be less than the LLDFs for the interior beams.
- When the Roadway width is less than 20ft, all beams are designed for one lane loaded only.
- In no case shall the LLDF be less than m N_L/N_b.

CALCULATIONS:

Shear LLDF Correction for Skew (Table 4.6.2.2.3c-1)

$$\text{Corr.} = 1.0 + 0.20\left(\frac{12.0Lt_s^{\,3}}{K_g}\right)^{0.3}\tan\theta$$

= 1.0 + 0.20 * [(12.0*50.4*8^3)/(1,271,611)]^0.3 * tan(30)

Corr. = 1.076

Check θ:	0° ≤ 30° ≤ 60°	OK
Check S:	3.5' ≤ 8.0' ≤ 16.0'	OK
Check L:	20' ≤ 50.4' ≤ 240'	OK
Check N_b:	6 ≥ 4	OK

Moment LLDF Correction for Skew (Table 4.6.2.2.2e-1)

Check θ: 30° ≤ 30° ≤ 60° 30°

Corr. = 1 - c_1(tan θ)^1.5

= 1 - 0.142(tan 30)^1.5

Corr. = 0.938

where: $$c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$$

c_1 = 0.142

INTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3a-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 1.076

mg = 0.750 * 1.076 = 0.807

Equation

$$g = 0.36 + \left(\frac{S}{25}\right)$$

g = 0.36 + (8 / 25) = 0.680

Modify for Skew:

skew correction = 1.076

g = 0.680 * 1.076 = 0.732

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 50.4' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int1} = 0.732

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 1.076

mg = 0.875 * 1.076 = 0.942

Equation

$$g = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0}$$

g = 0.2 + (8 / 12) - (8 / 35)^2.0 = 0.814

Modify for Skew:

skew correction = 1.076

g = 0.814 * 1.076 = 0.876

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int2+} = 0.876

TxDOT Policy states $gV_{interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gV_{interior}$ is the Maximum of: gV_{int1} and $m \cdot N_L \div N_b$.

>> TxDOT Policy states that if W ≥ 20ft, $gV_{interior}$ is the Maximum of: gV_{int1}, gV_{int2+}, $m \cdot N_L \div N_b$.

$gV_{interior}$ = 0.876

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INTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 0.938

mg = 0.750 * 0.938 = 0.704

Equation

$$g = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

g = 0.06 + (8/14)^0.4 * (8/50.4)^0.3 * (1,271,611/(12*50.4*8^3))^0.1 = 0.590

Modify for Skew:

skew correction = 0.938

g = 0.590 * 0.938 = 0.553

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 50.4' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Check K_g: 10,000 ≤ 1,271,611 ≤ 7,000,000 OK

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int1} = 0.553

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 0.938

mg = 0.875 * 0.938 = 0.821

Equation

$$g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

g = 0.075 + (8/9.5)^0.6 * (8/50.4)^0.2 * (1,271,611/(12*50.4*8^3))^0.1 = 0.794

Modify for Skew:

skew correction = 0.938

g = 0.794 * 0.938 = 0.745

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int2+} = 0.745

TxDOT Policy states $gM_{interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gM_{interior}$ is the Maximum of: gM_{int1} and $m \cdot N_L \div N_b$.

>> TxDOT Policy states that if W ≥ 20ft, $gM_{interior}$ is the Maximum of: gM_{int1}, gM_{int2+}, $m \cdot N_L \div N_b$.

$gM_{interior}$ = 0.745

EXTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.0 = 0.625

TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 1.076

mg = 0.625 * 1.076 = 0.673

Use Lever Rule, as per AASHTO LRFD Table 4.6.2.2.3b-1.

gV_{ext1} = 0.673

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 1.076

mg = 0.625 * 1.076 = 0.673

Equation

d_e = dist. b/w CL web to curb

d_e = OH - Rail Width

d_e = 3ft - 1ft = 2.0 ft

$$e = 0.6 + \left(\frac{d_e}{10}\right)$$

e = 0.6 + (2.0/10) = 0.800

$g = e^*gV_{int2+Eq}$

g = 0.800 * 0.876 = 0.701

Skew Correction is included in gV(interior).

Range of Applicability (ROA) Checks

Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: $-1.0' \le 2.0' \le 5.5'$ OK

Check N_b: $6 \neq 3$ OK

Use Equation from Table 4.6.2.2.3b-1 because all criteria is OK.

gV_{ext2+} = 0.701

TxDOT Policy states $gV_{Exterior}$ must be $\ge gV_{Interior}$

$gV_{interior}$ = 0.876

TxDOT Policy states $gV_{Exterior}$ must be $\ge m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

>> TxDOT Policy states that if OH ≤ S/2, $gV_{Exterior}$ is $gV_{Interior}$.

TxDOT Policy states that if OH > S/2 and W < 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, $gV_{Interior}$, and $m \cdot N_L \div N_b$.

TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, gV_{ext2+}, $gV_{Interior}$, and $m \cdot N_L \div N_b$.

$gV_{exterior}$ = 0.876

EXTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2d-1):

One Lane Loaded

Lever Rule

mg = 0.625 * 1.0 = 0.625 — TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 0.938

mg = 0.625 * 0.938 = 0.586

Use Lever Rule as per AASHTO LRFD Table 4.6.2.2.2d-1.

gM_{ext1} = 0.586

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 0.938

mg = 0.625 * 0.938 = 0.586

Equation

$$e = 0.77 + \left(\frac{d_e}{9.1}\right)$$

e = 0.77 + (2.0/9.1) = 0.990

$g = e^*gM_{int2+Eq}$

g = 0.99 * 0.745 = 0.738

Skew Correction included in gM(interior).

Range of Applicability (ROA) Checks — Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: $-1.0' \le 2.0' \le 5.5'$ OK

Check N_b: $6 \neq 3$ OK

Use Equation from Table 4.6.2.2.2d-1 because all criteria is OK.

gM_{ext2+} = 0.738

TxDOT Policy states $gM_{Exterior}$ must be $\ge gM_{Interior}$

$gM_{interior}$ = 0.745

TxDOT Policy states $gM_{Exterior}$ must be $\ge m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

>> TxDOT Policy states that if OH ≤ S/2, $gM_{Exterior}$ is $gM_{Interior}$.

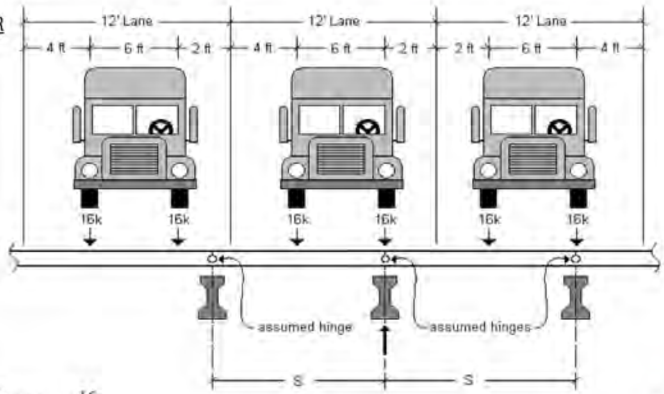
TxDOT Policy states that if OH > S/2 and W < 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, $gM_{Interior}$, and $m \cdot N_L \div N_b$.

TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, gM_{ext2+}, $gM_{Interior}$, and $m \cdot N_L \div N_b$.

$gM_{exterior}$ = 0.745

LEVER RULE S = 8.0 ft

INTERIOR



For S < 4:

One Lane = $\frac{16}{32}$ = 0.500

For 4 ≤ S < 6:

One Lane = $\frac{16}{32}$ = 0.500

Two Lanes = $\frac{16}{32}\left(1+\frac{S-4}{S}\right)$ = 0.750

>: **For 6 ≤ S < 10:**

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}\right)$ = 0.875

For 10 ≤ S < 12:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

For 12 ≤ S < 16:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

For 16 ≤ S < 18:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-16}{S}\right)$ = 0.000

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LEVER RULE S = 8.0 ft

INTERIOR (con't)

For 18 ≤ S < 22:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}\right)$ = -0.625

For 22 ≤ S ≤ 24:

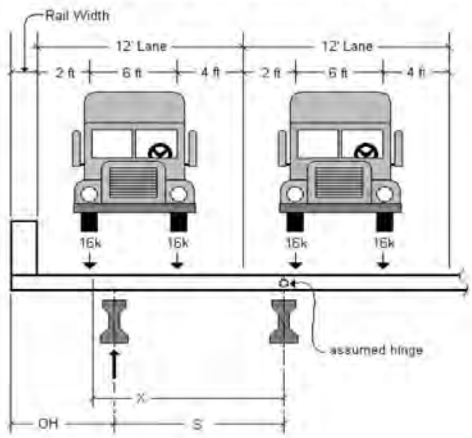
One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}+\frac{S-22}{S}\right)$ = -1.500

EXTERIOR



S = 8.0 ft
OH = 3.0 ft
Rail Width = RW = 1.0 ft
X = S+OH-RW-2ft = 8.0 ft

For X < 6:

One Lane = $\frac{16}{32}\left(\frac{X}{S}\right)$ = 0.500

>: **For 6 ≤ X < 12:**

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

For 12 ≤ X < 18:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}\right)$ = 0.375

LEVER RULE

EXTERIOR (con't) S = 8.0 ft OH = 3.0 ft

RW = 1.0 ft X = S+OH-RW-2ft = 8.0 ft

For 18 ≤ X < 24:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

For 24 ≤ X < 30:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}\right)$ = -1.250

For 30 ≤ X < 36:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

For 36 ≤ X < 42:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}\right)$ = -4.375

For 42 ≤ X ≤ 48:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}+\frac{X-42}{S}\right)$ = -6.500

INTERIOR			EXTERIOR		
One Lane Loaded	=	0.625	One Lane Loaded	=	0.625
Two Lanes Loaded	=	0.875	Two Lanes Loaded	=	0.625
Three Lanes Loaded	=	0.875	Three Lanes Loaded	=	0.625
Four Lanes Loaded	=	0.875	Four Lanes Loaded	=	0.625

4.3.15.4.2 *Span 2*

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LRFD Live Load Distribution Factors*

Live Load Distribution Factors are calculated according to AASHTO LRFD Bridge Design Specifications, 8th Edition (2017 with no interim revisions) as prescribed by TxDOT policies (LRFD Design Manual July 2018) and practices. The Lever Rule is used when outside the Range of Applicability. The Range of Applicability for the Skew Correction Factors is ignored.

INPUT:

Beam Type =	Tx54	
No. Beams, N_b =	6	
CL_{brg} to CL_{brg}, L =	106.75	ft
Beam Spacing, S =	8.00	ft
Avg. Skew Angle, θ =	30.00	deg
Slab Thickness, t_s =	8.00	in
Slab Overhang, OH =	3	ft
Rail Width, RW =	1	ft
Roadway Width, W =	44	ft
Number of Lanes, N_L =	3	

Deck Slab		
Conc wt =	0.145	k/ft^3
f'$_c$ =	4.0	ksi
E_{slab} =	3644	ksi

Beam		
weight =	0.145	k/ft^3
f'$_c$ =	8.5	ksi
E_{beam} =	5312	ksi
y_t =	30.49	in
A =	817.0	in^2
I =	299740	in^4

Longitudinal Stiffness Parameter: (4.6.2.2.1-1)

e_g (in) = 34.49 (dist. b/w cog of bm & deck)

n = 1.000

$K_g = n(I+Ae_g^2)$ = 1271611 in^4

*For typical cross sections (a,e,i,j & k). Table 4.6.2.2.1-1

RESULTS:

	Final LLDF
Interior Shear LLDF, $gV_{interior}$	0.891
Interior Moment LLDF, $gM_{interior}$	0.626
Exterior Shear LLDF, $gV_{exterior}$	0.891
Exterior Moment LLDF, $gM_{exterior}$	0.626

The Final LLDF may be modified according to the following TxDOT policies:

- Exterior beams use the interior LLDF when OH ≤ S/2.
- When OH > S/2 the exterior beam LLDF is determined by the lever rule for a single lane with a multiple presence factor of 1.0.
- In no case shall the LLDF for the exterior beams be less than the LLDFs for the interior beams.
- When the Roadway width is less than 20ft, all beams are designed for one lane loaded only.
- In no case shall the LLDF be less than $m \cdot N_L / N_b$

CALCULATIONS:

Shear LLDF Correction for Skew (Table 4.6.2.2.3c-1)

$$\text{Corr.} = 1.0 + 0.20\left(\frac{12.0Lt_s^{\,3}}{K_g}\right)^{0.3}\tan\theta$$

= 1.0 + 0.20 * [(12.0*106.8*8^3)/(1,271,611)]^0.3 * tan(30)

Corr. = 1.095

Check θ:	0° ≤ 30° ≤ 60°	OK
Check S:	3.5' ≤ 8.0' ≤ 16.0'	OK
Check L:	20' ≤ 106.8' ≤ 240'	OK
Check N_b:	6 ≥ 4	OK

Moment LLDF Correction for Skew (Table 4.6.2.2.2e-1)

Check θ: 30° ≤ 30° ≤ 60° 30°

Corr. = 1 - c_1(tan θ)^1.5

= 1 - 0.081(tan 30)^1.5

Corr. = 0.964

where: $$c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$$

c_1 = 0.081

INTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3a-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 1.095

mg = 0.750 * 1.095 = 0.821

Equation

$$g = 0.36 + \left(\frac{S}{25}\right)$$

g = 0.36 + (8 / 25) = 0.680

Modify for Skew:

skew correction = 1.095

g = 0.680 * 1.095 = 0.745

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 106.8' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int1} = 0.745

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 1.095

mg = 0.875 * 1.095 = 0.958

Equation

$$g = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0}$$

g = 0.2 + (8 / 12) - (8 / 35)^2.0 = 0.814

Modify for Skew:

skew correction = 1.095

g = 0.814 * 1.095 = 0.891

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int2+} = 0.891

TxDOT Policy states $gV_{Interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gV_{Interior}$ is the Maximum of: gV_{int1} and $m \cdot N_L \div N_b$.

>> TxDOT Policy states that if W ≥ 20ft, $gV_{Interior}$ is the Maximum of: gV_{int1}, gV_{int2+}, $m \cdot N_L \div N_b$.

$gV_{Interior}$ = 0.891

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INTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 0.964

mg = 0.750 * 0.964 = 0.723

Equation

$$g = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^{\,3}}\right)^{0.1}$$

g = 0.06 + (8/14)^0.4 * (8/106.8)^0.3 * (1,271,611/(12*106.8*8^3))^0.1 = 0.453

Modify for Skew:

skew correction = 0.964

g = 0.453 * 0.964 = 0.437

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 106.8' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Check K_g: 10,000 ≤ 1,271,611 ≤ 7,000,000 OK

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int1} = 0.437

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 0.964

mg = 0.875 * 0.964 = 0.844

Equation

$$g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^{\,3}}\right)^{0.1}$$

g = 0.075 + (8/9.5)^0.6 * (8/106.8)^0.2 * (1,271,611/(12*106.8*8^3))^0.1 = 0.649

Modify for Skew:

skew correction = 0.964

g = 0.649 * 0.964 = 0.626

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int2+} = 0.626

TxDOT Policy states $gM_{Interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gM_{Interior}$ is the Maximum of: gM_{int1} and $m \cdot N_L \div N_b$.

\>> TxDOT Policy states that if W ≥ 20ft, $gM_{Interior}$ is the Maximum of: gM_{int1}, gM_{int2+}, $m \cdot N_L \div N_b$.

$gM_{interior}$ = 0.626

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
BRIDGE	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:		Rev. 10/18 - (No Interim)
DIVISION	Descrip:	ITBC Design Example 2, Span 2			File:	Ex2_Span2_distribution_factors.xl			Sheet: 4 of 8

EXTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.0 = 0.625 — TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 1.095

mg = 0.625 * 1.095 = 0.684

Use Lever Rule, as per AASHTO LRFD Table 4.6.2.2.3b-1.

gV_{ext1} = 0.684

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 1.095

mg = 0.625 * 1.095 = 0.684

Equation

d_e = dist. b/w CL web to curb

d_e = OH - Rail Width

d_e = 3ft - 1ft = 2.0 ft

$$e = 0.6 + \left(\frac{d_e}{10}\right)$$

e = 0.6 + (2.0/10) = 0.800

$g = e{*}gV_{int2+Eq}$

g = 0.800 * 0.891 = 0.713

Skew Correction is included in gV(interior).

Range of Applicability (ROA) Checks — Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: $-1.0' \le 2.0' \le 5.5'$ OK

Check N_b: $6 \ne 3$ OK

Use Equation from Table 4.6.2.2.3b-1 because all criteria is OK.

gV_{ext2+} = 0.713

TxDOT Policy states $gV_{Exterior}$ must be $\ge gV_{Interior}$

$gV_{interior}$ = 0.891

TxDOT Policy states $gV_{Exterior}$ must be $\ge m{\cdot}N_L \div N_b$

$m{\cdot}N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

>> TxDOT Policy states that if OH ≤ S/2, $gV_{Exterior}$ is $gV_{Interior}$.

TxDOT Policy states that if OH > S/2 and W < 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, $gV_{interior}$, and $m{\cdot}N_L \div N_b$.

TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, gV_{ext2+}, $gV_{interior}$, and $m{\cdot}N_L \div N_b$.

$gV_{exterior}$ = 0.891

EXTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2d-1):

One Lane Loaded

Lever Rule

mg = 0.625 * 1.0 = 0.625 — TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 0.964

mg = 0.625 * 0.964 = 0.603

Use Lever Rule as per AASHTO LRFD Table 4.6.2.2.2d-1.

gM_{ext1} = 0.603

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 0.964

mg = 0.625 * 0.964 = 0.603

Equation

$$e = 0.77 + \left(\frac{d_e}{9.1}\right)$$

e = 0.77 + (2.0/9.1) = 0.990

$g = e^{*}gM_{int2+Eq}$

g = 0.99 * 0.626 = 0.620

Skew Correction included in gM(interior).

Range of Applicability (ROA) Checks — Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: $-1.0' \le 2.0' \le 5.5'$ OK

Check N_b: $6 \neq 3$ OK

Use Equation from Table 4.6.2.2.2d-1 because all criteria is OK.

gM_{ext2+} = 0.620

TxDOT Policy states $gM_{Exterior}$ must be $\ge gM_{Interior}$

$gM_{interior}$ = 0.626

TxDOT Policy states $gM_{Exterior}$ must be $\ge m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

>> TxDOT Policy states that if OH ≤ S/2, $gM_{Exterior}$ is $gM_{Interior}$.

TxDOT Policy states that if OH > S/2 and W < 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, $gM_{Interior}$, and $m \cdot N_L \div N_b$.

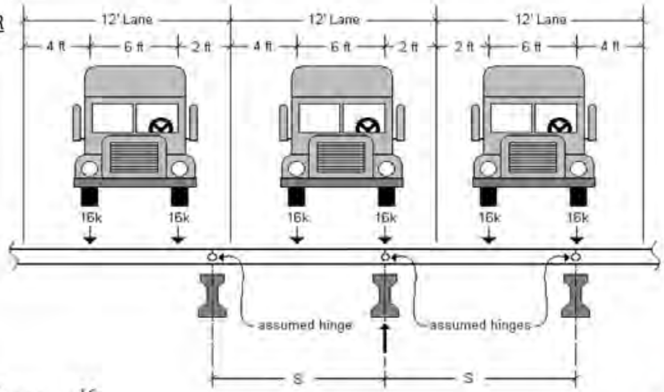
TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, gM_{ext2+}, $gM_{Interior}$, and $m \cdot N_L \div N_b$.

$gM_{exterior}$ = 0.626

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
BRIDGE	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:		Rev. 10/18 - (No Interim)
DIVISION	Descrip:	ITBC Design Example 2, Span 2			File:	Ex2_Span2_distribution_factors.xl			Sheet: 6 of 8

LEVER RULE S = 8.0 ft

INTERIOR



For S < 4:

One Lane = $\frac{16}{32}$ = 0.500

For 4 ≤ S < 6:

One Lane = $\frac{16}{32}$ = 0.500

Two Lanes = $\frac{16}{32}\left(1+\frac{S-4}{S}\right)$ = 0.750

>: **For 6 ≤ S < 10:**

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}\right)$ = 0.875

For 10 ≤ S < 12:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

For 12 ≤ S < 16:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

For 16 ≤ S < 18:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-16}{S}\right)$ = 0.000

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
BRIDGE	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:		Rev. 10/18 - (No Interim)
DIVISION	Descrip:	ITBC Design Example 2, Span 2			File:	Ex2_Span2_distribution_factors.xl			Sheet: 7 of 8

LEVER RULE S = 8.0 ft

INTERIOR (con't)

For 18 ≤ S < 22:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}\right)$ = -0.625

For 22 ≤ S ≤ 24:

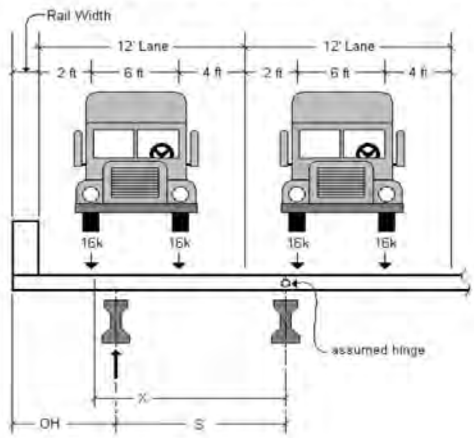
One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}+\frac{S-22}{S}\right)$ = -1.500

EXTERIOR



S = 8.0 ft
OH = 3.0 ft
Rail Width = RW = 1.0 ft
X = S+OH-RW-2ft = 8.0 ft

For X < 6:

One Lane = $\frac{16}{32}\left(\frac{X}{S}\right)$ = 0.500

>: **For 6 ≤ X < 12:**

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

For 12 ≤ X < 18:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}\right)$ = 0.375

TXDOT BRIDGE DIVISION	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:		Rev. 10/18 - (No Interim)
	Descrip:	ITBC Design Example 2, Span 2			File:	Ex2_Span2_distribution_factors.xl		Sheet:	8 of 8

LEVER RULE

EXTERIOR (con't) S = 8.0 ft OH = 3.0 ft
RW = 1.0 ft X = S+OH-RW-2ft = 8.0 ft

For 18 ≤ X < 24:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

For 24 ≤ X < 30:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}\right)$ = -1.250

For 30 ≤ X < 36:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

For 36 ≤ X < 42:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}\right)$ = -4.375

For 42 ≤ X ≤ 48:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}+\frac{X-42}{S}\right)$ = -6.500

INTERIOR			EXTERIOR		
One Lane Loaded	=	0.625	One Lane Loaded	=	0.625
Two Lanes Loaded	=	0.875	Two Lanes Loaded	=	0.625
Three Lanes Loaded	=	0.875	Three Lanes Loaded	=	0.625
Four Lanes Loaded	=	0.875	Four Lanes Loaded	=	0.625

4.3.15.5 Concrete Section Shear Capacity Spreadsheet

Texas Department of Transportation

County:	ANY	Descrip:	ITBC Design Example 2 - Bent 2
Highway:	ANY		
C-S-J:	XXXXXX	Design:	BRG
		Ck Dsn:	BRG
Bridge Division	Rev: 09/26/08	Date:	Aug-20

CONCRETE SECTION SHEAR CAPACITY BY AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, FOURTH EDITION, 2007

Resistance Factors: Units: **US**

ϕ_V =	0.9
ϕ_M =	0.9
ϕ_N =	0.75

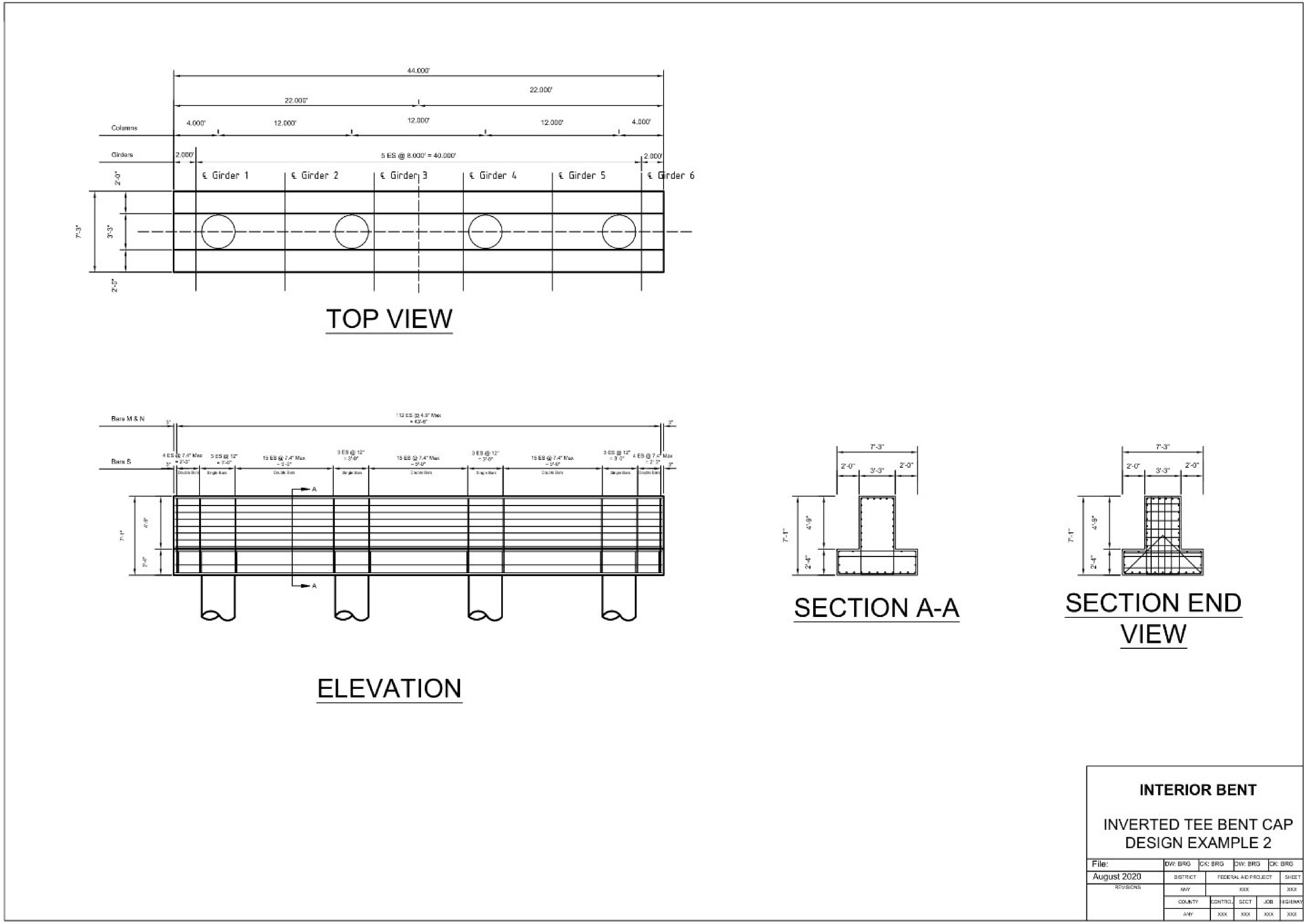
Concrete:		**Mild Steel:**		**Prestressed Steel:**	
f'c =	5 ksi	fy =	60 ksi	fpu =	270 ksi
Ec =	4070 ksi	Es =	29000 ksi	Ep =	28500 ksi

		SECTIONS							
	Units	**8**	**12**	**32**	**36**	**56**	**60**	**80**	**84**
Input Data									
Bending moment, **Mu**	kip-ft	499.7	718.4	592.8	394.2	394.2	592.8	718.4	500
Shear force, **Vu**	kip	237.2	243.5	133.6	452.1	234.9	251.6	137.6	422.2
Axial force, **Nu (+ if tensile)**	kip	0	0	0	0	0	0	0	0
Web width, **bv**	in	39.00	39.00	39.00	39.00	39.00	39.00	39.00	39.00
Shear depth, **dv**	in	80.53	80.53	80.53	80.53	80.53	80.53	80.53	80.53
Mild steel reinf. area, **As**	in^2	10.92	10.92	10.92	10.92	10.92	10.92	10.92	10.92
Conc area on tension side, **Ac**	in^2	1657.5	1657.5	1657.5	1657.5	1657.5	1657.5	1657.5	1657.5
Area of stirrups, **Av**	in^2	1.76	1.76	1.76	1.76	1.76	1.76	1.76	1.76
Stirrup spacing, **s**	in	7.4	7.4	7.4	7.4	7.4	7.4	7.4	7.4
Prestressed steel area, **Aps**	in^2	0	0	0	0	0	0	0	0
Prestress shear, **Vp**	kip	0	0	0	0	0	0	0	0
Average prestress, **fps**	ksi	0	0	0	0	0	0	0	0
Torsional moment, **Tu**	kip-ft	706	353	353	706	706	353	353	706
Shear flow area, **Ao**	in^2	2971.6	2971.6	2971.6	2971.6	2971.6	2971.6	2971.6	2971.6
Area of one leg of stirrup, **At**	in^2	0.44	0.44	0.44	0.44	0.44	0.44	0.44	0.44
Perimeter of stirrup, **Ph**	in	324	324	324	324	324	324	324	324
Calculated Values									
Vc	kip	543.7	541.5	605.8	494.9	543.7	537.0	601.4	494.9
Vs	kip	1669.4	1719.4	2009.4	1431.2	1669.4	1699.1	2000.9	1431.2
ϕVn	kip	**1992**	**2035**	**2354**	**1733**	**1992**	**2013**	**2342**	**1733**
ε_x		6.67E-04	6.83E-04	4.01E-04	1.00E-03	6.61E-04	7.02E-04	4.12E-04	1.00E-03
θ	deg	32.60	32.80	29.00	36.40	32.60	33.10	29.10	36.40
β		2.450	2.440	2.730	2.230	2.450	2.420	2.710	2.230
Req'd Shear reinf. **Av/S**	in^2/in	0.000	0.000	0.000	0.001	0.000	0.000	0.000	0.000
Req'd Torsion reinf. **At/S**	in^2/in	0.017	0.009	0.007	0.019	0.017	0.009	0.007	0.019
Maximum stirrup spacing, **Smax**	in	24.0	24.0	24.0	22.3	24.0	24.0	24.0	22.6
Conclusion									
Shear Reinforcing		OK	OK	OK	OK	OK	OK	OK	OK
Longitudinal Reinforcing		OK	OK	OK	OK	OK	OK	OK	OK

Note: Longitudinal Reinforcing check can be ignored for typical multi-column bent caps. For straddle bents with no overhangs, this check must be considered. Refer to LRFD 5.8.3.5 for further information.

If torsion is not being considered, leave last five rows of input data blank.

4.3.15.6 Bent Cap Details



4.4 INVERTED-T BENT CAP DESIGN EXAMPLE 3 (45° SKEW ANGLE)

Design example is in accordance with the AASHTO LRFD Bridge Design Specifications, 8th Ed. (2017) as prescribed by TxDOT Bridge Manual - LRFD (January 2020).

4.4.1 Design Parameters

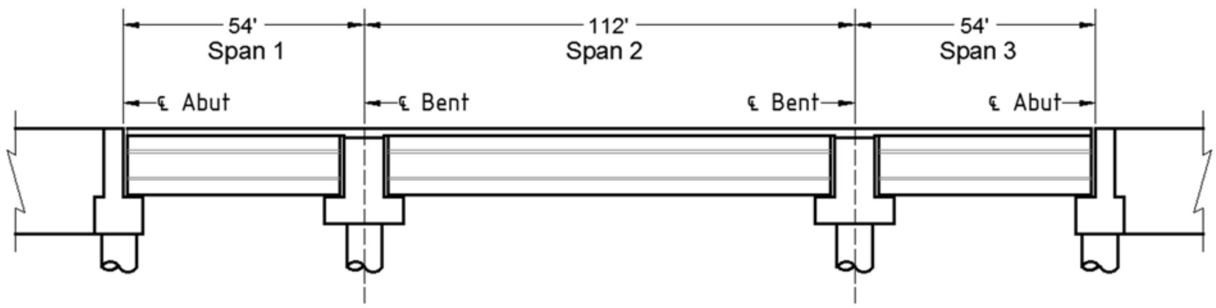


Figure 4.53 Spans of the Bridge with 45 Degrees Skewed ITBC

"AASHTO LRFD" refers to the ASSHTO LRFD Bridge Design Specification, 8th Ed. (2017)..

"BDM-LRFD" refers to the TxDOT Bridge Design Manual - LRFD (January 2020).

"TxSP" refers to TxDOT guidance, recommendations, and standard practice.

"Furlong & Mirza" refers to "Strength and Serviceability of Inverted T-Beam Bent Caps Subject to Combined Flexure, Shear, and Torsion", Center for Highway Research Research Report No. 153-1F, The University of Texas at Austin, August 1974.

Span 1

54' Type TX54 Girders (0.851 k/ft)

6 Girders Spaced @ 11.31' along the axis of bent with 3' overhangs

2" Haunch

Span 2

112' Type TX54 Girders (0.851 k/ft)

6 Girders Spaced @ 11.31' along the axis of bent with 3' overhangs

3.75" Haunch

Span 3

54' Type TX54 Girders (0.851 k/ft)

6 Girders Spaced @ 11.31' along the axis of bent with 3' overhangs

2" Haunch

All Spans

The basic bridge geometry can be found on the Bridge Layout located in the Appendices.

Deck is 46 ft wide

Type T551 Rail (0.382 k/ft)

8" Thick Slab (0.100 ksf)

Assume 2" Overlay @ 140 pcf (0.023 ksf)

(TxSP)

Use Class "C" Concrete

(BDM-LRFD, Ch. 4, Sect. 5, Materials)

$f'_c = 5$ ksi

$w_c = 150$ pcf (for weight)

$w_c = 145$ pcf (for Modulus of Elasticity calculation)

Grade 60 Reinforcing *(BDM-LRFD, Ch. 4, Sect. 5, Materials)*

$f_y = 60$ ksi

Bents

Use 36" Diameter Columns (Typical for Type TX54 Girders)

Define Variables

Back Span	**Forward Span**	
$Span1 = 54ft$	$Span2 = 112ft$	*Span Length*
$GdrSpa1 = 8ft$	$GdrSpa2 = 8ft$	*Girder Spacing (Normalized values)*
$GdrNo1 = 6$	$GdrNo2 = 6$	*Number of Girders in Span*
$GdrWt1 = 0.851klf$	$GdrWt2 = 0.851klf$	*Weight of Girder*
$Haunch1 = 2in$	$Haunch2 = 3.75in$	*Size of Haunch*

Bridge

$Skew = 45deg$ *Skew of Bents*

$BridgeW = 46ft$ *Width of Bridge Deck*

$RdwyW = 44ft$ *Width of Roadway*

$GirderD = 54in$ *Depth of Type TX54 Girder*

$BrgSeat = 1.5in$ *Bearing Seat Buildup*

$BrgPad = 2.75in$ *Bearing Pad Thickness*

$SlabThk = 8in$ *Thickness of Bridge Slab*

$OverlayThk = 2in$ *Thickness of Overlay*

$RailWt = 0.372klf$ *Weight of Rail*

$w_c = 0.150kcf$ *Unit Weight of Concrete for Loads*

$w_{Olay} = 0.140kcf$ *Unit Weigh of Overlay*

Bents

$f_c = 5ksi$ *Concrete Strength*

$w_{cE} = 0.145kcf$ *Unit Weight of Concrete for E_c*

$E_c = 33000 \cdot w_{cE}^{1.5} \cdot \sqrt{f_c}$ $E_c = 4074$ ksi *Modulus of Elasticity of Concrete (AASHTO LRFD Eq. C5.4.2.4-2)*

$f_y = 60ksi$ *Yield Strength of Reinforcement*

$E_s = 29000ksi$ *Modulus of Elasticity of Steel*

$D_{column} = 36in$ *Diameter of Columns*

Other Variables

Dynamic Load Allowance (AASHTO LRFD Table 3.6.2.1-1)

$\mathrm{IM} = 33\%$

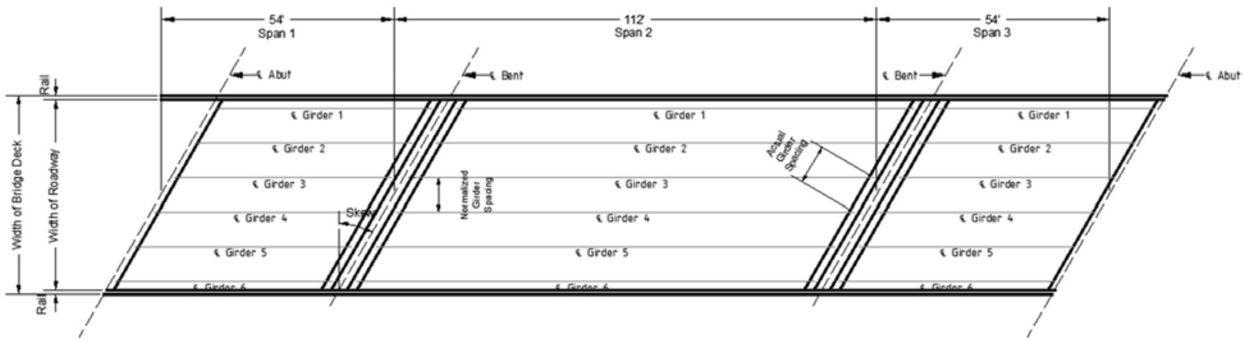


Figure 4.54Top View of the 45 Degrees Skewed ITBC with Spans and Girders

4.4.2 Determine Cap Dimensions

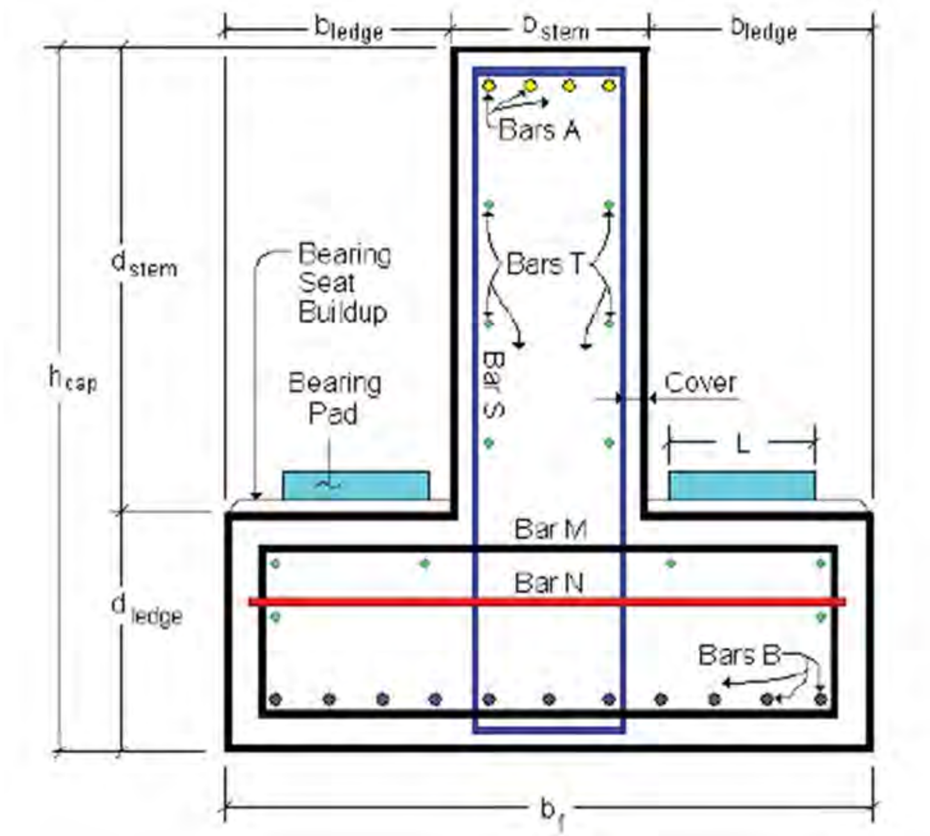


Figure 4.55 Section View of 45 Degrees Skewed ITBC

4.4.2.1 Stem Width

The stem is typically at least 3" wider than the Diameter of the Column (36") to allow for the extension of the column reinforcement into the Cap. (TxSP)

$b_{stem} = \text{at least } D_{column} + 3\text{in}$

Use: $b_{stem} = 42 \text{ in}$

4.4.2.2 Stem Height

Haunch2 is the larger of the two haunches.

Distance from Top of Slab to Top of Ledge:

$$D_{Slab\_to\_Ledge} = SlabThk + Haunch2 + GirderD + BrgPa$$

$D_{Slab\_to\_Ledge} = 70.00 \text{ in}$

$StemHaunch = 3.75 \text{ in}$

The top of the stem must be 2.5" below the bottom of the slab. (BDM-LRFD, Ch. 4, Sect. 5, Geometric Constraints)

Accounting for the 1/2" of bituminous fiber, the top of the stem must have at least 2" of haunch on it, but the haunch should not be less than either of the haunches of the adjacent spans.

$d_{stem} = D_{Slab\_to\_Ledge} - SlabThk - StemHaunch - 0.5in$

The stem must accommodate ½" of bituminous fiber.

$d_{stem} = 57.75$ in

Use: $d_{stem} = 57$ in

Round the Stem Height down to the nearest 1". (TxSP)

4.4.2.3 Ledge Width

The Ledge Width must be adequate for Bar M to develop fully.

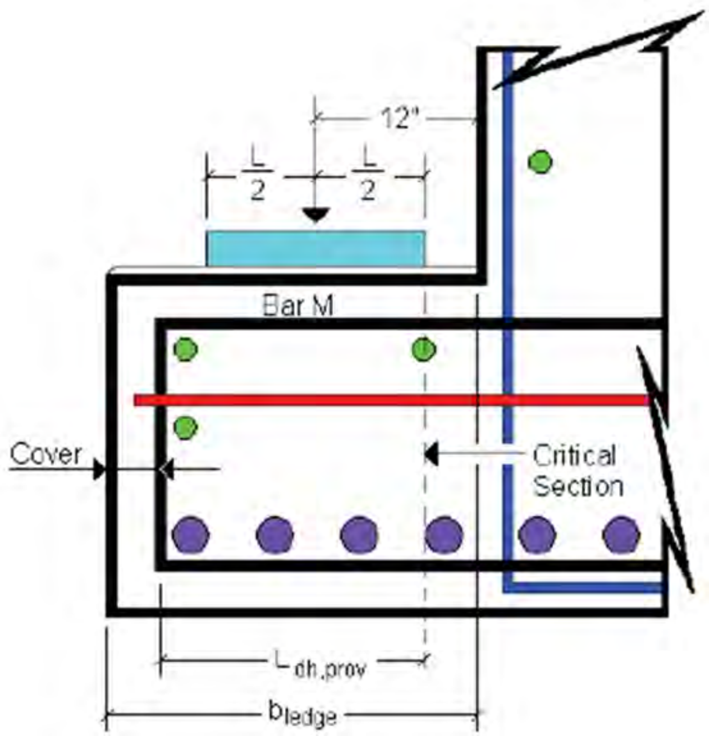


Figure 4.56 Ledge Section of 45 Degrees ITBC

"$L_{dh,prov}$" must be greater than or equal to "$L_{dh,req}$" for Bar M.

cover = 2.5 in

"cover" is measured from the center of the transverse bars.

L = 9 in

"L" is the length of the Bearing Pad along the girder. A typical type TX54 bearing pad is 9" × 21" for 45° skewed beents, as shown in the IGEB standard.

Determine the Required Development Length of Bar M:

Try # 7 Bar for Bar M.

$d_{bar\_M} = 0.875$ in

$A_{bar\_M} = 0.60\ in^2$

Basic Development Length

$$L_{dh} = \frac{38.0 \cdot d_{bar\_M}}{60} \cdot \left(\frac{f_y}{\sqrt{f_c}}\right) \qquad L_{dh} = 14.87 \text{ in}$$

(AASHTO LRFD Eq. 5.10.8.2.4a-2)

Modification Factors for L_{dh}:

(AASHTO LRFD 5.10.8.2.4b)

Is Top Cover greater than or equal to 2.5", and Side Cover greater than or equal to 2"?

$$\text{SideCover} = \text{cover} - \frac{d_{bar\_M}}{2} = 2.06\ \text{in}$$

$$\text{TopCover} = \text{cover} - \frac{d_{bar_M}}{2} = 2.06\ \text{in}$$

"Side Cover" and "Top Cover" are the clear cover on the side and top of the hook respectively. The dimension "cover" is measured from the center of Bar M.

No. Reinforcement Confinement Factor, $\lambda_{rc} = 1.0$

Coating Factor, $\lambda_{cw} = 1.0$

Excess Reinforcement Factor, $\lambda_{er} = 1.0$

Concrete Density Modification Factor, $\lambda = 1.0$ *(AASHTO LRFD 5.4.2.8)*

The Required Development Length: *(AASHTO LRFD 5.10.8.2.4a)*

$$L_{dh\_req} = \max(L_{dh} \cdot \left(\frac{\lambda_{rc} \cdot \lambda_{cw} \cdot \lambda_{er}}{\lambda}\right), 8 \cdot d_{bar\_M}, 6\text{in.})$$

Therefore,

$$L_{dh\_req} = 14.87\ \text{in}$$

$b_{ledge\_min} = L_{dh\_req} + \text{cover} + 12\text{in} - \frac{L}{2}$ $\qquad b_{ledge\_min} = 24.87\ \text{in}$

The distance from the face of the stem to the center of bearing is 12" for TxGirders (IGEB).

Use:

$$b_{ledge} = 25\ \text{in}$$

Width of Bottom Flange:

$b_f = 2 \cdot b_{ledge} + b_{stem}$ $\qquad b_f = 92\ \text{in}$

4.4.2.4 Ledge Depth

As a general rule of thumb, Ledge Depth is greater than or equal to 2'-3". This is the depth at which a bent from a typical bridge will pass the punching shear check.

Use a Ledge Depth of 28".

$$d_{ledge} = 28\ \text{in}$$

Total Depth of Cap:

$h_{cap} = d_{stem} + d_{ledge}$ $\qquad h_{cap} = 85\ \text{in}$

4.4.2.5 Summary of Cross Sectional Dimensions

$b_{stem} = 42\ \text{in}$

$d_{stem} = 57\ \text{in}$

$b_{ledge} = 25\ \text{in}$

$d_{ledge} = 28\ \text{in}$

$h_{cap} = 85\ \text{in}$

4.4.2.6 Length of Cap

First define Girder Spacing and End Distance:

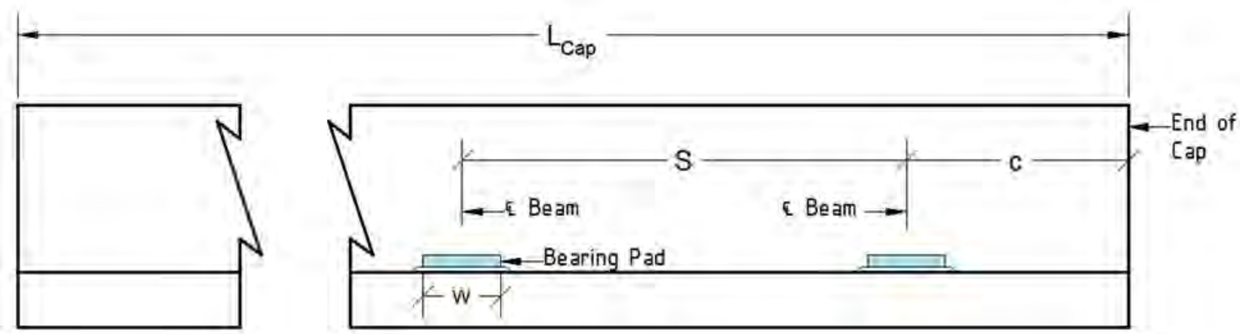


Figure 4.57 Elevation View of 45 Degrees Skewed ITBC

$S = 8\ \text{ft}$ *Girder Spacing*

$c = 2\ \text{ft}$ *"c" is the distance from the Center Line of the Exterior Girder to the Edge of the Cap measured along the Cap.*

$L_{Cap} = S \cdot (GdrNo1 - 1) + 2c$ $L_{Cap} = 44\ \text{ft}$ *Length of Cap*

TxDOT policy is as follows, "The edge distance between the exterior bearing pad and the end of the inverted T-beam shall not be less than 12in." (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria) replacing the statement in AASHTO LRFD 5.13.2.5.5 stating it shall not be less than d_f. Preferably, the stem should extend at least 3" beyond the edge of the bearing seat.

Bearing Pad Dimensions: *(IGEB standard)*

$L = 9\ \text{in}$ *Length of Bearing Pad*

$W = 21\ \text{in}$ *Width of Bearing Pad*

4.4.3 Cross Sectional Properties of Cap

$A_g = d_{ledge} \cdot b_f + d_{stem} \cdot b_{stem}$ $A_g = 4970\ \text{in}^2$

$ybar = \frac{d_{ledge} \cdot b_f \cdot \left(\frac{1}{2} d_{ledge}\right) + d_{stem} \cdot b_{stem} \cdot \left(d_{ledge} + \frac{1}{2} d_{stem}\right)}{A_g}$ $ybar = 34.5\ \text{in}$ *Distance from bottom of the cap to the center of gravity of the cap*

$$I_g = \frac{b_f \cdot d_{ledge}^3}{12} + b_f \cdot d_{ledge} \cdot \left(ybar - \frac{1}{2} d_{ledge}\right)^2 + \frac{b_{stem} \cdot d_{stem}^3}{12} + \cdots$$

$$b_{stem} \cdot d_{stem} \cdot \left[ybar - \left(d_{ledge} + \frac{1}{2} d_{stem}\right)\right]^2$$ $I_g = 3.06 \times 10^6\ \text{in}^4$

4.4.4 Cap Analysis

4.4.4.1 Cap Model

Assume:

4 Columns Spaced @ 12'-0"

The cap will be modeled as a continuous beam with simple supports using TxDOT's CAP18 program.

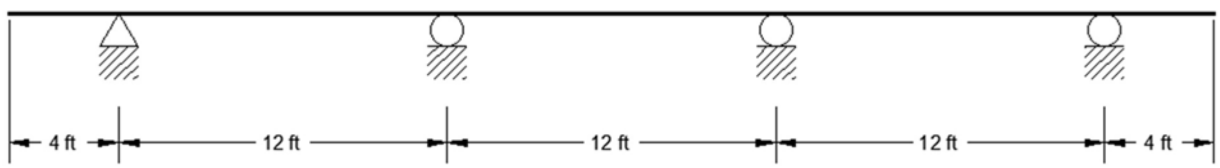


Figure 4.58 Continuous Beam Model for 45 Degrees Skewed ITBC

TxDOT does not consider frame action for typical multi-column bents (BDM-LRFD, Ch. 4, Sect. 5, Structural Analysis).

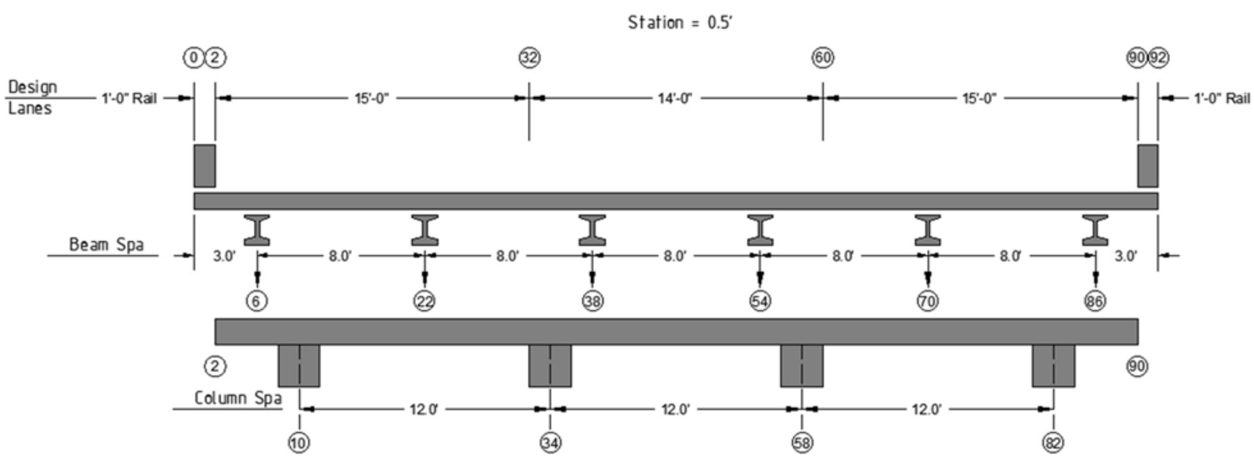


Figure 4.59 Cap 18 Model of 45 Degrees Skewed ITBC

The circled numbers in Figure 4.59 are the stations that will be used in the CAP 18 input file. One station is 0.5 ft in the direction perpendicular to the pgl, not parallel to the bent.

$$\text{station} = 0.5\ \text{ft}$$ *Station increment for CAP 18*

Recall:

$$E_c = 4074\ \text{ksi} \qquad I_g = 3.06 \times 10^6\ \text{in}^4$$

$$E_c I_g = 1.25 \times 10^{10}\ \text{kip} \cdot \text{in}^2 / \left(12\frac{\text{in}}{\text{ft}}\right)^2 \quad E_c I_g = 8.66 \times 10^7 \text{kip} \cdot \text{ft}^2$$

4.4.4.1.1 *Dead Load*

Values used in the following equations can be found on "4.4.1 Design Parameters"

SPAN 1

$$\text{Rail1} = \frac{2 \cdot \text{RailWt} \cdot \frac{\text{Span1}}{2}}{\min(\text{GdrNo1}, 6)} \qquad \text{Rail1} = 3.44 \frac{\text{kip}}{\text{girder}}$$

Rail Weight is distributed evenly among stringers, up to 3 stringers per rail (TxSP).

$$\text{Slab1} = w_c \cdot \text{GdrSpa1} \cdot \text{SlabThk} \cdot \frac{\text{Span1}}{2} \cdot 1.10 \qquad \text{Slab1} = 23.76 \frac{\text{kip}}{\text{girder}}$$

Increase slab DL by 10% to account for haunch and thickened slab ends.

$$\text{Girder1} = \text{GdrWt1} \cdot \frac{\text{Span1}}{2} \qquad \text{Girder1} = 22.98 \frac{\text{kip}}{\text{girder}}$$

$$\text{DLRxn1} = (\text{Rail1} + \text{Slab1} + \text{Girder1}) \qquad \text{DLRxn1} = 50.17 \frac{\text{kip}}{\text{girder}}$$

Overlay is calculated separetely, because it has different load factor than the rest of the dead loads.

$$\text{Overlay1} = w_{\text{Olay}} \cdot \text{GdrSpa1} \cdot \text{OverlayThk} \cdot \frac{\text{Span1}}{2} \qquad \text{Overlay1} = 5.04 \frac{\text{kip}}{\text{girder}}$$

Design for future overlay.

SPAN 2

$$\text{Rail2} = \frac{2 \cdot \text{RailWt} \cdot \frac{\text{Span2}}{2}}{\min(\text{GdrNo2}, 6)} \qquad \text{Rail2} = 7.13 \frac{\text{kip}}{\text{girder}}$$

$$\text{Slab2} = w_c \cdot \text{GdrSpa2} \cdot \text{SlabThk} \cdot \frac{\text{Span2}}{2} \cdot 1.10 \qquad \text{Slab2} = 49.28 \frac{\text{kip}}{\text{girder}}$$

$$\text{Girder2} = \text{GdrWt1} \cdot \frac{\text{Span2}}{2} \qquad \text{Girder2} = 47.66 \frac{\text{kip}}{\text{girder}}$$

$$\text{DLRxn2} = (\text{Rail2} + \text{Slab2} + \text{Girder2}) \qquad \text{DLRxn2} = 104.07 \frac{\text{kip}}{\text{girder}}$$

$$\text{Overlay2} = w_{\text{Olay}} \cdot \text{GdrSpa2} \cdot \text{OverlayThk} \cdot \frac{\text{Span2}}{2} \qquad \text{Overlay2} = 10.45 \frac{\text{kip}}{\text{girder}}$$

CAP

$$\text{Cap} = w_c \cdot A_g = 5.177 \frac{\text{kip}}{\text{ft}} \cdot \frac{0.5\text{ft}}{\text{station}} \qquad \text{Cap} = 2.589 \frac{\text{kip}}{\text{station}}$$

4.4.4.1.2 *Live Load*

AASHTO LRFD 3.6.1.2.2 and 3.6.1.2.4)

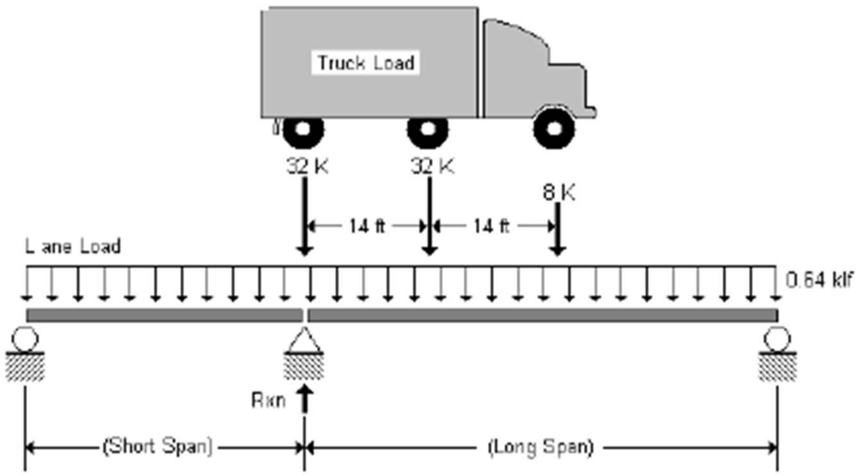


Figure 4.60 Live Load Model of 45 Degrees Skewed ITBC

$\text{LongSpan} = \max(\text{Span1}, \text{Span2})$ $\text{LongSpan} = 112\ \text{ft}$

$\text{ShortSpan} = \min(\text{Span1}, \text{Span2})$ $\text{ShortSpan} = 54\ \text{ft}$

$\text{IM} = 0.33$

$$\text{Lane} = 0.64\text{klf} \cdot \left(\frac{\text{LongSpan}+\text{ShortSpan}}{2}\right)$$

$$\text{Lane} = 53.12\,\frac{\text{kip}}{\text{lane}}$$

Use HL-93 Live Load. For maximum reaction at interior bents, "Design Truck" will always govern over "Design Tandem". For the maximum reaction when the long span is more than twice as long as the short span, place the rear (32 kip) axle over the support and the middle (32 kip) and front (8 kip) axles on the long span. For the maximum reaction when the long span is less than twice as long as the short span, place the middle (32 kip) axle over the support, the front (8 kip) axle on the short span and the rear (32 kip) axle on the long span.

$$\text{Truck} = 32\text{kip} + 32\text{kip} \cdot \left(\frac{\text{LongSpan}-14\text{ft}}{\text{LongSpan}}\right) + 8\text{kip} \cdot \left(\frac{\text{LongSpan}-28\text{ft}}{\text{LongSpan}}\right)$$

$$\text{Truck} = 66.00\,\frac{\text{ki}}{\text{lane}}$$

$$\text{LLRxn} = \text{Lane} + \text{Truck} \cdot (1 + \text{IM})$$

$$\text{LLRxn} = 140.90\,\frac{\text{kip}}{\text{lane}}$$

Combine "Design Truck" and "Design Lane" loadings (AASHTO LRFD 3.6.1.3). Dynamic load allowance, IM, does not apply to "Design Lane." (AASHTO LRFD 3.6.1.2.4)

$$P = 16.0\text{kip} \cdot (1 + IM)$$

$$P = 21.28\ \text{kip}$$

$$w = \frac{LLRxn - (2 \cdot P)}{10\text{ft}}$$

$$w = 9.83\frac{\text{kip}}{\text{ft}} \cdot \frac{0.5\text{ft}}{\text{station}}$$

$$w = 4.92\frac{\text{kip}}{\text{station}}$$

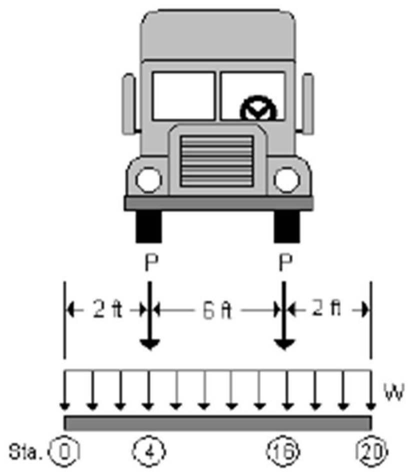


Figure 4.61 Live Load Model of 45 Degrees Skewed ITBC for CAP18

The Live Load is applied to the slab by two 16 kip wheel loads increased by the dynamic load allowance with the reminder of the live load distributed over a 10 ft (AASHTO LRFD 3.6.1.2.1) design lane width. (TxSP)

The Live Load applied to the slab is distributed to the beams assuming the slab is hinged at each beam except the outside beam. (BDM-LRFD, Ch. 4, Sect. 5, Structural Analysis)

4.4.4.1.3 *Cap 18 Data Input*

Input "Multiple Presence Factors" into CAP18 as "Load Reduction Factors".

Multiple Presence Factors, m (AASHTO LRFD Table 3.6.1.1.2-1)

No. of Lanes	Factor "m"
1	1.20
2	1.00
3	0.85
>3	0.65

Limit States (AASHTO LRFD 3.4.1)

The cap design need only consider Strength I, Service I, and Service I with DL (TxSP).

Strength I

Live Load and Dynamic Load Allowance	LL+IM = 1.75
Dead Load Components	DC = 1.25
Dead Load Wearing Surface (Overlay)	DW = 1.50

TxDOT allows the Overlay Factor to be reduced to 1.25 (TxSP), since overlay is typically used in design only to increase the safety factor, but in this example we will use DW=1.50.

Service I

Live Load and Dynamic Load Allowance	LL+IM = 1.00
Dead Load and Wearing Surface	DC & DW = 1.00

Dead Load

TxDOT considers Service level Dead Load only with a limit reinforcement stress of 22 ksi to minimize cracking. (BDM-LRFD, Chapter 4, Section 5, Design Criteria)

4.4.4.1.4 *Cap 18 Output*

	Max +M	**Max -M**
Dead Load:	$M_{posDL} = 379.0\ kip \cdot ft$	$M_{negDL} = -563.1\ kip \cdot ft$
Service Load:	$M_{posServ} = 721.8\ kip \cdot ft$	$M_{negServ} = -862.2\ kip \cdot ft$
Factored Load:	$M_{posUlt} = 1080.5\ kip \cdot ft$	$M_{negUlt} = -1238.4\ kip \cdot ft$

4.4.4.2 Girder Reactions on Ledge

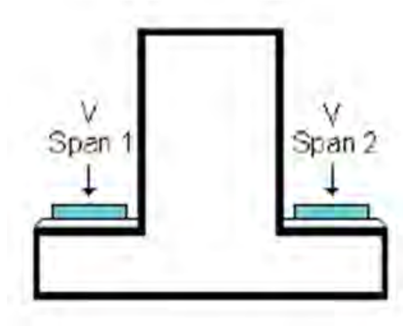


Figure 4.62 Girder Reactions on the Ledge of 45 Degrees Skewed ITBC

Dead Load

$\text{DLSpan1} = \text{Rail1} + \text{Slab1} + \text{Girder1}$ $\text{DLSpan1} = 50.17 \frac{\text{kip}}{\text{girder}}$

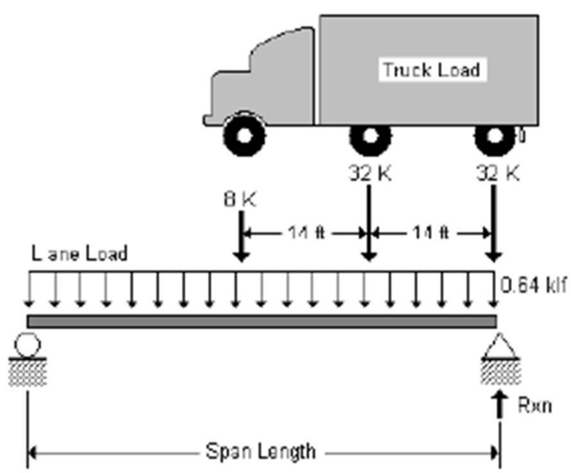
$\text{Overlay1} = 5.04 \frac{\text{kip}}{\text{girder}}$

$\text{DLSpan2} = \text{Rail2} + \text{Slab2} + \text{Girder2}$ $\text{DLSpan2} = 104.07 \frac{\text{kip}}{\text{girder}}$

$\text{Overlay2} = 10.45 \frac{\text{kip}}{\text{girder}}$

Live Load

Loads per Lane:



Use HL-93 Live Load. For maximum reaction at interior bents, "Design Truck" will always govern over "Design Tandem" for Spans greater than 26ft. For the maximum reaction, place the back (32 kips) axle over the support.

Figure 4.63 Live Load Model of 45 Degrees Skewed ITBC for Girder Reactions on Ledge

$\mathbf{LaneSpan1 = 0.64klf \cdot \left(\frac{Span1}{2}\right)}$ $\mathbf{LaneSpan1 = 17.28 \frac{kip}{lane}}$

$\mathbf{LaneSpan2 = 0.64klf \cdot \left(\frac{Span2}{2}\right)}$ $\mathbf{LaneSpan2 = 35.84 \frac{kip}{lane}}$

$$\mathbf{TruckSpan1} = \mathbf{32kip} + \mathbf{32kip} \cdot \left(\frac{\mathbf{Span1 - 14ft}}{\mathbf{Span1}}\right) + \mathbf{8kip} \cdot \left(\frac{\mathbf{Span1 - 28ft}}{\mathbf{Span1}}\right)$$

$$\text{TruckSpan1} = 59.56\,\frac{\text{kip}}{\text{lane}}$$

$$\mathbf{TruckSpan2} = \mathbf{32kip} + \mathbf{32kip} \cdot \left(\frac{\mathbf{Span2 - 14ft}}{\mathbf{Span2}}\right) + \mathbf{8kip} \cdot \left(\frac{\mathbf{Span2 - 28ft}}{\mathbf{Span2}}\right)$$

$$\text{TruckSpan2} = 66.00\,\frac{\text{kip}}{\text{lane}}$$

$$\text{IM} = 0.33$$

$$\text{LLRxnSpan1} = \text{LaneSpan1} + \text{TruckSpan1} \cdot (1 + \text{IM})$$

$$\text{LLRxnSpan1} = 96.49\,\frac{\text{kip}}{\text{lane}}$$

$$\text{LLRxnSpan2} = \text{LaneSpan2} + \text{TruckSpan2} \cdot (1 + \text{IM})$$

$$\text{LLRxnSpan2} = 123.62\,\frac{\text{kip}}{\text{girder}}$$

Combine "Design Truck" and "Design Lane" loadings (AASHTO LRFD 3.6.1.3).

Dynamic load allowance, IM, does not apply to "Design Lane." (AASHTO LRFD 3.6.1.2.4).

$$gV_{Span1\_Int} = 0.921$$

$$gV_{Span1\_Ext} = 0.921$$

$$gV_{Span2\_Int} = 0.947$$

$$gV_{Span2\_Ext} = 0.947$$

The Live Load Reactions are assumed to be the Shear Live Load Distribution Factor multiplied by the Live Load Reaction per Lane. The Shear Live Load Distribution Factor is calculated using the "LRFD Live Load Distribution Factors" Spreadsheet found in the Appendices.

The Exterior Girders must have a Live Load Distribution Factor equal to or greater than the Interior Girders. This is to accommodate a possible future bridge widening. Widening the bridge would cause the exterior girders to become interior girders

$$\text{LLSpan1Int} = gV_{Span1\_Int} \cdot \text{LLRxnSpan1} \qquad \text{LLSpan1Int} = 88.87\,\frac{\text{kip}}{\text{girder}}$$

$$\text{LLSpan1Ext} = gV_{Span1\_Ext} \cdot \text{LLRxnSpan1} \qquad \text{LLSpan1Ext} = 88.87\,\frac{\text{kip}}{\text{girder}}$$

$$\text{LLSpan2Int} = gV_{Span2\_Int} \cdot \text{LLRxnSpan2} \qquad \text{LLSpan2Int} = 117.07\,\frac{\text{kip}}{\text{girder}}$$

$$\text{LLSpan2Ext} = gV_{Span2\_Ext} \cdot \text{LLRxnSpan2} \qquad \text{LLSpan2Ext} = 117.07\,\frac{\text{kip}}{\text{girder}}$$

Span 1

Interior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{s\_Span1Int} = DLSpan1 + Overlay1 + LLSpan1Int$$

$$V_{s\_Span1Int} = 144 \text{ kip}$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{u\_Span1Int} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 + 1.75 \cdot LLSpan1Int$$

$$V_{u\_Span1Int} = 226 \text{ kip}$$

Exterior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{s\_Span1Ext} = DLSpan1 + Overlay1 + LLSpan1Ext$$

$$V_{s\_Span1Ext} = 144 \text{ kip}$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{u\_Span1Ext} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 + 1.75 \cdot LLSpan1Ext$$

$$V_{u\_Span1Ext} = 226 \text{ kip}$$

Span 2

Interior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{s\_Span2Int} = DLSpan2 + Overlay2 + LLSpan2Int$$

$$V_{s\_Span2Int} = 232 \text{ kip}$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{u\_Span2Int} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot LLSpan2Int$$

$$V_{u\_Span2Int} = 351 \text{ kip}$$

Exterior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{s\_Span2Ext} = DLSpan2 + Overlay2 + LLSpan2Ext$$

$$V_{s\_Span2Ext} = 232 \text{ kip}$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{u\_Span2Ext} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot LLSpan2Ext$$

$$V_{u\_Span2Ext} = 351 \text{ kip}$$

4.4.4.3 Torsional Loads

To maximize the torsion, the live load only acts on the longer span.

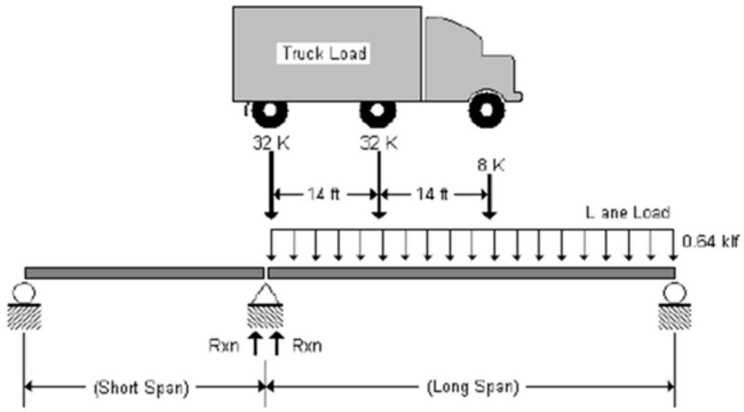


Figure 4.64 Live Load Model of 45 Degrees Skewed ITBC for Torsional Loads

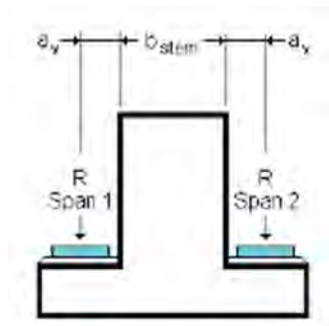


Figure 4.65. Loads on the Ledge of 45 Degrees Skewed ITBC for Torsion

$a_v = 12\ in$

"a_v" is the value for the distance from the face of the stem to the center of bearing for the girders. 12" is the typical values for TxGirders on ITBC (IGEB). The lever arm is the distance from the center line of bearing to the centerline of the cap.

$b_{stem} = 42\ in$

$LeverArm = a_v + \frac{1}{2} b_{stem}$ $\qquad LeverArm = 33\ in$

<u>Interior Girders</u>

Girder Reactions

$$R_{u\_Span1} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1$$

$$R_{u\_Span1} = 70\ kip$$

$$R_{u\_Span2} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2\_Int} \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)]$$

$$R_{u\_Span2} = 351\ kip$$

Torsional Load

$$T_{u\_Int} = \left|R_{u\_Span1} - R_{u\_Span2}\right| \cdot LeverArm$$

$$T_{u\_Int} = 773\ kip \cdot ft$$

Exterior Girders

Girder Reactions

$$R_{u\_Span1} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1$$

$$R_{u\_Span1} = 70\ kip$$

$$R_{u\_Span2} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2\_Ext} \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)]$$

$$R_{u\_Span2} = 351\ kip$$

Torsional Load

$$T_{u\_Ext} = \left|R_{u\_Span1} - R_{u\_Span2}\right| \cdot LeverArm$$

$$T_{u\_Ext} = 773\ kip \cdot ft$$

Torsion on Cap

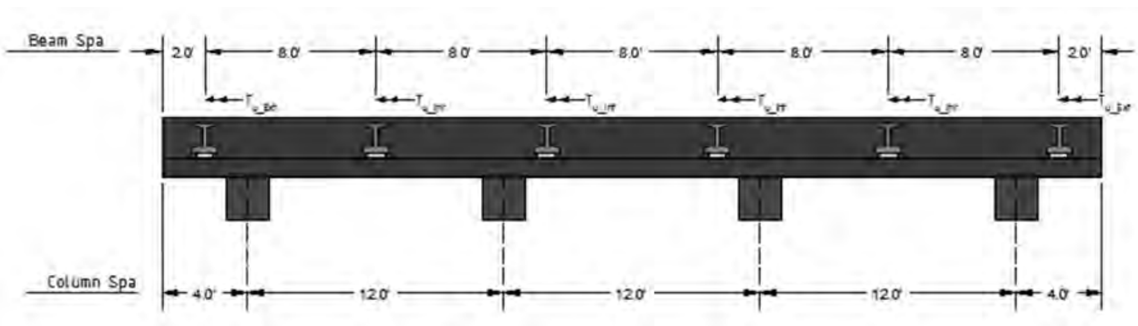


Figure 4.66 Elevation View of 45 Degrees Skewed ITBC with Torsion Loads

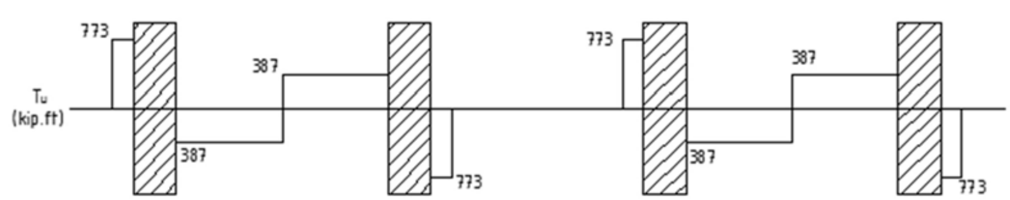


Figure 4.67 Torsion Diagram of 45 Degrees Skewed ITBC

Analyzed assuming Bents are torsionally rigid at Effective Face of Columns.

$T_u = 773\ kip \cdot ft$ *Maximum Torsion on Cap*

4.4.4.4 Load Summary

Ledge Loads

Interior Girder

Service Load

$V_{s\_Int} = \max(V_{s\_Span1Int}, V_{s\_Span2Int})$ $V_{s\_Int} = 231.60\ \text{kip}$

Factored Load

$V_{u\_Int} = \max(V_{u\_Span1Int}, V_{u\_Span2Int})$ $V_{u\_Int} = 350.64\ \text{kip}$

Exterior Girder

Service Load

$V_{s\_Ext} = \max(V_{s\_Span1Ext}, V_{s\_Span2Ext})$ $V_{s\_Ext} = 231.60\ \text{kip}$

Factored Load

$V_{u\_Ext} = \max(V_{u\_Span1Ext}, V_{u\_Span2Ext})$ $V_{u\_Ext} = 350.64\ \text{kip}$

Cap Loads

Positive Moment (From CAP18)

Dead Load: $M_{posDL} = 379.0\ \text{kip} \cdot \text{ft}$

Service Load: $M_{posServ} = 721.8\ \text{kip} \cdot \text{ft}$

Factored Load: $M_{posUlt} = 1080.5\ \text{kip} \cdot \text{ft}$

Negative Moment (From CAP18)

Dead Load: $M_{negDL} = -563.1\ \text{kip} \cdot \text{ft}$

Service Load: $M_{negServ} = -862.2\ \text{kip} \cdot \text{ft}$

Factored Load: $M_{negUlt} = -1238.4\ \text{kip} \cdot \text{ft}$

Maximum Torsion and Concurrent Shear and Moment (Strength I)

$T_u = 773\ \text{kip} \cdot \text{ft}$

$V_u = 462.8\ \text{kip}$

$M_u = 504.8\ \text{kip} \cdot \text{ft}$

Located two stations away from centerline of column.

V_u and M_u values are from CAP18

4.4.5 Locate and Describe Reinforcing

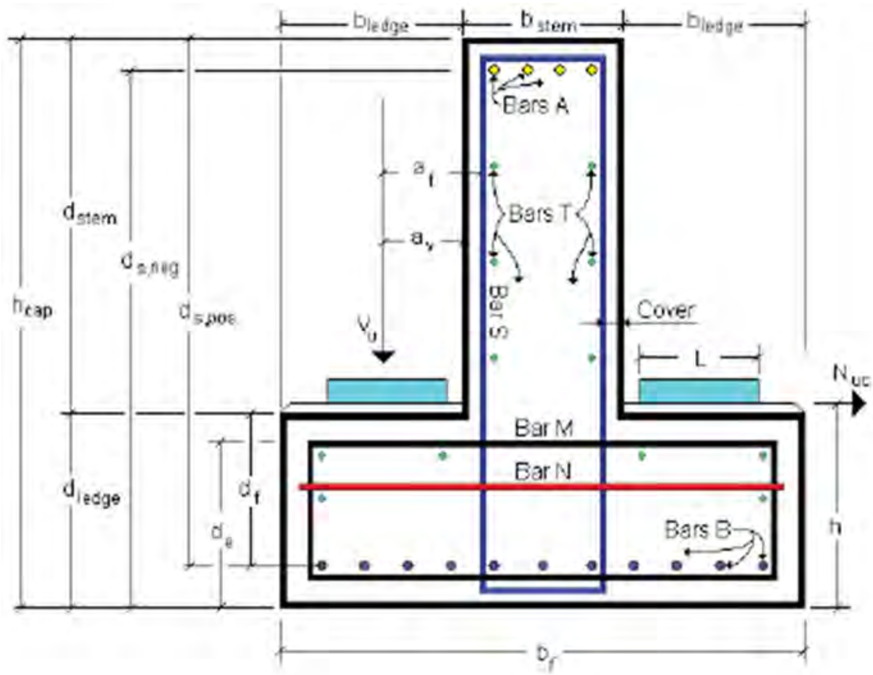


Figure 4.68 Section View of 45 Degrees Skewed ITBC

Recall:

$b_{stem} = 42 \text{ in}$

$d_{stem} = 57 \text{ in}$

$b_{ledge} = 25 \text{ in}$

$d_{ledge} = 28 \text{ in}$

$b_f = 92 \text{ in}$

$h_{cap} = 85 \text{ in}$

$\text{cover} = 2.5 \text{ in}$

4.4.5.1 Describe Reinforcing Bars

Use # 11 bars for Bar A

$A_{bar\_A} = 1.56\ in^2$ $d_{bar\_A} = 1.410\ in$

Use # 11 bars for Bar B

$A_{bar\_B} = 1.56\ in^2$ $d_{bar\_B} = 1.410\ in$

Use # 7 bars for Bar M

$A_{bar\_M} = 0.60\ in^2$ $d_{bar\_M} = 0.875 in$

Use # 7 bars for Bar N

$A_{bar\_N} = 0.60\ in^2$ $d_{bar\_N} = 0.875\ in$

Use # 6 bars for Bar S

$A_{bar\_S} = 0.44\ in^2$ $d_{bar\_S} = 0.75\ in$

Use # 6 bars for Bar T

$A_{bar\_T} = 0.44\ in^2$ $d_{bar\_T} = 0.75\ in$

In the calculation of b_{ledge}, # 7 Bar M was considered. Bar M must be # 7 or smaller to allow it fully develop.

To prevent confusion, use the same bar size for Bar N as Bar M.

4.4.5.2 Calculate Dimensions

$d_{s\_neg} = h_{cap} - cover - \frac{1}{2} d_{bar\_S} - \frac{1}{2} d_{bar\_A}$ $d_{s\_neg} = 81.42\ in$

$d_{s\_pos} = h_{cap} - cover - \frac{1}{2} \max(d_{bar\_S}, d_{bar\_M}) - \frac{1}{2} d_{bar\_B}$ $d_{s\_pos} = 81.36\ in$

$a_v = 12\ in$

$a_f = a_v + cover$ $a_f = 14.50\ in$

$d_e = d_{ledge} - cover$ $d_e = 25.50\ in$

$d_f = d_{ledge} - cover - \frac{1}{2} d_{bar\_M} - \frac{1}{2} d_{bar\_B}$ $d_f = 24.36\ in$

$h = d_{ledge} + BrgSeat$ $h = 29.50\ in$

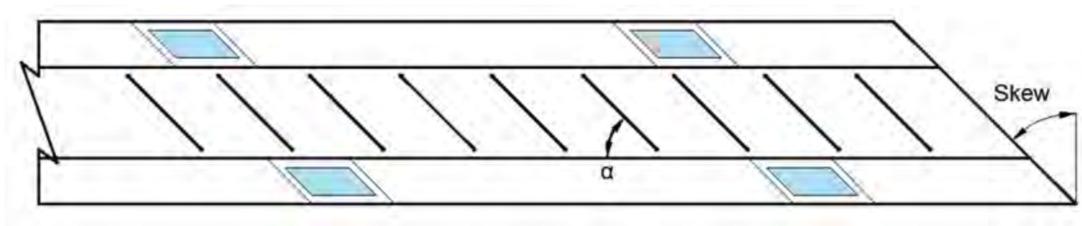


Figure 4.69 Plan View of 45 Degrees Skewed ITBC

$\alpha = 45 \text{ deg}$ *Angle of Bars S (Angle from the horizontal)*

Recall:

$L = 9 \text{ in}$ *Dimension of Bearing Pad*

$W = 21 \text{ in}$

4.4.6 Check Bearing

The load on the bearing pad propagates along a truncated pyramid whose top has the area A_1 and whose base has the area A_2. A_1 is the loaded area (the bearing pad area: L×W). A_2 is the area of the lowest rectangle contained wholly within the support (the Inverted Tee Cap). A_2 must not overlap the truncated pyramid of another load in either direction, nor can it extend beyond the edges of the cap in any direction.

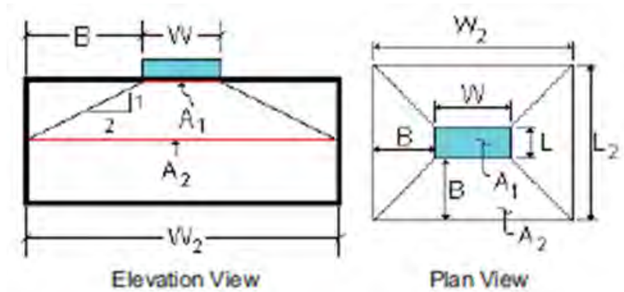


Figure 4.70 Bearing Check for 45 Degrees Skew Angle

Resistance Factor (ϕ) = 0.7 *(AASHTO LRFD 5.5.4.2)*

$A_1 = L \cdot W$ $A_1 = 189 \text{ in}^2$ *Area under Bearing Pad*

Interior Girders

$$B = \min\left[\left(b_{ledge} - a_v\right) - \frac{1}{2}L, \left(a_v + \frac{1}{2}b_{stem}\right) - \frac{1}{2}L, 2d_{ledge}, \frac{1}{2}S - \frac{1}{2}W\right]$$

"B" is the distance from perimeter of A_1 to the perimeter of A_2 as seen in the above figure

$B = 8.5 \text{ in.}$

$L_2 = L + 2 \cdot B$ $L_2 = 26.00 \text{ in}$

$W_2 = W + 2 \cdot B$ $W_2 = 38.00 \text{ in}$

$A_2 = L_2 \cdot W_2$ $A_2 = 988 \text{ in}^2$

Modification factor

$m = \min\left(\sqrt{\frac{A_2}{A_1}}, 2\right)$ = 2.29 and 2 $m = 2$ *AASHTO LRFD Eq. 5.6.5-3*

$\phi V_n = \phi \;\; 0.85 \;\; f_c \;\; A_1 \;\; m$ $\phi V_n = 1124.55$ kips *AASHTO LRFD Eqs. 5.6.5-1 and 5.6.5-2.*

$V_{u\_Int} = 350.64 < \phi V_n$ BearingChk = "OK!" *V_{u_int} from "4.4.4.4* Load Summary*".*

Exterior Girders

$$B = \min\left[\left(b_{ledge} - a_v\right) - \frac{1}{2}L, \left(a_v + \frac{1}{2}b_{stem}\right) - \frac{1}{2}L, 2d_{ledge}, \frac{1}{2}S - \frac{1}{2}W, c - \frac{1}{2}W\right]$$

B= 8.5 in. *"B" is the distance from perimeter of A_1 to the perimeter of A_2 as seen in the above figure*

$L_2 = L + 2 \;\; B$ $L_2 = 26.00$ in

$W_2 = W + 2 \;\; B$ $W_2 = 38.00$ in

$A_2 = L_2 \;\; W_2$ $A_2 = 988\ in^2$

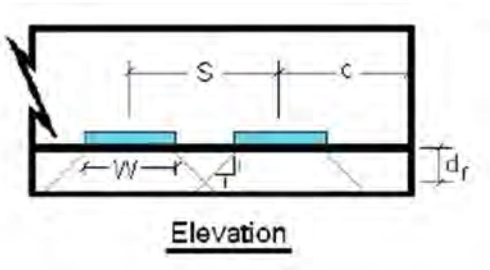
Modification factor

$m = \min\left(\sqrt{\frac{A_2}{A_1}}, 2\right)$ = 2.29 and 2 $m = 2$ *AASHTO LRFD Eq. 5.6.5-3*

$\phi V_n = \phi \;\; 0.85 \;\; f_c \;\; A_1 \;\; m$ $\phi V_n = 1124.55$ kips AASHTO LRFD Eqs. 5.6.5-1 and 5.6.5-2:

$V_{u\_ext} = 350.64$ kips $< \Phi V_n$ BearingChk= "OK!" V_{u_ext} from "*4.4.4.4* Load Summary*".*

4.4.7 Check Punching Shear



*AASHTO LRFD **5.8.4.3.4**, the truncated pyramids assumed as failure surfaces for punching shear shall not overlap.*

Figure 4.71 Punching Shear Check for 45 Degrees Skew Angle

Resistance Factor (ϕ) = 0.90 *AASHTO LRFD 5.5.4.2.*

Determine if the Shear Cones Intersect

Is $\frac{1}{2}S - \frac{1}{2}W \geq d_f$?

Yes. Therefore, shear cones do not intersect in the longitudinal direction of the cap.

$\frac{1}{2}S - \frac{1}{2}W = 37.5\text{ in}$

$d_f = 24.36\text{ in}$

TxDOT uses "d_f" instead of "d_e" for Punching Shear (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria). This is because "d_f" has traditionally been used for inverted tee bents and was sed in the Inverted Tee Research (Furiong % Mirza pg. 58).

Is $\frac{1}{2}b_{stem} + a_v - \frac{1}{2}L \geq d_f$?

Yes. Therefore, shear cones do not intersect in the transverse direction of the cap.

$\frac{1}{2}b_{stem} + a_v - \frac{1}{2}L = 28.5\text{ in}$

$d_f = 24.36$ in

Interior Girders

$V_n = 0.125\ \square\ \lambda\sqrt{f'_c}\, b_o\, d_f$ $V_n = 597.27\text{ kips}$ *AASHTO LRFD 5.8.4.3.4-3*

$b_o = W + 2L + 2d_f$ $b_o = 87.72\text{ in}$ *AASHTO LRFD 5.8.4.3.4-4*

$\phi V_n = 537.54\text{ kips}$

$V_{u\_Int} = 350.64\text{ kips} < \phi V_n$ PunchingShearChk= “OK!” *V_{u_int} from “4.4.4.4* Load Summary*”.*

Exterior Girders

$V_n = \min[(0.125 \cdot \sqrt{f_c} \cdot \left(\frac{1}{2}W + L + d_f + c\right) * d_f, 0.125 \cdot \sqrt{f_c} \cdot (W + 2L + 2d_f) * d_f)]$

$V_n = 462.04 \text{ kips}$

AASHTO LRFD 5.8.4.3.4-3 and 5.8.4.3.4-5

$\phi V_n = 415.84 \text{ kips}$

$V_{u\_ext} = 350.64 \text{ kips} < \phi V_n$

PunchingShearChk= "OK!"

V_{u_ext} from "4.4.4.4 Load Summary*".*

4.4.8 Check Shear Friction

Resistance Factor (ϕ) =0.90

AASHTO LRFD 5.5.4.2

Determine the Distribution Width

Interior Girders

$b_{s\_Int} = \min(W + 4a_v, S)$

"S" is the girder spacing.

= min (69 in, 96 in)

$b_{s\_Int} = 69 \text{ in}$

$A_{cv} = b_{s\_Int} \cdot d_e$

$A_{cv} = 1759.5 \text{ in2}$

Exterior Girders

$b_{s\_Ext} = \min(W + 4a_v, S, 2c)$

"S" is the girder spacing.

= min [69, 96, 48]

= 48 in

$A_{cv} = b_{s\_ext} \cdot d_e$

$A_{cv} = 1224 \text{ in2}$

Interior Girders

$V_n = \min(0.2 \cdot f_c \cdot A_{cv}, 0.8 \cdot A_{cv})$

$V_n = 1408 \text{ kips}$

AASHTO LRFD 5.8.4.2.2-1 and 5.8.4.2.2-2

= min (1759.5, 1408)

$\phi V_n = 1267 \text{ kips}$

$V_{u\_Int} = 350.64 \text{ kips} < \phi V_n$

ShearFrictionChk= "OK!"

V_{u_int} from "4.4.4.4 Load Summary".

Exterior Girders

$V_n = \min(0.2 \cdot f_c \cdot A_{cv}, 0.8 \cdot A_{cv})$ $= \min(1224, 979.2)$	V_n = 979.2 kips	*AASHTO LRFD 5.8.4.2.2-1 and 5.8.4.2.2-2*
$\phi V_n = 881$ kips		
$V_{u\_ext} = 350.64$ kips $< \phi V_n$	ShearFrictionChk= "OK!"	*V_{u_ext} from "4.4.4.4 Load Summary".*

4.4.9 Flexural Reinforcement for Negative Bending (Bars A)

$M_{dl} = \lvert M_{negDL} \rvert$	$M_{dl} = 563.1\ kip \cdot ft$
$M_s = \lvert M_{negServ} \rvert$	$M_s = 862.2\ kip \cdot ft$
$M_u = \lvert M_{negUlt} \rvert$	$M_u = 1238.4\ kip \cdot ft$

4.4.9.1 Minimum Flexural Reinforcement

Factored Flexural Resistance, M_r, must be greater than or equal to the lesser of $1.2M_{cr}$ (Cracking Moment) or $1.33M_u$ (Ultimate Moment).

$I_g = 3.06 \times 10^6\ in^4$		*Gross Moment of Inertia*
$h_{cap} = 85\ in$		*Depth of Cap*
$ybar = 34.5\ in$		*Distance to the Center of Gravity of the Cap from the bottom of the Cap*
$f_r = 0.24\sqrt{f_c}$	$f_r = 0.537\ ksi$	*Modulus of Rupture (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)*
$y_t = h_{cap} - ybar$	$y_t = 50.50\ in$	*Distance from Center of Gravity to extreme tension fiber*
$S = \frac{I_g}{y_t}$	$S = 6.06 \times 10^4\ in^3$	*Section Modulus for the extreme tension fiber*
$M_{cr} = S \cdot f_r \cdot \frac{1ft}{12in}$	$M_{cr} = 2711.8\ kip \cdot ft$	*Cracking Moment (AASHTO LRFD Eq. 5.6.3.3-1)*

M_f = minimum of:

$1.2M_{cr} = 3254.2\ kip \cdot ft$

$1.33M_u = 1647.1\ kip \cdot ft$

Design the lesser of $1.2M_{cr}$ or $1.33M_u$ when determining mininum area of steel required.

Thus, M_r must be greater than $M_f = 1647.1\ kip \cdot ft$

4.4.9.2 Moment Capacity Design

Try, 7 ~ #11's Top		
$BarANo = 7$		*Number of bars in tension*
$d_{bar\_A} = 1.410\ in$		*Diameter of main reinforcing bars*
$A_{bar\_A} = 1.56\ in^2$		*Area of main reinforcing bars*
$A_s = BarANo \cdot A_{bar\_A}$	$A_s = 10.92\ in^2$	*Area of steel in tension*
$d_{stirrup} = d_{bar\_S}$	$d_{stirrup} = 0.75\ in$	*Diameter of shear reinforcing bars*
$d = d_{s\_neg}$	$d = 81.42\ in$	
$b = b_f$	$b = 92\ in$	
$f_c = 5.0\ ksi$		*Compressive Strength of Concrete*
$f_y = 60\ ksi$		*Yield Strength of Rebar*
$\beta_1 = 0.85 - 0.05(f_c - 4ksi)$		*(AASHTO LRFD 5.6.2.2)*
Bounded by: $0.65 \le \beta_1 \le 0.85$	$\beta_1 = 0.80$	
$c = \frac{A_s f_y}{0.85 f_c \beta_1 b}$	$c = 2.09\ in$	*Depth of Cross Section under Compression under Ultimate Load (AASHTO LRFD Eq. 5.6.3.1.2-4)*

This "c" is the distance from the extreme compression fiber to the neutral axis, not the distance from the center of bearing of the last girder to the end of the cap.

$a = c \cdot \beta_1$	$a = 1.67\ in$	*Depth of Equivalent Stress Block (AASHTO LRFD 5.6.2.2)*

Note: "a" is less than "d_{ledge}". Therefore the equivalent stress block acts over a rectangular area. If "a" was greater than "d_{ledge}", it would act over a Tee shaped area.

$M_n = A_s f_y \left(d - \frac{a}{2}\right) \cdot \frac{1ft}{12in}$	$M_n = 4400\ kip \cdot ft$	*Nominal Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.2-1)*
$\varepsilon_s = 0.003 \cdot \frac{d-c}{c}$	$\varepsilon_s = 0.114$	*Strain in Reinforcing at Ultimate*
$\varepsilon_s > 0.005$		
FlexureBehavior = "Tension Controlled"		*(AASHTO LRFD 5.6.2.1)*
$\Phi_M = 0.90$		*(AASHTO LRFD 5.5.4.2)*
$M_r = \Phi_M M_n$	$M_r = 3960\ kip \cdot ft$	*Factored Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.1-1)*
$M_f = 1647.1\ kip \cdot ft < M_r$	MinReinfChk = "OK!"	
$M_u = 1238.4\ kip \cdot ft < M_r$	UltimateMom = "OK!"	

4.4.9.3 Check Serviceability

To find s_{max}:

For service loads, the stress on the cross-section is located as shown in Figure 4.72.

Modular Ratio:

$$n = \frac{E_s}{E_c} \qquad n = 7.12$$

Tension Reinforcement Ratio:

$$\rho = \frac{A_s}{b \cdot d} \qquad \rho = 0.00146$$

$$k = \sqrt{(2\rho n) + (\rho n)^2} - (\rho n) \qquad k = 0.134$$

$$d \cdot k = 10.91 \text{ in} \ < \ d_{ledge} = 28 \text{ in}$$

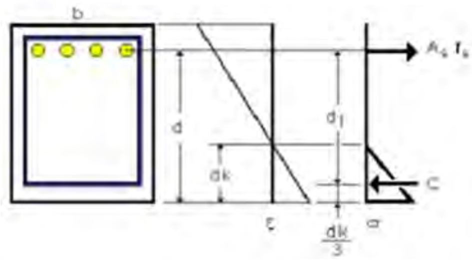


Figure 4.72 Stresses on the Cross Section for Service Loads of 45 Degrees Skewed ITBC

Therefore, the compression force acts over a rectangular area.

$$j = 1 - \frac{k}{3} \qquad j = 0.955$$

If the compression force does not act over rectangular area, j will be different.

$$f_{ss} = \frac{M_s}{A_s \cdot j \cdot d} \cdot \frac{12\text{in}}{1\text{ft}} \qquad f_{ss} = 12.2 \text{ ksi}$$

Service Load Bending Stress in outer layer of the reinforcing.

$$f_a = 0.6 f_y \qquad f_a = 36.00 \text{ ksi}$$

Allowable Bending Stress (BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

$f_{ss} < f_a$ ServiceStress = "OK!"

$$d_c = cover + \frac{1}{2} d_{stirrup} + \frac{1}{2} d_{bar\_A} \qquad d_c = 3.58 \text{ in}$$

Exposure Condition Factor:

$$\gamma_e = 1.00$$

For Class 1 Exposure Conditions. For areas where deicing chenicals are frequently used, design for Class 2 Exposure ($\gamma_e = 0.75$). (BDM-LRFD Ch. 4, Sect. 5, Design Criteria) (AASHTO LRFD Eq. 5.6.7-1)

$$\beta_s = 1 + \frac{d_c}{0.7(h_{cap} - d_c)} \qquad \beta_s = 1.06$$

$$s_{max} = \min\left(\frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c, 12\text{in.}\right) \qquad s_{max} = 12 \text{ in}$$

$$s_{Actual} = \frac{b_{stem} - 2d_c}{BarANo - 1} \qquad s_{Actual} = 5.81 \text{ in}$$

A good practice is to place a bar every 12 in along each surface of the bent. (TxSP)

$s_{actu} < s_{max}$ ServiceabilityCheck = "OK!

4.4.9.4 Check Dead Load

Check allowable M_{dl}: $f_{dl} = 22$ ksi

TxDOT limits dead load stress to 22 ksi, which is set to limit observed cracking under dead load.

$$M_a = A_s \cdot d \cdot j \cdot f_{dl} \cdot \frac{1\text{ft}}{12\text{in}} \qquad M_a = 1556.7 \text{ kip} \cdot \text{ft}$$

Allowable Dead Load Moment

$M_{dl} = 563.1 \text{ kip} \cdot \text{ft} \ < \ M_a$ DeadLoadMom = "OK!"

4.4.10 Flexural Reinforcement for Positive Bending (Bars B)

$M_{dl} = M_{posDL}$ — $M_{dl} = 379.0\ \text{kip} \cdot \text{ft}$

$M_s = M_{posServ}$ — $M_s = 721.8\ \text{kip} \cdot \text{ft}$

$M_u = M_{posUlt}$ — $M_u = 1080.5\ \text{kip} \cdot \text{ft}$

4.4.10.1 Minimum Flexural Reinforcement

Factored Flexural Resistance, M_r, must be greater than or equal to the lesser of $1.2M_{cr}$ (Cracking Moment) or $1.33M_u$ (Ultimate Moment).

$I_g = 3.06 \times 10^6\ \text{in}^4$ — *Gross Moment of Inertia*

$y_t = \text{ybar}$ — $y_t = 34.5\ \text{in}$ — *Distance to the Center of Gravity of the Cap from the top of the Cap*

$f_r = 0.24\sqrt{f_c}$ — $f_r = 0.537\ \text{ksi}$ — *Modulus of Rupture (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)*

$S = \frac{I_g}{y_t}$ — $S = 8.87 \times 10^4\ \text{in}^3$ — *Section Modulus for the extreme tension fiber*

$M_{cr} = S \cdot f_r \cdot \frac{1\text{ft}}{12\text{in}}$ — $M_{cr} = 3969.3\ \text{kip} \cdot \text{ft}$ — *Cracking Moment (AASHTO LRFD Eq. 5.6.3.3-1)*

M_f = minimum of:

$1.2M_{cr} = 4763.2\ \text{kip} \cdot \text{ft}$

$1.33M_u = 1437.1\ \text{kip} \cdot \text{ft}$

Design the lesser of $1.2M_{cr}$ *or* $1.33M_u$ *when determining mininum area of steel required.*

Thus, M_r must be greater than $M_f = 1437.1\ \text{kip} \cdot \text{ft}$

4.4.10.2 Moment Capacity Design

Try, 11 ~ #11's Bottom

$BarBNo = 11$		*Number of bars in tension*
$d_{bar\_B} = 1.41\ in$		*Diameter of main reinforcing bars*
$A_{bar\_B} = 1.56\ in^2$		*Area of main reinforcing bars*
$A_s = BarBNo \cdot A_{bar\_B}$	$A_s = 17.16\ in^2$	*Area of steel in tension*
$d = d_{s\_pos}$	$d = 81.36\ in$	
$b = b_{stem}$	$b = 42\ in$	
$f_c = 5.0\ ksi$		*Compressive Strength of Concrete*
$f_y = 60\ ksi$		*Yield Strength of Rebar*
$\beta_1 = 0.85 - 0.05(f_c - 4ksi)$		*(AASHTO LRFD 5.6.2.2)*
Bounded by: $0.65 \leq \beta_1 \leq 0.85$	$\beta_1 = 0.80$	
$c = \frac{A_s f_y}{0.85 f_c \beta_1 b}$	$c = 7.21\ in$	*Depth of Cross Section under Compression under Ultimate Load (AASHTO LRFD Eq. 5.6.3.1.2-4)*

This "c" is the distance from the extreme compression fiber to the neutral axis, not the distance from the center of bearing of the last girder to the end of the cap.

$a = c \cdot \beta_1$	$a = 5.77\ in$	*Depth of Equivalent Stress Block (AASHTO LRFD 5.6.2.2)*

Note: "a" is less than "d_{stem}". Therefore the equivalent stress block acts over a rectangular area. If "a" was greater than "d_{stem}", it would act over a Tee shaped area.

$M_n = A_s f_y \left(d - \frac{a}{2}\right) \cdot \frac{1ft}{12in}$	$M_n = 6733.2\ kip \cdot ft$	*Nominal Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.2-1)*
$\varepsilon_s = 0.003 \cdot \frac{d-c}{c}$	$\varepsilon_s = 0.031$	*Strain in Reinforcing at Ultimate*
$\varepsilon_s > 0.005$		
FlexureBehavior = "Tension Controlled"		*(AASHTO LRFD 5.6.2.1)*
$\Phi_M = 0.90$		*(AASHTO LRFD 5.5.4.2)*
$M_r = \Phi_M \cdot M_n$	$M_r = 6059.9\ kip \cdot ft$	*Factored Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.1-1)*

$M_f = 1437.1\ kip \cdot ft\ <\ M_r$ MinReinfChk = "OK!"

$M_u = 1080.5\ kip \cdot ft\ <\ M_r$ UltimateMom = "OK!"

4.4.10.3 Check Serviceability

To find s_{max}:

For service loads, the stress on the cross-section is located as shown in Figure 4.73.

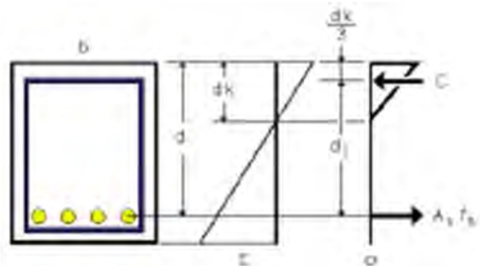


Figure 4.73 Stresses on the Cross Section for Bars B for Service Loads of 45 Degrees Skewed ITBC

Modular Ratio:

$$n = \frac{E_s}{E_c} \qquad n = 7.12$$

Tension Reinforcement Ratio:

$$\rho = \frac{A_s}{b \cdot d} \qquad \rho = 0.005$$

$$k = \sqrt{(2\rho n) + (\rho n)^2} - (\rho n) \qquad k = 0.234$$

$$d \cdot k = 19.04\ \text{in} \ < \ d_{stem} = 57.00\ \text{in}$$

Therefore, the compression force acts over a rectangular area.

$$j = 1 - \frac{k}{3} \qquad j = 0.922$$

If the compression force does not act over rectangular area, j will be different.

$$f_{ss} = \frac{M_s}{A_s \cdot j \cdot d} \cdot \frac{12\text{in}}{1\text{ft}} \qquad f_{ss} = 6.73\ \text{ksi}$$

Service Load Bending Stress in outer layer of the reinforcing.

$$f_a = 0.6 f_y \qquad f_a = 36.00\ \text{ksi}$$

Allowable Bending Stress (BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

$f_{ss} < f_a$ ServiceStress = "OK!"

$$d_c = \text{cover} + \frac{1}{2} d_{stirrup} + \frac{1}{2} d_{bar\_B} \qquad d_c = 3.64\ \text{in}$$

Exposure Condition Factor:

$$\gamma_e = 1.00$$

For Class 1 Exposure Conditions. For areas where deicing chenicals are frequently used, design for Class 2 Exposure ($\gamma_e = 0.75$). (BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

$$\beta_s = 1 + \frac{d_c}{0.7(h_{cap} - d_c)} \qquad \beta_s = 1.06$$

$$s_{max} = \min\left(\frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c, 12\text{in.}\right) \qquad s_{max} = 12\ \text{in}$$

(AASHTO LRFD Eq. 5.6.7-1)

A good practice is to place a bar every 12 in along each surface of the bent. (TxSP)

<u>Bars Inside Stirrup Bar S</u>

Try: BarBInsideSNo = 5

Number of Bars B that are inside Stirrup Bar S.

$$s_{Actual} = \frac{b_{stem} - 2\left(\text{cover} \ \ \frac{1}{2} d_{bar\_S} + \frac{1}{2} d_{bar\_B}\right)}{\text{BarBInsideSNo}-} \qquad s_{Actual} = 8.71\ \text{in}$$

$s_{actual} < s_{max}$ ServiceabilityCheck = "OK!"

Bars Outside Stirrup Bar S

Number of Bars B that are inside Stirrup Bar S.

$\mathrm{BarBOutsideSNo} = 11 - \mathrm{BarBInsideSNo}$

$\mathrm{BarBOutsideSNo} = 6$

$$s_{Actual} = \frac{2b_{ledge} + 2\left(cover\ \ \frac{1}{2}d_{bar\_S} + \frac{1}{2}d_{bar\_B} - cover\ \ \frac{1}{2}d_{bar\_M} - \frac{1}{2}d_{bar\_B}\right)}{\mathrm{BarBOutsideSNo}}$$

$s_{actual} = 8.31\ \mathrm{in}\ < s_{max}$ ServiceabilityCheck = "OK!"

4.4.10.4 Check Dead Load

TxDOT limits dead load stress to 22 ksi. This is due to observed cracking under dead load.

Check allowable M_{dl}: $f_{dl} = 22\ \mathrm{ksi}$

$M_a = A_s \cdot d \cdot j \cdot f_{dl} \cdot \frac{1\mathrm{ft}}{12\mathrm{in}}$ $M_a = 2360\ \mathrm{kip} \cdot \mathrm{ft}$ *Allowable Dead Load Moment*

$M_{dl} = 379.0\ \mathrm{kip} \cdot \mathrm{ft}\ < M_a$ DeadLoadMom = "OK!"

Flexural Steel Summary:

Use 7 ~ # 11 Bars on Top

& 11 ~ # 11 Bars on Bottom

4.4.11 Ledge Reinforcement (Bars M & N)

Try Bars M and Bars N at a 6.20" spacing.

$s_{bar\_M} = 6.20 \text{ in}$

$s_{bar\_N} = 6.20 \text{ in}$

Use trial and error to determine the spacing needed for the ledge reinforcing.

It is typical for Bars M & N to be paired together

4.4.11.1 Determine Distribution Widths

These distribution widths will be used on the following pages to determine the required ledge reinforcement per foot of cap.

Distribution Width for Shear (AASHTO LRFD 5.8.4.3.2)

Interior Girders

$b_{s\_Int} = \min(W + 4a_v, S)$

$b_{s\_Int} = 69.00 \text{ in}$

Exterior Girders

$b_{s\_Ext} = \min(W + 4a_v, 2c, S)$

$b_{s\_Ext} = 48.00 \text{ in}$

Note: These are the same distribution widths used for the Shear Friction check.

"S" is the girder spacing.

"c" is the distance from the center of bearing of the outside beam to the end of the ledge.

Distribution Width for Bending and Axial Loads (AASHTO LRFD 5.8.4.3.3)

Interior Girders

$b_{m\_Int} = \min(W + 5a_f, S)$

$b_{m\_Int} = 93.50 \text{ in}$

Exterior Girders

$b_{m\_Ext} = \min(W + 5a_f, 2c, S)$

$b_{m\_Ex} = 48.00 \text{ in}$

4.4.11.2 Reinforcing Required for Shear Friction

AASHTO LRFD 5.7.4.1

$\Phi = 0.90$ *(AASHTO LRFD 5.5.4)*

$\mu = 1.4$ $c_1 = 0\ \text{ksi}$ $P_c = 0\ \text{kip}$

Recall: $d_e = 25.50\ \text{in}$

"μ" is 1.4 for monolithically placed concrete. (AASHTO LRFD 5.7.4.4)

For clarity, the cohesion factor is labeled "c_1". This is to prevent confusion with "c", the distance from the last girder to the edge of the cap. c_1 is 0ksi for corbels and ledges. (AASHTO LRFD 5.7.4.4)

"P_c" is zero as there is no axial compression.

<u>Minimum Reinforcing</u> (AASHTO LRFD Eq. 5.7.4.2-1)

$$A_{vf\_min} = \frac{0.05\ \text{ksi} \cdot A_{cv}}{f_y}$$

$$A_{cv} = d_e \cdot b_s \quad \text{and} \quad a_{vf} = \frac{A_{vf}}{b_s}$$

$$a_{vf\_min} = \frac{0.05\text{ksi} \cdot d_e}{f_y} \qquad a_{vf\_min} = 0.26\,\frac{\text{in}^2}{\text{ft}}$$

Minimum Reinforcing required for Shear Friction

<u>Interior Girders</u>

$$A_{cv} = d_e \cdot b_{s\_Int} \qquad A_{cv} = 1759\ \text{in}^2$$

$$V_{u\_Int} = 350.6\ \text{kip}$$

From "4.4.4.4 Load Summary".

$$V_n = c_1 A_{cv} + \mu\left(A_{vf} f_y + P_c\right)$$

(AASHTO LRFD Eq. 5.7.4.3-3)

$$\Phi V_n \geq V_u$$

(AASHTO LRFD Eq. 5.7.4.3-1 & AASHTO LRFD Eq. 5.7.4.3-2)

$$\Phi \cdot \left[c_1 A_{cv} + \mu\left(A_{vf} f_y + P_c\right)\right] \geq V_u$$

$$A_{vf} = \frac{\dfrac{\dfrac{V_{u\_Int}}{\Phi} - c_1 A_{cv}}{\mu} - P_c}{f_y} \qquad A_{vf} = 4.64\ \text{in}^2$$

Required Reinforcing for Shear Friction

$$a_{vf\_Int} = \frac{A_{vf}}{b_{s\_Int}} \qquad a_{vf\_Int} = 0.81\,\frac{\text{in}^2}{\text{ft}}$$

Required Reinforcing for Shear Friction per foot length of cap

Exterior Girders

$A_{cv} = d_e \cdot b_{s\_Ext}$ $\quad$ $A_{cv} = 1224\ in^2$

$V_{u\_Ext} = 350.6\ kip$ $\quad$ *From "4.4.4.4 Load Summary".*

$V_n = c_1 A_{cv} + \mu(A_{vf} f_y + P_c)$ $\quad$ *(AASHTO LRFD Eq. 5.7.4.3-3)*

$\Phi V_n \geq V_u$ $\quad$ *(AASHTO LRFD Eq. 5.7.4.3-1 & AASHTO LRFD Eq. 5.7.4.3-2)*

$\Phi \cdot [c_1 A_{cv} + \mu(A_{vf} f_y + P_c)] \geq V_u$

$A_{vf} = \dfrac{\dfrac{\frac{V_{u\_Ext}}{\Phi} - c_1 A_{cv}}{\mu} - P_c}{f_y}$ $\quad$ $A_{vf} = 4.64\ in^2$ $\quad$ *Required Reinforcing for Shear Friction*

$a_{vf\_Ext} = \dfrac{A_{vf}}{b_{s\_Ext}}$ $\quad$ $a_{vf\_Ext} = 1.16 \dfrac{in^2}{ft}$ $\quad$ *Required Reinforcing for Shear Friction per foot length of cap*

4.4.11.3 Reinforcing Required for Flexure

AASHTO LRFD 5.8.4.2.1

Recall: $h = 29.50\ in$ $\quad$ $d_e = 25.50\ in$ $\quad$ $a_v = 12\ in$ $\quad$ *From "4.4.5.2 Calculate Dimensions"*

Interior Girders

$V_{u\_Int} = 350.6\ kip$ $\quad$ *From "4.4.4.4 Load Summary".*

$N_{uc\_Int} = 0.2 \cdot V_{u\_Int}$ $\quad$ $N_{uc\_Int} = 70.1\ kip$ $\quad$ *(AASHTO LRFD 5.8.4.2.1)*

$M_{u\_Int} = V_{u\_Int} \cdot a_v + N_{uc\_Int}(h - d_e)$ $\quad$ $M_{u\_Int} = 374\ kip \cdot ft$ $\quad$ *(AASHTO LRFD Eq. 5.8.4.2.1-1)*

Use the following equations to solve for A_f:

$\Phi M_n \geq M_{u\_Int}$ $\quad$ *(AASHTO LRFD Eq. 1.3.2.1-1)*

$M_n = A_f f_y \left(d_e - \frac{a}{2}\right)$ $\quad$ *(AASHTO LRFD Eq.5.6.3.2.2-1)*

$c = \dfrac{A_f f_y}{\alpha_1 f_c \beta_1 b_{m\_Int}}$ $\quad$ *(AASHTO LRFD Eq. 5.6.3.1.2-4)*

$\alpha_1 = 0.85$

$\beta_1 = 0.80$ $\quad$ *(AASHTO LRFD 5.6.2.2)*

$a = c\beta_1$

$0.75 \leq \Phi = 0.65 + 0.15\left(\frac{d_e}{c} - 1\right) \leq 0.90$ $\quad$ *AASHTO LRFD 5.5.4.2*

Solve for A_f: $\quad$ $A_f = 3.29\ in^2$ $\quad$ *Required Reinforcing for Flexure*

$a_{f\_Int} = \dfrac{A_f}{b_{m\_Int}}$ $\quad$ $a_{f\_Int} = 0.42 \dfrac{in^2}{ft}$ $\quad$ *Required Reinforcing for Flexure per foot length of cap*

Exterior Girders

$V_{u\_Ext} = 350.6 \text{ kip}$ — *From "4.4.4.4 Load Summary".*

$N_{uc\_Ext} = 0.2 \cdot V_{u\_Ext}$ $\quad N_{uc\_Ext} = 70.1 \text{ kip}$ — *(AASHTO LRFD 5.8.4.2.1)*

$M_{u\_Ext} = V_{u\_Ext} \cdot a_v + N_{uc\_Ext}(h - d_e)$ $\quad M_{u\_Ext} = 374 \text{ kip} \cdot \text{ft}$ — *(AASHTO LRFD Eq. 5.8.4.2.1-1)*

Use the following equations to solve for A_f:

$\Phi M_n \geq M_{u\_Ext}$ — *(AASHTO LRFD Eq. 1.3.2.1-1)*

$M_n = A_f f_y \left(d_e - \frac{a}{2}\right)$ — *(AASHTO LRFD Eq.5.6.3.2.2-1)*

$c = \frac{A_f f_y}{\alpha_1 f_c \beta_1 b_{m\_Ext}}$ — *(AASHTO LRFD Eq. 5.6.3.1.2-4)*

$\alpha_1 = 0.85$

$\beta_1 = 0.80$ — *(AASHTO LRFD 5.6.2.2)*

$a = c\beta_1$

$0.75 \leq \Phi = 0.65 + 0.15\left(\frac{d_e}{c} - 1\right) \leq 0.90$ — *AASHTO LRFD 5.5.4.2*

Solve for A_f: $\quad A_f = 3.32 \text{ in}^2$ — *Required Reinforcing for Flexure*

$a_{f\_Ext} = \frac{A_f}{b_{m\_Ext}}$ $\quad a_{f\_Ext} = 0.83 \frac{\text{in}^2}{\text{ft}}$ — *Required Reinforcing for Flexure per foot length of cap*

4.4.11.4 Reinforcing Required for Axial Tension

(AASHTO LRFD 5.8.4.2.2)

$\Phi = 0.90$ — *AASHTO LRFD 5.5.4.2*

Interior Girders:

$N_{uc\_Int} = 0.2V_{u\_Int}$ $\quad N_{uc\_Int} = 70.1 \text{ kip}$

$A_n = \frac{N_{uc\_Int}}{\Phi f_y}$ $\quad A_n = 1.30 \text{ in}^2$ — *Required Reinforcing for Axial Tension*

$a_{n\_Int} = \frac{A_n}{b_{m\_Int}}$ $\quad a_{n\_Int} = 0.17 \frac{\text{in}^2}{\text{ft}}$ — *Required Reinforcing for Axial Tension per foot length of cap*

Exterior Girders:

$N_{uc\_Ext} = 0.2V_{u\_Int}$ $\quad N_{uc\_Ext} = 70.1 \text{ kip}$

$A_n = \frac{N_{uc\_Ext}}{\Phi f_y}$ $\quad A_n = 1.29 \text{ in}^2$ — *Required Reinforcing for Axial Tension*

$a_{n\_Ext} = \frac{A_n}{b_{m\_Ext}}$ $\quad a_{n\_Ext} = 0.32 \frac{\text{in}^2}{\text{ft}}$ — *Required Reinforcing for Axial Tension per foot length of cap*

4.4.11.5 Minimum Reinforcing

(AASHTO LRFD 5.8.4.2.1)

$$a_{s\_min} = 0.04\frac{f_c}{f_y}\,d_e \qquad a_{s\_min} = 1.02\,\frac{in^2}{ft}$$ *Minimum Required Reinforcing*

4.4.11.6 Check Required Reinforcing

Actual Reinforcing:

$$a_s = \frac{A_{bar\_M}}{s_{bar\_M}} \qquad a_s = 1.16\,\frac{in^2}{ft}$$ *Primary Ledge Reinforcing Provided*

$$a_h = \frac{A_{bar\_N}}{s_{bar\_N}} \qquad a_h = 1.16\,\frac{in^2}{ft}$$ *Auxiliary Ledge Reinforcing Provided*

Checks: $A_s \geq A_{s\_min}$ *(AASHTO LRFD 5.8.4.2.1)*

$A_s \geq A_f + A_n$ *(AASHTO LRFD 5.8.4.2.2)*

$A_s \geq \frac{2A_{vf}}{3} + A_n$ *(AASHTO LRFD Eq. 5.8.4.2.2-5)*

$A_h \geq 0.5(A_s - A_n)$ *(AASHTO LRFD Eq. 5.8.4.2.2-6)*

Check Interior Girders:

Bar M:

Check if: $a_s \geq a_{s\_min}$ *(AASHTO LRFD 5.8.4.2.1)*

$a_s \geq a_{f\_Int} + a_{n\_Int}$ *(AASHTO LRFD 5.8.4.2.2)*

$a_s \geq \frac{2a_{vf\_Int}}{3} + a_{n\_Int}$ *(AASHTO LRFD Eq. 5.8.4.2.2-5)*

$$a_s = 1.16\,\frac{in^2}{ft}$$

$$a_{s\_min} = 1.02\,\frac{in^2}{ft} \quad < \quad a_s$$

$$a_{f\_Int} + a_{n\_Int} = 0.59\,\frac{in^2}{ft} \quad < \quad a_s$$

$$\frac{2a_{vf\_Int}}{3} + a_{n\_Int} = 0.71\,\frac{in^2}{ft} \quad < \quad a_s$$

BarMCheck = “OK!”

Bar N:

Check if: $a_h \geq 0.5 \cdot (a_s - a_{n\_Int})$ *(AASHTO LRFD Eq. 5.8.4.2.2-6)*

$a_s =$ The maximum of:

$a_{f\_Int} + a_{n\_Int}$

$\frac{2a_{vf\_Int}}{3} + a_{n\_Int}$

$$a_s = 0.71\,\frac{in^2}{ft}$$

"a_s" in this equation is the steel required for Bar M, based on the requirements for Bar M in AASHTO LRFD 5.8.4.2.2. This is derived from the suggestion that Ah should not be less than $A_f/2$ nor less than $A_{vf}/3$ (Furlong & Mirza pg. 73 & 74)

$$0.5 \cdot (a_s - a_{n\_Int}) = 0.28 \frac{in^2}{ft} < a_h$$

BarNCheck = “OK!”

Check Exterior Girders:

Bar M:

Check if: $a_s \geq a_{s\_min}$ *(AASHTO LRFD 5.8.4.2.1)*

$a_s \geq a_{f\_Ext} + a_{n\_Ext}$ *(AASHTO LRFD 5.8.4.2.2)*

$a_s \geq \frac{2a_{vf\_Ext}}{3} + a_{n\_Ext}$ *(AASHTO LRFD Eq. 5.8.4.2.2-5)*

$$a_s = 1.16 \frac{in^2}{ft}$$

$$a_{s\_min} = 1.02 \frac{in^2}{ft} < a_s$$

$$a_{f\_Ext} + a_{n\_Ext} = 1.15 \frac{in^2}{ft} < a_s$$

$$\frac{2a_{vf\_Ext}}{3} + a_{n\_Ext} = 1.09 \frac{in^2}{ft} < a_s$$

BarMCheck = “OK!”

Bar N:

Check if: $a_h \geq 0.5 \cdot (a_s - a_{n\_Ext})$ *(AASHTO LRFD Eq. 5.8.4.2.2-6)*

a_s = The maximum of:

$a_{f\_Ext} + a_{n\_Ex}$

$\frac{2a_{vf\_Ext}}{3} + a_{n\_Ext}$

$$a_s = 1.15 \frac{in^2}{ft}$$

"a_s" in this equation is the steel required for Bar M, based on the requirements for Bar M in AASHTO LRFD 5.8.4.2.2. This is derived from the suggestion that Ah should not be less than $A_f/2$ nor less than $A_{vf}/3$ (Furlong & Mirza pg. 73 & 74)

$$0.5 \cdot (a_s - a_{n\_Ext}) = 0.42 \frac{in^2}{ft} < a_h$$

BarNCheck = “OK!”

Ledge Reinforcement Summary:

Use # 7 primary ledge reinforcing @ 6.20” maximum spacing

& # 7 auxiliary ledge reinforcing @ 6.20” maximum spacing

4.4.12 Hanger Reinforcement (Bars S)

Try Double # 6 Stirrups at a 7.20" spacing.

Use trial and error to determine the spacing needed for the hanger reinforcing.

$s_{bar\_S} = 7.20\ \text{in}$

$A_{hr} = 2\text{stirrups} \cdot A_{bar\_S}$ $\qquad A_{hr} = 0.88\ \text{in}^2$

$A_v = 2\text{legs} \cdot A_{hr}$ $\qquad A_v = 1.76\ \text{in}^2$

4.4.12.1 <u>Check Minimum Transverse Reinforcement</u>

$b_v = b_{stem}$ $\qquad b_v = 42\ \text{in}$

$A_{v\_min} = 0.0316\lambda\sqrt{f_c}\frac{b_v \cdot s_{bar\_S}}{f_y}$ *(AASHTO LRFD Eq. 5.7.2.5-1)*

(AASHTO LRFD 5.4.2.8)

$\lambda = 1.0$ for normal weight concrete

$A_{v\_min} = 0.36\ \text{in}^2$

$A_v > A_{v\_min}$ $\qquad$ MinimumSteelCheck = "OK!"

4.4.12.2 <u>Check Service Limit State</u>

AASHTO LRFD 5.8.4.3.5 with notifications from BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

<u>Interior Girders</u>

TxDOT uses "2/3 f_y" from the original research (Furlong & Mirza Eq. 5.4) instead of "0.5 f_y" from AASHTO LRFD Eq. 5.8.4.3.5-1. (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)

V_{all} = minimum of:

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{bar\_S}} \cdot (W + 3a_v) = 235\ \text{kip}$$

Bounded by: $(W + 3a_v) \le \min(S, 2c)$

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{bar\_S}} \cdot S = 469\ \text{kip}$$

(BDM-LRFD Ch.4, Sect. 5, Design Criteria modified to limit the distribution width to the girder spacing. This will prevent distribution widths from overlapping)

$V_{all} = 235\ \text{kip}$

$V_{s\_Int} = 231.6\ \text{kip} < V_{all}$ $\qquad$ ServiceCheck = "OK!"

Exterior Girders

TxDOT uses "2/3 f_y" from the original research (Furlong & Mirza Eq. 5.4) instead of "0.5 f_y" from AASHTO LRFD Eq. 5.8.4.3.5-1. (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)

V_{all} = minimum of:

V_{all} for the Interior Girder

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{bar\_S}} \cdot \left(\frac{W + 3a_v}{2} + c\right) = 235 \text{ kip}$$

Bounded by: $(W + 3a_v) \leq \min(S, 2c)$

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{bar\_S}} \cdot \left(\frac{S}{2} + c\right) = 352 \text{ kip}$$

(BDM-LRFD Ch.4, Sect. 5, Design Criteria Modified to limit the distribution width to half the girder spacing and the distance to the edge of the cap. This will prevent distribution widths from overlapping or extending over the edge of the cap.)

$V_{all} = 235$ kip

$V_{s\_Ext} = 231.6 \text{ kip} < V_{all}$ ServiceCheck = "OK!"

4.4.12.3 Check Strength Limit State

(AASHTO LRFD 5.8.4.3.5)

$\Phi = 0.90$ *(AASHTO LRFD Eq. 5.5.4.2)*

Interior Girders:

V_n = minimum of:

$$\frac{A_{hr} \cdot f_y}{s_{bar\_S}} \cdot S = 704 \text{ kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-2)*

$$\left(0.063\sqrt{f_c} \cdot b_f \cdot d_f\right) + \frac{A_{hr} \cdot f_y}{s_{bar\_S}} (W + 2d_f) = 827 \text{ kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-3)*

$V_n = 704$ kip

$\Phi V_n = 634$ kip

$V_{u\_Int} = 350.6 \text{ kip} < \Phi V_n$ UltimateCheck = "OK!"

Exterior Girders:

V_n = minimum of:

V_n for the Interior Girder

$$\frac{A_{hr} \cdot f_y}{s_{bar\_S}} \cdot \left(\frac{S}{2} + c\right) = 528 \text{ kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-2)*

$$\left(0.063\sqrt{f_c} \cdot b_f \cdot d_f\right) + \frac{A_{hr} \cdot f_y}{s_{bar\_S}} \left(\frac{W + 2d_f}{2} + c\right) = 747 \text{ kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-3)*

(These equations are modified to limit the distribution width to the edge of the cap)

$V_n = 528$ kip

$\Phi V_n = 475$ kip

$V_{u\_Ext} = 350.6 \text{ kip} < \Phi V_n$ UltimateCheck = "OK!"

4.4.12.4 Check Combined Shear and Torsion

The following calculations are for Station 36. All critical locations must be checked. See the Concrete Section Shear Capacity spreadsheet in the appendices for calculations at other locations. Shear and Moment were calculated using the CAP 18 program.

$M_u = 504.8\ \text{kip} \cdot \text{ft}$ $V_u = 462.8\ \text{kip}$ $N_u = 0\ \text{kip}$ $T_u = 773\ \text{kip} \cdot \text{ft}$

Recall:

$\beta_1 = 0.80$ $f_y = 60\ \text{ksi}$

$f_c = 5.0\ \text{ksi}$ $E_s = 29000\ \text{ksi}$

$b_f = 92\ \text{in}$ $h_{cap} = 85\ \text{in}$ $b_{stem} = 42\ \text{in}$ $h = 29.50\ \text{in}$

$b_v = b_{stem}$ $b_v = 42\ \text{in}$

Find d_v:

(AASHTO LRFD 5.7.2.8)

Shears are maximum near the column faces. In these regions the cap is in negative bending with tension in the top of the cap. Therefore, the calculations are based on the steel in the top of the bent cap.

$A_s = A_{bar\_A} \cdot \text{BarANo}$ $A_s = 10.92\ \text{in}^2$

$c = \frac{A_s f_y}{0.85 f_c \beta_1 b_f}$ $c = 2.10\ \text{in}$

$a = c \cdot \beta_1$ $a = 1.68\ \text{in}$

$d_s = d_{s\_neg}$ $d_s = 81.42\ \text{in}$

$M_n = A_s f_y \left(d_s - \frac{a}{2}\right)$ $M_n = 4400\ \text{kip} \cdot \text{f}$

$A_{ps} = 0\ \text{in}^2$

$d_e = \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y}$ $d_e = 81.42\ \text{in}$ *(AASHTO LRFD Eq. 5.7.2.8-2)*

d_v = maximum of:

$\frac{M_n}{A_s f_y + A_{ps} f_{ps}} = 80.59\ \text{in}$

$0.9 d_e = 73.28\ \text{in}$

$0.72 h = 21.24\ \text{in}$

$d_v = 80.59\ \text{in}$

The method for calculating θ and β used in this design example are from AASHTO LRFD Appendix B5. The method from AASHTO LRFD 5.7.3.4.2 may be used instead. The method from 5.7.3.4.2 is based on the method from Appendix B5; however, it is less accurate and more conservative (often excessively conservative). The method from Appendix B5 is preferred because it is more accurate, but it requires iterating to a solution.

Determine θ and β:

$\Phi_V = 0.90$

(AASHTO LRFD Eq. 5.5.4.2)

$v_u = \frac{|V_u - (\Phi_V \cdot V_p)|}{\Phi_V \cdot b_v \cdot d_v}$ $v_u = 0.15$ ksi

Shear Stress on the Concrete (AASHTO LRFD Eq. 5.7.2.8-1)

$\frac{v_u}{f_c} = 0.03$

Using Table B5.2-1 with $\frac{v_u}{f_c} = 0.03$ and $\varepsilon_x = 0.001$

$\theta = 36.4$ deg and $\beta = 2.23$

Determining θ and β is an iterative process, therefore, assume initial shear strain value ε_x of 0.001 per LRFD B5.2 and then verify that the assumption was valid.

$$\varepsilon_x = \frac{\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})}$$

Strain halfway between the compressive and tensile resultants (AASHTO LRFD Eq. B5.2-3) If $\varepsilon_x < 0$, then use equation B5.2-5 and re-solve for ε_x.

where $|M_u| = 504.8$ kip · ft must be > $|V_u - V_p|d_v = 3108$ kip · ft

$\varepsilon_x = 1.23 \times 10^{-3} > 1.00 \times 10^{-3}$

use $\varepsilon_x = 1.00 \times 10^{-3}$.

For values of ε_x greater than 0.001, the tensile strain in the reinforcing, ε_t is greater than 0.002. ($\varepsilon_t = 2\varepsilon_x - \varepsilon_c$, where ε_c is < 0) Grade 60 steel yields at a strain of 60 ksi / 29,000 ksi = 0.002. By limiting the tensile strain in the steel to the yield strain and using the Modulus of Elasticity of the steel prior to yield, this limits the tensile stress of the steel to the yield stress. ε_x has not changed from the assumed value, therefore no iterations are required.

$V_p = 0$ kip

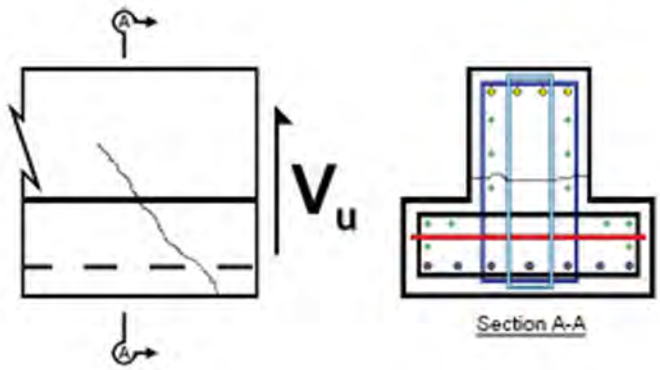
"V_p" is zero as there is no prestressing.

$A_c = b_{stem} \cdot \frac{h_{cap}}{2}$ $A_c = 1785$ in^2

$s = s_{bar\_S}$ $s = 7.20$ in

(AASHTO LRFD B5.2) "A_c" is the area of concrete on the flexural tension side of the cap, from the extreme tension fiber to one half the cap depth.

"A_c" is needed if AASHTO LRFD Eq. B5.2-3 is negative.



The transverse reinforcement, "A_v", is double closed stirrups. The failure surface intersects four stirrup legs, therefore the area of the shear steel is four times the stirrup bar's area (0.44in2). See the sketch of the failure plane to the left.

Figure 4.74 Failure Surface of 45 Degrees Skewed ITBC for Combined Shear and Torsion

$A_v = 2legs \cdot 2stirrups \cdot A_{bar\_S}$ $\qquad A_v = 1.76\ in^2$

$A_t = 1leg \cdot A_{bar\_S}$ $\qquad A_t = 0.44\ in^2$

$A_{oh} = (d_{stem}) \cdot (b_{stem} - 2cover) + (d_{ledge} - 2cover) \cdot (b_f - 2cover)$

$A_{oh} = 4110\ in^2$

$A_0 = 0.85A_{oh}$ $\qquad A_0 = 3493.5in^2$

$p_h = (b_{stem} - 2cover) + 2(b_{ledge}) + (b_f - 2cover) + 2(h_{cap} - 2cover)$

$p_h = 334\ in$

<u>Equivalent Shear Force</u>

$$V_{u\_Eq} = \sqrt{V_u^2 + \left(\frac{0.9p_hT_u}{2A_0}\right)^2} \qquad V_{u\_Eq} = 611.1\ kip$$ *(AASHTO LRFD Eq. B.5.2-1)*

<u>Shear Steel Required</u>

$V_n =$ the lesser of:

$V_c + V_s + V_p$ *(AASHTO LRFD Eq. 5.7.3.3-1)*

$0.25 \cdot f_c \cdot b_v \cdot d_v + V_p$ *(AASHTO LRFD Eq. 5.7.3.3-2)*

Check maximum ΦV_n for section:

$$\Phi V_{n\_max} = \Phi \cdot (0.25 \cdot f_c \cdot b_v \cdot d_v + V_p)$$

$\Phi V_{n\_max} = 3808\ kip$

$V_u = 462.8\ kip < \Phi V_{n\_max}$ MaxShearCheck = "OK!"

Calculate required shear steel:

$$V_u < \Phi V_n$$ *(AASHTO LRFD Eq. 1.3.2.1-1)*

$$V_c = 0.0316 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v \qquad V_c = 533\ \text{kip}$$ *(AASHTO LRFD Eq. 5.7.3.3-3)*

$$V_u < \Phi_V \cdot (V_c + V_s + V_p)$$

$$V_s = \frac{A_v \cdot f_y \cdot d_v \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha}{s_{req}}$$ *(AASHTO LRFD Eq. 5.7.3.3-4)*

$$a_{v\_req} = \frac{\frac{V_u}{\Phi_V} - V_c - V_p}{f_y \cdot d_v \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha} \qquad a_{v\_req} = 0.00\,\frac{in^2}{ft}$$

<u>Torsional Steel Required</u>

$$\Phi_T = 0.9$$ *(AASHTO LRFD 5.5.4.2)*

$$T_u \le \Phi_T T_n$$ *(AASHTO LRFD Eq. 1.3.2.1-1)*

$$T_n = \frac{2A_o A_t f_y \cot\theta}{s_{bar\_S}}$$ *(AASHTO LRFD Eq. 5.7.3.6.2-1)*

$$a_{t\_req} = \frac{T_u}{\Phi_T 2A_o f_y \cot\theta} \qquad a_{t\_req} = 0.22\,\frac{in^2}{ft}$$

<u>Total Required Transverse Steel</u>

The transverse reinforcement is designed for the side of the section where the effects of shear and torsion are additive. (AASHTO LRFD C5.7.3.6.1)

$$a_{req} = a_{v\_req} + 2sides \cdot a_{t\_req} \qquad a_{req} = 0.44\,\frac{in^2}{ft}$$

$$a_{prov} = \frac{A_v}{s_{bar\_S}} \qquad a_{prov} = 2.93\,\frac{in^2}{ft}$$

$$a_{prov} > a_{req}$$ TransverseSteelCheck = “OK!”

<u>Longitudinal Reinforcement</u>

$$A_{ps}f_{ps} + A_s f_y \ge \frac{|M_u|}{\Phi d_v} + \frac{0.5N_u}{\Phi} + \cdots$$ *(AASHTO LRFD Eq. 5.7.3.6.3-1)*

$$\cot\theta\sqrt{\left(\left|\frac{V_u}{\Phi} - V_p\right| - 0.5V_s\right)^2 + \left(\frac{0.45p_h T_u}{2A_o\Phi}\right)^2}$$

$$V_s = a_{t\_req} \cdot f_y \cdot d_v \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha$$ *(AASHTO LRFD Eq. 5.7.3.3-4)*

$$\text{Bounded By: } V_s < \frac{V_u}{\Phi_V} \qquad V_s = 514.2\ \text{kip}$$ *(AASHTO LRFD Eq. 5.7.3.5-1)*

$$\frac{|M_u|}{\Phi_f d_v} + \frac{0.5N_u}{\Phi_c} + \cot\theta\sqrt{\left(\left|\frac{V_u}{\Phi_V} - V_p\right| - 0.5V_s\right)^2 + \left(\frac{0.45p_h T_u}{2A_o\Phi_T}\right)^2} = 544\ \text{kip}$$

Provided Force:

$$A_s f_y = 655.2\ \text{kip} > 544\ \text{kip}$$ LongitudinalReinfChk = “OK!”

4.4.12.5 Maximum Spacing of Transverse Reinforcement *(AASHTO LRFD 5.7.2.6)*

Shear Stress

$v_u = \frac{|V_u - \Phi_V V_p|}{\Phi_V b_v d_v}$ $v_u = 0.15$ ksi *(AASHTO LRFD Eq. 5.7.2.8-1)*

$0.125 \cdot f_c = 0.625$ ksi

If $v_u < 0.125 \cdot f_c$ *(AASHTO LRFD Eq. 5.7.2.6-1)*

$s_{max} = \min(0.8d_v, 24in)$

If $v_u \geq 0.125 \cdot f_c$ *(AASHTO LRFD Eq. 5.7.2.6-2)*

$s_{max} = \min(0.4d_v, 12in)$

Since $v_u < 0.125 \cdot f_c$ $s_{max} = 24.00$ in

TxDOT limits the maximum transverse reinforcement spacing to 12". *(BDM-LRFD, Ch. 4, Sect. 5, Detailing)*

$s_{max} = 12.00$ in

$s_{bar\_S} = 7.20$ in $< s_{max}$ SpacingCheck= "OK!"

Hanger Reinforcement Summary:

Use double # 6 stirrups @ 7.20" maximum spacing

4.4.13 End Reinforcements (Bars U1, U2, U3, and G)

Extra vertical, horizontal, and diagonal reinforcing at the end surfaces is provided to reduce the maximum crack widths. According to the parametric analysis, it is recommended to place #6 U1 Bars, U2 Bars, and U3 Bars at the end faces and #7 G Bars at approximately 6in. spacing at the first 30” to 35” of the end of bent cap. U1 Bars are the vertical end reinforcements, U2 Bars and U3 Bars are the horizontal end reinforcements at the stem and the ledge, respectively. G Bars are the diagonal end reinforcement.

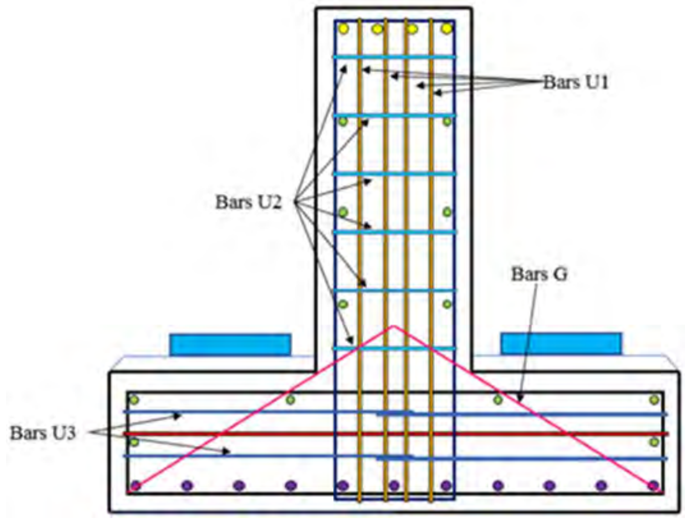


Figure 4.75 End Face Section View of 45 Degrees Skewed ITBC

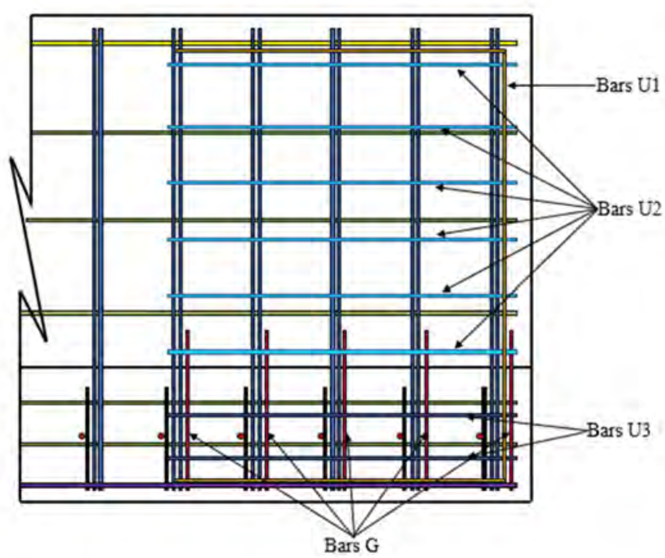


Figure 4.76 End Face Elevation View of 45 Degrees Skewed ITBC

4.4.14 Skin Reinforcement (Bars T)

Try 7 ~ # 6 bars in Stem and 3 ~ # 6 bars in Ledge on each side

$A_{bar\_T} = 0.44 \text{ in}^2$

$NoTBarsStem = 7$

$NoTBarsLedge = 3$

"a" must be within $\frac{2}{3}d_e$.

(AASHTO LRFD 5.13.2.4.1)

$\frac{2}{3}d_e = 17.00 \text{ in}$

TxDOT typically uses: $a = 6 \text{ in}$

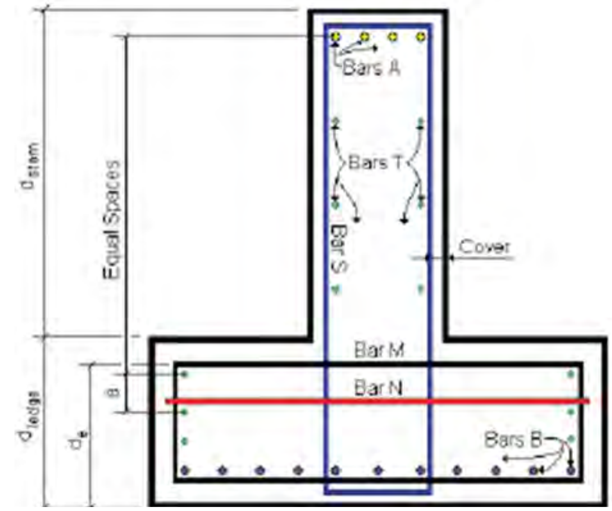


Figure 4.77 Section View for T Bars of 45 Degrees Skewed ITBC

4.4.14.1 <u>Required Area of Skin Reinforcement</u>

(AASHTO LRFD 5.6.7)

(AASHTO LRFD Eq. 5.6.7-3)

$A_{sk\_Req} = 0.012 \cdot (d - 30)$ $\qquad A_{sk\_Req} = 0.62 \frac{\text{in}^2}{\text{ft}}$

A_{sk} need not be greater than one quarter of the main reinforcing ($A_s/4$)per side face within $d/2$ of the main reinforcing. (AASHTO LRFD 5.6.7)

"d" is the distance from the extreme compression fiber to the centroid of the extreme tension steel element. In this example design, $d = d_{s\_pos} = 81.36$ in.

$$A_{sk\_max} = \max\left(\frac{\frac{A_{bar\_A} \cdot BarANo}{4}}{\frac{d_{s\_neg}}{2}}, \frac{\frac{A_{bar\_B} \cdot BarBNo}{4}}{\frac{d_{s\_pos}}{2}}\right)$$

$$A_{sk\_max} = 1.27 \frac{\text{in}^2}{\text{ft}}$$

$$A_{skReq} = \min(A_{sk\_Req}, A_{sk\_max})$$

$$A_{skReq} = 0.62 \frac{\text{in}^2}{\text{ft}}$$

4.4.14.2 <u>Required Spacing of Skin Reinforcement</u>

(AASHTO LRFD 5.6.7)

s_{req} = minimum of:

$\frac{A_{bar\_T}}{A_{skReq}} = 8.52 \text{ in}$

$\frac{d_{s\_neg}}{6} = 13.57 \text{ in}$

$\frac{d_{s\_pos}}{6} = 13.56 \text{ in}$

& 12 in

$$s_{req} = 8.52 \text{ in}$$

4.4.14.3 Actual Spacing of Skin Reinforcement

Check T Bars spacing in Stem:

$$h_{top} = d_{stem} - \left(\text{cover} + \frac{d_{bar\_S}}{2} + \frac{d_{bar\_A}}{2}\right) + \left(\text{cover} + \frac{d_{bar\_M}}{2} + \frac{d_{bar\_T}}{2}\right)$$

$$h_{top} = 56.73 \text{ in}$$

$$s_{skStem} = \frac{h_{top}}{NoTBarsStem+1}$$

$$s_{skStem} = 7.09 \text{ in}$$

$s_{skStem} < s_{req}$ SkinSpacing = “OK!”

Check T Bars spacing in Ledge:

$$h_{bot} = d_{ledge} - \left(\text{cover} + \frac{d_{bar\_M}}{2} + \frac{d_{bar\_T}}{2}\right) - \left(\text{cover} + \frac{d_{bar\_S}}{2} + \frac{d_{bar\_B}}{2}\right)$$

$$h_{bot} = 21.11 \text{ in}$$

$$s_{skLedge} = \frac{h_{bot} - a}{NoTBarsLedge}$$

$$s_{skLedge} = 7.56 \text{ in}$$

$s_{skLedge} < s_{req}$ SkinSpacing = “OK!”

Check if “a” is less than s_{req}

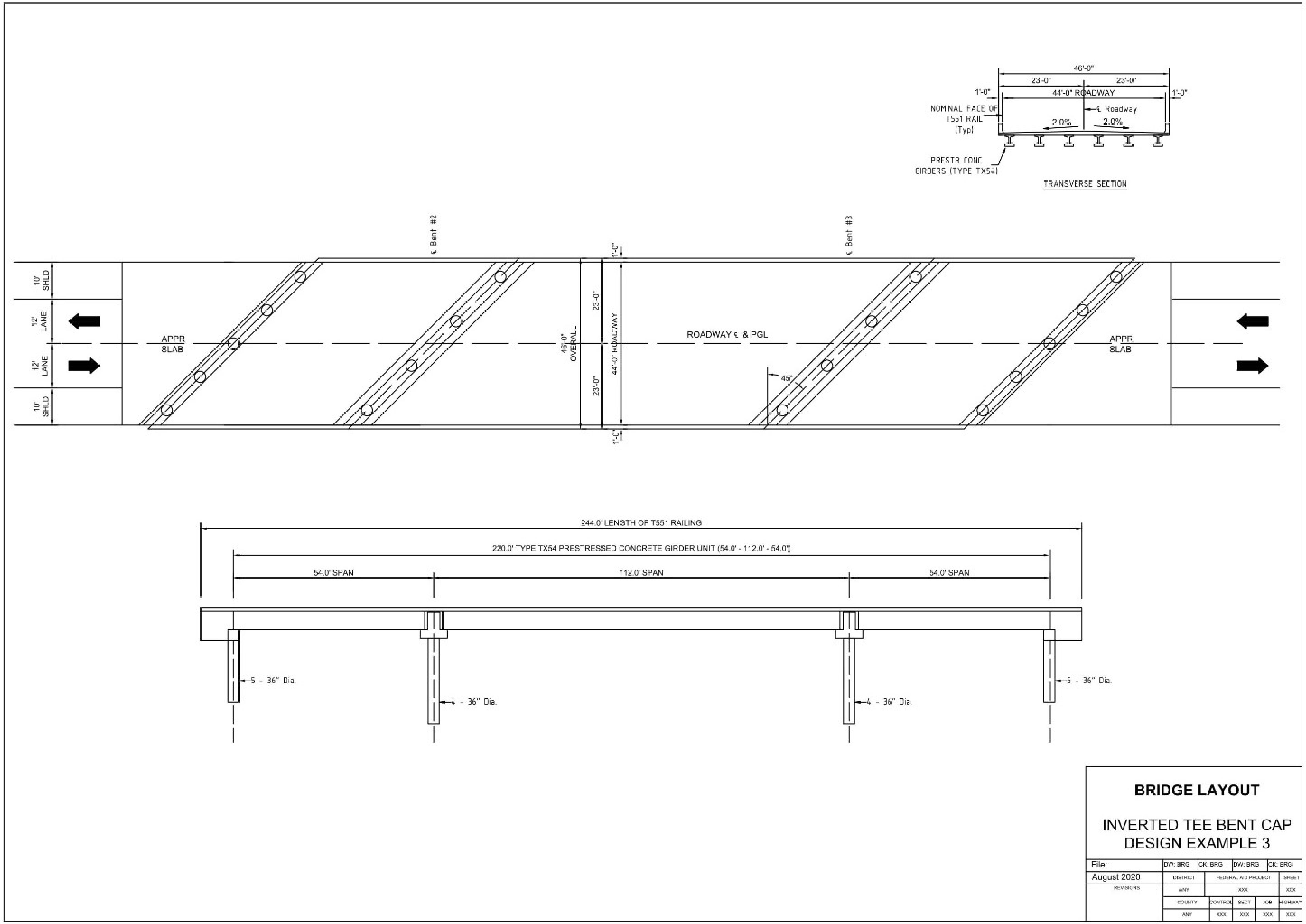
$a = 6 \text{ in} < s_{req}$ SkinSpacing = “OK!”

Skin Reinforcement Summary:

Use 7 ~ # 6 bars in Stem and 3 ~ # 6 bars in Ledge on each side

4.4.15 Design Details and Drawings

4.4.15.1 Bridge layout



4.4.15.2 CAP 18 Input File

```
$File                        Proj               User  Date (Today
$ Num     County      Highway  Num     CSJ      Init     if Blank) Comment
$XXXX  XXXXXXXXXXXXX  XXXXXX  XXXX  XXXX-XX-XXX  XXX  XXXXXXXXXXXX XXXXXXX
00001  ___County____  Highwy  Pro#  0000-00-000  BRG               Comment
$Header Card 2 ---------------------------------------------------------------
$XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
CAP18 Version 6.00 ITBC Design Example 3, Skew = 45.00
$Problem Card ----------------------------------------------------------------
$Prob E   XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
   1 E 0 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay)
$TABLE 1 - CONTROL DATA ------------------------------------------------------
$                Enter 1 to keep:       Number cards   Options:
$                 Env Tab2 Tab3 Tab4     on Table 4  Envelope  Print   Skew Angle
$                  X    X    X    X              XX    X        XX     XXXXXXXXXX
                                                 16                         45.0
$TABLE 2 - CONSTANTS ---------------------------------------------------------
$   TABLE 2a                                                Anly Opt (1=Working,
$                               |-Movable Load Data--|  2=Load Factor,3=Both)
$                Num  Increment    |Num  Start Stop Step|Anly|  Load Factors:
$                Inc   Length      |Inc  Sta   Sta  Size| Opt| Dead      Live
$                 XX XXXXXXXXX       XXX  XXX  XXX    X    X XXXXXXXX XXXXXXXX
                  92      0.5         20    2   70    1    3    1.25      1.75
$   TABLE 2b
$     Overlay    Max #|-----------Live Load Reduction Factors---------|
$   Load Factor  Lanes| 1 lane   2 lanes   3 lanes   4 lanes   5 lanes
$     XXXXX          X  XXXX      XXXX      XXXX      XXXX      XXXX
      1.50           3  1.2       1.0       0.85      0.65      0.65
$TABLE 3 - LIST OF STATIONS --------------------------------------------------
$      Number of input values for               Str - Stringers, Sup - Supports
$                Lane Str  Sup  MCP  VCP        MCP - Moment Control Points
$                 XX   XX   XX   XX   XX        VCP - Shear Control Points
  (Num Inputs)     3    6    4   11    8
$      Left Lane Boundary Stations
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Lane Left)      2   32   60
$      Right Lane Boundary Stations
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Lane Right)    32   60   90
$      Station of Stringers (two rows max, may be at tenths of stations, XX.X)
$               XXXX XXXX XXXX XXXX XXXX XXXX XXXX XXXX XXXX XXXX
  (Stringers)      6   22   38   54   70   86
$      Station of Supports (two rows max)
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Supports)      10   34   58   82
$      Moment Control Point Stations (two rows max)
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Mom CP)         6   10   22   34   38   46   54   58   70   82
  (Mom CP)        86
$      Shear Control Point Stations (two rows max)
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Shear CP)       8   12   32   36   56   60   80   84
$TABLE 4 - STIFFNESS AND LOAD DATA -------------------------------------------
$                               Bending   Sidewalk,  Cap &
$                Station  1 if  Stiffness  Slab      Stringer  Moving    Overlay
$Comments       From  To Cont'd  of Cap    Loads     Loads     Loads     Loads,DW
$XXXXXXXXXXXXXX  XXX  XXX    X XXXXXXXXX XXXXXXXXX XXXXXXXXX XXXXXXXXX XXXXXXXXX
(CAP EI & DL)      2   90      8.66E+07            -2.589
(DL Span1, Bm1)    6    6                          -50.17              -5.04
(DL Span1, Bm2)   22   22                          -50.17              -5.04
(DL Span1, Bm3)   38   38                          -50.17              -5.04
(DL Span1, Bm4)   54   54                          -50.17              -5.04
(DL Span1, Bm5)   70   70                          -50.17              -5.04
(DL Span1, Bm6)   86   86                          -50.17              -5.04
(DL Span2, Bm1)    6    6                          -104.1              -10.5
(DL Span2, Bm2)   22   22                          -104.1              -10.5
(DL Span2, Bm3)   38   38                          -104.1              -10.5
(DL Span2, Bm4)   54   54                          -104.1              -10.5
(DL Span2, Bm5)   70   70                          -104.1              -10.5
(DL Span2, Bm6)   86   86                          -104.1              -10.5
(Dist. Lane Ld)    0   20                                    -4.92
(Conc. Lane Ld)    4    4                                    -21.3
(Conc. Lane Ld)   16   16                                    -21.3
```

4.4.15.3 CAP 18 Output File

```
AUG 11, 2020     TEXAS DEPARTMENT OF TRANSPORTATION (TxDOT)        PAGE  1
CAP18                 BENT CAP ANALYSIS      Ver.  6.2  (Jul, 2011)

   PSF             HIGHWAY  PD-    CONTROL-   CODED
    NO     COUNTY       NO    IPE  SECTION-JOB   BY    DATE
  00001  ___County____  Highwy  Pro#  0000-00-000  BRG  AUG 11, 2020 Comment

CAP18 Version 6.00 ITBC Design Example 3, Skew = 45.00
PROB     1   (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay


ENGLISH SYSTEM UNITS

TABLE 1.  CONTROL DATA

   OPTION TO PRINT TABLE SRS (1=YES)                          0

                              ENVELOPES    TABLE NUMBER
                              OF MAXIMUMS   2   3   4
   KEEP FROM PRECEDING PROBLEM (1=YES)           0      0   0   0
   CARDS INPUT THIS PROBLEM                            16

   OPTION TO CLEAR ENVELOPES BEFORE LANE LOADINGS (1=YES)             0

   OPTION TO OMIT PRINT FOR TABLES (TABLE DESIGNATIONS IN PARENTHESES)
     -1(4A), -2(5) -3(4A,5), -4(4A,5,6), -5(4A,5,6,7):             0

   SKEW ANGLE, DEGREES                                45.000


TABLE 2.  CONSTANTS

   NUMBER OF INCREMENTS FOR SLAB AND CAP                         92
   INCREMENT LENGTH, FT                                0.500
   NUMBER OF INCREMENTS FOR MOVABLE LOAD                          20
   START POSITION OF MOVABLE-LOAD STA ZERO                        2
   STOP POSITION OF MOVABLE-LOAD STA ZERO                        70
   NUMBER OF INCREMENTS BETWEEN EACH POSITION OF MOVABLE LOAD          1

   ANALYSIS OPTION (1=WORKING STRESS, 2=LOAD FACTOR, 3=BOTH)          3

   LOAD FACTOR FOR DEAD LOAD                             1.25
   LOAD FACTOR FOR OVERLAY LOAD                            1.50
   LOAD FACTOR FOR LIVE LOAD                            1.75

   MAXIMUM NUMBER OF LANES TO BE LOADED SIMULTANEOUSLY              3

   LIST OF LOAD COEFFICIENTS CORRESPONDING TO NUMBER OF LANES LOADED
       1           2           3           4           5
     1.200       1.000       0.850
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 3. LISTS OF STATIONS

	NUM OF LANES	NUM OF STRINGERS	NUM OF SUPPORTS	NUM MOM CONTR PTS	NUM SHEAR CONTR PTS
TOTAL	3	6	4	11	8

LANE LEFT 2 32 60

LANE RIGHT 32 60 90
STRINGERS 6.0 22.0 38.0 54.0 70.0 86.0
SUPPORTS 10 34 58 82
MOM CONTR 6 10 22 34 38 46 54 58 70 82
86
SHEAR CONTR 8 12 32 36 56 60 80 84

TABLE 4. STIFFNESS AND LOAD DATA

STA FROM	STA TO	FIXED-OR-MOVABLE CONTD IF=1	CAP BENDING STIFFNESS (K-FT*FT)	SIDEWALK, SLAB LOADS (K)	STRINGER, CAP LOADS (K)	OVERLAY LOADS (K)	MOVABLE-POSITION SLAB LOADS (K)
2	90	0	86600000.000	0.000	-2.589	0.000	0.000
6	6	0	0.000	0.000	-50.170	-5.040	0.000
22	22	0	0.000	0.000	-50.170	-5.040	0.000
38	38	0	0.000	0.000	-50.170	-5.040	0.000
54	54	0	0.000	0.000	-50.170	-5.040	0.000
70	70	0	0.000	0.000	-50.170	-5.040	0.000
86	86	0	0.000	0.000	-50.170	-5.040	0.000
6	6	0	0.000	0.000	-104.100	-10.500	0.000
22	22	0	0.000	0.000	-104.100	-10.500	0.000
38	38	0	0.000	0.000	-104.100	-10.500	0.000
54	54	0	0.000	0.000	-104.100	-10.500	0.000
70	70	0	0.000	0.000	-104.100	-10.500	0.000
86	86	0	0.000	0.000	-104.100	-10.500	0.000
0	20	0	0.000	0.000	0.000	0.000	-4.920
4	4	0	0.000	0.000	0.000	0.000	-21.300
16	16	0	0.000	0.000	0.000	0.000	-21.300

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 4A. DEAD LOAD RESULTS (WORKING STRESS)

STA	DIST X (FT)	DEFLECTION (FT)	MOMENT (K-FT)	SHEAR (K)
-1	-0.71	0.000000	0.0	0.0
0	0.00	0.000000	0.0	0.0
1	0.71	-0.000087	0.0	0.0
2	1.41	-0.000076	0.0	-0.9
3	2.12	-0.000065	-1.3	-3.7
4	2.83	-0.000055	-5.2	-7.3
5	3.54	-0.000044	-11.7	-11.0
6	4.24	-0.000033	-20.7	-99.6
7	4.95	-0.000023	-152.4	-188.1
8	5.66	-0.000013	-286.7	-191.8
9	6.36	-0.000005	-423.7	-195.4
10	7.07	0.000000	-563.1	-33.0
11	7.78	0.000002	-470.4	129.3
12	8.49	0.000002	-380.2	125.7
13	9.19	-0.000001	-292.7	122.0
14	9.90	-0.000005	-207.7	118.4
15	10.61	-0.000011	-125.3	114.7
16	11.31	-0.000017	-45.5	111.0
17	12.02	-0.000024	31.8	107.4
18	12.73	-0.000030	106.4	103.7
19	13.44	-0.000036	178.4	100.0
20	14.14	-0.000041	247.9	96.4
21	14.85	-0.000044	314.7	92.7
22	15.56	-0.000046	379.0	4.2
23	16.26	-0.000045	320.6	-84.4
24	16.97	-0.000042	259.6	-88.1
25	17.68	-0.000038	196.1	-91.7
26	18.38	-0.000033	129.9	-95.4
27	19.09	-0.000027	61.2	-99.1
28	19.80	-0.000021	-10.2	-102.7
29	20.51	-0.000015	-84.1	-106.4
30	21.21	-0.000009	-160.6	-110.0
31	21.92	-0.000004	-239.7	-113.7
32	22.63	-0.000001	-321.4	-117.4
33	23.33	0.000001	-405.7	-121.0
34	24.04	0.000000	-492.5	44.5
35	24.75	-0.000004	-342.7	210.1
36	25.46	-0.000009	-195.4	206.4
37	26.16	-0.000016	-50.8	202.8
38	26.87	-0.000023	91.3	114.2
39	27.58	-0.000029	110.7	25.6
40	28.28	-0.000035	127.6	22.0
41	28.99	-0.000040	141.8	18.3
42	29.70	-0.000045	153.4	14.6
43	30.41	-0.000048	162.5	11.0

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 4A. DEAD LOAD RESULTS (WORKING STRESS)

STA	DIST X (FT)	DEFLECTION (FT)	MOMENT (K-FT)	SHEAR (K)
44	31.11	-0.000051	169.0	7.3
45	31.82	-0.000052	172.9	3.7
46	32.53	-0.000053	174.2	0.0
47	33.23	-0.000052	172.9	-3.7
48	33.94	-0.000051	169.0	-7.3
49	34.65	-0.000048	162.5	-11.0
50	35.36	-0.000045	153.4	-14.6
51	36.06	-0.000040	141.8	-18.3
52	36.77	-0.000035	127.6	-22.0
53	37.48	-0.000029	110.7	-25.6
54	38.18	-0.000023	91.3	-114.2
55	38.89	-0.000016	-50.8	-202.8
56	39.60	-0.000009	-195.4	-206.4
57	40.31	-0.000004	-342.7	-210.1
58	41.01	0.000000	-492.5	-44.5
59	41.72	0.000001	-405.7	121.0
60	42.43	-0.000001	-321.4	117.4
61	43.13	-0.000004	-239.7	113.7
62	43.84	-0.000009	-160.6	110.0
63	44.55	-0.000015	-84.1	106.4
64	45.25	-0.000021	-10.2	102.7
65	45.96	-0.000027	61.2	99.1
66	46.67	-0.000033	129.9	95.4
67	47.38	-0.000038	196.1	91.7
68	48.08	-0.000042	259.6	88.1
69	48.79	-0.000045	320.6	84.4
70	49.50	-0.000046	379.0	-4.2
71	50.20	-0.000044	314.7	-92.7
72	50.91	-0.000041	247.9	-96.4
73	51.62	-0.000036	178.4	-100.0
74	52.33	-0.000030	106.4	-103.7
75	53.03	-0.000024	31.8	-107.4
76	53.74	-0.000017	-45.5	-111.0
77	54.45	-0.000011	-125.3	-114.7
78	55.15	-0.000005	-207.7	-118.4
79	55.86	-0.000001	-292.7	-122.0
80	56.57	0.000002	-380.2	-125.7
81	57.28	0.000002	-470.4	-129.3
82	57.98	0.000000	-563.1	33.0
83	58.69	-0.000005	-423.7	195.4
84	59.40	-0.000013	-286.7	191.8
85	60.10	-0.000023	-152.4	188.1
86	60.81	-0.000033	-20.7	99.6
87	61.52	-0.000044	-11.7	11.0
88	62.23	-0.000055	-5.2	7.3
89	62.93	-0.000065	-1.3	3.7
90	63.64	-0.000076	0.0	0.9

```
  91     64.35     -0.000087      0.0       0.0
  92     65.05      0.000000      0.0       0.0
  93     65.76      0.000000      0.0       0.0

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PROB     1  (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)


TABLE 5.  MULTI-LANE LOADING SUMMARY    ( WORKING STRESS )
        ( *--CRITICAL NUMBER OF LANE LOADS)

    MOMENT ( FT-K )
    ------------------------------------------------------------------------
    AT    DEAD LD   LANE   POSITIVE   LOAD AT    LANE   NEGATIVE   LOAD AT
    STA    EFFECT   ORDER  MAXIMUM    LANE STA   ORDER  MAXIMUM    LANE STA
    ------------------------------------------------------------------------

     6    -20.7
              0      0.0              0     0.0
              1      0.0              1     0.0
              2      0.0              2     0.0
              3      0.0              3     0.0
              0*                      0*

    10    -563.1
              0      0.0              0   -249.3     1   2
              1      0.0              1   -249.3     1   2
              2      0.0              2     0.0
              3      0.0              3     0.0
              0*                      0*

    22     379.0
              0    285.7     0  13    0     -47.2     2  36
              1    284.5     1  12    1     -47.2     2  36
              2     13.2     3  62    2       0.0
              3      0.0              3     0.0
              0*                      0*

    34    -492.5
              0     26.4     3  62    0    -192.8     0  18
              1     26.4     3  62    1    -164.8     1  12
              2      0.0              2   -119.8     2  32
              3      0.0              3     0.0
              0*                      2*

    38      91.3
              0    118.2     2  32    0     -83.2     1   9
              1    118.2     2  32    1     -83.2     1   9
              2      4.5     3  62    2       0.0
              3      0.0              3     0.0
              0*                      0*

    46     174.2
              0     98.1     2  36    0     -39.3     1   9
              1     98.1     2  36    1     -39.3     1   9
              2      0.0              2     -39.3     3  63
              3      0.0              3     0.0
              0*                      2*

    54      91.3
              0    118.2     2  40    0     -83.2     3  63
              1    118.2     2  40    1     -83.2     3  63
              2      4.5     1  10    2       0.0
              3      0.0              3     0.0
              0*                      0*

    58    -492.5
              0     26.4     1   9    0    -192.8     0  54
              1     26.4     1   9    1    -164.8     3  60
              2      0.0              2   -119.8     2  40
              3      0.0              3     0.0
              0*                      2*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

```
MOMENT ( FT-K )
----------------------------------------------------------------------
AT   DEAD LD  LANE   POSITIVE  LOAD AT   LANE   NEGATIVE  LOAD AT
STA  EFFECT   ORDER  MAXIMUM   LANE STA  ORDER  MAXIMUM   LANE STA
----------------------------------------------------------------------

70   379.0
          0    285.7   0  59    0    -47.2    2  36
          1    284.5   3  60    1    -47.2    2  36
          2     13.2   1   9    2      0.0
          3      0.0            3      0.0
          0*                    0*

82   -563.1
          0      0.0            0   -249.3    3  70
          1      0.0            1   -249.3    3  70
          2      0.0            2      0.0
          3      0.0            3      0.0
          0*                    0*

86   -20.7
          0      0.0            0      0.0
          1      0.0            1      0.0
          2      0.0            2      0.0
          3      0.0            3      0.0
          0*                    0*
```

PROB   1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

```
   SHEAR ( K )
   ------------------------------------------------------------------
   AT   DEAD LD  LANE  POSITIVE  LOAD AT  LANE  NEGATIVE  LOAD AT
   STA   EFFECT  ORDER MAXIMUM   LANE STA  ORDER MAXIMUM   LANE STA
   ------------------------------------------------------------------

    8   -191.8
             0     0.0            0   -88.1    1   2
             1     0.0            1   -88.1    1   2
             2     0.0            2     0.0
             3     0.0            3     0.0
             0*                   0*

   12    125.7
             0    44.8    1   6   0    -5.6    2  36
             1    44.8    1   6   1    -5.6    2  36
             2     1.6    3  62   2     0.0
             3     0.0            3     0.0
             0*                   0*

   32   -117.4
             0     1.6    3  62   0   -54.6    0  15
             1     1.6    3  62   1   -53.0    1  12
             2     0.0            2   -11.2    2  32
             3     0.0            3     0.0
             0*                   0*

   36    206.4
             0    87.6    0  28   0    -7.8    3  63
             1    84.1    2  32   1    -7.8    3  63
             2    30.7    1  12   2     0.0
             3     0.0            3     0.0
             2*                   0*

   56   -206.4
             0     7.8    1   9   0   -87.6    0  44
             1     7.8    1   9   1   -84.1    2  40
             2     0.0            2   -30.7    3  60
             3     0.0            3     0.0
             0*                   2*

   60    117.4
             0    54.6    0  57   0    -1.6    1   9
             1    53.0    3  60   1    -1.6    1   9
             2    11.2    2  40   2     0.0
             3     0.0            3     0.0
             0*                   0*

   80   -125.7
             0     5.6    2  36   0   -44.8    3  66
             1     5.6    2  36   1   -44.8    3  66
             2     0.0            2    -1.6    1   9
             3     0.0            3     0.0
             0*                   0*

   84    191.8
             0    88.1    3  70   0     0.0
             1    88.1    3  70   1     0.0
             2     0.0            2     0.0
             3     0.0            3     0.0
             0*                   0*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

```
 REACTION ( K )
 ----------------------------------------------------------------------
 AT   DEAD LD  LANE  POSITIVE  LOAD AT   LANE  NEGATIVE  LOAD AT
 STA   EFFECT  ORDER  MAXIMUM   LANE STA  ORDER  MAXIMUM   LANE STA
 ----------------------------------------------------------------------

 10   332.1
          0    127.9    1  2   0     -5.6    2  36
          1    127.9    1  2   1     -5.6    2  36
          2      1.6    3  62  2      0.0
          3      0.0           3      0.0
          0*                   0*

 34   338.4
          0    117.1    0  22  0     -9.3    3  63
          1     95.3    2  32  1     -9.3    3  63
          2     83.6    1  12  2      0.0
          3      0.0           3      0.0
          2*                   0*

 58   338.4
          0    117.1    0  50  0     -9.3    1   9
          1     95.3    2  40  1     -9.3    1   9
          2     83.6    3  60  2      0.0
          3      0.0           3      0.0
          2*                   0*

 82   332.1
          0    127.9    3  70  0     -5.6    2  36
          1    127.9    3  70  1     -5.6    2  36
          2      1.6    1  9   2      0.0
          3      0.0           3      0.0
          0*                   0*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
-1	-0.71	0.0	0.0	0.0	0.0
0	0.00	0.0	0.0	0.0	0.0
1	0.71	0.0	0.0	0.0	0.0
2	1.41	0.0	0.0	-0.9	-0.9
3	2.12	-1.3	-1.3	-3.7	-3.7
4	2.83	-5.2	-5.2	-7.3	-7.3
5	3.54	-11.7	-11.7	-11.0	-11.0
6	4.24	-20.7	-20.7	-99.6	-152.4
7	4.95	-152.4	-227.2	-188.1	-293.9
8	5.66	-286.7	-436.3	-191.8	-297.5
9	6.36	-423.7	-648.0	-195.4	-301.2
10	7.07	-563.1	-862.2	-16.1	-62.1
11	7.78	-451.8	-735.8	183.1	122.7
12	8.49	-336.6	-611.9	179.5	119.0
13	9.19	-223.0	-490.6	175.8	115.3
14	9.90	-112.0	-371.9	172.1	111.7
15	10.61	-3.1	-255.7	168.5	108.0
16	11.31	104.9	-142.2	164.8	104.4
17	12.02	211.8	-31.3	161.2	100.7
18	12.73	317.6	68.6	157.5	97.0
19	13.44	421.4	136.0	153.8	93.4
20	14.14	523.6	200.7	150.2	89.7
21	14.85	623.6	262.8	146.5	86.1
22	15.56	721.8	322.4	20.1	-9.0
23	16.26	617.7	258.8	-82.5	-150.0
24	16.97	511.2	192.4	-86.2	-153.6
25	17.68	402.7	123.0	-89.9	-157.3
26	18.38	291.8	50.8	-93.5	-160.9
27	19.09	179.5	-24.5	-97.2	-164.6
28	19.80	73.6	-102.4	-100.8	-168.3
29	20.51	-32.8	-182.8	-104.5	-171.9
30	21.21	-134.2	-266.5	-108.2	-175.6
31	21.92	-212.0	-388.3	-111.8	-179.3
32	22.63	-292.3	-515.3	-115.5	-182.9
33	23.33	-375.3	-645.0	-119.2	-186.6
34	24.04	-460.8	-777.2	88.3	27.0
35	24.75	-317.6	-546.1	324.9	200.8
36	25.46	-176.9	-342.7	321.2	197.1
37	26.16	20.9	-171.0	317.6	193.5
38	26.87	233.2	-8.5	172.6	104.9
39	27.58	248.4	17.5	34.9	16.3
40	28.28	261.4	40.9	31.3	12.7
41	28.99	272.3	61.7	27.6	9.0
42	29.70	281.0	74.8	23.9	5.3

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
43	30.41	287.1	83.8	20.3	1.7
44	31.11	290.6	90.3	16.6	-2.0
45	31.82	291.5	94.2	13.0	-5.6
46	32.53	291.9	95.5	9.3	-9.3
47	33.23	291.5	94.2	5.6	-13.0
48	33.94	290.6	90.3	2.0	-16.6
49	34.65	287.1	83.8	-1.7	-20.3
50	35.36	281.0	74.8	-5.3	-23.9
51	36.06	272.3	61.7	-9.0	-27.6
52	36.77	261.4	40.9	-12.7	-31.3
53	37.48	248.4	17.5	-16.3	-34.9
54	38.18	233.2	-8.5	-104.9	-172.6
55	38.89	20.9	-171.0	-193.5	-317.6
56	39.60	-176.9	-342.7	-197.1	-321.2
57	40.31	-317.6	-546.1	-200.8	-324.9
58	41.01	-460.8	-777.2	-27.0	-88.3
59	41.72	-375.3	-645.0	186.6	119.2
60	42.43	-292.3	-515.3	182.9	115.5
61	43.13	-212.0	-388.3	179.3	111.8
62	43.84	-134.2	-266.5	175.6	108.2
63	44.55	-32.8	-182.8	171.9	104.5
64	45.25	73.6	-102.4	168.3	100.8
65	45.96	179.5	-24.5	164.6	97.2
66	46.67	291.8	50.8	160.9	93.5
67	47.38	402.7	123.0	157.3	89.9
68	48.08	511.2	192.4	153.6	86.2
69	48.79	617.7	258.8	150.0	82.5
70	49.50	721.8	322.4	9.0	-20.1
71	50.20	623.6	262.8	-86.1	-146.5
72	50.91	523.6	200.7	-89.7	-150.2
73	51.62	421.4	136.0	-93.4	-153.8
74	52.33	317.6	68.6	-97.0	-157.5
75	53.03	211.8	-31.3	-100.7	-161.2
76	53.74	104.9	-142.2	-104.4	-164.8
77	54.45	-3.1	-255.7	-108.0	-168.5
78	55.15	-112.0	-371.9	-111.7	-172.1
79	55.86	-223.0	-490.6	-115.3	-175.8
80	56.57	-336.6	-611.9	-119.0	-179.5
81	57.28	-451.8	-735.8	-122.7	-183.1
82	57.98	-563.1	-862.2	62.1	16.1
83	58.69	-423.7	-648.0	301.2	195.4
84	59.40	-286.7	-436.3	297.5	191.8
85	60.10	-152.4	-227.2	293.9	188.1
86	60.81	-20.7	-20.7	152.4	99.6

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
87	61.52	-11.7	-11.7	11.0	11.0
88	62.23	-5.2	-5.2	7.3	7.3
89	62.93	-1.3	-1.3	3.7	3.7
90	63.64	0.0	0.0	0.9	0.9
91	64.35	0.0	0.0	0.0	0.0
92	65.05	0.0	0.0	0.0	0.0
93	65.76	0.0	0.0	0.0	0.0

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 7. MAXIMUM SUPPORT REACTIONS (WORKING STRESS)

STA	DIST X (FT)	MAX + REACT (K)	MAX - REACT (K)
10	7.07	485.6	325.4
34	24.04	517.3	327.3
58	41.01	517.3	327.3
82	57.98	485.6	325.4

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 5. MULTI-LANE LOADING SUMMARY (LOAD FACTOR)
(*--CRITICAL NUMBER OF LANE LOADS)

MOMENT (FT-K)

AT STA	DEAD LD EFFECT	LANE ORDER	POSITIVE MAXIMUM	LOAD AT LANE STA	LANE ORDER	NEGATIVE MAXIMUM	LOAD AT LANE STA
6	-25.9						
		0	0.0		0	0.0	
		1	0.0		1	0.0	
		2	0.0		2	0.0	
		3	0.0		3	0.0	
		0*			0*		
10	-714.9						
		0	0.0		0	-436.2	1 2
		1	0.0		1	-436.2	1 2
		2	0.0		2	0.0	
		3	0.0		3	0.0	
		0*			0*		
22	480.6						
		0	499.9	0 13	0	-82.6	2 36
		1	497.9	1 12	1	-82.6	2 36
		2	23.1	3 62	2	0.0	
		3	0.0		3	0.0	
		0*			0*		
34	-623.9						
		0	46.2	3 62	0	-337.3	0 18
		1	46.2	3 62	1	-288.5	1 12
		2	0.0		2	-209.6	2 32
		3	0.0		3	0.0	
		0*			2*		
38	116.9						
		0	206.9	2 32	0	-145.6	1 9
		1	206.9	2 32	1	-145.6	1 9
		2	8.0	3 62	2	0.0	
		3	0.0		3	0.0	
		0*			0*		
46	220.4						
		0	171.6	2 36	0	-68.9	1 9
		1	171.6	2 36	1	-68.9	1 9
		2	0.0		2	-68.9	3 63
		3	0.0		3	0.0	
		0*			2*		
54	116.9						
		0	206.9	2 40	0	-145.6	3 63
		1	206.9	2 40	1	-145.6	3 63
		2	8.0	1 10	2	0.0	
		3	0.0		3	0.0	
		0*			0*		
58	-623.9						
		0	46.2	1 9	0	-337.3	0 54
		1	46.2	1 9	1	-288.5	3 60
		2	0.0		2	-209.6	2 40
		3	0.0		3	0.0	
		0*			2*		

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

MOMENT (FT-K)

AT STA	DEAD LD EFFECT	LANE ORDER	POSITIVE MAXIMUM	LOAD AT LANE STA	LANE ORDER	NEGATIVE MAXIMUM	LOAD AT LANE STA
70	480.6						
		0	499.9	0 59	0	-82.6	2 36
		1	497.9	3 60	1	-82.6	2 36
		2	23.1	1 9	2	0.0	
		3	0.0		3	0.0	
		0*			0*		
82	-714.9						
		0	0.0		0	-436.2	3 70
		1	0.0		1	-436.2	3 70
		2	0.0		2	0.0	
		3	0.0		3	0.0	
		0*			0*		
86	-25.9						
		0	0.0		0	0.0	
		1	0.0		1	0.0	
		2	0.0		2	0.0	
		3	0.0		3	0.0	
		0*			0*		

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

```
 SHEAR ( K )
 ----------------------------------------------------------------------
 AT   DEAD LD  LANE  POSITIVE  LOAD AT   LANE  NEGATIVE  LOAD AT
 STA   EFFECT  ORDER MAXIMUM   LANE STA  ORDER MAXIMUM   LANE STA
 ----------------------------------------------------------------------

  8   -243.6
          0     0.0           0   -154.2   1  2
          1     0.0           1   -154.2   1  2
          2     0.0           2     0.0
          3     0.0           3     0.0
          0*                  0*

 12    159.2
          0    78.4    1  6   0     -9.7    2  36
          1    78.4    1  6   1     -9.7    2  36
          2     2.7    3 62   2      0.0
          3     0.0           3     0.0
          0*                  0*

 32   -148.5
          0     2.7    3 62   0    -95.6    0  15
          1     2.7    3 62   1    -92.7    1  12
          2     0.0           2   -19.5   2  32
          3     0.0           3     0.0
          0*                  0*

 36    261.9
          0   153.2    0 28   0    -13.6    3  63
          1   147.2    2 32   1    -13.6    3  63
          2    53.7    1 12   2      0.0
          3     0.0           3     0.0
          2*                  0*

 56   -261.9
          0    13.6    1  9   0   -153.2    0  44
          1    13.6    1  9   1   -147.2    2  40
          2     0.0           2   -53.7   3  60
          3     0.0           3     0.0
          0*                  2*

 60    148.5
          0    95.6    0 57   0     -2.7    1  9
          1    92.7    3 60   1     -2.7    1  9
          2    19.5    2 40   2      0.0
          3     0.0           3     0.0
          0*                  0*

 80   -159.2
          0     9.7    2 36   0    -78.4    3  66
          1     9.7    2 36   1    -78.4    3  66
          2     0.0           2    -2.7   1  9
          3     0.0           3     0.0
          0*                  0*

 84    243.6
          0   154.2    3 70   0      0.0
          1   154.2    3 70   1      0.0
          2     0.0           2     0.0
          3     0.0           3     0.0
          0*                  0*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

REACTION (K)

AT STA	DEAD LD EFFECT	LANE ORDER	POSITIVE MAXIMUM	LOAD AT LANE STA	LANE ORDER	NEGATIVE MAXIMUM	LOAD AT LANE STA
10	421.1						
		0	223.8	1 2	0	-9.7	2 36
		1	223.8	1 2	1	-9.7	2 36
		2	2.7	3 62	2	0.0	
		3	0.0		3	0.0	
		0*			0*		
34	428.7						
		0	205.0	0 22	0	-16.3	3 63
		1	166.8	2 32	1	-16.3	3 63
		2	146.3	1 12	2	0.0	
		3	0.0		3	0.0	
		2*			0*		
58	428.7						
		0	205.0	0 50	0	-16.3	1 9
		1	166.8	2 40	1	-16.3	1 9
		2	146.3	3 60	2	0.0	
		3	0.0		3	0.0	
		2*			0*		
82	421.1						
		0	223.8	3 70	0	-9.7	2 36
		1	223.8	3 70	1	-9.7	2 36
		2	2.7	1 9	2	0.0	
		3	0.0		3	0.0	
		0*			0*		

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
-1	-0.71	0.0	0.0	0.0	0.0
0	0.00	0.0	0.0	0.0	0.0
1	0.71	0.0	0.0	0.0	0.0
2	1.41	0.0	0.0	-1.1	-1.1
3	2.12	-1.6	-1.6	-4.6	-4.6
4	2.83	-6.5	-6.5	-9.2	-9.2
5	3.54	-14.6	-14.6	-13.7	-13.7
6	4.24	-25.9	-25.9	-126.4	-218.9
7	4.95	-193.3	-324.2	-239.0	-424.1
8	5.66	-363.9	-625.6	-243.6	-428.7
9	6.36	-537.8	-930.4	-248.2	-433.2
10	7.07	-714.9	-1238.4	-12.6	-93.0
11	7.78	-565.0	-1061.9	257.9	152.1
12	8.49	-406.9	-888.7	253.3	147.5
13	9.19	-250.4	-718.7	248.8	142.9
14	9.90	-97.2	-552.0	244.2	138.4
15	10.61	53.6	-388.5	239.6	133.8
16	11.31	204.3	-228.2	235.0	129.2
17	12.02	354.3	-71.1	230.5	124.6
18	12.73	503.5	67.8	225.9	120.1
19	13.44	650.7	151.1	221.3	115.5
20	14.14	796.2	231.2	216.7	110.9
21	14.85	939.3	308.0	212.1	106.3
22	15.56	1080.5	381.5	33.3	-17.6
23	16.26	926.2	298.1	-104.0	-222.0
24	16.97	769.1	211.2	-108.6	-226.6
25	17.68	609.7	120.3	-113.2	-231.2
26	18.38	447.6	25.7	-117.7	-235.7
27	19.09	284.0	-72.9	-122.3	-240.3
28	19.80	133.3	-174.7	-126.9	-244.9
29	20.51	-17.3	-279.8	-131.5	-249.5
30	21.21	-157.7	-389.2	-136.1	-254.0
31	21.92	-255.5	-564.2	-140.6	-258.6
32	22.63	-356.6	-746.9	-145.2	-263.2
33	23.33	-460.9	-932.8	-149.8	-267.8
34	24.04	-568.4	-1122.0	133.4	26.0
35	24.75	-389.9	-789.9	467.4	250.2
36	25.46	-214.5	-504.8	462.8	245.6
37	26.16	62.0	-273.8	458.2	241.1
38	26.87	365.1	-57.8	247.0	128.4
39	27.58	382.0	-22.0	48.3	15.8
40	28.28	396.5	10.5	43.7	11.2
41	28.99	408.3	39.8	39.2	6.6
42	29.70	417.7	56.8	34.6	2.0
43	30.41	423.9	68.2	30.0	-2.5
44	31.11	426.8	76.3	25.4	-7.1

45	31.82	426.5	81.1	20.9	-11.7
46	32.53	426.4	82.7	16.3	-16.3
47	33.23	426.5	81.1	11.7	-20.9
48	33.94	426.8	76.3	7.1	-25.4

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PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
49	34.65	423.9	68.2	2.5	-30.0
50	35.36	417.7	56.8	-2.0	-34.6
51	36.06	408.3	39.8	-6.6	-39.2
52	36.77	396.5	10.5	-11.2	-43.7
53	37.48	382.0	-22.0	-15.8	-48.3
54	38.18	365.1	-57.8	-128.4	-247.0
55	38.89	62.0	-273.8	-241.1	-458.2
56	39.60	-214.5	-504.8	-245.6	-462.8
57	40.31	-389.9	-789.9	-250.2	-467.4
58	41.01	-568.4	-1122.0	-26.0	-133.4
59	41.72	-460.9	-932.8	267.8	149.8
60	42.43	-356.6	-746.9	263.2	145.2
61	43.13	-255.5	-564.2	258.6	140.6
62	43.84	-157.7	-389.2	254.0	136.1
63	44.55	-17.3	-279.8	249.5	131.5
64	45.25	133.3	-174.7	244.9	126.9
65	45.96	284.0	-72.9	240.3	122.3
66	46.67	447.6	25.7	235.7	117.7
67	47.38	609.7	120.3	231.2	113.2
68	48.08	769.1	211.2	226.6	108.6
69	48.79	926.2	298.1	222.0	104.0
70	49.50	1080.5	381.5	17.6	-33.3
71	50.20	939.3	308.0	-106.3	-212.1
72	50.91	796.2	231.2	-110.9	-216.7
73	51.62	650.7	151.1	-115.5	-221.3
74	52.33	503.5	67.8	-120.1	-225.9
75	53.03	354.3	-71.1	-124.6	-230.5
76	53.74	204.3	-228.2	-129.2	-235.0
77	54.45	53.6	-388.5	-133.8	-239.6
78	55.15	-97.2	-552.0	-138.4	-244.2
79	55.86	-250.4	-718.7	-142.9	-248.8
80	56.57	-406.9	-888.7	-147.5	-253.3
81	57.28	-565.0	-1061.9	-152.1	-257.9
82	57.98	-714.9	-1238.4	93.0	12.6
83	58.69	-537.8	-930.4	433.2	248.2
84	59.40	-363.9	-625.6	428.7	243.6
85	60.10	-193.3	-324.2	424.1	239.0
86	60.81	-25.9	-25.9	218.9	126.4
87	61.52	-14.6	-14.6	13.7	13.7
88	62.23	-6.5	-6.5	9.2	9.2
89	62.93	-1.6	-1.6	4.6	4.6

90	63.64	0.0	0.0	1.1	1.1
91	64.35	0.0	0.0	0.0	0.0
92	65.05	0.0	0.0	0.0	0.0

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
93	65.76	0.0	0.0	0.0	0.0

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 7. MAXIMUM SUPPORT REACTIONS (LOAD FACTOR)

STA	DIST X (FT)	MAX + REACT (K)	MAX - REACT (K)
10	7.07	689.7	409.4
34	24.04	741.8	409.2
58	41.01	741.8	409.2
82	57.98	689.7	409.4

4.4.15.4 Live Load Distribution Factor Spreadsheet

4.4.15.4.1 *Spans 1 & 3*

TXDOT BRIDGE DIVISION	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:		Rev. 10/18 - (No Interim)
	Descrip:	ITBC Design Example 3, Span 1 & 3			File:	Ex3_Span1_distribution_factors.xl			Sheet: 1 of 8

LRFD Live Load Distribution Factors*

Live Load Distribution Factors are calculated according to AASHTO LRFD Bridge Design Specifications, 8th Edition (2017 with no interim revisions) as prescribed by TxDOT policies (LRFD Design Manual July 2018) and practices. The Lever Rule is used when outside the Range of Applicability. The Range of Applicability for the Skew Correction Factors is ignored.

INPUT:

Beam Type =	Tx54	
No. Beams, N_b =	6	
CL_{brg} to CL_{brg}, L =	50.25	ft
Beam Spacing, S =	8.00	ft
Avg. Skew Angle, θ =	45.00	deg
Slab Thickness, t_s =	8.00	in
Slab Overhang, OH =	3	ft
Rail Width, RW =	1	ft
Roadway Width, W =	44	ft
Number of Lanes, N_L =	3	

Deck Slab		
Conc wt =	0.145	k/ft^3
f'_c =	4.0	ksi
E_{slab} =	3644	ksi

Beam		
weight =	0.145	k/ft^3
f'_c =	8.5	ksi
E_{beam} =	5312	ksi
y_t =	30.49	in
A =	817.0	in^2
I =	299740	in^4

Longitudinal Stiffness Parameter: (4.6.2.2.1-1)

e_g (in) = 34.49 (dist. b/w cog of bm & deck)

n = 1.000

$K_g = n(I+Ae_g^2)$ = 1271611 in^4

**For typical cross sections (a,e,i,j & k), Table 4.6.2.2.1-1*

RESULTS:

	Final LLDF
Interior Shear LLDF, $gV_{interior}$	0.921
Interior Moment LLDF, $gM_{interior}$	0.682
Exterior Shear LLDF, $gV_{exterior}$	0.921
Exterior Moment LLDF, $gM_{exterior}$	0.682

The Final LLDF may be modified according to the following TxDOT policies:

- Exterior beams use the interior LLDF when OH ≤ S/2.
- When OH > S/2 the exterior beam LLDF is determined by the lever rule for a single lane with a multiple presence factor of 1.0.
- In no case shall the LLDF for the exterior beams be less than the LLDFs for the interior beams.
- When the Roadway width is less than 20ft. all beams are designed for one lane loaded only.
- In no case shall the LLDF be less than $m \cdot N_L \div N_b$.

CALCULATIONS:

Shear LLDF Correction for Skew (Table 4.6.2.2.3c-1)

$$\text{Corr.} = 1.0 + 0.20\left(\frac{12.0Lt_s^{\,3}}{K_g}\right)^{0.3}\tan\theta$$

= 1.0 + 0.20 * [(12.0*50.3*8^3)/(1,271,611)]^0.3 * tan(45)

Corr. = 1.131

Check θ:	0° ≤ 45° ≤ 60°	OK
Check S:	3.5' ≤ 8.0' ≤ 16.0'	OK
Check L:	20' ≤ 50.3' ≤ 240'	OK
Check N_b:	6 ≥ 4	OK

Moment LLDF Correction for Skew (Table 4.6.2.2.2e-1)

Check θ: 30° ≤ 45° ≤ 60° 45°

Corr. = 1 - c_1(tan θ)^1.5

= 1 - 0.142(tan 45)^1.5

Corr. = 0.858

where: $$c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$$

c_1 = 0.142

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
BRIDGE	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:		Rev. 10/18 - (No Interim)
DIVISION	Descrip:	ITBC Design Example 3, Span 1 & 3			File:	Ex3_Span1_distribution_factors.xl			Sheet: 2 of 8

INTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3a-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 1.131

mg = 0.750 * 1.131 = 0.848

Equation

$$g = 0.36 + \left(\frac{S}{25}\right)$$

g = 0.36 + (8 / 25) = 0.680

Modify for Skew:

skew correction = 1.131

g = 0.680 * 1.131 = 0.769

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 50.3' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int1} = 0.769

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 1.131

mg = 0.875 * 1.131 = 0.990

Equation

$$g = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0}$$

g = 0.2 + (8 / 12) - (8 / 35)^2.0 = 0.814

Modify for Skew:

skew correction = 1.131

g = 0.814 * 1.131 = 0.921

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int2+} = 0.921

TxDOT Policy states $gV_{Interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gV_{Interior}$ is the Maximum of: gV_{int1} and $m \cdot N_L \div N_b$.

>> TxDOT Policy states that if W ≥ 20ft, $gV_{Interior}$ is the Maximum of: gV_{int1}, gV_{int2+}, $m \cdot N_L \div N_b$.

$gV_{Interior}$ = 0.921

INTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 0.858

mg = 0.750 * 0.858 = 0.644

Equation

$$g = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

g = 0.06 + (8/14)^0.4 * (8/50.3)^0.3 * (1,271,611/(12*50.3*8^3))^0.1 = 0.591

Modify for Skew:

skew correction = 0.858

g = 0.591 * 0.858 = 0.507

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 50.3' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Check K_g: 10,000 ≤ 1,271,611 ≤ 7,000,000 OK

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int1} = 0.507

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 0.858

mg = 0.875 * 0.858 = 0.751

Equation

$$g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

g = 0.075 + (8/9.5)^0.6 * (8/50.3)^0.2 * (1,271,611/(12*50.3*8^3))^0.1 = 0.795

Modify for Skew:

skew correction = 0.858

g = 0.795 * 0.858 = 0.682

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int2+} = 0.682

TxDOT Policy states $gM_{interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gM_{interior}$ is the Maximum of: gM_{int1} and $m \cdot N_L \div N_b$.

>> TxDOT Policy states that if W ≥ 20ft, $gM_{interior}$ is the Maximum of: gM_{int1}, gM_{int2+}, $m \cdot N_L \div N_b$.

$gM_{interior}$ = 0.682

EXTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.0 = 0.625

TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 1.131

mg = 0.625 * 1.131 = 0.707

Use Lever Rule, as per AASHTO LRFD Table 4.6.2.2.3b-1.

gV_{ext1} = 0.707

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 1.131

mg = 0.625 * 1.131 = 0.707

Equation

d_e = dist. b/w CL web to curb

d_e = OH - Rail Width

d_e = 3ft - 1ft = 2.0 ft

$$e = 0.6 + \left(\frac{d_e}{10}\right)$$

e = 0.6 + (2.0/10) = 0.800

$g = e^*gV_{int2+Eq}$

g = 0.800 * 0.921 = 0.737

Skew Correction is included in gV(interior).

Range of Applicability (ROA) Checks

Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: $-1.0' \le 2.0' \le 5.5'$ OK

Check N_b: $6 \ne 3$ OK

Use Equation from Table 4.6.2.2.3b-1 because all criteria is OK.

gV_{ext2+} = 0.737

TxDOT Policy states $gV_{Exterior}$ must be $\ge gV_{Interior}$

$gV_{interior}$ = 0.921

TxDOT Policy states $gV_{Exterior}$ must be $\ge m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

>> TxDOT Policy states that if OH ≤ S/2, $gV_{Exterior}$ is $gV_{Interior}$.

TxDOT Policy states that if OH > S/2 and W < 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, $gV_{interior}$, and $m \cdot N_L \div N_b$.

TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, gV_{ext2+}, $gV_{interior}$, and $m \cdot N_L \div N_b$.

$gV_{exterior}$ = 0.921

EXTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2d-1):

One Lane Loaded

Lever Rule

mg = 0.625 * 1.0 = 0.625 — TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 0.858

mg = 0.625 * 0.858 = 0.536

Use Lever Rule as per AASHTO LRFD Table 4.6.2.2.2d-1.

gM_{ext1} = 0.536

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 0.858

mg = 0.625 * 0.858 = 0.536

Equation

$$e = 0.77 + \left(\frac{d_e}{9.1}\right)$$

e = 0.77 + (2.0/9.1) = 0.990

$g = e^{*}gM_{int2+Eq}$

g = 0.99 * 0.682 = 0.675

Skew Correction included in gM(interior).

Range of Applicability (ROA) Checks — Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: $-1.0' \le 2.0' \le 5.5'$ OK

Check N_b: $6 \ne 3$ OK

Use Equation from Table 4.6.2.2.2d-1 because all criteria is OK.

gM_{ext2+} = 0.675

TxDOT Policy states $gM_{Exterior}$ must be $\ge gM_{Interior}$

$gM_{interior}$ = 0.682

TxDOT Policy states $gM_{Exterior}$ must be $\ge m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

>> TxDOT Policy states that if OH ≤ S/2, $gM_{Exterior}$ is $gM_{Interior}$.

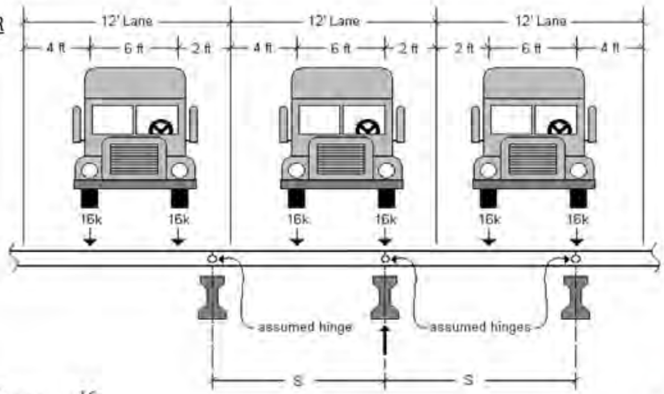
TxDOT Policy states that if OH > S/2 and W < 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, $gM_{Interior}$, and $m \cdot N_L \div N_b$.

TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, gM_{ext2+}, $gM_{Interior}$, and $m \cdot N_L \div N_b$.

$gM_{exterior}$ = 0.682

LEVER RULE S = 8.0 ft

INTERIOR



For S < 4:

One Lane = $\frac{16}{32}$ = 0.500

For 4 ≤ S < 6:

One Lane = $\frac{16}{32}$ = 0.500

Two Lanes = $\frac{16}{32}\left(1+\frac{S-4}{S}\right)$ = 0.750

>: For 6 ≤ S < 10:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}\right)$ = 0.875

For 10 ≤ S < 12:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

For 12 ≤ S < 16:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

For 16 ≤ S < 18:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-16}{S}\right)$ = 0.000

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
BRIDGE	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:		Rev. 10/18 - (No Interim)
DIVISION	Descrip:	ITBC Design Example 3, Span 1 & 3			File:	Ex3_Span1_distribution_factors.xl		Sheet:	7 of 8

LEVER RULE S = 8.0 ft

<u>INTERIOR</u> (con't)

For 18 ≤ S < 22:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}\right)$ = -0.625

For 22 ≤ S ≤ 24:

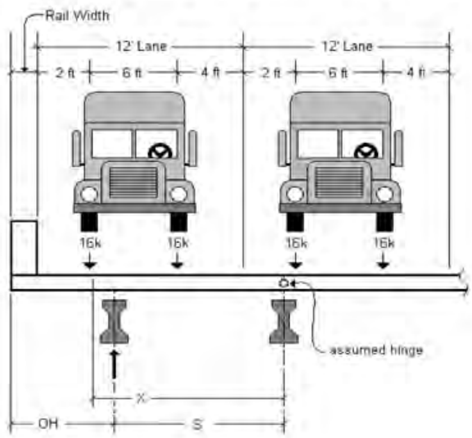
One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}+\frac{S-22}{S}\right)$ = -1.500

<u>EXTERIOR</u>



S = 8.0 ft
OH = 3.0 ft
Rail Width = RW = 1.0 ft
X = S+OH-RW-2ft = 8.0 ft

For X < 6:

One Lane = $\frac{16}{32}\left(\frac{X}{S}\right)$ = 0.500

>: **For 6 ≤ X < 12:**

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

For 12 ≤ X < 18:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}\right)$ = 0.375

LEVER RULE

EXTERIOR (con't) S = 8.0 ft OH = 3.0 ft
RW = 1.0 ft X = S+OH-RW-2ft = 8.0 ft

For 18 ≤ X < 24:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

For 24 ≤ X < 30:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}\right)$ = -1.250

For 30 ≤ X < 36:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

For 36 ≤ X < 42:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}\right)$ = -4.375

For 42 ≤ X ≤ 48:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}+\frac{X-42}{S}\right)$ = -6.500

INTERIOR			EXTERIOR		
One Lane Loaded	=	0.625	One Lane Loaded	=	0.625
Two Lanes Loaded	=	0.875	Two Lanes Loaded	=	0.625
Three Lanes Loaded	=	0.875	Three Lanes Loaded	=	0.625
Four Lanes Loaded	=	0.875	Four Lanes Loaded	=	0.625

4.4.15.4.2 *Span 2*

TXDOT BRIDGE DIVISION	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:		Rev. 10/18 - (No Interim)
	Descrip:	ITBC Design Example 3, Span 2			File:	Ex3_Span2_distribution_factors.xl		Sheet:	1 of 8

LRFD Live Load Distribution Factors*

Live Load Distribution Factors are calculated according to AASHTO LRFD Bridge Design Specifications, 8th Edition (2017 with no interim revisions) as prescribed by TxDOT policies (LRFD Design Manual July 2018) and practices. The Lever Rule is used when outside the Range of Applicability. The Range of Applicability for the Skew Correction Factors is ignored.

INPUT:

Beam Type =	Tx54	
No. Beams, N_b =	6	
CL_{brg} to CL_{brg}, L =	106.50	ft
Beam Spacing, S =	8.00	ft
Avg. Skew Angle, θ =	45.00	deg
Slab Thickness, t_s =	8.00	in
Slab Overhang, OH =	3	ft
Rail Width, RW =	1	ft
Roadway Width, W =	44	ft
Number of Lanes, N_L =	3	

Deck Slab		
Conc wt =	0.145	k/ft^3
f'_c =	4.0	ksi
E_{slab} =	3644	ksi

Beam		
weight =	0.145	k/ft^3
f'_c =	8.5	ksi
E_{beam} =	5312	ksi
y_t =	30.49	in
A =	817.0	in^2
I =	299740	in^4

Longitudinal Stiffness Parameter: (4.6.2.2.1-1)

e_g (in) = 34.49 (dist. b/w cog of bm & deck)

n = 1.000

$K_g = n(I + Ae_g^2)$ = 1271611 in^4

*For typical cross sections (a,e,i,j & k). Table 4.6.2.2.1-1

RESULTS:

	Final LLDF
Interior Shear LLDF, $gV_{interior}$	**0.947**
Interior Moment LLDF, $gM_{interior}$	**0.596**
Exterior Shear LLDF, $gV_{exterior}$	**0.947**
Exterior Moment LLDF, $gM_{exterior}$	**0.596**

The Final LLDF may be modified according to the following TxDOT policies:

- Exterior beams use the interior LLDF when OH ≤ S/2.
- When OH > S/2 the exterior beam LLDF is determined by the lever rule for a single lane with a multiple presence factor of 1.0.
- In no case shall the LLDF for the exterior beams be less than the LLDFs for the interior beams.
- When the Roadway width is less than 20ft, all beams are designed for one lane loaded only.
- In no case shall the LLDF be less than $m \cdot N_L \div N_b$.

CALCULATIONS:

Shear LLDF Correction for Skew (Table 4.6.2.2.3c-1)

$$\text{Corr.} = 1.0 + 0.20\left(\frac{12.0Lt_s^{\,3}}{K_g}\right)^{0.3}\tan\theta$$

= 1.0 + 0.20 * [(12.0*106.5*8^3)/(1,271,611)]^0.3 * tan(45)

Corr. = 1.164

Check θ:	0° ≤ 45° ≤ 60°	OK
Check S:	3.5' ≤ 8.0' ≤ 16.0'	OK
Check L:	20' ≤ 106.5' ≤ 240'	OK
Check N_b:	6 ≥ 4	OK

Moment LLDF Correction for Skew (Table 4.6.2.2.2e-1)

Check θ: 30° ≤ 45° ≤ 60° 45°

Corr. = 1 - c_1(tan θ)^1.5

= 1 - 0.081(tan 45)^1.5

Corr. = 0.919

where: $$c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$$

c_1 = 0.081

INTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3a-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 1.164

mg = 0.750 * 1.164 = 0.873

Equation

$$g = 0.36 + \left(\frac{S}{25}\right)$$

g = 0.36 + (8 / 25) = 0.680

Modify for Skew:

skew correction = 1.164

g = 0.680 * 1.164 = 0.792

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 106.5' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int1} = 0.792

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 1.164

mg = 0.875 * 1.164 = 1.019

Equation

$$g = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0}$$

g = 0.2 + (8 / 12) - (8 / 35)^2.0 = 0.814

Modify for Skew:

skew correction = 1.164

g = 0.814 * 1.164 = 0.947

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int2+} = 0.947

TxDOT Policy states $gV_{Interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gV_{Interior}$ is the Maximum of: gV_{int1} and $m \cdot N_L \div N_b$.

>> TxDOT Policy states that if W ≥ 20ft, $gV_{Interior}$ is the Maximum of: gV_{int1}, gV_{int2+}, $m \cdot N_L \div N_b$.

$gV_{interior}$ = 0.947

INTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 0.919

mg = 0.750 * 0.919 = 0.689

Equation

$$g = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

g = 0.06 + (8/14)^0.4 * (8/106.5)^0.3 * (1,271,611/(12*106.5*8^3))^0.1 = 0.453

Modify for Skew:

skew correction = 0.919

g = 0.453 * 0.919 = 0.416

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 106.5' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Check K_g: 10,000 ≤ 1,271,611 ≤ 7,000,000 OK

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int1} = 0.416

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 0.919

mg = 0.875 * 0.919 = 0.804

Equation

$$g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

g = 0.075 + (8/9.5)^0.6 * (8/106.5)^0.2 * (1,271,611/(12*106.5*8^3))^0.1 = 0.649

Modify for Skew:

skew correction = 0.919

g = 0.649 * 0.919 = 0.596

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int2+} = 0.596

TxDOT Policy states $gM_{interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gM_{interior}$ is the Maximum of: gM_{int1} and $m \cdot N_L \div N_b$.

>> TxDOT Policy states that if W ≥ 20ft, $gM_{interior}$ is the Maximum of: gM_{int1}, gM_{int2+}, $m \cdot N_L \div N_b$.

$gM_{interior}$ = 0.596

EXTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.0 = 0.625 — TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 1.164

mg = 0.625 * 1.164 = 0.728

Use Lever Rule, as per AASHTO LRFD Table 4.6.2.2.3b-1.

gV_{ext1} = 0.728

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 1.164

mg = 0.625 * 1.164 = 0.728

Equation

d_e = dist. b/w CL web to curb

d_e = OH - Rail Width

d_e = 3ft - 1ft = 2.0 ft

$$e = 0.6 + \left(\frac{d_e}{10}\right)$$

e = 0.6 + (2.0/10) = 0.800

$g = e{*}gV_{int2+Eq}$

g = 0.800 * 0.947 = 0.758

Skew Correction is included in gV(interior).

Range of Applicability (ROA) Checks — Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: $-1.0' \le 2.0' \le 5.5'$ OK

Check N_b: $6 \ne 3$ OK

Use Equation from Table 4.6.2.2.3b-1 because all criteria is OK.

gV_{ext2+} = 0.758

TxDOT Policy states $gV_{Exterior}$ must be $\ge gV_{interior}$

$gV_{interior}$ = 0.947

TxDOT Policy states $gV_{Exterior}$ must be $\ge m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

>> TxDOT Policy states that if OH ≤ S/2, $gV_{Exterior}$ is $gV_{interior}$.

TxDOT Policy states that if OH > S/2 and W < 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, $gV_{interior}$, and $m \cdot N_L \div N_b$.

TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, gV_{ext2+}, $gV_{interior}$, and $m \cdot N_L \div N_b$.

$gV_{exterior}$ = 0.947

EXTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2d-1):

One Lane Loaded

Lever Rule

mg = 0.625 * 1.0 = 0.625 — TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 0.919

mg = 0.625 * 0.919 = 0.574

Use Lever Rule as per AASHTO LRFD Table 4.6.2.2.2d-1.

gM_{ext1} = 0.574

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 0.919

mg = 0.625 * 0.919 = 0.574

Equation

$$e = 0.77 + \left(\frac{d_e}{9.1}\right)$$

e = 0.77 + (2.0/9.1) = 0.990

$g = e^*gM_{int2+Eq}$

g = 0.99 * 0.596 = 0.590

Skew Correction included in gM(interior).

Range of Applicability (ROA) Checks — Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: $-1.0' \le 2.0' \le 5.5'$ OK

Check N_b: $6 \neq 3$ OK

Use Equation from Table 4.6.2.2.2d-1 because all criteria is OK.

gM_{ext2+} = 0.590

TxDOT Policy states $gM_{Exterior}$ must be $\ge gM_{Interior}$

$gM_{interior}$ = 0.596

TxDOT Policy states $gM_{Exterior}$ must be $\ge m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

>> TxDOT Policy states that if OH ≤ S/2, $gM_{Exterior}$ is $gM_{Interior}$.

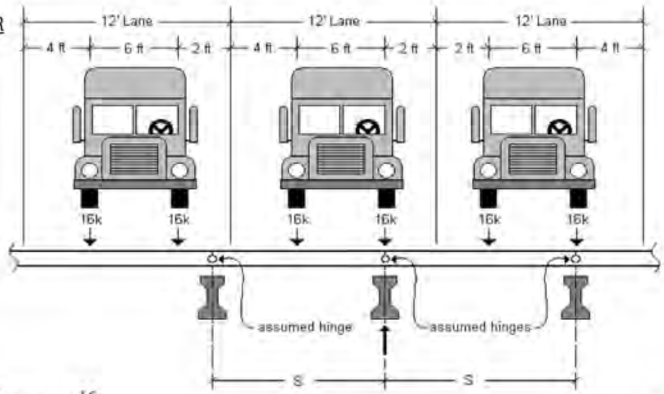
TxDOT Policy states that if OH > S/2 and W < 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, $gM_{Interior}$, and $m \cdot N_L \div N_b$.

TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, gM_{ext2+}, $gM_{Interior}$, and $m \cdot N_L \div N_b$.

$gM_{exterior}$ = 0.596

LEVER RULE S = 8.0 ft

INTERIOR



For S < 4:

One Lane = $\frac{16}{32}$ = 0.500

For 4 ≤ S < 6:

One Lane = $\frac{16}{32}$ = 0.500

Two Lanes = $\frac{16}{32}\left(1+\frac{S-4}{S}\right)$ = 0.750

>: **For 6 ≤ S < 10:**

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}\right)$ = 0.875

For 10 ≤ S < 12:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

For 12 ≤ S < 16:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

For 16 ≤ S < 18:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-16}{S}\right)$ = 0.000

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
BRIDGE	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:		Rev. 10/18 - (No Interim)
DIVISION	Descrip:	ITBC Design Example 3, Span 2			File:	Ex3_Span2_distribution_factors.xl		Sheet:	7 of 8

LEVER RULE S = 8.0 ft

INTERIOR (con't)

For 18 ≤ S < 22:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}\right)$ = -0.625

For 22 ≤ S ≤ 24:

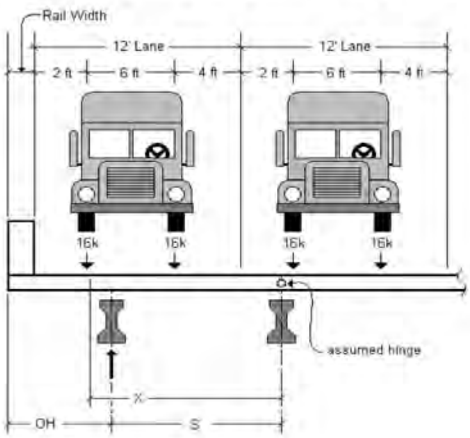
One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}+\frac{S-22}{S}\right)$ = -1.500

EXTERIOR



S = 8.0 ft
OH = 3.0 ft
Rail Width = RW = 1.0 ft
X = S+OH-RW-2ft = 8.0 ft

For X < 6:

One Lane = $\frac{16}{32}\left(\frac{X}{S}\right)$ = 0.500

>: For 6 ≤ X < 12:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

For 12 ≤ X < 18:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}\right)$ = 0.375

LEVER RULE

EXTERIOR (con't) S = 8.0 ft OH = 3.0 ft
RW = 1.0 ft X = S+OH-RW-2ft = 8.0 ft

For 18 ≤ X < 24:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

For 24 ≤ X < 30:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}\right)$ = -1.250

For 30 ≤ X < 36:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

For 36 ≤ X < 42:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}\right)$ = -4.375

For 42 ≤ X ≤ 48:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}+\frac{X-42}{S}\right)$ = -6.500

INTERIOR			EXTERIOR		
One Lane Loaded	=	0.625	One Lane Loaded	=	0.625
Two Lanes Loaded	=	0.875	Two Lanes Loaded	=	0.625
Three Lanes Loaded	=	0.875	Three Lanes Loaded	=	0.625
Four Lanes Loaded	=	0.875	Four Lanes Loaded	=	0.625

4.4.15.5 Concrete Section Shear Capacity Spreadsheet

Texas Department of Transportation

County:	ANY	Descrip:	ITBC Design Example 3 - Bent 2		
Highway:	ANY				
C-S-J:	XXXXXXX	Design:	BRG	Ck Dsn:	BRG
Bridge Division	Rev: 09/26/08			Date:	Aug-20

CONCRETE SECTION SHEAR CAPACITY BY AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, FOURTH EDITION, 2007

Resistance Factors: Units: **US**

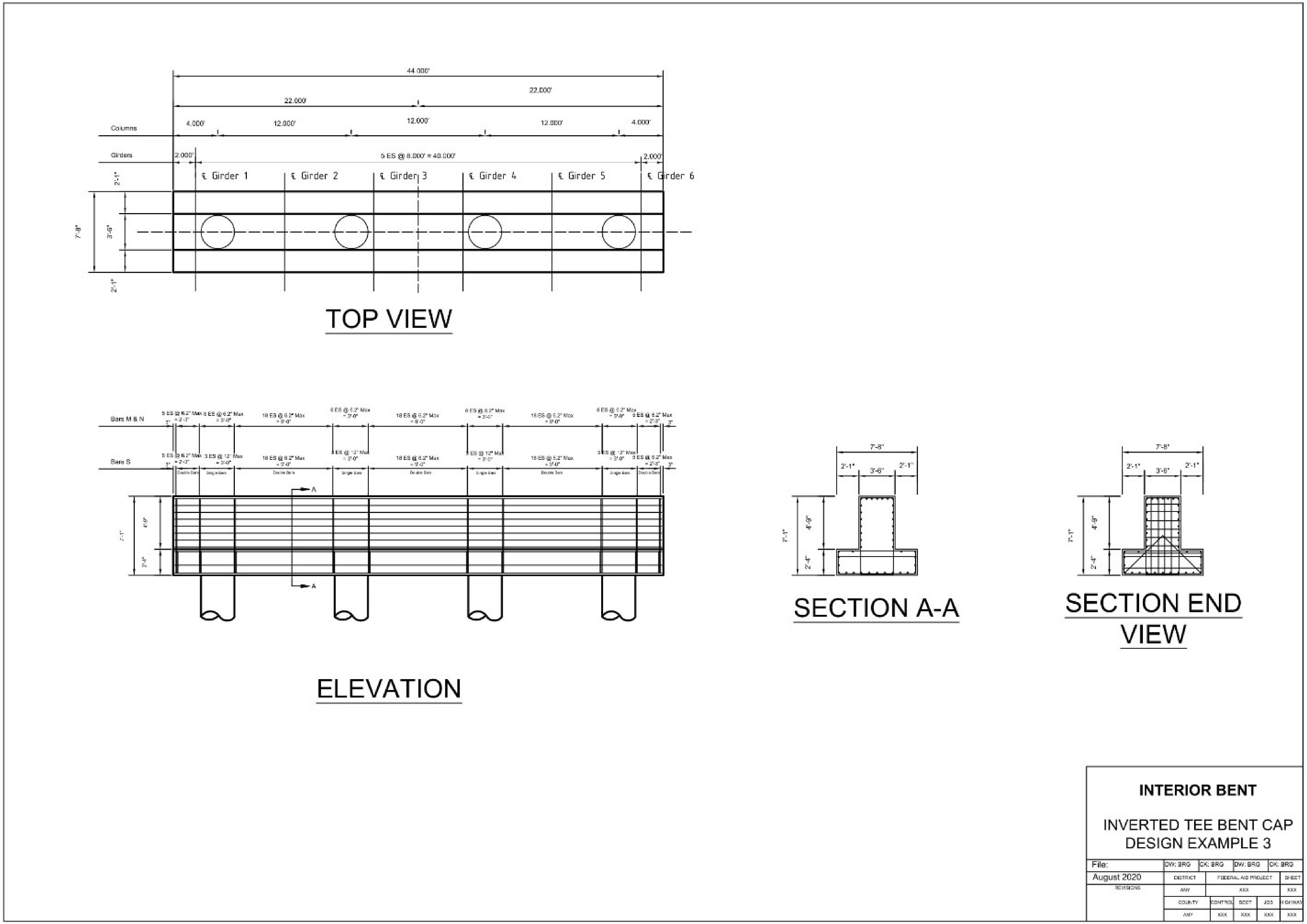
ϕ_V =	0.9
ϕ_M =	0.9
ϕ_N =	0.75

Concrete:			Mild Steel:			Prestressed Steel:		
f'c =	5	ksi	fy =	60	ksi	fpu =	270	ksi
Ec =	4070	ksi	Es =	29000	ksi	Ep =	28500	ksi

		SECTIONS							
	Units	8	12	32	36	56	60	80	84
Input Data									
Bending moment, **Mu**	kip-ft	625.6	888.7	746.9	504.8	504.8	746.9	888.7	626
Shear force, **Vu**	kip	243.6	253.3	145.2	462.8	245.6	263.2	147.5	428.7
Axial force, **Nu (+ if tensile)**	kip	0	0	0	0	0	0	0	0
Web width, **bv**	in	42.00	42.00	42.00	42.00	42.00	42.00	42.00	42.00
Shear depth, **dv**	in	80.59	80.59	80.59	80.59	80.59	80.59	80.59	80.59
Mild steel reinf. area, **As**	in^2	10.92	10.92	10.92	10.92	10.92	10.92	10.92	10.92
Conc area on tension side, **Ac**	in^2	1785	1785	1785	1785	1785	1785	1785	1785
Area of stirrups, **Av**	in^2	1.76	1.76	1.76	1.76	1.76	1.76	1.76	1.76
Stirrup spacing, **s**	in	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
Prestressed steel area, **Aps**	in^2	0	0	0	0	0	0	0	0
Prestress shear, **Vp**	kip	0	0	0	0	0	0	0	0
Average prestress, **fps**	ksi	0	0	0	0	0	0	0	0
Torsional moment, **Tu**	kip-ft	773	387	387	773	773	387	387	773
Shear flow area, **Ao**	in^2	3493.5	3493.5	3493.5	3493.5	3493.5	3493.5	3493.5	3493.5
Area of one leg of stirrup, **At**	in^2	0.44	0.44	0.44	0.44	0.44	0.44	0.44	0.44
Perimeter of stirrup, **Ph**	in	334	334	334	334	334	334	334	334
Calculated Values									
Vc	kip	583.6	578.8	641.0	533.3	581.2	574.0	641.0	533.3
Vs	kip	1715.2	1753.6	2037.8	1484.3	1708.2	1731.8	2029.6	1484.3
ϕVn	kip	**2069**	**2099**	**2411**	**1816**	**2060**	**2075**	**2404**	**1816**
ε_x		6.83E-04	7.07E-04	4.33E-04	1.00E-03	6.88E-04	7.31E-04	4.39E-04	1.00E-03
θ	deg	32.80	33.10	29.40	36.40	32.90	33.42	29.50	36.40
β		2.440	2.420	2.680	2.230	2.430	2.400	2.680	2.230
Req'd Shear reinf. **Av/S**	in^2/in	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Req'd Torsion reinf. **At/S**	in^2/in	0.016	0.008	0.007	0.018	0.016	0.008	0.007	0.018
Maximum stirrup spacing, **Smax**	in	24.0	24.0	24.0	24.0	24.0	24.0	24.0	24.0
Conclusion									
Shear Reinforcing		OK	OK	OK	OK	OK	OK	OK	OK
Longitudinal Reinforcing		OK	OK	OK	OK	OK	OK	OK	OK

Note: Longitudinal Reinforcing check can be ignored for typical multi-column bent caps. For straddle bents with no overhangs, this check must be considered. Refer to LRFD 5.8.3.5 for further information.
If torsion is not being considered, leave last five rows of input data blank.

4.4.15.6 <u>Bent Cap Details</u>



4.5 INVERTED-T BENT CAP DESIGN EXAMPLE 4 (60° SKEW ANGLE)

Design example is in accordance with the AASHTO LRFD Bridge Design Specifications, 8th Ed. (2017) as prescribed by TxDOT Bridge Manual - LRFD (January 2020).

4.5.1 Design Parameters

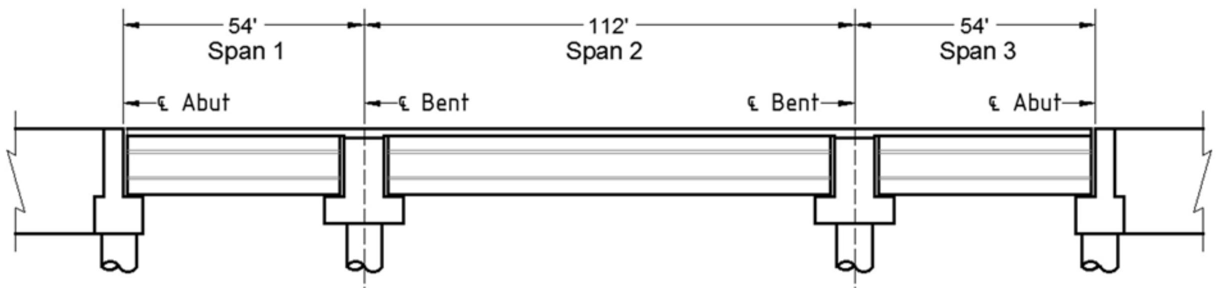


Figure 4.78 Spans of the Bridge with 60 Degrees Skewed ITBC

"AASHTO LRFD" refers to the ASSHTO LRFD Bridge Design Specification, 8th Ed. (2017)..

"BDM-LRFD" refers to the TxDOT Bridge Design Manual - LRFD (January 2020).

"TxSP" refers to TxDOT guidance, recommendations, and standard practice.

"Furlong & Mirza" refers to "Strength and Serviceability of Inverted T-Beam Bent Caps Subject to Combined Flexure, Shear, and Torsion", Center for Highway Research Research Report No. 153-1F, The University of Texas at Austin, August 1974.

Span 1

54' Type TX54 Girders (0.851 k/ft)

6 Girders Spaced @ 16' along the axis of bent with 3' overhangs

2" Haunch

Span 2

112' Type TX54 Girders (0.851 k/ft)

6 Girders Spaced @ 16' along the axis of bent with 3' overhangs

3.75" Haunch

Span 3

54' Type TX54 Girders (0.851 k/ft)

6 Girders Spaced @ 16' along the axis of bent with 3' overhangs

2" Haunch

All Spans

The basic bridge geometry can be found on the Bridge Layout located in the Appendices.

Deck is 46 ft wide

Type T551 Rail (0.382 k/ft)

8" Thick Slab (0.100 ksf)

Assume 2" Overlay @ 140 pcf (0.023 ksf)

(TxSP)

Use Class "C" Concrete

(BDM-LRFD, Ch. 4, Sect. 5, Materials)

$f'_c = 5$ ksi

$w_c = 150$ pcf (for weight)

$w_c = 145$ pcf (for Modulus of Elasticity calculation)

Grade 60 Reinforcing *(BDM-LRFD, Ch. 4, Sect. 5, Materials)*

$f_y = 60 \text{ ksi}$

Bents

Use 36" Diameter Columns (Typical for Type TX54 Girders)

Define Variables

Back Span	**Forward Span**	
$\text{Span1} = 54\text{ft}$	$\text{Span2} = 112\text{ft}$	*Span Length*
$\text{GdrSpa1} = 8\text{ft}$	$\text{GdrSpa2} = 8\text{ft}$	*Girder Spacing (Normalized values)*
$\text{GdrNo1} = 6$	$\text{GdrNo2} = 6$	*Number of Girders in Span*
$\text{GdrWt1} = 0.851\text{klf}$	$\text{GdrWt2} = 0.851\text{klf}$	*Weight of Girder*
$\text{Haunch1} = 2\text{in}$	$\text{Haunch2} = 3.75\text{in}$	*Size of Haunch*

Bridge

$\text{Skew} = 60\text{deg}$ *Skew of Bents*

$\text{BridgeW} = 46\text{ft}$ *Width of Bridge Deck*

$\text{RdwyW} = 44\text{ft}$ *Width of Roadway*

$\text{GirderD} = 54\text{in}$ *Depth of Type TX54 Girder*

$\text{BrgSeat} = 1.5\text{in}$ *Bearing Seat Buildup*

$\text{BrgPad} = 2.75\text{in}$ *Bearing Pad Thickness*

$\text{SlabThk} = 8\text{in}$ *Thickness of Bridge Slab*

$\text{OverlayThk} = 2\text{in}$ *Thickness of Overlay*

$\text{RailWt} = 0.372\text{klf}$ *Weight of Rail*

$w_c = 0.150\text{kcf}$ *Unit Weight of Concrete for Loads*

$w_{Olay} = 0.140\text{kcf}$ *Unit Weigh of Overlay*

Bents

$f_c = 5\text{ksi}$ *Concrete Strength*

$w_{cE} = 0.145\text{kcf}$ *Unit Weight of Concrete for E_c*

$E_c = 33000 \cdot w_{cE}^{1.5} \cdot \sqrt{f_c}$ $\quad E_c = 4074 \text{ ksi}$ *Modulus of Elasticity of Concrete (AASHTO LRFD Eq. C5.4.2.4-2)*

$f_y = 60\text{ksi}$ *Yield Strength of Reinforcement*

$E_s = 29000\text{ksi}$ *Modulus of Elasticity of Steel*

$D_{column} = 36\text{in}$ *Diameter of Columns*

Other Variables

$IM = 33\%$

Dynamic Load Allowance
(AASHTO LRFD Table 3.6.2.1-1)

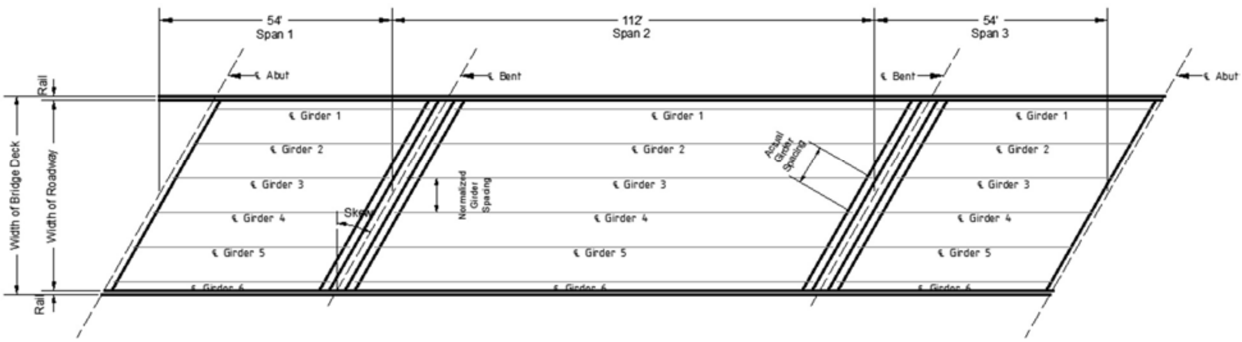


Figure 4.79 Top View of the 60 Degrees Skewed ITBC with Spans and Girders

4.5.2 Determine Cap Dimensions

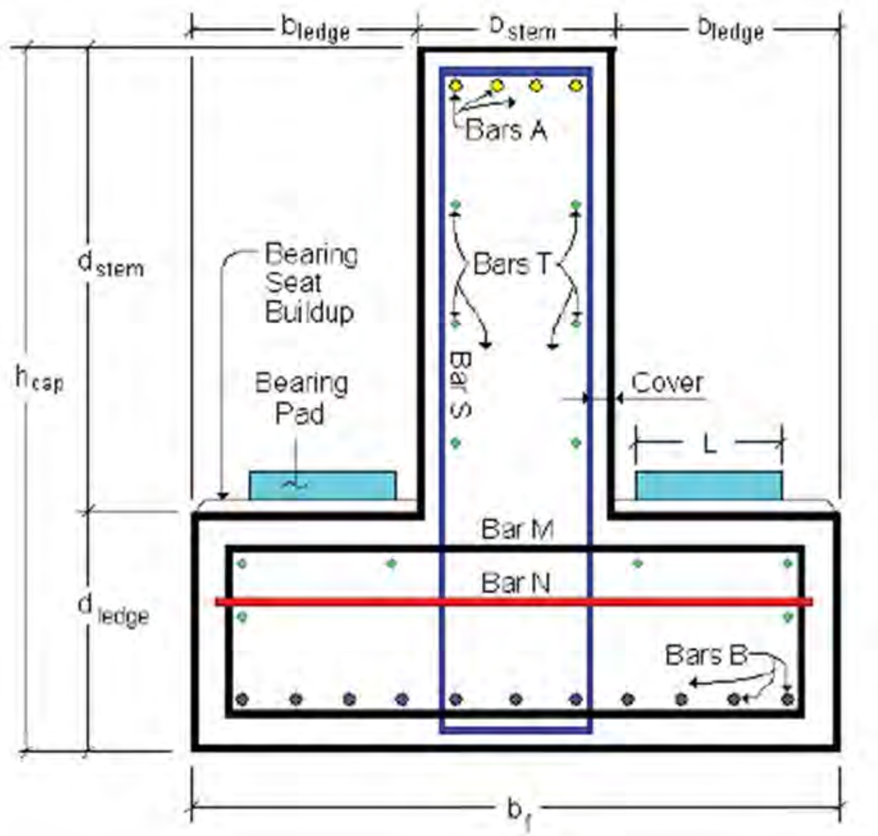


Figure 4.80 Section View of 60 Degrees Skewed ITBC

4.5.2.1 <u>Stem Width</u>

The stem is typically at least 3" wider than the Diameter of the Column (36") to allow for the extension of the column reinforcement into the Cap. (TxSP)

b_{stem} = at least $D_{column} + 3in$

Use: $b_{stem} = 42\ in$

4.5.2.2 <u>Stem Height</u>

Distance from Top of Slab to Top of Ledge:

Haunch2 is the larger of the two haunches.

$$D_{Slab\_to\_Ledge} = SlabThk + Haunch2 + GirderD + BrgPa$$

$D_{Slab\_to\_Ledge} = 70.00\ in$

$StemHaunch = 3.75\ in$

The top of the stem must be 2.5" below the bottom of the slab. (BDM-LRFD, Ch. 4, Sect. 5, Geometric Constraints)

Accounting for the 1/2" of bituminous fiber, the top of the stem must have at least 2" of haunch on it, but the haunch should not be less than either of the haunches of the adjacent spans.

$d_{stem} = D_{Slab\_to\_Ledge} - SlabThk - StemHaunch - 0.5in$

The stem must accommodate ½" of bituminous fiber.

$d_{stem} = 57.75 \text{ in}$

Use: $d_{stem} = 57 \text{ in}$

Round the Stem Height down to the nearest 1". (TxSP)

4.5.2.3 Ledge Width

The Ledge Width must be adequate for Bar M to develop fully.

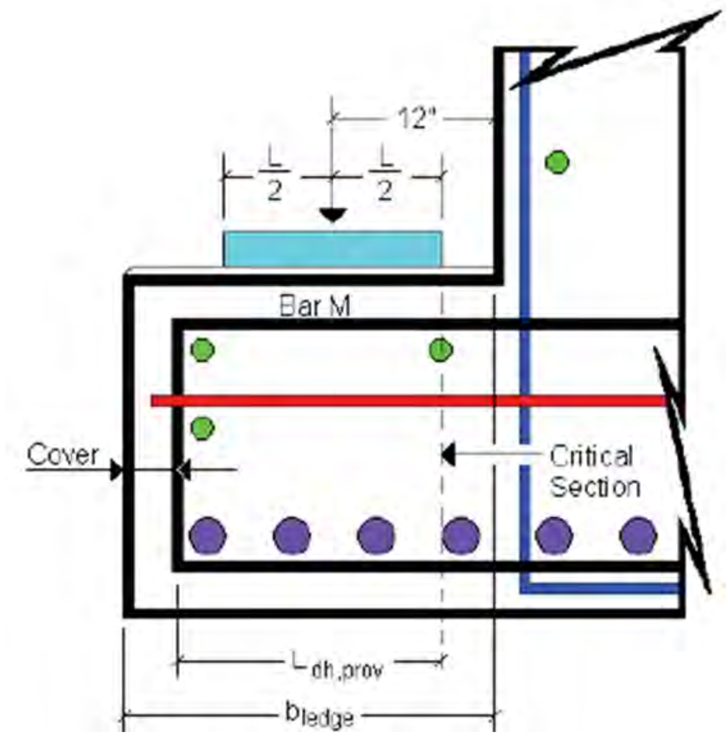


Figure 4.81 Ledge Section of 60 Degrees ITBC

"$L_{dh,prov}$" must be greater than or equal to "$L_{dh,req}$" for Bar M.

$cover = 2.5 \text{ in}$

"cover" is measured from the center of the transverse bars.

$L = 15 \text{ in}$

"L" is the length of the Bearing Pad along the girder. A typical type TX54 bearing pad is circular 15" Dia. for 60° skewed beents, as shown in the IGEB standard.

Determine the Required Development Length of Bar M:

Try # 7 Bar for Bar M.

$d_{bar\_M} = 0.875 \text{ in}$

$A_{bar\_M} = 0.60 \text{ in}^2$

Basic Development Length

$L_{dh} = \frac{38.0 \cdot d_{bar\_M}}{60} \cdot \left(\frac{f_y}{\sqrt{f_c}}\right)$ $L_{dh} = 14.87 \text{ in}$ *(AASHTO LRFD Eq. 5.10.8.2.4a-2)*

Modification Factors for L_{dh}: *(AASHTO LRFD 5.10.8.2.4b)*

Is Top Cover greater than or equal to 2.5", and Side Cover greater than or equal to 2"?

$$\text{SideCover} = \text{cover} - \frac{d_{bar\_M}}{2} = 2.06 \text{ in}$$

$$\text{TopCover} = \text{cover} - \frac{d_{bar_M}}{2} = 2.06 \text{ in}$$

"Side Cover" and "Top Cover" are the clear cover on the side and top of the hook respectively. The dimension "cover" is measured from the center of Bar M.

No. Reinforcement Confinement Factor, $\lambda_{rc} = 1.0$

Coating Factor, $\lambda_{cw} = 1.0$

Excess Reinforcement Factor, $\lambda_{er} = 1.0$

Concrete Density Modification Factor, $\lambda = 1.0$ *(AASHTO LRFD 5.4.2.8)*

The Required Development Length: *(AASHTO LRFD 5.10.8.2.4a)*

$$L_{dh\_req} = \max(L_{dh} \cdot \left(\frac{\lambda_{rc} \cdot \lambda_{cw} \cdot \lambda_{er}}{\lambda}\right), 8 \cdot d_{bar\_M}, 6\text{in.})$$

Therefore,

$$L_{dh\_req} = 14.87 \text{ in}$$

$b_{ledge\_min} = L_{dh\_req} + \text{cover} + 12\text{in} - \frac{L}{2}$ $\qquad$ $b_{ledge\_min} = 21.87 \text{ in}$

The distance from the face of the stem to the center of bearing is 12" for TxGirders (IGEB).

Use:

$$b_{ledge} = 25 \text{ in}$$

Width of Bottom Flange:

$b_f = 2 \cdot b_{ledge} + b_{stem}$ $\qquad$ $b_f = 92 \text{ in}$

4.5.2.4 Ledge Depth

As a general rule of thumb, Ledge Depth is greater than or equal to 2'-3". This is the depth at which a bent from a typical bridge will pass the punching shear check.

Use a Ledge Depth of 28".

$$d_{ledge} = 28 \text{ in}$$

Total Depth of Cap:

$h_{cap} = d_{stem} + d_{ledge}$ $\qquad$ $h_{cap} = 85 \text{ in}$

4.5.2.5 Summary of Cross Sectional Dimensions

$b_{stem} = 42 \text{ in}$

$d_{stem} = 57 \text{ in}$

$b_{ledge} = 25 \text{ in}$

$d_{ledge} = 28 \text{ in}$

$h_{cap} = 85 \text{ in}$

4.5.2.6 Length of Cap

First define Girder Spacing and End Distance:

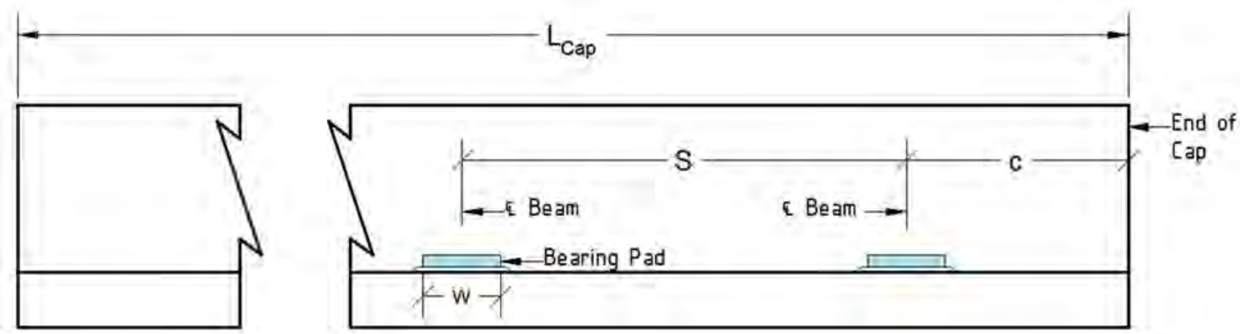


Figure 4.82 Elevation View of 60 Degrees Skewed ITBC

$S = 8\ \text{ft}$ — *Girder Spacing*

$c = 2\ \text{ft}$ — *"c" is the distance from the Center Line of the Exterior Girder to the Edge of the Cap measured along the Cap.*

$L_{Cap} = S \cdot (GdrNo1 - 1) + 2c$ $\quad L_{Cap} = 44\ \text{ft}$ — *Length of Cap*

TxDOT policy is as follows, "The edge distance between the exterior bearing pad and the end of the inverted T-beam shall not be less than 12in." (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria) replacing the statement in AASHTO LRFD 5.13.2.5.5 stating it shall not be less than d_f. Preferably, the stem should extend at least 3" beyond the edge of the bearing seat.

Bearing Pad Dimensions: *(IGEB standard)*

$L = 15\ \text{in}$ — *Length of Bearing Pad*

$W = 15\ \text{in}$ — *Width of Bearing Pad*

4.5.3 Cross Sectional Properties of Cap

$A_g = d_{ledge} \cdot b_f + d_{stem} \cdot b_{stem}$ $\quad A_g = 4970\ \text{in}^2$

$$ybar = \frac{d_{ledge} \cdot b_f \cdot \left(\frac{1}{2} d_{ledge}\right) + d_{stem} \cdot b_{stem} \cdot \left(d_{ledge} + \frac{1}{2} d_{stem}\right)}{A_g} \quad ybar = 34.5\ \text{in}$$

Distance from bottom of the cap to the center of gravity of the cap

$$I_g = \frac{b_f \cdot d_{ledge}^3}{12} + b_f \cdot d_{ledge} \cdot \left(ybar - \frac{1}{2} d_{ledge}\right)^2 + \frac{b_{stem} \cdot d_{stem}^3}{12} + \cdots$$

$$b_{stem} \cdot d_{stem} \cdot \left[ybar - \left(d_{ledge} + \frac{1}{2} d_{stem}\right)\right]^2 \quad I_g = 3.06 \times 10^6\ \text{in}^4$$

4.5.4 Cap Analysis

4.5.4.1 Cap Model

Assume:

4 Columns Spaced @ 12'-0"

The cap will be modeled as a continuous beam with simple supports using TxDOT's CAP18 program.

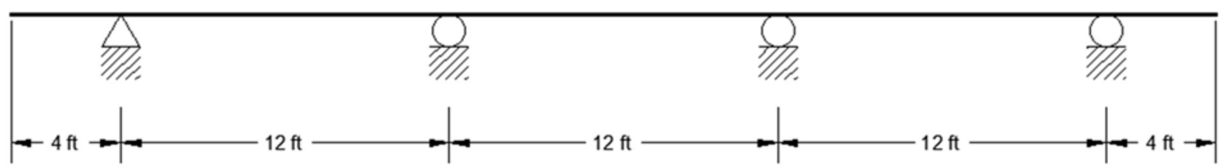


Figure 4.83 Continuous Beam Model for 60 Degrees Skewed ITBC

TxDOT does not consider frame action for typical multi-column bents (BDM-LRFD, Ch. 4, Sect. 5, Structural Analysis).

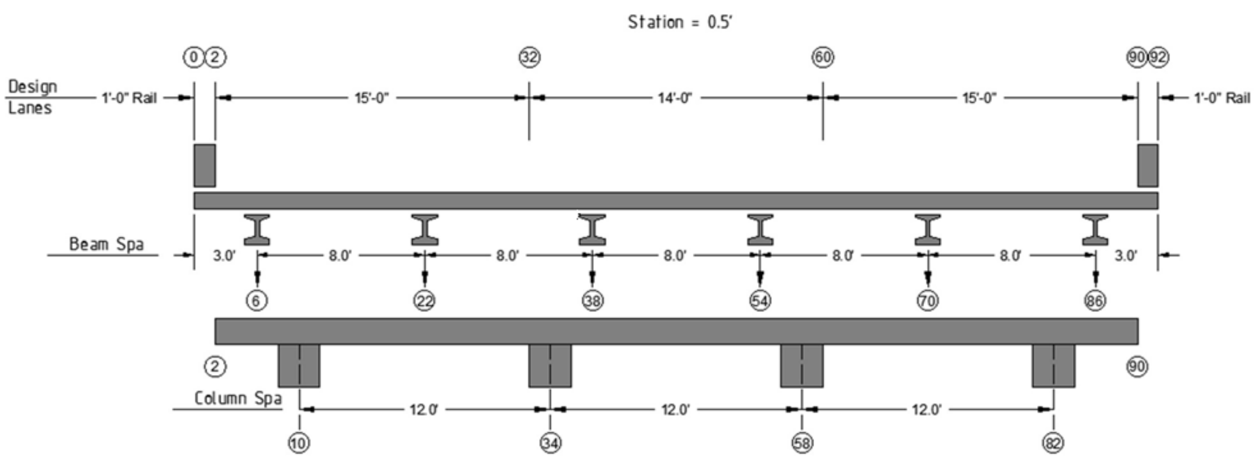


Figure 4.84 Cap 18 Model of 60 Degrees Skewed ITBC

The circled numbers in Figure 4.84 are the stations that will be used in the CAP 18 input file. One station is 0.5 ft in the direction perpendicular to the pgl, not parallel to the bent.

$$\text{station} = 0.5\text{ ft}$$ *Station increment for CAP 18*

Recall:

$$E_c = 4074\text{ ksi} \qquad I_g = 3.06 \times 10^6\text{ in}^4$$

$$E_cI_g = 1.25 \times 10^{10}\text{ kip}\cdot\text{in}^2/\left(12\frac{\text{in}}{\text{ft}}\right)^2 \quad E_cI_g = 8.66 \times 10^7\text{kip}\cdot\text{ft}^2$$

4.5.4.1.1 Dead Load

Values used in the following equations can be found on "4.5.1 Design Parameters"

SPAN 1

$\text{Rail1} = \frac{2 \cdot \text{RailWt} \cdot \frac{\text{Span1}}{2}}{\min(\text{GdrNo1},6)}$ $\text{Rail1} = 3.44 \frac{\text{kip}}{\text{girder}}$

Rail Weight is distributed evenly among stringers, up to 3 stringers per rail (TxSP).

$\text{Slab1} = w_c \cdot \text{GdrSpa1} \cdot \text{SlabThk} \cdot \frac{\text{Span1}}{2} \cdot 1.10$ $\text{Slab1} = 23.76 \frac{\text{kip}}{\text{girder}}$

Increase slab DL by 10% to account for haunch and thickened slab ends.

$\text{Girder1} = \text{GdrWt1} \cdot \frac{\text{Span1}}{2}$ $\text{Girder1} = 22.98 \frac{\text{kip}}{\text{girder}}$

$\text{DLRxn1} = (\text{Rail1} + \text{Slab1} + \text{Girder1})$ $\text{DLRxn1} = 50.17 \frac{\text{kip}}{\text{girder}}$

Overlay is calculated separetely, because it has different load factor than the rest of the dead loads.

$\text{Overlay1} = w_{\text{Olay}} \cdot \text{GdrSpa1} \cdot \text{OverlayThk} \cdot \frac{\text{Span1}}{2}$ $\text{Overlay1} = 5.04 \frac{\text{kip}}{\text{girder}}$

Design for future overlay.

SPAN 2

$\text{Rail2} = \frac{2 \cdot \text{RailWt} \cdot \frac{\text{Span2}}{2}}{\min(\text{GdrNo2},6)}$ $\text{Rail2} = 7.13 \frac{\text{kip}}{\text{girder}}$

$\text{Slab2} = w_c \cdot \text{GdrSpa2} \cdot \text{SlabThk} \cdot \frac{\text{Span2}}{2} \cdot 1.10$ $\text{Slab2} = 49.28 \frac{\text{kip}}{\text{girder}}$

$\text{Girder2} = \text{GdrWt1} \cdot \frac{\text{Span2}}{2}$ $\text{Girder2} = 47.66 \frac{\text{kip}}{\text{girder}}$

$\text{DLRxn2} = (\text{Rail2} + \text{Slab2} + \text{Girder2})$ $\text{DLRxn2} = 104.07 \frac{\text{kip}}{\text{girder}}$

$\text{Overlay2} = w_{\text{Olay}} \cdot \text{GdrSpa2} \cdot \text{OverlayThk} \cdot \frac{\text{Span2}}{2}$ $\text{Overlay2} = 10.45 \frac{\text{kip}}{\text{girder}}$

CAP

$\text{Cap} = w_c \cdot A_g = 5.177 \frac{\text{kip}}{\text{ft}} \cdot \frac{0.5\text{ft}}{\text{station}}$ $\text{Cap} = 2.589 \frac{\text{kip}}{\text{station}}$

4.5.4.1.2 Live Load

AASHTO LRFD 3.6.1.2.2 and 3.6.1.2.4)

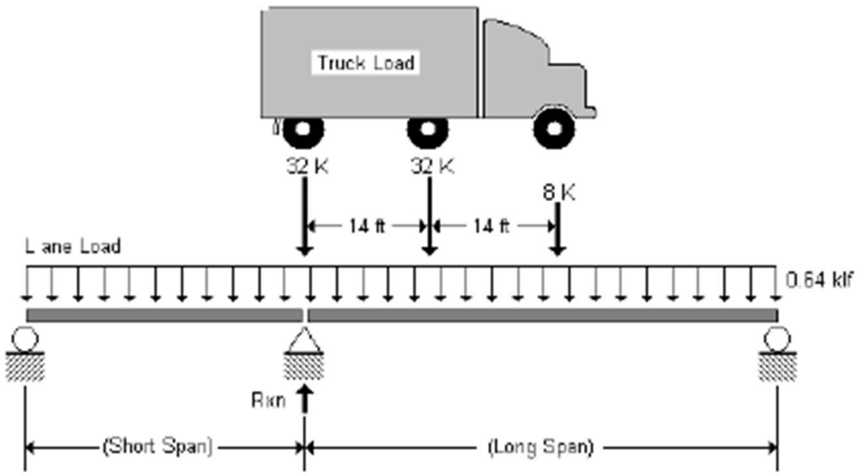


Figure 4.85 Live Load Model of 60 Degrees Skewed ITBC

$\text{LongSpan} = \max(\text{Span1}, \text{Span2})$ $\qquad$ $\text{LongSpan} = 112\ \text{ft}$

$\text{ShortSpan} = \min(\text{Span1}, \text{Span2})$ $\qquad$ $\text{ShortSpan} = 54\ \text{ft}$

$\text{IM} = 0.33$

$$\text{Lane} = 0.64\text{klf} \cdot \left(\frac{\text{LongSpan}+\text{ShortSpan}}{2}\right)$$

$$\text{Lane} = 53.12\,\frac{\text{kip}}{\text{lane}}$$

$$\text{Truck} = 32\text{kip} + 32\text{kip} \cdot \left(\frac{\text{LongSpan}-14\text{ft}}{\text{LongSpan}}\right) + 8\text{kip} \cdot \left(\frac{\text{LongSpan}-28\text{ft}}{\text{LongSpan}}\right)$$

$$\text{Truck} = 66.00\,\frac{\text{kip}}{\text{lane}}$$

Use HL-93 Live Load. For maximum reaction at interior bents, "Design Truck" will always govern over "Design Tandem". For the maximum reaction when the long span is more than twice as long as the short span, place the rear (32 kip) axle over the support and the middle (32 kip) and front (8 kip) axles on the long span. For the maximum reaction when the long span is less than twice as long as the short span, place the middle (32 kip) axle over the support, the front (8 kip) axle on the short span and the rear (32 kip) axle on the long span.

$$\text{LLRxn} = \text{Lane} + \text{Truck} \cdot (1 + \text{IM})$$

$$\text{LLRxn} = 140.90\,\frac{\text{kip}}{\text{lane}}$$

Combine "Design Truck" and "Design Lane" loadings (AASHTO LRFD 3.6.1.3). Dynamic load allowance, IM, does not apply to "Design Lane." (AASHTO LRFD 3.6.1.2.4)

$$P = 16.0kip \cdot (1 + IM)$$

$$P = 21.28\,kip$$

$$w = \frac{LLRxn - (2 \cdot P)}{10ft}$$

$$w = 9.83\,\frac{kip}{ft} \cdot \frac{0.5ft}{station}$$

$$w = 4.92\,\frac{kip}{station}$$

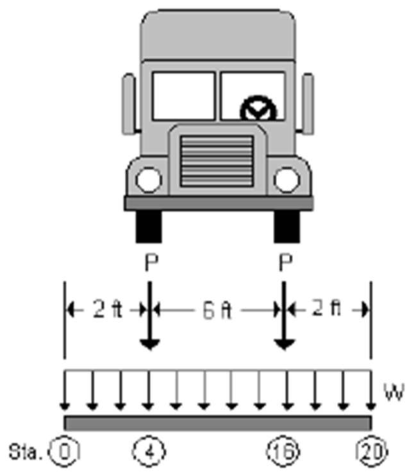


Figure 4.86 Live Load Model of 60 Degrees Skewed ITBC for CAP18

The Live Load is applied to the slab by two 16 kip wheel loads increased by the dynamic load allowance with the reminder of the live load distributed over a 10 ft (AASHTO LRFD 3.6.1.2.1) design lane width. (TxSP)

The Live Load applied to the slab is distributed to the beams assuming the slab is hinged at each beam except the outside beam. (BDM-LRFD, Ch. 4, Sect. 5, Structural Analysis)

4.5.4.1.3 Cap 18 Data Input

Input "Multiple Presence Factors" into CAP18 as "Load Reduction Factors".

Multiple Presence Factors, m (AASHTO LRFD Table 3.6.1.1.2-1)

No. of Lanes	Factor "m"
1	1.20
2	1.00
3	0.85
>3	0.65

Limit States (AASHTO LRFD 3.4.1)

The cap design need only consider Strength I, Service I, and Service I with DL (TxSP).

Strength I

Live Load and Dynamic Load Allowance	LL+IM = 1.75
Dead Load Components	DC = 1.25
Dead Load Wearing Surface (Overlay)	DW = 1.50

TxDOT allows the Overlay Factor to be reduced to 1.25 (TxSP), since overlay is typically used in design only to increase the safety factor, but in this example we will use DW=1.50.

Service I

Live Load and Dynamic Load Allowance	LL+IM = 1.00
Dead Load and Wearing Surface	DC & DW = 1.00

Dead Load

TxDOT considers Service level Dead Load only with a limit reinforcement stress of 22 ksi to minimize cracking. (BDM-LRFD, Chapter 4, Section 5, Design Criteria)

4.5.4.1.4 Cap 18 Output

	Max +M	**Max -M**
Dead Load:	$M_{posDL} = 582.2 \text{ kip} \cdot \text{ft}$	$M_{negDL} = -844.9 \text{ kip} \cdot \text{ft}$
Service Load:	$M_{posServ} = 1067.0 \text{ kip} \cdot \text{ft}$	$M_{negServ} = -1267.9 \text{ kip} \cdot \text{ft}$
Factored Load:	$M_{posUlt} = 1585.8 \text{ kip} \cdot \text{ft}$	$M_{negUlt} = -1812.0 \text{ kip} \cdot \text{ft}$

4.5.4.2 Girder Reactions on Ledge

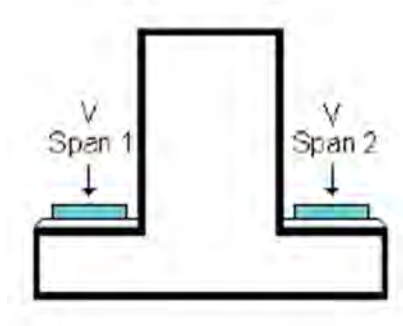


Figure 4.87 Girder Reactions on the Ledge of 60 Degrees Skewed ITBC

Dead Load

$\text{DLSpan1} = \text{Rail1} + \text{Slab1} + \text{Girder1}$ $\qquad$ $\text{DLSpan1} = 50.17\,\frac{\text{kip}}{\text{girder}}$

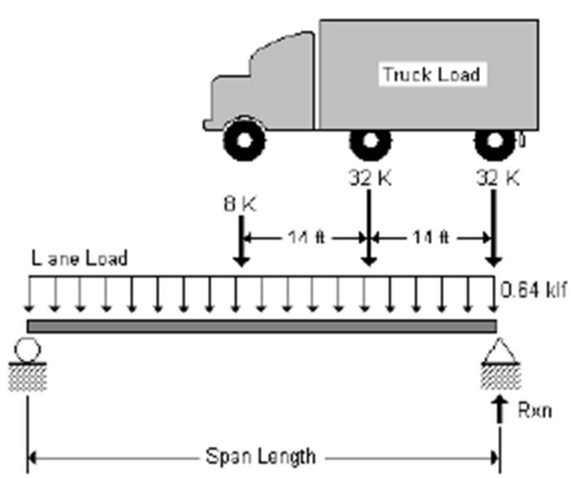
$\text{Overlay1} = 5.04\,\frac{\text{kip}}{\text{girder}}$

$\text{DLSpan2} = \text{Rail2} + \text{Slab2} + \text{Girder2}$ $\qquad$ $\text{DLSpan2} = 104.07\,\frac{\text{kip}}{\text{girder}}$

$\text{Overlay2} = 10.45\,\frac{\text{kip}}{\text{girder}}$

Live Load

Loads per Lane:



Use HL-93 Live Load. For maximum reaction at interior bents, "Design Truck" will always govern over "Design Tandem" for Spans greater than 26ft. For the maximum reaction, place the back (32 kips) axle over the support.

Figure 4.88 Live Load Model of 60 Degrees Skewed ITBC for Girder Reactions on Ledge

$$\textbf{LaneSpan1} = \mathbf{0.64klf} \cdot \left(\frac{\textbf{Span1}}{\mathbf{2}}\right) \qquad \textbf{LaneSpan1} = \mathbf{17.28}\,\frac{\textbf{kip}}{\textbf{lane}}$$

$$\textbf{LaneSpan2} = \mathbf{0.64klf} \cdot \left(\frac{\textbf{Span2}}{\mathbf{2}}\right) \qquad \textbf{LaneSpan2} = \mathbf{35.84}\,\frac{\textbf{kip}}{\textbf{lane}}$$

$$\mathbf{TruckSpan1} = \mathbf{32kip} + \mathbf{32kip} \cdot \left(\frac{\mathbf{Span1-14ft}}{\mathbf{Span1}}\right) + \mathbf{8kip} \cdot \left(\frac{\mathbf{Span1-28ft}}{\mathbf{Span1}}\right)$$

$$\text{TruckSpan1} = 59.56\,\frac{\text{kip}}{\text{lane}}$$

$$\mathbf{TruckSpan2} = \mathbf{32kip} + \mathbf{32kip} \cdot \left(\frac{\mathbf{Span2-14ft}}{\mathbf{Span2}}\right) + \mathbf{8kip} \cdot \left(\frac{\mathbf{Span2-28ft}}{\mathbf{Span2}}\right)$$

$$\text{TruckSpan2} = 66.00\,\frac{\text{kip}}{\text{lane}}$$

$$\text{IM} = 0.33$$

$$\text{LLRxnSpan1} = \text{LaneSpan1} + \text{TruckSpan1} \cdot (1 + \text{IM})$$

$$\text{LLRxnSpan1} = 96.49\,\frac{\text{kip}}{\text{lane}}$$

$$\text{LLRxnSpan2} = \text{LaneSpan2} + \text{TruckSpan2} \cdot (1 + \text{IM})$$

$$\text{LLRxnSpan2} = 123.62\,\frac{\text{kip}}{\text{girder}}$$

Combine "Design Truck" and "Design Lane" loadings (AASHTO LRFD 3.6.1.3).

Dynamic load allowance, IM, does not apply to "Design Lane." (AASHTO LRFD 3.6.1.2.4).

$$gV_{Span1\_Int} = 0.999$$

$$gV_{Span1\_Ext} = 0.999$$

$$gV_{Span2\_Int} = 1.045$$

$$gV_{Span2\_Ext} = 1.045$$

The Live Load Reactions are assumed to be the Shear Live Load Distribution Factor multiplied by the Live Load Reaction per Lane. The Shear Live Load Distribution Factor is calculated using the "LRFD Live Load Distribution Factors" Spreadsheet found in the Appendices.

The Exterior Girders must have a Live Load Distribution Factor equal to or greater than the Interior Girders. This is to accommodate a possible future bridge widening. Widening the bridge would cause the exterior girders to become interior girders

$$\text{LLSpan1Int} = gV_{Span1\_Int} \cdot \text{LLRxnSpan1} \qquad \text{LLSpan1Int} = 96.40\,\frac{\text{kip}}{\text{girder}}$$

$$\text{LLSpan1Ext} = gV_{Span1\_Ext} \cdot \text{LLRxnSpan1} \qquad \text{LLSpan1Ext} = 96.40\,\frac{\text{kip}}{\text{girder}}$$

$$\text{LLSpan2Int} = gV_{Span2\_Int} \cdot \text{LLRxnSpan2} \qquad \text{LLSpan2Int} = 129.18\,\frac{\text{kip}}{\text{girder}}$$

$$\text{LLSpan2Ext} = gV_{Span2\_Ext} \cdot \text{LLRxnSpan2} \qquad \text{LLSpan2Ext} = 129.18\,\frac{\text{kip}}{\text{girder}}$$

<u>Span 1</u>

Interior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{s\_Span1Int} = DLSpan1 + Overlay1 + LLSpan1Int$$

$$V_{s\_Span1Int} = 152 \text{ kip}$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{u\_Span1Int} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 + 1.75 \cdot LLSpan1Int$$

$$V_{u\_Span1Int} = 239 \text{ kip}$$

Exterior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{s\_Span1Ext} = DLSpan1 + Overlay1 + LLSpan1Ext$$

$$V_{s\_Span1Ext} = 152 \text{ kip}$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{u\_Span1Ext} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 + 1.75 \cdot LLSpan1Ext$$

$$V_{u\_Span1Ext} = 239 \text{ kip}$$

Span 2

Interior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{s\_Span2Int} = DLSpan2 + Overlay2 + LLSpan2Int$$

$$V_{s\_Span2Int} = 244 \text{ kip}$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{u\_Span2Int} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot LLSpan2Int$$

$$V_{u\_Span2Int} = 372 \text{ kip}$$

Exterior Girder

Service Load (Service I Limit State, AASHTO LRFD 3.4.1)

$$V_{s\_Span2Ext} = DLSpan2 + Overlay2 + LLSpan2Ext$$

$$V_{s\_Span2Ext} = 244 \text{ kip}$$

Factored Load (Strength I Limit State, AASHTO LRFD 3.4.1)

$$V_{u\_Span2Ext} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot LLSpan2Ext$$

$$V_{u\_Span2Ext} = 372 \text{ kip}$$

4.5.4.3 Torsional Loads

To maximize the torsion, the live load only acts on the longer span.

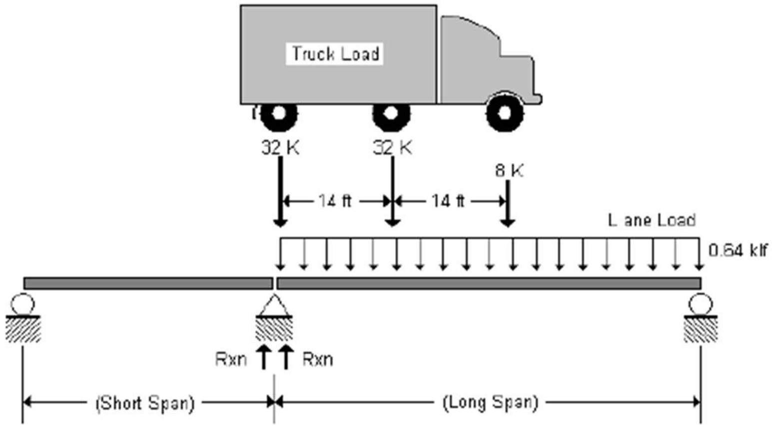


Figure 4.89 Live Load Model of 60 Degrees Skewed ITBC for Torsional Loads

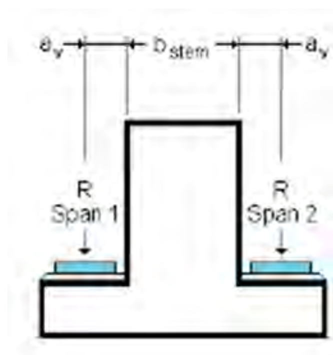


Figure 4.90 Loads on the Ledge of 60 Degrees Skewed ITBC for Torsion

$a_v = 12 \text{ in}$

"a_v" is the value for the distance from the face of the stem to the center of bearing for the girders. 12" is the typical values for TxGirders on ITBC (IGEB). The lever arm is the distance from the center line of bearing to the centerline of the cap.

$b_{stem} = 42 \text{ in}$

$LeverArm = a_v + \frac{1}{2} b_{stem}$ $\qquad LeverArm = 33 \text{ in}$

Interior Girders

Girder Reactions

$$R_{u\_Span1} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1$$

$$R_{u\_Span1} = 70 \text{ kip}$$

$$R_{u\_Span2} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2\_Int} \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)]$$

$$R_{u\_Span2} = 372 \text{ kip}$$

Torsional Load

$$T_{u\_Int} = \left|R_{u\_Span1} - R_{u\_Span2}\right| \cdot LeverArm$$

$$T_{u\_Int} = 830\ kip \cdot ft$$

<u>Exterior Girders</u>

Girder Reactions

$$R_{u\_Span1} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1$$

$$R_{u\_Span1} = 70\ kip$$

$$R_{u\_Span2} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2\_Ext} \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)]$$

$$R_{u\_Span2} = 372\ kip$$

Torsional Load

$$T_{u\_Ext} = \left|R_{u\_Span1} - R_{u\_Span2}\right| \cdot LeverArm$$

$$T_{u\_Ext} = 830\ kip \cdot ft$$

Torsion on Cap

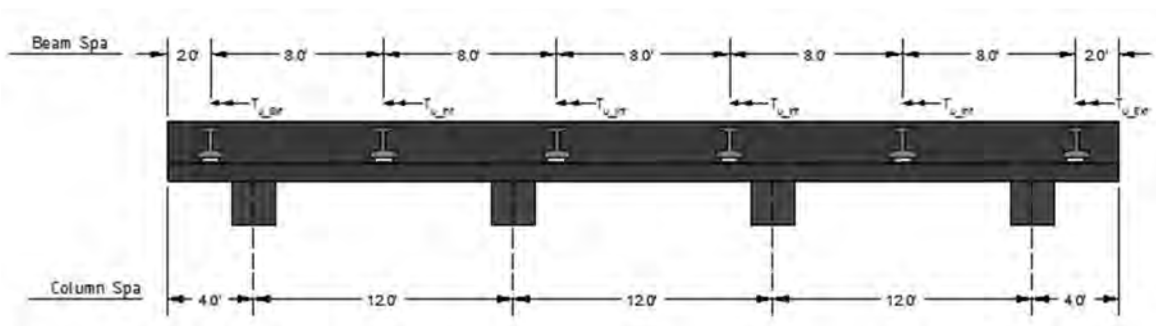


Figure 4.91 Elevation View of 60 Degrees Skewed ITBC with Torsion Loads

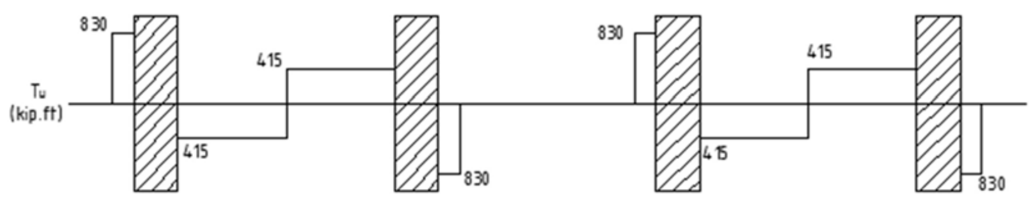


Figure 4.92 Torsion Diagram of 60 Degrees Skewed ITBC

Analyzed assuming Bents are torsionally rigid at Effective Face of Columns.

$T_u = 830\ kip \cdot ft$ *Maximum Torsion on Cap*

4.5.4.4 Load Summary

Ledge Loads

Interior Girder

Service Load

$V_{s\_Int} = \max(V_{s\_Span1Int}, V_{s\_Span2Int})$ $\quad$ $V_{s\_Int} = 243.7\ \text{kip}$

Factored Load

$V_{u\_Int} = \max(V_{u\_Span1Int}, V_{u\_Span2Int})$ $\quad$ $V_{u\_Int} = 371.8\ \text{kip}$

Exterior Girder

Service Load

$V_{s\_Ext} = \max(V_{s\_Span1Ext}, V_{s\_Span2Ext})$ $\quad$ $V_{s\_Ext} = 243.7\ \text{kip}$

Factored Load

$V_{u\_Ext} = \max(V_{u\_Span1Ext}, V_{u\_Span2Ext})$ $\quad$ $V_{u\_Ext} = 371.8\ \text{kip}$

Cap Loads

Positive Moment (From CAP18)

Dead Load: $M_{posDL} = 582.2\ \text{kip} \cdot \text{ft}$

Service Load: $M_{posServ} = 1067.0\ \text{kip} \cdot \text{ft}$

Factored Load: $M_{posUlt} = 1585.8\ \text{kip} \cdot \text{ft}$

Negative Moment (From CAP18)

Dead Load: $M_{negDL} = -844.9\ \text{kip} \cdot \text{ft}$

Service Load: $M_{negServ} = -1267.9\ \text{kip} \cdot \text{ft}$

Factored Load: $M_{negUlt} = -1812.0\ \text{kip} \cdot \text{ft}$

Maximum Torsion and Concurrent Shear and Moment (Strength I)

$T_u = 830\ \text{kip} \cdot \text{ft}$

$V_u = 481.8\ \text{kip}$

$M_u = 769.1\ \text{kip} \cdot \text{ft}$

Located two stations away from centerline of column.

V_u and M_u values are from CAP18

4.5.5 Locate and Describe Reinforcing

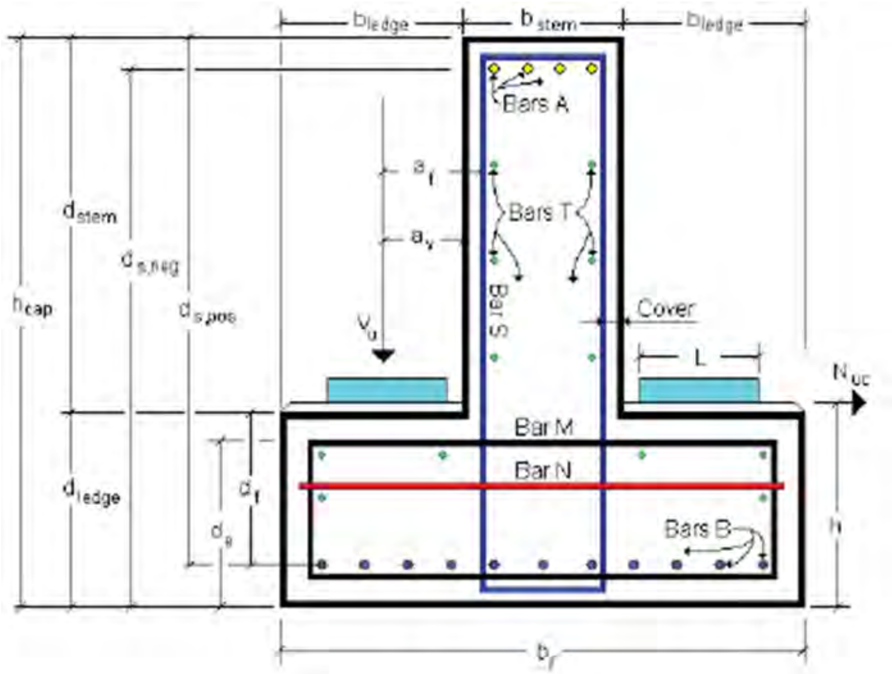


Figure 4.93 Section View of 60 Degrees Skewed ITBC

Recall:

$b_{stem} = 42 \text{ in}$

$d_{stem} = 57 \text{ in}$

$b_{ledge} = 25 \text{ in}$

$d_{ledge} = 28 \text{ in}$

$b_f = 92 \text{ in}$

$h_{cap} = 85 \text{ in}$

$\text{cover} = 2.5 \text{ in}$

4.5.5.1 Describe Reinforcing Bars

Use # 11 bars for Bar A

$A_{bar\_A} = 1.56\ in^2$ $d_{bar\_A} = 1.410\ in$

Use # 11 bars for Bar B

$A_{bar\_B} = 1.56\ in^2$ $d_{bar\_B} = 1.410\ in$

Use # 7 bars for Bar M

$A_{bar\_M} = 0.60\ in^2$ $d_{bar\_M} = 0.875 in$

Use # 7 bars for Bar N

$A_{bar\_N} = 0.60\ in^2$ $d_{bar\_N} = 0.875\ in$

Use # 6 bars for Bar S

$A_{bar\_S} = 0.44\ in^2$ $d_{bar\_S} = 0.75\ in$

Use # 6 bars for Bar T

$A_{bar\_T} = 0.44\ in^2$ $d_{bar\_T} = 0.75\ in$

In the calculation of b_{ledge}, # 7 Bar M was considered. Bar M must be # 7 or smaller to allow it fully develop.

To prevent confusion, use the same bar size for Bar N as Bar M.

4.5.5.2 Calculate Dimensions

$d_{s\_neg} = h_{cap} - cover - \frac{1}{2} d_{bar\_S} - \frac{1}{2} d_{bar\_A}$ $d_{s\_neg} = 81.42\ in$

$d_{s\_pos} = h_{cap} - cover - \frac{1}{2} max(d_{ba\ \_S}, d_{bar\_M})\ - \frac{1}{2} d_{bar\_B}$ $d_{s\_pos} = 81.36\ in$

$a_v = 12\ in$

$a_f = a_v + cover$ $a_f = 14.50\ in$

$d_e = d_{ledge} - cover$ $d_e = 25.50\ in$

$d_f = d_{ledge} - cover - \frac{1}{2} d_{bar\_M} - \frac{1}{2} d_{bar\_B}$ $d_f = 24.36\ in$

$h = d_{ledge} + BrgSeat$ $h = 29.50\ in$

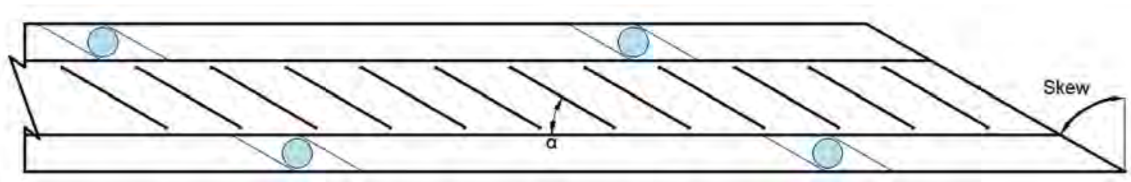


Figure 4.94 Plan View of 60 Degrees Skewed ITBC

$\alpha = 30\ \text{deg}$ *Angle of Bars S (Angle from the horizontal)*

Recall:

$L = 15\ \text{in}$ *Dimension of Bearing Pad (15" Dia. Circular Bearing Pad)*

$W = 15\ \text{in}$

4.5.6 Check Bearing

The load on the bearing pad propagates along a truncated pyramid whose top has the area A_1 and whose base has the area A_2. A_1 is the loaded area (the bearing pad area: LxW). A_2 is the area of the lowest rectangle contained wholly within the support (the Inverted Tee Cap). A_2 must not overlap the truncated pyramid of another load in either direction, nor can it extend beyond the edges of the cap in any direction.

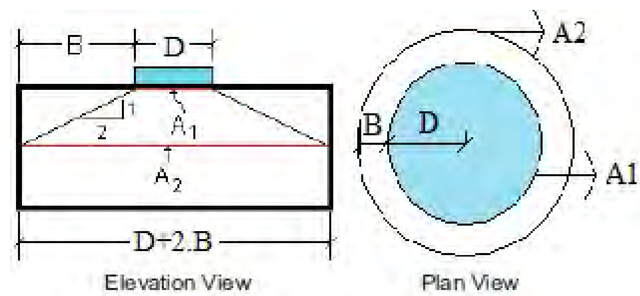


Figure 4.95 Bearing Check for 60 Degrees Skew Angle

Resistance Factor (ϕ) = 0.7 *(AASHTO LRFD 5.5.4.2)*

$A_1 = \frac{\pi}{4}\, d_1{}^2$ $\quad d_1 = 15in,\ A_1 = 176.71\ in^2$ *Area under Bearing Pad*

Interior Girders

$$B = \min\left[\left(b_{ledge} - a_v\right) - \frac{1}{2}L, \left(a_v + \frac{1}{2}b_{stem}\right) - \frac{1}{2}L, 2d_{ledge}, \frac{1}{2}S - \frac{1}{2}W\right]$$

"B" is the distance from perimeter of A_1 to the perimeter of A_2 as seen in the above figure

$B = 5.5\ \text{in.}$

Diameter of truncated area, d_2 = = d_1 + 2·B $\quad d_2$= 26 in

Base of the truncated pyramid, $A_2 = \frac{\pi}{4}\, d_2{}^2$ $\quad A_2$=530.93 in^2

Modification factor

$m = \min\left(\sqrt{\frac{A_2}{A_1}}, 2\right) = 1.73$ and 2 $m = 1.73$ *AASHTO LRFD Eq. 5.6.5-3*

$\phi V_n = \phi \ \ 0.85 \ \ f_c \ \ A_1 \ \ m$ $\phi V_n = 909.48$ kips *AASHTO LRFD Eqs. 5.6.5-1 and 5.6.5-2.*

$V_{u\_Int} = 371.8 < \phi V_n$ BearingChk = "OK!" *V_{u_int} from "4.5.4.4 Load Summary".*

Exterior Girders

$$B = \min\left[\left(b_{ledge} - a_v\right) - \frac{1}{2}L, \left(a_v + \frac{1}{2}b_{stem}\right) - \frac{1}{2}L, 2d_{ledge}, \frac{1}{2}S - \frac{1}{2}W, c - \frac{1}{2}W\right]$$

B= 5.5 in. *"B" is the distance from perimeter of A_1 to the perimeter of A_2 as seen in the above figure*

Diameter of truncated area, $d_2 = = d_1 + 2 \cdot B$ $d_2 = 26$ in

Base of the truncated pyramid, $A_2 = \frac{\pi}{4}\, d_2{}^2$ $A_2 = 530.93\ in^2$

Modification factor

$m = \min\left(\sqrt{\frac{A_2}{A_1}}, 2\right) = 1.73$ and 2 $m = 1.73$ *AASHTO LRFD Eq. 5.6.5-3*

$\phi V_n = \phi \ \ 0.85 \ \ f_c \ \ A_1 \ \ m$ $\phi V_n = 909.48$ kips *AASHTO LRFD Eqs. 5.6.5-1 and 5.6.5-2:*

$V_{u\_ext} = 371.8 \text{ kips} < \Phi V_n$ BearingChk= "OK!" *V_{u_ext} from "4.5.4.4 Load Summary".*

4.5.7 Check Punching Shear

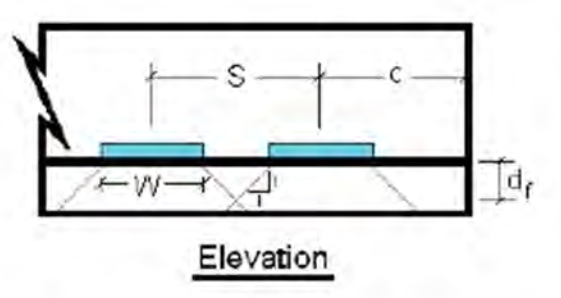


Figure 4.96 Punching Shear Check for 60 Degrees Skew Angle

*AASHTO LRFD **5.8.4.3.4**, the truncated pyramids assumed as failure surfaces for punching shear shall not overlap.*

Resistance Factor (ϕ) = 0.90

AASHTO LRFD 5.5.4.2.

Determine if the Shear Cones Intersect

Is $\frac{1}{2}S - \frac{1}{2}W \geq d_f$?

Yes. Therefore, shear cones do not intersect in the longitudinal direction of the cap.

$\frac{1}{2}S - \frac{1}{2}W = 40.5 \text{ in}$

$d_f = 24.36 \text{ in}$

TxDOT uses "d_f" instead of "d_e" for Punching Shear (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria). This is because "d_f" has traditionally been used for inverted tee bents and was sed in the Inverted Tee Research (Furiong % Mirza pg. 58).

Is $\frac{1}{2}b_{stem} + a_v - \frac{1}{2}L \geq d_f$?

Yes. Therefore, shear cones do not intersect in the transverse direction of the cap.

$\frac{1}{2}b_{stem} + a_v - \frac{1}{2}L = 25.5 \text{ in}$

$d_f = 24.36 \text{ in}$

Interior Girders

$V_n = 0.125 \, ⍰ \, \lambda \sqrt{f'_c} \, b_o \, d_f$ $\quad V_n = 581.39 \text{ kips}$ *AASHTO LRFD 5.8.4.3.4-3*

$b_o = \frac{\pi}{2} * (D + d_f) + D$ $\quad b_o = 76.82 \, in$ *AASHTO LRFD 5.8.4.3.4-4*

$\phi V_n = 523.25 \text{ kips}$

$V_{u\_Int} = 371.25 \text{ kips} < \phi V_n$ PunchingShearChk= "OK!"

V_{u_int} from "*4.5.4.4 Load Summary*".

Exterior Girders

$V_n = \min[0.125 \cdot \sqrt{f_c} \cdot \left(\frac{\pi}{4} \cdot (D + d_f) + \frac{D}{2} + c\right) \cdot d_f, 0.125 \cdot \sqrt{f_c} \cdot \frac{\pi}{2} \cdot (D + d_f) + D]$ $V_n = 424.96$ kips *AASHTO LRFD 5.8.4.3.4-3 and 5.8.4.3.4-5*

$\phi V_n = 382.46$ kips

$V_{u\_ext} = 371.8 \text{ kips} < \phi V_n$ PunchingShearChk= "OK!" *V_{u_ext} from "4.5.4.4 Load Summary".*

4.5.8 Check Shear Friction

Resistance Factor (ϕ) =0.90 *AASHTO LRFD 5.5.4.2*

Determine the Distribution Width

<u>Interior Girders</u>

$b_{s\_Int} = \min(W + 4a_v, S)$ *"S" is the girder spacing.*

= min (63 in, 96 in)

$b_{s\_Int} = 63$ in

$A_{cv} = b_{s\_Int} \cdot d_e$ $A_{cv} = 1606.5\ in^2$

<u>Exterior Girders</u>

$b_{s\_Ext} = \min(W + 4a_v, S, 2c)$ *"S" is the girder spacing.*

= min [69, 96, 48]

= 48 in

$A_{cv} = b_{s\_ext} \cdot d_e$ $A_{cv} = 1224$ in2

Interior Girders

$V_n = \min(0.2 \cdot f_c \cdot A_{cv}, 0.8 \cdot A_{cv})$ $V_n = 1285.2$ kips *AASHTO LRFD 5.8.4.2.2-1 and 5.8.4.2.2-2*

= min (1606.5, 1285.2)

$\phi V_n = 1156.68$ kips

$V_{u\_Int} = 371.68 \text{ kips} < \phi V_n$ ShearFrictionChk= "OK!" *V_{u_int} from "4.5.4.4 Load Summary"*

.

Exterior Girders

$V_n = \min(0.2 \cdot f_c \cdot A_{cv}, 0.8 \cdot A_{cv})$ = min (1224, 979.2)	V_n = 979.2 kips	*AASHTO LRFD 5.8.4.2.2-1 and 5.8.4.2.2-2*
$\phi V_n = 881$ kips		
$V_{u\_ext} = 371.81$ kips $< \phi V_n$	ShearFrictionChk= "OK!"	V_{u_ext} *from "4.5.4.4 Load Summary".*

4.5.9 Flexural Reinforcement for Negative Bending (Bars A)

$M_{dl} = \lvert M_{negDL} \rvert$	$M_{dl} = 844.9\ kip \cdot ft$
$M_s = \lvert M_{negServ} \rvert$	$M_s = 1267.9\ kip \cdot ft$
$M_u = \lvert M_{negUlt} \rvert$	$M_u = 1812.0\ kip \cdot ft$

4.5.9.1 Minimum Flexural Reinforcement

Factored Flexural Resistance, M_r, must be greater than or equal to the lesser of $1.2M_{cr}$ (Cracking Moment) or $1.33M_u$ (Ultimate Moment).

$I_g = 3.06 \times 10^6\ in^4$		*Gross Moment of Inertia*
$h_{cap} = 85\ in$		*Depth of Cap*
$ybar = 34.5\ in$		*Distance to the Center of Gravity of the Cap from the bottom of the Cap*
$f_r = 0.24\sqrt{f_c}$	$f_r = 0.537\ ksi$	*Modulus of Rupture (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)*
$y_t = h_{cap} - ybar$	$y_t = 50.50\ in$	*Distance from Center of Gravity to extreme tension fiber*
$S = \frac{I_g}{y_t}$	$S = 6.06 \times 10^4\ in^3$	*Section Modulus for the extreme tension fiber*
$M_{cr} = S \cdot f_r \cdot \frac{1ft}{12in}$	$M_{cr} = 2711.8\ kip \cdot ft$	*Cracking Moment (AASHTO LRFD Eq. 5.6.3.3-1)*

M_f = minimum of:

$1.2M_{cr} = 3254.2\ kip \cdot ft$

$1.33M_u = 2410.0\ kip \cdot ft$

Design the lesser of $1.2M_{cr}$ or $1.33M_u$ when determining mininum area of steel required.

Thus, M_r must be greater than $M_f = 2410\ kip \cdot ft$

4.5.9.2 Moment Capacity Design

Try, 7 ~ #11's Top

$BarANo = 7$		*Number of bars in tension*
$d_{bar\_A} = 1.410\ in$		*Diameter of main reinforcing bars*
$A_{bar\_A} = 1.56\ in^2$		*Area of main reinforcing bars*
$A_s = BarANo \cdot A_{bar\_A}$	$A_s = 10.92\ in^2$	*Area of steel in tension*
$d_{stirrup} = d_{bar\_S}$	$d_{stirrup} = 0.75\ in$	*Diameter of shear reinforcing bars*
$d = d_{s\_neg}$	$d = 81.42\ in$	
$b = b_f$	$b = 92\ in$	
$f_c = 5.0\ ksi$		*Compressive Strength of Concrete*
$f_y = 60\ ksi$		*Yield Strength of Rebar*
$\beta_1 = 0.85 - 0.05(f_c - 4ksi)$		*(AASHTO LRFD 5.6.2.2)*
Bounded by: $0.65 \leq \beta_1 \leq 0.85$	$\beta_1 = 0.80$	
$c = \frac{A_s f_y}{0.85\ {}_c\beta_1 b}$	$c = 2.09\ in$	*Depth of Cross Section under Compression under Ultimate Load (AASHTO LRFD Eq. 5.6.3.1.2-4)*

This "c" is the distance from the extreme compression fiber to the neutral axis, not the distance from the center of bearing of the last girder to the end of the cap.

$a = c \cdot \beta_1$	$a = 1.67\ in$	*Depth of Equivalent Stress Block (AASHTO LRFD 5.6.2.2)*

Note: "a" is less than "d_{ledge}". Therefore the equivalent stress block acts over a rectangular area. If "a" was greater than "d_{ledge}", it would act over a Tee shaped area.

$M_n = A_s f_y \left(d - \frac{a}{2}\right) \cdot \frac{1ft}{12in}$	$M_n = 4400\ kip \cdot ft$	*Nominal Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.2-1)*
$\varepsilon_s = 0.003 \cdot \frac{d-c}{c}$	$\varepsilon_s = 0.114$	*Strain in Reinforcing at Ultimate*
$\varepsilon_s > 0.005$		
FlexureBehavior = "Tension Controlled"		*(AASHTO LRFD 5.6.2.1)*
$\Phi_M = 0.90$		*(AASHTO LRFD 5.5.4.2)*
$M_r = \Phi_M M_n$	$M_r = 3960\ kip \cdot ft$	*Factored Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.1-1)*
$M_f = 2410\ kip \cdot ft < M_r$	MinReinfChk = "OK!"	
$M_u = 1812\ kip \cdot ft < M_r$	UltimateMom = "OK!"	

4.5.9.3 Check Serviceability

For service loads, the stress on the cross-section is located as shown in Figure 4.97.

To find s_{max}:

Modular Ratio:

$$n = \frac{E_s}{E_c} \qquad n = 7.12$$

Tension Reinforcement Ratio:

$$\rho = \frac{A_s}{b \cdot d} \qquad \rho = 0.00146$$

$$k = \sqrt{(2\rho n) + (\rho n)^2} - (\rho n) \qquad k = 0.134$$

$$d \cdot k = 10.91\ \text{in} \ < \ d_{ledge} = 28\ \text{in}$$

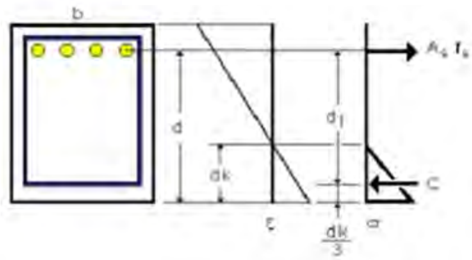


Figure 4.97 Stresses on the Cross Section for Service Loads of 60 Degrees Skewed ITBC

Therefore, the compression force acts over a rectangular area.

$$j = 1 - \frac{k}{3} \qquad j = 0.955$$

If the compression force does not act over rectangular area, j will be different.

$$f_{ss} = \frac{M_s}{A_s \cdot j \cdot d} \cdot \frac{12\text{in}}{1\text{ft}} \qquad f_{ss} = 17.92\ \text{ksi}$$

Service Load Bending Stress in outer layer of the reinforcing.

$$f_a = 0.6 f_y \qquad f_a = 36.00\ \text{ksi}$$

Allowable Bending Stress (BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

$f_{ss} < f_a$ ServiceStress = “OK!”

$$d_c = \text{cover} + \frac{1}{2} d_{stirrup} + \frac{1}{2} d_{bar\_A} \qquad d_c = 3.58\ \text{in}$$

Exposure Condition Factor:

$$\gamma_e = 1.00$$

For Class 1 Exposure Conditions. For areas where deicing chenicals are frequently used, design for Class 2 Exposure ($\gamma_e = 0.75$). (BDM-LRFD Ch. 4, Sect. 5, Design Criteria) (AASHTO LRFD Eq. 5.6.7-1)

$$\beta_s = 1 + \frac{d_c}{0.7(h_{cap} - d_c)} \qquad \beta_s = 1.06$$

$$s_{max} = \min\left(\frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c, 12\text{in.}\right) \qquad s_{max} = 12\ \text{in}$$

$$s_{Actual} = \frac{b_{stem} - 2d_c}{\text{BarANo} -} \qquad s_{Actual} = 5.81\ \text{in}$$

A good practice is to place a bar every 12 in along each surface of the bent. (TxSP)

$s_{actual} < s_{max}$ ServiceabilityCheck = “OK!

4.5.9.4 Check Dead Load

Check allowable M_{dl}: $f_{dl} = 22\ \text{ksi}$

TxDOT limits dead load stress to 22 ksi, which is set to limit observed cracking under dead load.

$$M_a = A_s \cdot d \cdot j \cdot f_{dl} \cdot \frac{1\text{ft}}{12\text{in}} \qquad M_a = 1556.7\ \text{kip} \cdot \text{ft}$$

Allowable Dead Load Moment

$M_{dl} = 844.9\ \text{kip} \cdot \text{ft} \ < \ M_a$ DeadLoadMom = “OK!”

4.5.10 Flexural Reinforcement for Positive Bending (Bars B)

$M_{dl} = M_{posDL}$	$M_{dl} = 582.2\ \text{kip} \cdot \text{ft}$
$M_s = M_{posServ}$	$M_s = 1067.0\ \text{kip} \cdot \text{ft}$
$M_u = M_{posUlt}$	$M_u = 1585.8\ \text{kip} \cdot \text{ft}$

4.5.10.1 Minimum Flexural Reinforcement

Factored Flexural Resistance, M_r, must be greater than or equal to the lesser of $1.2M_{cr}$ (Cracking Moment) or $1.33M_u$ (Ultimate Moment).

$I_g = 3.06 \times 10^6\ \text{in}^4$		*Gross Moment of Inertia*
$y_t = \text{ybar}$	$y_t = 34.5\ \text{in}$	*Distance to the Center of Gravity of the Cap from the top of the Cap*
$f_r = 0.24\sqrt{f_c}$	$f_r = 0.537\ \text{ksi}$	*Modulus of Rupture (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)*
$S = \frac{I_g}{y_t}$	$S = 8.87 \times 10^4\ \text{in}^3$	*Section Modulus for the extreme tension fiber*
$M_{cr} = S \cdot f_r \cdot \frac{1\text{ft}}{12\text{in}}$	$M_{cr} = 3969.3\ \text{kip} \cdot \text{ft}$	*Cracking Moment (AASHTO LRFD Eq. 5.6.3.3-1)*

M_f = minimum of:

$1.2M_{cr} = 4763.2\ \text{kip} \cdot \text{ft}$

$1.33M_u = 2109.1\ \text{kip} \cdot \text{ft}$

Design the lesser of $1.2M_{cr}$ or $1.33M_u$ when determining mininum area of steel required.

Thus, M_r must be greater than $M_f = 2109.1\ \text{kip} \cdot \text{ft}$

4.5.10.2 Moment Capacity Design

Try, 11 ~ #11's Bottom — *Number of bars in tension*

$BarBNo = 11$

$d_{bar\_B} = 1.41\ in$ — *Diameter of main reinforcing bars*

$A_{bar\_B} = 1.56\ in^2$ — *Area of main reinforcing bars*

$A_s = BarBNo \cdot A_{bar\_B}$ $\quad A_s = 17.16\ in^2$ — *Area of steel in tension*

$d = d_{s\_pos}$ $\quad d = 81.36\ in$

$b = b_{stem}$ $\quad b = 42\ in$

$f_c = 5.0\ ksi$ — *Compressive Strength of Concrete*

$f_y = 60\ ksi$ — *Yield Strength of Rebar*

$\beta_1 = 0.85 - 0.05(f_c - 4ksi)$ — *(AASHTO LRFD 5.6.2.2)*

Bounded by: $0.65 \le \beta_1 \le 0.85$ $\quad \beta_1 = 0.80$

$c = \frac{A_s f_y}{0.85\ {}_c\beta_1 b}$ $\quad c = 7.21\ in$ — *Depth of Cross Section under Compression under Ultimate Load (AASHTO LRFD Eq. 5.6.3.1.2-4)*

This "c" is the distance from the extreme compression fiber to the neutral axis, not the distance from the center of bearing of the last girder to the end of the cap.

$a = c \cdot \beta_1$ $\quad a = 5.77\ in$ — *Depth of Equivalent Stress Block (AASHTO LRFD 5.6.2.2)*

Note: "a" is less than "d_{stem}". Therefore the equivalent stress block acts over a rectangular area. If "a" was greater than "d_{stem}", it would act over a Tee shaped area.

$M_n = A_s f_y \left(d - \frac{a}{2}\right) \cdot \frac{1ft}{12in}$ $\quad M_n = 6733.2\ kip \cdot ft$ — *Nominal Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.2-1)*

$\varepsilon_s = 0.003 \cdot \frac{d-c}{c}$ $\quad \varepsilon_s = 0.031$ — *Strain in Reinforcing at Ultimate*

$\varepsilon_s > 0.005$

FlexureBehavior = "Tension Controlled" — *(AASHTO LRFD 5.6.2.1)*

$\Phi_M = 0.90$ — *(AASHTO LRFD 5.5.4.2)*

$M_r = \Phi_M \cdot M_n$ $\quad M_r = 6059.9\ kip \cdot ft$ — *Factored Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.1-1)*

$M_f = 2109.1\ kip \cdot ft < M_r$ $\quad$ MinReinfChk = "OK!"

$M_u = 1585.8\ kip \cdot ft < M_r$ $\quad$ UltimateMom = "OK!"

4.5.10.3 Check Serviceability

To find s_{max}:

For service loads, the stress on the cross-section is located as shown in Figure 4.98.

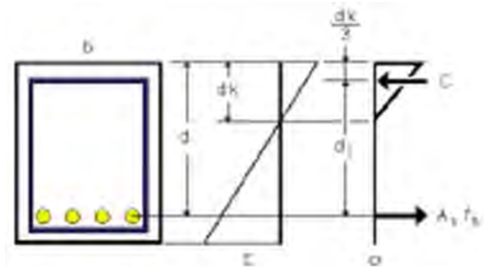


Figure 4.98 Stresses on the Cross Section for Bars B for Service Loads of 60 Degrees Skewed ITBC

Modular Ratio:

$$n = \frac{E_s}{E_c} \qquad n = 7.12$$

Tension Reinforcement Ratio:

$$\rho = \frac{A_s}{b \cdot d} \qquad \rho = 0.005$$

$$k = \sqrt{(2\rho n) + (\rho n)^2} - (\rho n) \qquad k = 0.234$$

$d \cdot k = 19.04\ \text{in} < d_{stem} = 57.00\ \text{in}$

Therefore, the compression force acts over a rectangular area.

$$j = 1 - \frac{k}{3} \qquad j = 0.922$$

If the compression force does not act over rectangular area, j will be different.

$$f_{ss} = \frac{M_s}{A_s \cdot j \cdot d} \cdot \frac{12in}{1ft} \qquad f_{ss} = 9.95\ \text{ksi}$$

Service Load Bending Stress in outer layer of the reinforcing.

$$f_a = 0.6 f_y \qquad f_a = 36.00\ \text{ksi}$$

Allowable Bending Stress (BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

$f_{ss} < f_a$ ServiceStress = "OK!"

$$d_c = cover + \frac{1}{2} d_{stirrup} + \frac{1}{2} d_{bar\_B} \qquad d_c = 3.64\ \text{in}$$

Exposure Condition Factor:

$$\gamma_e = 1.00$$

For Class 1 Exposure Conditions. For areas where deicing chenicals are frequently used, design for Class 2 Exposure ($\gamma_e = 0.75$). (BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

$$\beta_s = 1 + \frac{d_c}{0.7(h_{cap} - d_c)} \qquad \beta_s = 1.06$$

$$s_{max} = \min\left(\frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c, 12in.\right) \qquad s_{max} = 12\ \text{in}$$

(AASHTO LRFD Eq. 5.6.7-1)

A good practice is to place a bar every 12 in along each surface of the bent. (TxSP)

Bars Inside Stirrup Bar S

Try: BarBInsideSNo = 5

Number of Bars B that are inside Stirrup Bar S.

$$s_{Actual} = \frac{b_{stem} - 2\left(cover + \frac{1}{2} d_{bar\_S} + \frac{1}{2} d_{bar\_B}\right)}{BarBInsideSNo -} \qquad s_{Actual} = 8.71\ \text{in}$$

$s_{actual} < s_{max}$ ServiceabilityCheck = "OK

Bars Outside Stirrup Bar S

$BarBOutsideSNo = 11 - BarBInsideSNo$

Number of Bars B that are inside Stirrup Bar S.

$BarBOutsideSNo = 6$

$$s_{Actual} = \frac{2b_{ledge} + 2\left(cover \ \ \frac{1}{2}d_{bar\_S} + \frac{1}{2}d_{bar\_B} - cove \ \ \ \frac{1}{2}d_{bar\_M} - \frac{1}{2}d_{bar\_B}\right)}{BarBOutsideSNo}$$

$s_{actual} = 8.31\ in\ < s_{max}$ ServiceabilityCheck = "OK

4.5.10.4 Check Dead Load

Check allowable M_{dl}: $f_{dl} = 22\ ksi$

TxDOT limits dead load stress to 22 ksi. This is due to observed cracking under dead load.

$M_a = A_s \cdot d \cdot j \cdot f_{dl} \cdot \frac{1ft}{12in}$ $M_a = 2360\ kip \cdot ft$

Allowable Dead Load Moment

$M_{dl} = 582.2\ kip \cdot ft\ < M_a$ DeadLoadMom = "OK!"

Flexural Steel Summary:

Use 7 ~ # 11 Bars on Top

& 11 ~ # 11 Bars on Bottom

4.5.11 Ledge Reinforcement (Bars M & N)

Try Bars M and Bars N at a 5.80" spacing.

Use trial and error to determine the spacing needed for the ledge reinforcing.

It is typical for Bars M & N to be paired together

$$s_{bar\_M} = 5.80 \text{ in}$$

$$s_{bar\_N} = 5.80 \text{ in}$$

4.5.11.1 Determine Distribution Widths

These distribution widths will be used on the following pages to determine the required ledge reinforcement per foot of cap.

Distribution Width for Shear (AASHTO LRFD 5.8.4.3.2)

Note: These are the same distribution widths used for the Shear Friction check.

"S" is the girder spacing.

Interior Girders

$$b_{s\_Int} = \min(W + 4a_v, S)$$

$$b_{s\_Int} = 63.00 \text{ in}$$

Exterior Girders

"c" is the distance from the center of bearing of the outside beam to the end of the ledge.

$$b_{s\_Ext} = \min(W + 4a_v, 2c, S)$$

$$b_{s\_Ext} = 48.00 \text{ in}$$

Distribution Width for Bending and Axial Loads (AASHTO LRFD 5.8.4.3.3)

Interior Girders

$$b_{m\_Int} = \min(W + 5a_f, S)$$

$$b_{m\_Int} = 87.50 \text{ in}$$

Exterior Girders

$$b_{m\_Ext} = \min(W + 5a_f, 2c, S)$$

$$b_{m\_Ext} = 48.00 \text{ in}$$

4.5.11.2 Reinforcing Required for Shear Friction

AASHTO LRFD 5.7.4.1

$\Phi = 0.90$

(AASHTO LRFD 5.5.4)

$\mu = 1.4$ $c_1 = 0\ \text{ksi}$ $P_c = 0\ \text{kip}$

"μ" is 1.4 for monolithically placed concrete. (AASHTO LRFD 5.7.4.4)

Recall: $d_e = 25.50\ \text{in}$

For clarity, the cohesion factor is labeled "c_1". This is to prevent confusion with "c", the distance from the last girder to the edge of the cap. c_1 is 0ksi for corbels and ledges. (AASHTO LRFD 5.7.4.4)

"P_c" is zero as there is no axial compression.

Minimum Reinforcing (AASHTO LRFD Eq. 5.7.4.2-1)

$$A_{vf\_min} = \frac{0.05\ \text{ksi} \cdot A_{cv}}{f_y}$$

$$A_{cv} = d_e \cdot b_s \quad \text{and} \quad a_{vf} = \frac{A_{vf}}{b_s}$$

$$a_{vf\_min} = \frac{0.05\text{ksi} \cdot d_e}{f_y} \qquad a_{vf\_min} = 0.26\,\frac{\text{in}^2}{\text{ft}}$$

Minimum Reinforcing required for Shear Friction

Interior Girders

$$A_{cv} = d_e \cdot b_{s\_Int} \qquad A_{cv} = 1606.5\ \text{in}^2$$

$$V_{u\_Int} = 371.8\ \text{kip}$$

From "4.5.4.4 Load Summary".

$$V_n = c_1 A_{cv} + \mu\left(A_{vf} f_y + P_c\right)$$

(AASHTO LRFD Eq. 5.7.4.3-3)

$$\Phi V_n \geq V_u$$

(AASHTO LRFD Eq. 5.7.4.3-1 & AASHTO LRFD Eq. 5.7.4.3-2)

$$\Phi \cdot \left[c_1 A_{cv} + \mu\left(A_{vf} f_y + P_c\right)\right] \geq V_u$$

$$A_{vf} = \frac{\frac{\frac{V_{u\_Int}}{\Phi} - c_1 A_{cv}}{\mu} - P_c}{f_y} \qquad A_{vf} = 4.92\ \text{in}^2$$

Required Reinforcing for Shear Friction

$$a_{vf\_Int} = \frac{A_{vf}}{b_{s\_Int}} \qquad a_{vf\_Int} = 0.94\,\frac{\text{in}^2}{\text{ft}}$$

Required Reinforcing for Shear Friction per foot length of cap

Exterior Girders

$A_{cv} = d_e \cdot b_{s\_Ext}$ $A_{cv} = 1224\ in^2$

$V_{u\_Ext} = 371.8\ kip$ *From "4.5.4.4 Load Summary".*

$V_n = c_1 A_{cv} + \mu(A_{vf} f_y + P_c)$ *(AASHTO LRFD Eq. 5.7.4.3-3)*

$\Phi V_n \geq V_u$ *(AASHTO LRFD Eq. 5.7.4.3-1 & AASHTO LRFD Eq. 5.7.4.3-2)*

$\Phi \cdot [c_1 A_{cv} + \mu(A_{vf} f_y + P_c)] \geq V_u$

$A_{vf} = \frac{\frac{\frac{V_{u\_Ext}}{\Phi} - c_1 A_{cv}}{\mu} - P_c}{f_y}$ $A_{vf} = 4.92\ in^2$ *Required Reinforcing for Shear Friction*

$a_{vf\_Ext} = \frac{A_{vf}}{b_{s\_Ext}}$ $a_{vf\_Ext} = 1.23 \frac{in^2}{ft}$ *Required Reinforcing for Shear Friction per foot length of cap*

4.5.11.3 Reinforcing Required for Flexure

AASHTO LRFD 5.8.4.2.1

Recall: $h = 29.50\ in$ $d_e = 25.50\ in$ $a_v = 12\ in$ *From "4.5.5.2 Calculate Dimensions"*

Interior Girders

$V_{u\_Int} = 371.8\ kip$ *From "4.5.4.4 Load Summary".*

$N_{uc\_Int} = 0.2 \cdot V_{u\_Int}$ $N_{uc\_Int} = 74.4\ kip$ *(AASHTO LRFD 5.8.4.2.1)*

$M_{u\_Int} = V_{u\_Int} \cdot a_v + N_{uc\_Int}(h - d_e)$ $M_{u\_Int} = 397\ kip \cdot ft$ *(AASHTO LRFD Eq. 5.8.4.2.1-1)*

Use the following equations to solve for A_f:

$\Phi M_n \geq M_{u\_Int}$ *(AASHTO LRFD Eq. 1.3.2.1-1)*

$M_n = A_f f_y \left(d_e - \frac{a}{2}\right)$ *(AASHTO LRFD Eq.5.6.3.2.2-1)*

$c = \frac{A_f f_y}{\alpha_1 f_c \beta_1 b_{m\_Int}}$ *(AASHTO LRFD Eq. 5.6.3.1.2-4)*

$\alpha_1 = 0.85$

$\beta_1 = 0.80$ *(AASHTO LRFD 5.6.2.2)*

$a = c\beta_1$

$0.75 \leq \Phi = 0.65 + 0.15\left(\frac{d_e}{c} - 1\right) \leq 0.90$ *AASHTO LRFD 5.5.4.2*

Solve for A_f: $A_f = 3.50\ in^2$ *Required Reinforcing for Flexure*

$a_{f\_Int} = \frac{A_f}{b_{m\_Int}}$ $a_{f\_Int} = 0.48 \frac{in^2}{ft}$ *Required Reinforcing for Flexure per foot length of cap*

Exterior Girders

$V_{u\_Ext} = 371.8\ \text{kip}$ *From "4.5.4.4 Load Summary".*

$N_{uc\_Ext} = 0.2 \cdot V_{u\_Ext}$ $N_{uc\_Ext} = 74.4\ \text{kip}$ *(AASHTO LRFD 5.8.4.2.1)*

$M_{u\_Ext} = V_{u\_Ext} \cdot a_v + N_{uc\_Ext}(h - d_e)$ $M_{u\_Ext} = 397\ \text{kip} \cdot \text{ft}$ *(AASHTO LRFD Eq. 5.8.4.2.1-1)*

Use the following equations to solve for A_f:

$\Phi M_n \geq M_{u\_Ext}$ *(AASHTO LRFD Eq. 1.3.2.1-1)*

$M_n = A_f f_y \left(d_e - \frac{a}{2}\right)$ *(AASHTO LRFD Eq.5.6.3.2.2-1)*

$c = \frac{A_f f_y}{\alpha_1 f_c \beta_1 b_{m\_Ext}}$ *(AASHTO LRFD Eq. 5.6.3.1.2-4)*

$\alpha_1 = 0.85$

$\beta_1 = 0.80$ *(AASHTO LRFD 5.6.2.2)*

$a = c\beta_1$

$0.75 \leq \Phi = 0.65 + 0.15\left(\frac{d_e}{c} - 1\right) \leq 0.90$ *AASHTO LRFD 5.5.4.2*

Solve for A_f: $A_f = 3.53\ \text{in}^2$ *Required Reinforcing for Flexure*

$a_{f\_Ext} = \frac{A_f}{b_{m\_Ext}}$ $a_{f\_Ext} = 0.88 \frac{\text{in}^2}{\text{ft}}$ *Required Reinforcing for Flexure per foot length of cap*

4.5.11.4 Reinforcing Required for Axial Tension

(AASHTO LRFD 5.8.4.2.2)

$\Phi = 0.90$ *AASHTO LRFD 5.5.4.2*

Interior Girders:

$N_{uc\_Int} = 0.2 V_{u\_Int}$ $N_{uc\_Int} = 74.4\ \text{kip}$

$A_n = \frac{N_{uc\_Int}}{\Phi f_y}$ $A_n = 1.38\ \text{in}^2$ *Required Reinforcing for Axial Tension*

$a_{n\_Int} = \frac{A_n}{b_{m\_Int}}$ $a_{n\_Int} = 0.19 \frac{\text{in}^2}{\text{ft}}$ *Required Reinforcing for Axial Tension per foot length of cap*

Exterior Girders:

$N_{uc\_Ext} = 0.2 V_{u\_Int}$ $N_{uc\_Ext} = 74.4\ \text{kip}$

$A_n = \frac{N_{uc\_Ext}}{\Phi f_y}$ $A_n = 1.38\ \text{in}^2$ *Required Reinforcing for Axial Tension*

$a_{n\_Ext} = \frac{A_n}{b_{m\_Ext}}$ $a_{n\_Ext} = 0.35 \frac{\text{in}^2}{\text{ft}}$ *Required Reinforcing for Axial Tension per foot length of cap*

4.5.11.5 Minimum Reinforcing *(AASHTO LRFD 5.8.4.2.1)*

$a_{s\_min} = 0.04 \frac{f_c}{f_y} d_e$ $a_{s\_min} = 1.02 \frac{in^2}{ft}$ *Minimum Required Reinforcing*

4.5.11.6 Check Required Reinforcing

Actual Reinforcing:

$a_s = \frac{A_{bar\_M}}{s_{bar\_M}}$ $a_s = 1.24 \frac{in^2}{ft}$ *Primary Ledge Reinforcing Provided*

$a_h = \frac{A_{bar\_N}}{s_{bar\_N}}$ $a_h = 1.24 \frac{in^2}{ft}$ *Auxiliary Ledge Reinforcing Provided*

Checks: $A_s \geq A_{s\_min}$ *(AASHTO LRFD 5.8.4.2.1)*

$A_s \geq A_f + A_n$ *(AASHTO LRFD 5.8.4.2.2)*

$A_s \geq \frac{2A_{vf}}{3} + A_n$ *(AASHTO LRFD Eq. 5.8.4.2.2-5)*

$A_h \geq 0.5(A_s - A_n)$ *(AASHTO LRFD Eq. 5.8.4.2.2-6)*

Check Interior Girders:

Bar M:

Check if: $a_s \geq a_{s\_min}$ *(AASHTO LRFD 5.8.4.2.1)*

$a_s \geq a_{f\_Int} + a_{n\_Int}$ *(AASHTO LRFD 5.8.4.2.2)*

$a_s \geq \frac{2a_{vf\_Int}}{3} + a_{n\_Int}$ *(AASHTO LRFD Eq. 5.8.4.2.2-5)*

$a_s = 1.24 \frac{in^2}{ft}$

$a_{s\_min} = 1.02 \frac{in^2}{ft} < a_s$

$a_{f\_Int} + a_{n\_Int} = 0.67 \frac{in^2}{ft} < a_s$

$\frac{2a_{vf\_Int}}{3} + a_{n\_Int} = 0.82 \frac{in^2}{ft} < a_s$

BarMCheck = “OK!”

Bar N:

Check if: $a_h \geq 0.5 \cdot (a_s - a_{n\_Int})$ *(AASHTO LRFD Eq. 5.8.4.2.2-6)*

$a_s =$ The maximum of:

$a_{f\_Int} + a_{n\_Int}$

$\frac{2a_{vf\_Int}}{3} + a_{n\_Int}$

$a_s = 0.82 \frac{in^2}{ft}$

"a_s" in this equation is the steel required for Bar M, based on the requirements for Bar M in AASHTO LRFD 5.8.4.2.2. This is derived from the suggestion that Ah should not be less than $A_f/2$ nor less than $A_{vf}/3$ (Furlong & Mirza pg. 73 & 74)

$0.5 \cdot (a_s - a_{n\_Int}) = 0.32 \frac{in^2}{ft} < a_h$

BarNCheck = "OK!"

Check Exterior Girders:

Bar M:

Check if: $a_s \geq a_{s\_min}$ *(AASHTO LRFD 5.8.4.2.1)*

$a_s \geq a_{f\_Ext} + a_{n\_Ext}$ *(AASHTO LRFD 5.8.4.2.2)*

$a_s \geq \frac{2a_{vf\_Ext}}{3} + a_{n\_Ext}$ *(AASHTO LRFD Eq. 5.8.4.2.2-5)*

$a_s = 1.24 \frac{in^2}{ft}$

$a_{s\_min} = 1.02 \frac{in^2}{ft} < a_s$

$a_{f\_Ext} + a_{n\_Ext} = 1.23 \frac{in^2}{ft} < a_s$

$\frac{2a_{vf\_Ext}}{3} + a_{n\_Ext} = 1.17 \frac{in^2}{ft} < a_s$

BarMCheck = "OK!"

Bar N:

Check if: $a_h \geq 0.5 \cdot (a_s - a_{n\_Ext})$ *(AASHTO LRFD Eq. 5.8.4.2.2-6)*

$a_s =$ The maximum of:

$a_{f\_Ext} + a_{n\_Ex}$

$\frac{2a_{vf\_Ext}}{3} + a_{n\_Ext}$

$a_s = 1.15 \frac{in^2}{ft}$

"a_s" in this equation is the steel required for Bar M, based on the requirements for Bar M in AASHTO LRFD 5.8.4.2.2. This is derived from the suggestion that Ah should not be less than $A_f/2$ nor less than $A_{vf}/3$ (Furlong & Mirza pg. 73 & 74)

$0.5 \cdot (a_s - a_{n\_Ext}) = 0.42 \frac{in^2}{ft} < a_h$

BarNCheck = "OK!"

Ledge Reinforcement Summary:

Use # 7 primary ledge reinforcing @ 5.80" maximum spacing

& # 7 auxiliary ledge reinforcing @ 5.80" maximum spacing

4.5.12 Hanger Reinforcement (Bars S)

Try Double # 6 Stirrups at a 6.80" spacing.

Use trial and error to determine the spacing needed for the hanger reinforcing.

$s_{bar\_S} = 6.80\ \text{in}$

$A_{hr} = 2\text{stirrups} \cdot A_{bar\_S}$ $\qquad A_{hr} = 0.88\ \text{in}^2$

$A_v = 2\text{legs} \cdot A_{hr}$ $\qquad A_v = 1.76\ \text{in}^2$

4.5.12.1 <u>Check Minimum Transverse Reinforcement</u>

$b_v = b_{stem}$ $\qquad b_v = 42\ \text{in}$

$A_{v\_min} = 0.0316\lambda\sqrt{f_c}\frac{b_v \cdot s_{bar\_S}}{f_y}$ *(AASHTO LRFD Eq. 5.7.2.5-1)*

$\lambda = 1.0$ for normal weight concrete *(AASHTO LRFD 5.4.2.8)*

$A_{v\_min} = 0.34\ \text{in}^2$

$A_v > A_{v\_min}$ $\qquad$ MinimumSteelCheck = "OK!"

4.5.12.2 <u>Check Service Limit State</u>

AASHTO LRFD 5.8.4.3.5 with notifications from BDM-LRFD Ch. 4, Sect. 5, Design Criteria)

<u>Interior Girders</u>

TxDOT uses "2/3 f_y" from the original research (Furlong & Mirza Eq. 5.4) instead of "0.5 f_y" from AASHTO LRFD Eq. 5.8.4.3.5-1. (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)

V_{all} = minimum of:

$$\frac{A_{hr}\cdot\left(\frac{2}{3}f_y\right)}{s_{bar\_S}} \cdot (W + 3a_v) = 249\ \text{kip}$$

Bounded by: $(W + 3a_v) \leq \min(S, 2c)$

$$\frac{A_{hr}\cdot\left(\frac{2}{3}f_y\right)}{s_{bar\_S}} \cdot S = 497\ \text{kip}$$

(BDM-LRFD Ch.4, Sect. 5, Design Criteria modified to limit the distribution width to the girder spacing. This will prevent distribution widths from overlapping)

$V_{all} = 249\ \text{kip}$

$V_{s\_Int} = 243.7\ \text{kip} < V_{all}$ $\qquad$ ServiceCheck = "OK!"

Exterior Girders

TxDOT uses "2/3 f_y" from the original research (Furlong & Mirza Eq. 5.4) instead of "0.5 f_y" from AASHTO LRFD Eq. 5.8.4.3.5-1. (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria)

V_{all} = minimum of:

V_{all} for the Interior Girder

$$\frac{A_{hr}\cdot\left(\frac{2}{3}f_y\right)}{s_{bar\_S}}\cdot\left(\frac{W+3a_v}{2}+c\right)=249\ \text{kip}$$

Bounded by: $(W+3a_v)\le\min(S,2c)$

$$\frac{A_{hr}\cdot\left(\frac{2}{3}f_y\right)}{s_{bar\_S}}\cdot\left(\frac{S}{2}+c\right)=373\ \text{kip}$$

(BDM-LRFD Ch.4, Sect. 5, Design Criteria Modified to limit the distribution width to half the girder spacing and the distance to the edge of the cap. This will prevent distribution widths from overlapping or extending over the edge of the cap.)

$V_{all}=249\ \text{kip}$

$V_{s\_Ext}=243.7\ \text{kip}\ <\ V_{all}$ ServiceCheck = "OK!"

4.5.12.3 Check Strength Limit State

(AASHTO LRFD 5.8.4.3.5)

$\Phi=0.90$ *(AASHTO LRFD Eq. 5.5.4.2)*

Interior Girders:

V_n = minimum of:

$$\frac{A_{hr}\cdot f_y}{s_{bar\_S}}\cdot S=745\ \text{kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-2)*

$$\left(0.063\sqrt{f_c}\cdot b_f\cdot d_f\right)+\frac{A_{hr}\cdot f_y}{s_{bar\_S}}(W+2d_f)=810\ \text{kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-3)*

$V_n=745\ \text{kip}$

$\Phi V_n=670\ \text{kip}$

$V_{u\_Int}=371.8\ \text{kip}\ <\ \Phi V_n$ UltimateCheck = "OK!"

Exterior Girders:

V_n = minimum of:

V_n for the Interior Girder

$$\frac{A_{hr}\cdot f_y}{s_{bar\_S}}\cdot\left(\frac{S}{2}+c\right)=560\ \text{kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-2)*

$$\left(0.063\sqrt{f_c}\cdot b_f\cdot d_f\right)+\frac{A_{hr}\cdot f_y}{s_{bar\_S}}\left(\frac{W+2d_f}{2}+c\right)=808\ \text{kip}$$ *(AASHTO LRFD Eq. 5.8.4.3.5-3)*

(These equations are modified to limit the distribution width to the edge of the cap)

$V_n=560\ \text{kip}$

$\Phi V_n=504\ \text{kip}$

$V_{u\_Ext}=371.8\ \text{kip}\ <\ \Phi V_n$ UltimateCheck = "OK!"

4.5.12.4 Check Combined Shear and Torsion

The following calculations are for Station 36. All critical locations must be checked. See the Concrete Section Shear Capacity spreadsheet in the appendices for calculations at other locations. Shear and Moment were calculated using the CAP 18 program.

$M_u = 769.1\ \text{kip} \cdot \text{ft}$ $V_u = 481.8\ \text{kip}$ $N_u = 0\ \text{kip}$ $T_u = 830\ \text{kip} \cdot \text{ft}$

Recall:

$\beta_1 = 0.80$ $f_y = 60\ \text{ksi}$

$f_c = 5.0\ \text{ksi}$ $E_s = 29000\ \text{ksi}$

$b_f = 92\ \text{in}$ $h_{cap} = 85\ \text{in}$ $b_{ste} = 42\ \text{in}$ $h = 29.50\ \text{in}$

$b_v = b_{stem}$ $b_v = 42\ \text{in}$

Find d_v:

(AASHTO LRFD 5.7.2.8)

Shears are maximum near the column faces. In these regions the cap is in negative bending with tension in the top of the cap. Therefore, the calculations are based on the steel in the top of the bent cap.

$A_s = A_{bar\_A} \cdot \text{BarANo}$ $A_s = 10.92\ \text{in}^2$

$c = \frac{A_s f_y}{0.85\ {}_c\beta_1 b_f}$ $c = 2.10\ \text{in}$

$a = c \cdot \beta_1$ $a = 1.68\ \text{in}$

$d_s = d_{s\_neg}$ $d_s = 81.42\ \text{in}$

$M_n = A_s f_y \left(d_s - \frac{a}{2}\right)$ $M_n = 4400\ \text{kip} \cdot \text{ft}$

$A_{ps} = 0\ \text{in}^2$

$d_e = \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y}$ $d_e = 81.42\ \text{in}$ *(AASHTO LRFD Eq. 5.7.2.8-2)*

$d_v =$ maximum of:

$$\frac{M_n}{A_s f_y + A_{ps} f_{ps}} = 80.59\ \text{in}$$

$$0.9 d_e = 73.28\ \text{in}$$

$$0.72 h = 21.24\ \text{in}$$

$d_v = 80.59\ \text{in}$

The method for calculating θ and β used in this design example are from AASHTO LRFD Appendix B5. The method from AASHTO LRFD 5.7.3.4.2 may be used instead. The method from 5.7.3.4.2 is based on the method from Appendix B5; however, it is less accurate and more conservative (often excessively conservative). The method from Appendix B5 is preferred because it is more accurate, but it requires iterating to a solution.

Determine θ and β:

$\Phi_V = 0.90$ *(AASHTO LRFD Eq. 5.5.4.2)*

$v_u = \frac{|V_u-(\Phi_V \cdot V_p)|}{\Phi_V \cdot b_v \cdot d_v}$ $v_u = 0.16\ \text{ksi}$ *Shear Stress on the Concrete (AASHTO LRFD Eq. 5.7.2.8-1)*

$\frac{v_u}{f_c} = 0.03$

Using Table B5.2-1 with $\frac{v_u}{f_c} = 0.03$ and $\varepsilon_x = 0.001$

$\theta = 36.4\ \text{deg}$ and $\beta = 2.23$

Determining θ and β is an iterative process, therefore, assume initial shear strain value ε_x of 0.001 per LRFD B5.2 and then verify that the assumption was valid.

$\varepsilon_x = \frac{\frac{|M_u|}{d_v}+0.5N_u+0.5|V_u-V_p|\cot\theta-A_{ps}f_{po}}{2(E_sA_s+E_pA_{ps})}$

where $|M_u| = 769.1\ \text{kip}\cdot\text{ft}$ must be > $|V_u - V_p|d_v = 3236\ \text{kip}\cdot\text{ft}$

$\varepsilon_x = 1.23 \times 10^{-3} > 1.00 \times 10^{-3}$

use $\varepsilon_x = 1.00 \times 10^{-3}$.

Strain halfway between the compressive and tensile resultants (AASHTO LRFD Eq. B5.2-3) If ε_x< 0, then use equation B5.2-5 and re-solve for ε_x.

For values of ε_x greater than 0.001, the tensile strain in the reinforcing, $\varepsilon_t$is greater than 0.002. (ε_t= 2ε_x - ε_c, where ε_c is < 0) Grade 60 steel yields at a strain of 60 ksi / 29,000 ksi = 0.002. By limiting the tensile strain in the steel to the yield strain and using the Modulus of Elasticity of the steel prior to yield, this limits the tensile stress of the steel to the yield stress. $\varepsilon_x$has not changed from the assumed

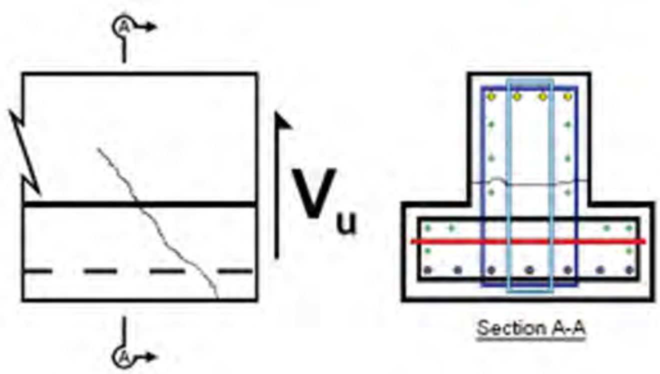
$V_p = 0\ \text{kip}$ *"V_p" is zero as there is no prestressing.*

$A_c = b_{stem} \cdot \frac{h_{cap}}{2}$ $A_c = 1785\ \text{in}^2$

$s = s_{bar\_S}$ $s = 6.80\ \text{in}$

(AASHTO LRFD B5.2) "A_c" is the area of concrete on the flexural tension side of the cap, from the extreme tension fiber to one half the cap depth.

"A_c" is needed if AASHTO LRFD Eq. B5.2-3 is negative.



The transverse reinforcement, "A_v", is double closed stirrups. The failure surface intersects four stirrup legs, therefore the area of the shear steel is four times the stirrup bar's area (0.44in2). See the sketch of the failure plane to the left.

Figure 4.99 Failure Surface of 60 Degrees Skewed ITBC for Combined Shear and Torsion

$A_v = 2legs \cdot 2stirrups \cdot A_{bar\_S}$ $\qquad A_v = 1.76\ in^2$

$A_t = 1leg \cdot A_{bar\_S}$ $\qquad A_t = 0.44\ in^2$

$A_{oh} = (d_{stem}) \cdot (b_{stem} - 2cover) + \left(d_{ledge} - 2cover\right) \cdot (b_f - 2cover)$

$A_{oh} = 4110\ in^2$

$A_0 = 0.85A_{oh}$ $\qquad A_0 = 3493.5in^2$

$p_h = (b_{stem} - 2cover) + 2\left(b_{ledge}\right) + (b_f - 2cover) + 2\left(h_{cap} - 2cover\right)$

$p_h = 334\ in$

<u>Equivalent Shear Force</u>

$$V_{u\_Eq} = \sqrt{V_u^2 + \left(\frac{0.9p_hT_u}{2A_0}\right)^2} \qquad V_{u\_Eq} = 624.3\ kip$$ *(AASHTO LRFD Eq. B.5.2-1)*

<u>Shear Steel Required</u>

V_n = the lesser of:

$V_c + V_s + V_p$ *(AASHTO LRFD Eq. 5.7.3.3-1)*

$0.25 \cdot f_c \cdot b_v \cdot d_v + V_p$ *(AASHTO LRFD Eq. 5.7.3.3-2)*

Check maximum ΦV_n for section:

$\Phi V_{n\_max} = \Phi \cdot \left(0.25 \cdot f_c \cdot b_v \cdot d_v + V_p\right)$

$\Phi V_{n\_max} = 3808\ kip$

$V_u = 481.8\ kip < \Phi V_{n\_max}$ MaxShearCheck = “OK!”

Calculate required shear steel:

$V_u < \Phi V_n$ *(AASHTO LRFD Eq. 1.3.2.1-1)*

$V_c = 0.0316 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v$ $V_c = 533\ \text{kip}$ *(AASHTO LRFD Eq. 5.7.3.3-3)*

$V_u < \Phi_V \cdot (V_c + V_s + V_p)$

$V_s = \frac{A_v \cdot f_y \cdot d_v \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha}{s_{req}}$ *(AASHTO LRFD Eq. 5.7.3.3-4)*

$a_{v\_req} = \frac{\frac{V_u}{\Phi_V} - V_c - V_p}{f_y \cdot d_v \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha}$ $a_{v\_req} = 0.004 \frac{in^2}{ft}$

Torsional Steel Required

$\Phi_T = 0.9$ *(AASHTO LRFD 5.5.4.2)*

$T_u \leq \Phi_T T_n$ *(AASHTO LRFD Eq. 1.3.2.1-1)*

$T_n = \frac{2A_o A_t f_y \cot\theta}{s_{bar\_S}}$ *(AASHTO LRFD Eq. 5.7.3.6.2-1)*

$a_{t\_req} = \frac{T_u}{\Phi_T 2A_o f_y \cot\theta}$ $a_{t\_req} = 0.23 \frac{in^2}{ft}$

Total Required Transverse Steel

$a_{req} = a_{v\_req} + 2sides \cdot a_{t\_req}$ $a_{req} = 0.46 \frac{in^2}{ft}$

$a_{prov} = \frac{A_v}{s_{bar\_S}}$ $a_{prov} = 3.10 \frac{in^2}{ft}$

The transverse reinforcement is designed for the side of the section where the effects of shear and torsion are additive. (AASHTO LRFD C5.7.3.6.1)

$a_{prov} > a_{req}$ TransverseSteelCheck = “OK!”

Longitudinal Reinforcement

$$A_{ps}f_{ps} + A_s f_y \geq \frac{|M_u|}{\Phi d_v} + \frac{0.5N_u}{\Phi} + \cdots$$ *(AASHTO LRFD Eq. 5.7.3.6.3-1)*

$$\cot\theta\sqrt{\left(\left|\frac{V_u}{\Phi} - V_p\right| - 0.5V_s\right)^2 + \left(\frac{0.45 p_h T_u}{2A_0 \Phi}\right)^2}$$

$V_s = a_{t\_req} \cdot f_y \cdot d_v \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha$ *(AASHTO LRFD Eq. 5.7.3.3-4)*

Bounded By: $V_s < \frac{V_u}{\Phi_V}$ $V_s = 535.3\ \text{kip}$ *(AASHTO LRFD Eq. 5.7.3.5-1)*

$$\frac{|M_u|}{\Phi_f d_v} + \frac{0.5N_u}{\Phi_c} + \cot\theta\sqrt{\left(\left|\frac{V_u}{\Phi_V} - V_p\right| - 0.5V_s\right)^2 + \left(\frac{0.45\ \ _h T_u}{2A_0 \Phi_T}\right)^2} = 614\ \text{kip}$$

Provided Force:

$A_s f_y = 655.2\ \text{kip} > 614\ \text{kip}$ LongitudinalReinfChk = “OK!”

4.5.12.5 Maximum Spacing of Transverse Reinforcement *(AASHTO LRFD 5.7.2.6)*

Shear Stress

$v_u = \frac{|V_u - \Phi_V V_p|}{\Phi_V b_v d_v}$ $v_u = 0.16$ ksi *(AASHTO LRFD Eq. 5.7.2.8-1)*

$0.125 \cdot f_c = 0.625$ ksi

If $v_u < 0.125 \cdot f_c$ *(AASHTO LRFD Eq. 5.7.2.6-1)*

$s_{max} = \min(0.8 d_v, 24in)$

If $v_u \geq 0.125 \cdot f_c$ *(AASHTO LRFD Eq. 5.7.2.6-2)*

$s_{max} = \min(0.4 d_v, 12in)$

Since $v_u < 0.125 \cdot f_c$ $s_{max} = 24.00$ in

TxDOT limits the maximum transverse reinforcement spacing to 12”. *(BDM-LRFD, Ch. 4, Sect. 5, Detailing)*

$s_{max} = 12.00$ in

$s_{bar\_S} = 6.80$ in $< s_{max}$ SpacingCheck= “OK!”

Hanger Reinforcement Summary:

Use double # 6 stirrups @ 6.80” maximum spacing

4.5.13 End Reinforcements (Bars U1, U2, U3, and G)

Extra vertical, horizontal, and diagonal reinforcing at the end surfaces is provided to reduce the maximum crack widths. According to the parametric analysis, it is recommended to place #6 U1 Bars, U2 Bars, and U3 Bars at the end faces and #7 G Bars at approximately 6in. spacing at the first 30" to 35" of the end of bent cap. U1 Bars are the vertical end reinforcements, U2 Bars and U3 Bars are the horizontal end reinforcements at the stem and the ledge, respectively. G Bars are the diagonal end reinforcement.

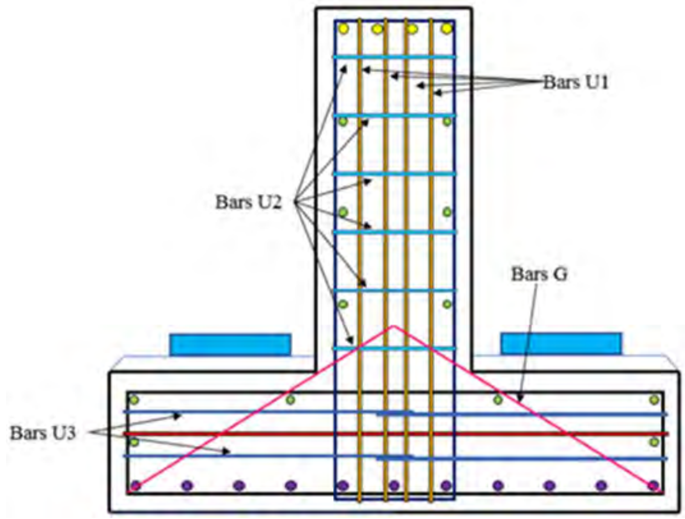


Figure 4.100 End Face Section View of 60 Degrees ITBC

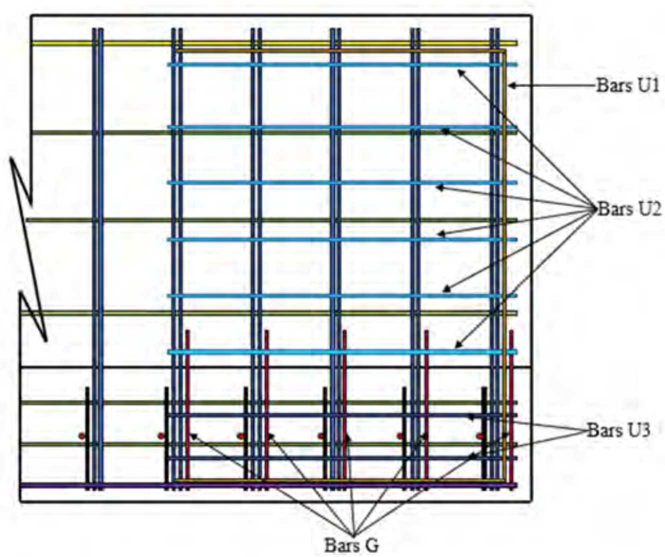


Figure 4.101 End Face Elevation View of 60 Degrees ITBC

4.5.14 Skin Reinforcement (Bars T)

Try 7 ~ # 6 bars in Stem and 3 ~ # 6 bars in Ledge on each side

$A_{bar\_T} = 0.44\ in^2$

$NoTBarsStem = 7$

$NoTBarsLedge = 3$

"a" must be within $\frac{2}{3}d_e$.

(AASHTO LRFD 5.13.2.4.1)

$\frac{2}{3}d_e = 17.00\ in$

TxDOT typically uses: $a = 6\ in$

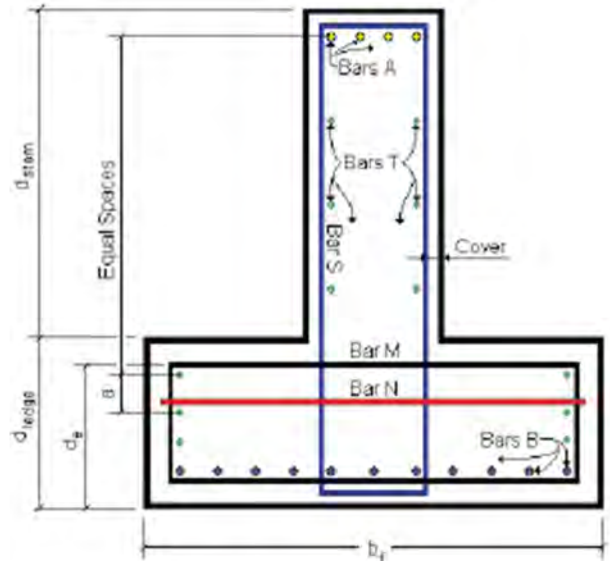


Figure 4.102 Section View for T Bars of 60 Degrees Skewed ITBC

4.5.14.1 Required Area of Skin Reinforcement

(AASHTO LRFD 5.6.7)

(AASHTO LRFD Eq. 5.6.7-3)

$A_{sk\_Req} = 0.012 \cdot (d - 30)$ $\qquad A_{sk\_Req} = 0.62 \frac{in^2}{ft}$

A_{sk} need not be greater than one quarter of the main reinforcing ($A_s/4$)per side face within $d/2$ of the main reinforcing. (AASHTO LRFD 5.6.7)

"d" is the distance from the extreme compression fiber to the centroid of the extreme tension steel element. In this example design, $d = d_{s\_pos} = 81.36$ in.

$$A_{sk\_max} = \max\left(\frac{\frac{A_{bar\_A} \cdot BarANo}{4}}{\frac{d_{s\_neg}}{2}}, \frac{\frac{A_{bar\_B} \cdot BarBNo}{4}}{\frac{d_{s\_pos}}{2}}\right)$$

$$A_{sk\_max} = 1.27 \frac{in^2}{ft}$$

$$A_{skReq} = \min(A_{sk\_Req}, A_{sk\_max})$$

$$A_{skReq} = 0.62 \frac{in^2}{ft}$$

4.5.14.2 Required Spacing of Skin Reinforcement

(AASHTO LRFD 5.6.7)

s_{req} = minimum of:

$\frac{A_{bar\_T}}{A_{skReq}} = 8.52\ in$

$\frac{d_{s\_neg}}{6} = 13.57\ in$

$\frac{d_{s\_pos}}{6} = 13.56\ \text{in}$

& 12 in

$s_{req} = 8.52\ \text{in}$

4.5.14.3 Actual Spacing of Skin Reinforcement

Check T Bars spacing in Stem:

$$h_{top} = d_{stem} - \left(\text{cover} + \frac{d_{bar\_S}}{2} + \frac{d_{bar\_A}}{2}\right) + \left(\text{cover} + \frac{d_{bar\_M}}{2} + \frac{d_{bar\_T}}{2}\right)$$

$h_{top} = 56.73\ \text{in}$

$$s_{skStem} = \frac{h_{top}}{\text{NoTBarsStem+}}$$

$s_{skStem} = 7.09\ \text{in}$

$s_{skStem} < s_{req}$ SkinSpacing = "OK!"

Check T Bars spacing in Ledge:

$$h_{bot} = d_{ledge} - \left(\text{cover} + \frac{d_{bar\_M}}{2} + \frac{d_{bar\_T}}{2}\right) - \left(\text{cover} + \frac{d_{bar\_S}}{2} + \frac{d_{bar\_B}}{2}\right)$$

$h_{bot} = 21.11\ \text{in}$

$$s_{skLedge} = \frac{h_{bot} - a}{\text{NoTBarsLedge}}$$

$s_{skLedge} = 7.56\ \text{in}$

$s_{skLedge} < s_{req}$ SkinSpacing = "OK!"

Check if "a" is less than s_{req}

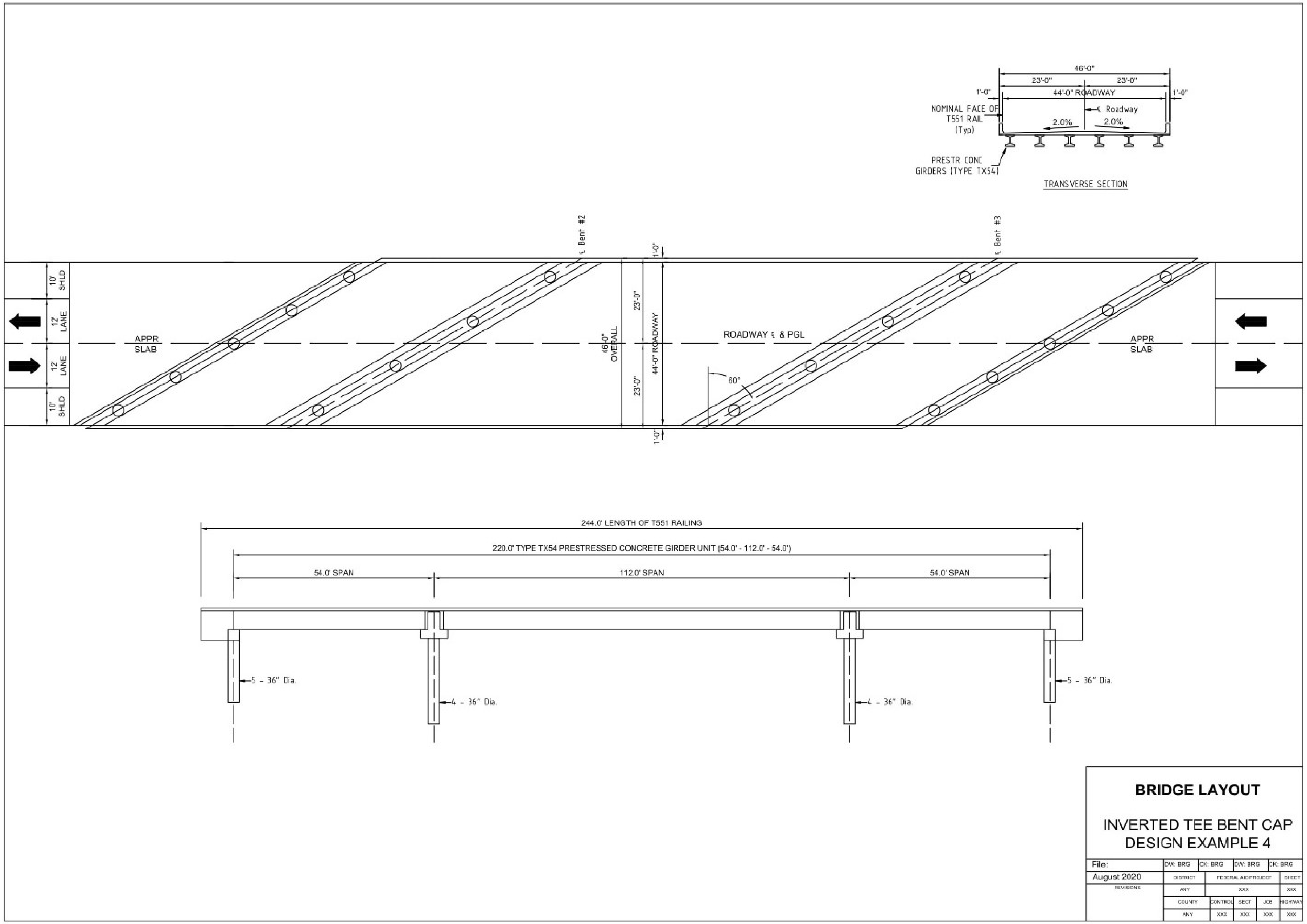
$a = 6\ \text{in} < s_{req}$ SkinSpacing = "OK!"

Skin Reinforcement Summary:

Use 7 ~ # 6 bars in Stem and 3 ~ # 6 bars in Ledge on each side

4.5.15 Design Details and Drawings

4.5.15.1 Bridge Layout



4.5.15.2 AP 18 Input File

```
$File                        Proj              User  Date (Today
$ Num     County      Highway  Num     CSJ      Init     if Blank) Comment
$XXXX  XXXXXXXXXXXXX  XXXXXX  XXXX  XXXX-XX-XXX  XXX  XXXXXXXXXXXX XXXXXXX
00001  ___County____  Highwy  Pro#  0000-00-000  BRG              Comment
$Header Card 2 ----------------------------------------------------------------
$XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
CAP18 Version 6.00 ITBC Design Example 4, Skew = 60.00
$Problem Card -----------------------------------------------------------------
$Prob E   XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
    1 E 0 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay)
$TABLE 1 - CONTROL DATA -------------------------------------------------------
$                Enter 1 to keep:       Number cards   Options:
$                 Env Tab2 Tab3 Tab4     on Table 4  Envelope  Print   Skew Angle
$                  X    X    X    X              XX    X         XX    XXXXXXXXXX
                                                 16                         60.0
$TABLE 2 - CONSTANTS ----------------------------------------------------------
$   TABLE 2a                                                Anly Opt (1=Working,
$                                 |-Movable Load Data--|  2=Load Factor,3=Both)
$                Num  Increment    |Num  Start Stop Step|Anly|  Load Factors:
$                Inc   Length      |Inc  Sta    Sta  Size| Opt| Dead      Live
$                 XX XXXXXXXXX       XXX  XXX  XXX    X    X XXXXXXXX XXXXXXXX
                  92     0.5          20    2   70    1    3    1.25     1.75
$   TABLE 2b
$    Overlay    Max #|-----------Live Load Reduction Factors---------|
$  Load Factor  Lanes| 1 lane   2 lanes   3 lanes   4 lanes   5 lanes
$    XXXXX          X  XXXX      XXXX      XXXX      XXXX      XXXX
     1.50           3  1.2       1.0       0.85      0.65      0.65
$TABLE 3 - LIST OF STATIONS ---------------------------------------------------
$     Number of input values for              Str - Stringers, Sup - Supports
$                Lane Str  Sup  MCP  VCP        MCP - Moment Control Points
$                 XX   XX   XX   XX   XX        VCP - Shear Control Points
  (Num Inputs)     3    6    4   11    8
$     Left Lane Boundary Stations
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Lane Left)      2   32   60
$     Right Lane Boundary Stations
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Lane Right)    32   60   90
$     Station of Stringers (two rows max, may be at tenths of stations, XX.X)
$               XXXX XXXX XXXX XXXX XXXX XXXX XXXX XXXX XXXX XXXX
  (Stringers)      6   22   38   54   70   86
$     Station of Supports (two rows max)
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Supports)      10   34   58   82
$     Moment Control Point Stations (two rows max)
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Mom CP)         6   10   22   34   38   46   54   58   70   82
  (Mom CP)        86
$     Shear Control Point Stations (two rows max)
$                XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX  XXX
  (Shear CP)       8   12   32   36   56   60   80   84
$TABLE 4 - STIFFNESS AND LOAD DATA --------------------------------------------
$                              Bending   Sidewalk,  Cap &
$                Station  1 if  Stiffness  Slab      Stringer  Moving    Overlay
$Comments       From  To Cont'd  of Cap    Loads      Loads     Loads    Loads,DW
$XXXXXXXXXXXXXX  XXX  XXX    X XXXXXXXXX XXXXXXXXX XXXXXXXXX XXXXXXXXX XXXXXXXXX
(CAP EI & DL)      2   90       8.66E+07            -2.589
(DL Span1, Bm1)    6    6                           -50.17              -5.04
(DL Span1, Bm2)   22   22                           -50.17              -5.04
(DL Span1, Bm3)   38   38                           -50.17              -5.04
(DL Span1, Bm4)   54   54                           -50.17              -5.04
(DL Span1, Bm5)   70   70                           -50.17              -5.04
(DL Span1, Bm6)   86   86                           -50.17              -5.04
(DL Span2, Bm1)    6    6                           -104.1              -10.5
(DL Span2, Bm2)   22   22                           -104.1              -10.5
(DL Span2, Bm3)   38   38                           -104.1              -10.5
(DL Span2, Bm4)   54   54                           -104.1              -10.5
(DL Span2, Bm5)   70   70                           -104.1              -10.5
(DL Span2, Bm6)   86   86                           -104.1              -10.5
(Dist. Lane Ld)    0   20                                     -4.92
(Conc. Lane Ld)    4    4                                     -21.3
(Conc. Lane Ld)   16   16                                     -21.3
```

4.5.15.3 CAP 18 Output File

```
AUG 12, 2020     TEXAS DEPARTMENT OF TRANSPORTATION (TxDOT)        PAGE  1
CAP18                   BENT CAP ANALYSIS       Ver.  6.2  (Jul, 2011)

   PSF             HIGHWAY  PD-    CONTROL-   CODED
    NO     COUNTY       NO   IPE  SECTION-JOB  BY    DATE
  00001  ___County____  Highwy  Pro#  0000-00-000  BRG  AUG 12, 2020 Comment

CAP18 Version 6.00 ITBC Design Example 4, Skew = 60.00
PROB     1   (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay


ENGLISH SYSTEM UNITS

TABLE 1.  CONTROL DATA

   OPTION TO PRINT TABLE SRS (1=YES)                          0

                                   ENVELOPES    TABLE NUMBER
                                   OF MAXIMUMS   2   3   4
   KEEP FROM PRECEDING PROBLEM (1=YES)             0      0   0   0
   CARDS INPUT THIS PROBLEM                             16

   OPTION TO CLEAR ENVELOPES BEFORE LANE LOADINGS (1=YES)             0

   OPTION TO OMIT PRINT FOR TABLES (TABLE DESIGNATIONS IN PARENTHESES)
     -1(4A), -2(5) -3(4A,5), -4(4A,5,6), -5(4A,5,6,7):            0

   SKEW ANGLE, DEGREES                              60.000


TABLE 2.  CONSTANTS

   NUMBER OF INCREMENTS FOR SLAB AND CAP                         92
   INCREMENT LENGTH, FT                             0.500
   NUMBER OF INCREMENTS FOR MOVABLE LOAD                          20
   START POSITION OF MOVABLE-LOAD STA ZERO                        2
   STOP POSITION OF MOVABLE-LOAD STA ZERO                        70
   NUMBER OF INCREMENTS BETWEEN EACH POSITION OF MOVABLE LOAD          1

   ANALYSIS OPTION (1=WORKING STRESS, 2=LOAD FACTOR, 3=BOTH)           3

   LOAD FACTOR FOR DEAD LOAD                             1.25
   LOAD FACTOR FOR OVERLAY LOAD                            1.50
   LOAD FACTOR FOR LIVE LOAD                            1.75

   MAXIMUM NUMBER OF LANES TO BE LOADED SIMULTANEOUSLY             3

   LIST OF LOAD COEFFICIENTS CORRESPONDING TO NUMBER OF LANES LOADED
       1           2           3           4           5
     1.200       1.000       0.850
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 3. LISTS OF STATIONS

	NUM OF LANES	NUM OF STRINGERS	NUM OF SUPPORTS	NUM MOM CONTR PTS	NUM SHEAR CONTR PTS
TOTAL	3	6	4	11	8

LANE LEFT 2 32 60

LANE RIGHT 32 60 90
STRINGERS 6.0 22.0 38.0 54.0 70.0 86.0
SUPPORTS 10 34 58 82
MOM CONTR 6 10 22 34 38 46 54 58 70 82
86
SHEAR CONTR 8 12 32 36 56 60 80 84

TABLE 4. STIFFNESS AND LOAD DATA

STA FROM	STA TO	CONTD IF=1	CAP BENDING STIFFNESS (K-FT*FT)	SIDEWALK, SLAB LOADS (K)	STRINGER, CAP LOADS (K)	OVERLAY LOADS (K)	MOVABLE-POSITION SLAB LOADS (K)
			FIXED-OR-MOVABLE ------------ FIXED-POSITION DATA ------------				
2	90	0	86600000.000	0.000	-2.589	0.000	0.000
6	6	0	0.000	0.000	-50.170	-5.040	0.000
22	22	0	0.000	0.000	-50.170	-5.040	0.000
38	38	0	0.000	0.000	-50.170	-5.040	0.000
54	54	0	0.000	0.000	-50.170	-5.040	0.000
70	70	0	0.000	0.000	-50.170	-5.040	0.000
86	86	0	0.000	0.000	-50.170	-5.040	0.000
6	6	0	0.000	0.000	-104.100	-10.500	0.000
22	22	0	0.000	0.000	-104.100	-10.500	0.000
38	38	0	0.000	0.000	-104.100	-10.500	0.000
54	54	0	0.000	0.000	-104.100	-10.500	0.000
70	70	0	0.000	0.000	-104.100	-10.500	0.000
86	86	0	0.000	0.000	-104.100	-10.500	0.000
0	20	0	0.000	0.000	0.000	0.000	-4.920
4	4	0	0.000	0.000	0.000	0.000	-21.300
16	16	0	0.000	0.000	0.000	0.000	-21.300

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 4A. DEAD LOAD RESULTS (WORKING STRESS)

STA	DIST X (FT)	DEFLECTION (FT)	MOMENT (K-FT)	SHEAR (K)
-1	-1.00	0.000000	0.0	0.0
0	0.00	0.000000	0.0	0.0
1	1.00	-0.000237	0.0	0.0
2	2.00	-0.000208	0.0	-1.3
3	3.00	-0.000178	-2.6	-5.2
4	4.00	-0.000148	-10.4	-10.4
5	5.00	-0.000119	-23.3	-15.5
6	6.00	-0.000090	-41.4	-105.6
7	7.00	-0.000061	-234.5	-195.7
8	8.00	-0.000035	-432.8	-200.9
9	9.00	-0.000014	-636.3	-206.1
10	10.00	0.000000	-844.9	-30.6
11	11.00	0.000004	-697.5	144.8
12	12.00	0.000000	-555.3	139.6
13	13.00	-0.000011	-418.3	134.5
14	14.00	-0.000026	-286.4	129.3
15	15.00	-0.000045	-159.7	124.1
16	16.00	-0.000065	-38.2	118.9
17	17.00	-0.000086	78.2	113.7
18	18.00	-0.000106	189.3	108.6
19	19.00	-0.000124	295.3	103.4
20	20.00	-0.000138	396.1	98.2
21	21.00	-0.000148	491.7	93.0
22	22.00	-0.000152	582.2	3.0
23	23.00	-0.000150	497.6	-87.1
24	24.00	-0.000141	407.9	-92.3
25	25.00	-0.000128	313.0	-97.5
26	26.00	-0.000112	213.0	-102.7
27	27.00	-0.000093	107.7	-107.8
28	28.00	-0.000072	-2.7	-113.0
29	29.00	-0.000052	-118.3	-118.2
30	30.00	-0.000033	-239.1	-123.4
31	31.00	-0.000017	-365.1	-128.6
32	32.00	-0.000005	-496.2	-133.7
33	33.00	0.000001	-632.6	-138.9
34	34.00	0.000000	-774.1	43.9
35	35.00	-0.000010	-544.7	226.8
36	36.00	-0.000026	-320.5	221.6
37	37.00	-0.000046	-101.5	216.4
38	38.00	-0.000068	112.3	126.3
39	39.00	-0.000087	151.1	36.2
40	40.00	-0.000106	184.8	31.1
41	41.00	-0.000122	213.3	25.9
42	42.00	-0.000135	236.6	20.7
43	43.00	-0.000146	254.7	15.5

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 4A. DEAD LOAD RESULTS (WORKING STRESS)

STA	DIST X (FT)	DEFLECTION (FT)	MOMENT (K-FT)	SHEAR (K)
44	44.00	-0.000154	267.6	10.4
45	45.00	-0.000159	275.4	5.2
46	46.00	-0.000160	278.0	0.0
47	47.00	-0.000159	275.4	-5.2
48	48.00	-0.000154	267.6	-10.4
49	49.00	-0.000146	254.7	-15.5
50	50.00	-0.000135	236.6	-20.7
51	51.00	-0.000122	213.3	-25.9
52	52.00	-0.000106	184.8	-31.1
53	53.00	-0.000087	151.1	-36.2
54	54.00	-0.000068	112.3	-126.3
55	55.00	-0.000046	-101.5	-216.4
56	56.00	-0.000026	-320.5	-221.6
57	57.00	-0.000010	-544.7	-226.8
58	58.00	0.000000	-774.1	-43.9
59	59.00	0.000001	-632.6	138.9
60	60.00	-0.000005	-496.2	133.7
61	61.00	-0.000017	-365.1	128.6
62	62.00	-0.000033	-239.1	123.4
63	63.00	-0.000052	-118.3	118.2
64	64.00	-0.000072	-2.7	113.0
65	65.00	-0.000093	107.7	107.8
66	66.00	-0.000112	213.0	102.7
67	67.00	-0.000128	313.0	97.5
68	68.00	-0.000141	407.9	92.3
69	69.00	-0.000150	497.6	87.1
70	70.00	-0.000152	582.2	-3.0
71	71.00	-0.000148	491.7	-93.0
72	72.00	-0.000138	396.1	-98.2
73	73.00	-0.000124	295.3	-103.4
74	74.00	-0.000106	189.3	-108.6
75	75.00	-0.000086	78.2	-113.7
76	76.00	-0.000065	-38.2	-118.9
77	77.00	-0.000045	-159.7	-124.1
78	78.00	-0.000026	-286.4	-129.3
79	79.00	-0.000011	-418.3	-134.5
80	80.00	0.000000	-555.3	-139.6
81	81.00	0.000004	-697.5	-144.8
82	82.00	0.000000	-844.9	30.6
83	83.00	-0.000014	-636.3	206.1
84	84.00	-0.000035	-432.8	200.9
85	85.00	-0.000061	-234.5	195.7
86	86.00	-0.000090	-41.4	105.6
87	87.00	-0.000119	-23.3	15.5
88	88.00	-0.000148	-10.4	10.4
89	89.00	-0.000178	-2.6	5.2
90	90.00	-0.000208	0.0	1.3

```
  91     91.00     -0.000237      0.0       0.0
  92     92.00      0.000000      0.0       0.0
  93     93.00      0.000000      0.0       0.0
```


PROB   1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

```
TABLE 5.  MULTI-LANE LOADING SUMMARY    ( WORKING STRESS )
       ( *--CRITICAL NUMBER OF LANE LOADS)

   MOMENT ( FT-K )
   -----------------------------------------------------------------------
   AT    DEAD LD   LANE   POSITIVE   LOAD AT    LANE   NEGATIVE   LOAD AT
   STA    EFFECT   ORDER  MAXIMUM    LANE STA   ORDER  MAXIMUM    LANE STA
   -----------------------------------------------------------------------

    6    -41.4
            0      0.0             0      0.0
            1      0.0             1      0.0
            2      0.0             2      0.0
            3      0.0             3      0.0
            0*                     0*

   10   -844.9
            0      0.0             0    -352.5     1   2
            1      0.0             1    -352.5     1   2
            2      0.0             2      0.0
            3      0.0             3      0.0
            0*                     0*

   22    582.2
            0    404.0    0  13    0     -66.7     2  36
            1    402.4    1  12    1     -66.7     2  36
            2     18.7    3  62    2      0.0
            3      0.0             3      0.0
            0*                     0*

   34   -774.1
            0     37.4    3  62    0    -272.6     0  18
            1     37.4    3  62    1    -233.1     1  12
            2      0.0             2    -169.4     2  32
            3      0.0             3      0.0
            0*                     2*

   38    112.3
            0    167.2    2  32    0    -117.6     1   9
            1    167.2    2  32    1    -117.6     1   9
            2      6.4    3  62    2      0.0
            3      0.0             3      0.0
            0*                     0*

   46    278.0
            0    138.7    2  36    0     -55.6     1   9
            1    138.7    2  36    1     -55.6     1   9
            2      0.0             2     -55.6     3  63
            3      0.0             3      0.0
            0*                     2*

   54    112.3
            0    167.2    2  40    0    -117.6     3  63
            1    167.2    2  40    1    -117.6     3  63
            2      6.4    1  10    2      0.0
            3      0.0             3      0.0
            0*                     0*

   58   -774.1
            0     37.4    1   9    0    -272.6     0  54
            1     37.4    1   9    1    -233.1     3  60
            2      0.0             2    -169.4     2  40
            3      0.0             3      0.0
            0*                     2*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

MOMENT (FT-K)

AT STA	DEAD LD EFFECT	LANE ORDER	POSITIVE MAXIMUM	LOAD AT LANE STA	LANE ORDER	NEGATIVE MAXIMUM	LOAD AT LANE STA
70	582.2						
		0	404.0	0 59	0	-66.7	2 36
		1	402.4	3 60	1	-66.7	2 36
		2	18.7	1 9	2	0.0	
		3	0.0		3	0.0	
		0*			0*		
82	-844.9						
		0	0.0		0	-352.5	3 70
		1	0.0		1	-352.5	3 70
		2	0.0		2	0.0	
		3	0.0		3	0.0	
		0*			0*		
86	-41.4						
		0	0.0		0	0.0	
		1	0.0		1	0.0	
		2	0.0		2	0.0	
		3	0.0		3	0.0	
		0*			0*		

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

SHEAR (K)

AT STA	DEAD LD EFFECT	LANE ORDER	POSITIVE MAXIMUM	LOAD AT LANE STA	LANE ORDER	NEGATIVE MAXIMUM	LOAD AT LANE STA
8	-200.9						
		0	0.0		0	-88.1	1 2
		1	0.0		1	-88.1	1 2
		2	0.0		2	0.0	
		3	0.0		3	0.0	
		0*			0*		
12	139.6						
		0	44.8	1 6	0	-5.6	2 36
		1	44.8	1 6	1	-5.6	2 36
		2	1.6	3 62	2	0.0	
		3	0.0		3	0.0	
		0*			0*		
32	-133.7						
		0	1.6	3 62	0	-54.6	0 15
		1	1.6	3 62	1	-53.0	1 12
		2	0.0		2	-11.2	2 32
		3	0.0		3	0.0	
		0*			0*		
36	221.6						
		0	87.6	0 28	0	-7.8	3 63
		1	84.1	2 32	1	-7.8	3 63
		2	30.7	1 12	2	0.0	
		3	0.0		3	0.0	
		2*			0*		
56	-221.6						
		0	7.8	1 9	0	-87.6	0 44
		1	7.8	1 9	1	-84.1	2 40
		2	0.0		2	-30.7	3 60
		3	0.0		3	0.0	
		0*			2*		
60	133.7						
		0	54.6	0 57	0	-1.6	1 9
		1	53.0	3 60	1	-1.6	1 9
		2	11.2	2 40	2	0.0	
		3	0.0		3	0.0	
		0*			0*		
80	-139.6						
		0	5.6	2 36	0	-44.8	3 66
		1	5.6	2 36	1	-44.8	3 66
		2	0.0		2	-1.6	1 9
		3	0.0		3	0.0	
		0*			0*		
84	200.9						
		0	88.1	3 70	0	0.0	
		1	88.1	3 70	1	0.0	
		2	0.0		2	0.0	
		3	0.0		3	0.0	
		0*			0*		

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

```
  REACTION ( K )
  ------------------------------------------------------------------------
  AT    DEAD LD  LANE   POSITIVE   LOAD AT   LANE   NEGATIVE   LOAD AT
  STA    EFFECT  ORDER  MAXIMUM    LANE STA  ORDER  MAXIMUM    LANE STA
  ------------------------------------------------------------------------

   10    361.2
            0    127.9    1  2    0    -5.6    2 36
            1    127.9    1  2    1    -5.6    2 36
            2      1.6    3 62    2     0.0
            3      0.0            3     0.0
            0*                    0*

   34    376.0
            0    117.1    0 22    0    -9.3    3 63
            1     95.3    2 32    1    -9.3    3 63
            2     83.6    1 12    2     0.0
            3      0.0            3     0.0
            2*                    0*

   58    376.0
            0    117.1    0 50    0    -9.3    1  9
            1     95.3    2 40    1    -9.3    1  9
            2     83.6    3 60    2     0.0
            3      0.0            3     0.0
            2*                    0*

   82    361.2
            0    127.9    3 70    0    -5.6    2 36
            1    127.9    3 70    1    -5.6    2 36
            2      1.6    1  9    2     0.0
            3      0.0            3     0.0
            0*                    0*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
-1	-1.00	0.0	0.0	0.0	0.0
0	0.00	0.0	0.0	0.0	0.0
1	1.00	0.0	0.0	0.0	0.0
2	2.00	0.0	0.0	-1.3	-1.3
3	3.00	-2.6	-2.6	-5.2	-5.2
4	4.00	-10.4	-10.4	-10.4	-10.4
5	5.00	-23.3	-23.3	-15.5	-15.5
6	6.00	-41.4	-41.4	-105.6	-158.5
7	7.00	-234.5	-340.3	-195.7	-301.4
8	8.00	-432.8	-644.3	-200.9	-306.6
9	9.00	-636.3	-953.5	-206.1	-311.8
10	10.00	-844.9	-1267.9	-13.7	-59.6
11	11.00	-671.3	-1072.8	198.6	138.1
12	12.00	-493.5	-882.9	193.4	133.0
13	13.00	-319.7	-698.2	188.3	127.8
14	14.00	-151.1	-518.6	183.1	122.6
15	15.00	13.0	-344.2	177.9	117.4
16	16.00	174.5	-175.0	172.7	112.3
17	17.00	332.8	-10.9	167.5	107.1
18	18.00	488.0	135.9	162.4	101.9
19	19.00	638.9	235.3	157.2	96.7
20	20.00	786.0	329.4	152.0	91.5
21	21.00	928.5	418.3	146.8	86.4
22	22.00	1067.0	502.1	18.9	-10.2
23	23.00	917.7	410.2	-85.3	-152.7
24	24.00	763.7	312.8	-90.4	-157.9
25	25.00	605.2	209.7	-95.6	-163.0
26	26.00	441.9	101.0	-100.8	-168.2
27	27.00	275.0	-13.5	-106.0	-173.4
28	28.00	115.8	-133.1	-111.2	-178.6
29	29.00	-45.8	-257.9	-116.3	-183.8
30	30.00	-201.8	-388.8	-121.5	-188.9
31	31.00	-325.9	-575.3	-126.7	-194.1
32	32.00	-455.1	-770.5	-131.9	-199.3
33	33.00	-589.6	-970.9	-137.0	-204.5
34	34.00	-729.2	-1176.6	87.7	26.4
35	35.00	-509.1	-832.4	341.6	217.5
36	36.00	-294.2	-528.8	336.4	212.3
37	37.00	-0.1	-271.5	331.2	207.1
38	38.00	312.9	-28.9	184.8	117.0
39	39.00	345.8	19.3	45.5	26.9
40	40.00	374.1	62.2	40.4	21.8
41	41.00	397.8	100.0	35.2	16.6
42	42.00	416.9	125.3	30.0	11.4

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
43	43.00	430.9	143.4	24.8	6.2
44	44.00	439.6	156.4	19.7	1.1
45	45.00	443.3	164.1	14.5	-4.1
46	46.00	444.5	166.7	9.3	-9.3
47	47.00	443.3	164.1	4.1	-14.5
48	48.00	439.6	156.4	-1.1	-19.7
49	49.00	430.9	143.4	-6.2	-24.8
50	50.00	416.9	125.3	-11.4	-30.0
51	51.00	397.8	100.0	-16.6	-35.2
52	52.00	374.1	62.2	-21.8	-40.4
53	53.00	345.8	19.3	-26.9	-45.5
54	54.00	312.9	-28.9	-117.0	-184.8
55	55.00	-0.1	-271.5	-207.1	-331.2
56	56.00	-294.2	-528.8	-212.3	-336.4
57	57.00	-509.1	-832.4	-217.5	-341.6
58	58.00	-729.2	-1176.6	-26.4	-87.7
59	59.00	-589.6	-970.9	204.5	137.0
60	60.00	-455.1	-770.5	199.3	131.9
61	61.00	-325.9	-575.3	194.1	126.7
62	62.00	-201.8	-388.8	188.9	121.5
63	63.00	-45.8	-257.9	183.8	116.3
64	64.00	115.8	-133.1	178.6	111.2
65	65.00	275.0	-13.5	173.4	106.0
66	66.00	441.9	101.0	168.2	100.8
67	67.00	605.2	209.7	163.0	95.6
68	68.00	763.7	312.8	157.9	90.4
69	69.00	917.7	410.2	152.7	85.3
70	70.00	1067.0	502.1	10.2	-18.9
71	71.00	928.5	418.3	-86.4	-146.8
72	72.00	786.0	329.4	-91.5	-152.0
73	73.00	638.9	235.3	-96.7	-157.2
74	74.00	488.0	135.9	-101.9	-162.4
75	75.00	332.8	-10.9	-107.1	-167.5
76	76.00	174.5	-175.0	-112.3	-172.7
77	77.00	13.0	-344.2	-117.4	-177.9
78	78.00	-151.1	-518.6	-122.6	-183.1
79	79.00	-319.7	-698.2	-127.8	-188.3
80	80.00	-493.5	-882.9	-133.0	-193.4
81	81.00	-671.3	-1072.8	-138.1	-198.6
82	82.00	-844.9	-1267.9	59.6	13.7
83	83.00	-636.3	-953.5	311.8	206.1
84	84.00	-432.8	-644.3	306.6	200.9
85	85.00	-234.5	-340.3	301.4	195.7
86	86.00	-41.4	-41.4	158.5	105.6

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
87	87.00	-23.3	-23.3	15.5	15.5
88	88.00	-10.4	-10.4	10.4	10.4
89	89.00	-2.6	-2.6	5.2	5.2
90	90.00	0.0	0.0	1.3	1.3
91	91.00	0.0	0.0	0.0	0.0
92	92.00	0.0	0.0	0.0	0.0
93	93.00	0.0	0.0	0.0	0.0

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 7. MAXIMUM SUPPORT REACTIONS (WORKING STRESS)

STA	DIST X (FT)	MAX + REACT (K)	MAX - REACT (K)
10	10.00	514.7	354.6
34	34.00	554.9	364.9
58	58.00	554.9	364.9
82	82.00	514.7	354.6

```
PROB     1  (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)


TABLE 5.  MULTI-LANE LOADING SUMMARY    ( LOAD FACTOR)
       ( *--CRITICAL NUMBER OF LANE LOADS)

    MOMENT ( FT-K )
    ----------------------------------------------------------------------
    AT    DEAD LD   LANE   POSITIVE   LOAD AT    LANE   NEGATIVE   LOAD AT
    STA    EFFECT   ORDER  MAXIMUM    LANE STA   ORDER  MAXIMUM    LANE STA
    ----------------------------------------------------------------------

     6    -51.8
              0     0.0              0     0.0
              1     0.0              1     0.0
              2     0.0              2     0.0
              3     0.0              3     0.0
              0*                     0*

    10  -1071.7
              0     0.0              0   -616.9    1   2
              1     0.0              1   -616.9    1   2
              2     0.0              2     0.0
              3     0.0              3     0.0
              0*                     0*

    22    737.4
              0   707.0    0  13     0    -116.8    2  36
              1   704.1    1  12     1    -116.8    2  36
              2    32.7    3  62     2      0.0
              3     0.0              3     0.0
              0*                     0*

    34   -979.2
              0    65.4    3  62     0    -477.1    0  18
              1    65.4    3  62     1    -408.0    1  12
              2     0.0              2   -296.4    2  32
              3     0.0              3     0.0
              0*                     2*

    38    144.3
              0   292.6    2  32     0    -205.9    1   9
              1   292.6    2  32     1    -205.9    1   9
              2    11.2    3  62     2      0.0
              3     0.0              3     0.0
              0*                     0*

    46    351.4
              0   242.7    2  36     0     -97.4    1   9
              1   242.7    2  36     1     -97.4    1   9
              2     0.0              2    -97.4    3  63
              3     0.0              3     0.0
              0*                     2*

    54    144.3
              0   292.6    2  40     0    -205.9    3  63
              1   292.6    2  40     1    -205.9    3  63
              2    11.2    1  10     2      0.0
              3     0.0              3     0.0
              0*                     0*

    58   -979.2
              0    65.4    1   9     0    -477.1    0  54
              1    65.4    1   9     1    -408.0    3  60
              2     0.0              2   -296.4    2  40
              3     0.0              3     0.0
              0*                     2*
```

PROB   1   (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

```
   MOMENT ( FT-K )
   ----------------------------------------------------------------------
   AT    DEAD LD   LANE   POSITIVE   LOAD AT    LANE   NEGATIVE   LOAD AT
   STA    EFFECT   ORDER  MAXIMUM    LANE STA   ORDER  MAXIMUM    LANE STA
   ----------------------------------------------------------------------

    70     737.4
             0     707.0     0  59     0     -116.8      2  36
             1     704.1     3  60     1     -116.8      2  36
             2      32.7     1   9     2        0.0
             3       0.0               3      0.0
             0*                        0*

    82   -1071.7
             0       0.0               0    -616.9    3  70
             1       0.0               1    -616.9    3  70
             2       0.0               2      0.0
             3       0.0               3      0.0
             0*                        0*

    86     -51.8
             0       0.0               0      0.0
             1       0.0               1      0.0
             2       0.0               2      0.0
             3       0.0               3      0.0
             0*                        0*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

SHEAR (K)

AT STA	DEAD LD EFFECT	LANE ORDER	POSITIVE MAXIMUM	LOAD AT LANE STA	LANE ORDER	NEGATIVE MAXIMUM	LOAD AT LANE STA
8	-255.0						
		0	0.0		0	-154.2	1 2
		1	0.0		1	-154.2	1 2
		2	0.0		2	0.0	
		3	0.0		3	0.0	
		0*			0*		
12	176.7						
		0	78.4	1 6	0	-9.7	2 36
		1	78.4	1 6	1	-9.7	2 36
		2	2.7	3 62	2	0.0	
		3	0.0		3	0.0	
		0*			0*		
32	-168.9						
		0	2.7	3 62	0	-95.6	0 15
		1	2.7	3 62	1	-92.7	1 12
		2	0.0		2	-19.5	2 32
		3	0.0		3	0.0	
		0*			0*		
36	280.9						
		0	153.2	0 28	0	-13.6	3 63
		1	147.2	2 32	1	-13.6	3 63
		2	53.7	1 12	2	0.0	
		3	0.0		3	0.0	
		2*			0*		
56	-280.9						
		0	13.6	1 9	0	-153.2	0 44
		1	13.6	1 9	1	-147.2	2 40
		2	0.0		2	-53.7	3 60
		3	0.0		3	0.0	
		0*			2*		
60	168.9						
		0	95.6	0 57	0	-2.7	1 9
		1	92.7	3 60	1	-2.7	1 9
		2	19.5	2 40	2	0.0	
		3	0.0		3	0.0	
		0*			0*		
80	-176.7						
		0	9.7	2 36	0	-78.4	3 66
		1	9.7	2 36	1	-78.4	3 66
		2	0.0		2	-2.7	1 9
		3	0.0		3	0.0	
		0*			0*		
84	255.0						
		0	154.2	3 70	0	0.0	
		1	154.2	3 70	1	0.0	
		2	0.0		2	0.0	
		3	0.0		3	0.0	
		0*			0*		

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

```
 REACTION ( K )
 ----------------------------------------------------------------------
 AT   DEAD LD  LANE  POSITIVE  LOAD AT   LANE  NEGATIVE  LOAD AT
 STA  EFFECT   ORDER MAXIMUM   LANE STA  ORDER MAXIMUM   LANE STA
 ----------------------------------------------------------------------

 10   457.5
           0   223.8   1  2    0    -9.7    2 36
           1   223.8   1  2    1    -9.7    2 36
           2     2.7   3 62    2     0.0
           3     0.0           3     0.0
           0*                  0*

 34   475.7
           0   205.0   0 22    0   -16.3    3 63
           1   166.8   2 32    1   -16.3    3 63
           2   146.3   1 12    2     0.0
           3     0.0           3     0.0
           2*                  0*

 58   475.7
           0   205.0   0 50    0   -16.3    1  9
           1   166.8   2 40    1   -16.3    1  9
           2   146.3   3 60    2     0.0
           3     0.0           3     0.0
           2*                  0*

 82   457.5
           0   223.8   3 70    0    -9.7    2 36
           1   223.8   3 70    1    -9.7    2 36
           2     2.7   1  9    2     0.0
           3     0.0           3     0.0
           0*                  0*
```

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
-1	-1.00	0.0	0.0	0.0	0.0
0	0.00	0.0	0.0	0.0	0.0
1	1.00	0.0	0.0	0.0	0.0
2	2.00	0.0	0.0	-1.6	-1.6
3	3.00	-3.2	-3.2	-6.5	-6.5
4	4.00	-12.9	-12.9	-12.9	-12.9
5	5.00	-29.1	-29.1	-19.4	-19.4
6	6.00	-51.8	-51.8	-134.0	-226.5
7	7.00	-297.1	-482.1	-248.5	-433.6
8	8.00	-548.8	-918.9	-255.0	-440.0
9	9.00	-807.0	-1362.2	-261.5	-446.5
10	10.00	-1071.7	-1812.0	-9.6	-90.0
11	11.00	-839.4	-1542.1	277.3	171.4
12	12.00	-597.4	-1278.8	270.8	165.0
13	13.00	-359.6	-1021.9	264.3	158.5
14	14.00	-128.3	-771.5	257.8	152.0
15	15.00	97.6	-527.5	251.4	145.6
16	16.00	321.6	-290.0	244.9	139.1
17	17.00	542.6	-59.0	238.4	132.6
18	18.00	760.7	144.5	232.0	126.1
19	19.00	973.9	267.4	225.5	119.7
20	20.00	1183.0	383.9	219.0	113.2
21	21.00	1386.6	493.8	212.5	106.7
22	22.00	1585.8	597.3	31.8	-19.1
23	23.00	1365.1	476.9	-107.4	-225.4
24	24.00	1138.6	349.5	-113.9	-231.9
25	25.00	907.0	214.8	-120.4	-238.4
26	26.00	669.5	72.9	-126.8	-244.8
27	27.00	428.2	-76.6	-133.3	-251.3
28	28.00	203.0	-232.5	-139.8	-257.8
29	29.00	-23.8	-394.9	-146.3	-264.2
30	30.00	-238.0	-565.4	-152.7	-270.7
31	31.00	-394.0	-830.5	-159.2	-277.2
32	32.00	-556.5	-1108.4	-165.7	-283.7
33	33.00	-725.4	-1392.8	-172.1	-290.1
34	34.00	-900.8	-1683.6	132.6	25.2
35	35.00	-626.4	-1192.1	488.2	271.1
36	36.00	-358.5	-769.1	481.8	264.6
37	37.00	50.5	-424.3	475.3	258.1
38	38.00	495.4	-102.8	262.1	143.6
39	39.00	533.5	-38.0	61.6	29.0
40	40.00	566.2	20.4	55.1	22.6
41	41.00	593.4	72.3	48.6	16.1
42	42.00	615.2	104.9	42.2	9.6
43	43.00	630.6	127.5	35.7	3.1
44	44.00	639.4	143.7	29.2	-3.3

45	45.00	641.9	153.4	22.7	-9.8
46	46.00	642.7	156.6	16.3	-16.3
47	47.00	641.9	153.4	9.8	-22.7
48	48.00	639.4	143.7	3.3	-29.2

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
49	49.00	630.6	127.5	-3.1	-35.7
50	50.00	615.2	104.9	-9.6	-42.2
51	51.00	593.4	72.3	-16.1	-48.6
52	52.00	566.2	20.4	-22.6	-55.1
53	53.00	533.5	-38.0	-29.0	-61.6
54	54.00	495.4	-102.8	-143.6	-262.1
55	55.00	50.5	-424.3	-258.1	-475.3
56	56.00	-358.5	-769.1	-264.6	-481.8
57	57.00	-626.4	-1192.1	-271.1	-488.2
58	58.00	-900.8	-1683.6	-25.2	-132.6
59	59.00	-725.4	-1392.8	290.1	172.1
60	60.00	-556.5	-1108.4	283.7	165.7
61	61.00	-394.0	-830.5	277.2	159.2
62	62.00	-238.0	-565.4	270.7	152.7
63	63.00	-23.8	-394.9	264.2	146.3
64	64.00	203.0	-232.5	257.8	139.8
65	65.00	428.2	-76.6	251.3	133.3
66	66.00	669.5	72.9	244.8	126.8
67	67.00	907.0	214.8	238.4	120.4
68	68.00	1138.6	349.5	231.9	113.9
69	69.00	1365.1	476.9	225.4	107.4
70	70.00	1585.8	597.3	19.1	-31.8
71	71.00	1386.6	493.8	-106.7	-212.5
72	72.00	1183.0	383.9	-113.2	-219.0
73	73.00	973.9	267.4	-119.7	-225.5
74	74.00	760.7	144.5	-126.1	-232.0
75	75.00	542.6	-59.0	-132.6	-238.4
76	76.00	321.6	-290.0	-139.1	-244.9
77	77.00	97.6	-527.5	-145.6	-251.4
78	78.00	-128.3	-771.5	-152.0	-257.8
79	79.00	-359.6	-1021.9	-158.5	-264.3
80	80.00	-597.4	-1278.8	-165.0	-270.8
81	81.00	-839.4	-1542.1	-171.4	-277.3
82	82.00	-1071.7	-1812.0	90.0	9.6
83	83.00	-807.0	-1362.2	446.5	261.5
84	84.00	-548.8	-918.9	440.0	255.0
85	85.00	-297.1	-482.1	433.6	248.5
86	86.00	-51.8	-51.8	226.5	134.0
87	87.00	-29.1	-29.1	19.4	19.4
88	88.00	-12.9	-12.9	12.9	12.9
89	89.00	-3.2	-3.2	6.5	6.5

90	90.00	0.0	0.0	1.6	1.6
91	91.00	0.0	0.0	0.0	0.0
92	92.00	0.0	0.0	0.0	0.0

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA	DIST X (FT)	MAX + MOM (FT-K)	MAX - MOM (FT-K)	MAX + SHEAR (K)	MAX - SHEAR (K)
93	93.00	0.0	0.0	0.0	0.0

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'lay
(CONTINUED)

TABLE 7. MAXIMUM SUPPORT REACTIONS (LOAD FACTOR)

STA	DIST X (FT)	MAX + REACT (K)	MAX - REACT (K)
10	10.00	726.1	445.8
34	34.00	788.8	456.2
58	58.00	788.8	456.2
82	82.00	726.1	445.8

4.5.15.4 Live Load Distribution Factor Spreadsheet

4.5.15.4.1 Spans 1 & 3

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
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LRFD Live Load Distribution Factors*

Live Load Distribution Factors are calculated according to AASHTO LRFD Bridge Design Specifications, 8th Edition (2017 with no interim revisions) as prescribed by TxDOT policies (LRFD Design Manual July 2018) and practices. The Lever Rule is used when outside the Range of Applicability. The Range of Applicability for the Skew Correction Factors is ignored.

INPUT:

Beam Type =	Tx54	
No. Beams, N_b =	6	
CL_{brg} to CL_{brg}, L =	50.25	ft
Beam Spacing, S =	8.00	ft
Avg. Skew Angle, θ =	60.00	deg
Slab Thickness, t_s =	8.00	in
Slab Overhang, OH =	3	ft
Rail Width, RW =	1	ft
Roadway Width, W =	44	ft
Number of Lanes, N_L =	3	

Deck Slab		
Conc wt =	0.145	k/ft^3
f'_c =	4.0	ksi
E_{slab} =	3644	ksi

Beam		
weight =	0.145	k/ft^3
f'_c =	8.5	ksi
E_{beam} =	5312	ksi
y_t =	30.49	in
A =	817.0	in^2
I =	299740	in^4

Longitudinal Stiffness Parameter: (4.6.2.2.1-1)

e_g (in) = 34.49 (dist. b/w cog of bm & deck)

n = 1.000

$K_g = n(I+Ae_g^2)$ = 1271611 in^4

*For typical cross sections (a,e,i,j & k). Table 4.6.2.2.1-1

RESULTS:

	Final LLDF
Interior Shear LLDF, $gV_{interior}$	0.999
Interior Moment LLDF, $gM_{interior}$	0.537
Exterior Shear LLDF, $gV_{exterior}$	0.999
Exterior Moment LLDF, $gM_{exterior}$	0.537

The Final LLDF may be modified according to the following TxDOT policies:

- Exterior beams use the interior LLDF when OH ≤ S/2.
- When OH > S/2 the exterior beam LLDF is determined by the lever rule for a single lane with a multiple presence factor of 1.0.
- In no case shall the LLDF for the exterior beams be less than the LLDFs for the interior beams.
- When the Roadway width is less than 20ft, all beams are designed for one lane loaded only.
- In no case shall the LLDF be less than $m \cdot N_L \div N_b$.

CALCULATIONS:

Shear LLDF Correction for Skew (Table 4.6.2.2.3c-1)

$$\text{Corr.} = 1.0 + 0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}\tan\theta$$

= 1.0 + 0.20 * [(12.0*50.3*8^3)/(1,271,611)]^0.3 * tan(60)

Corr. = 1.227

Check θ:	0° ≤ 60° ≤ 60°	OK
Check S:	3.5' ≤ 8.0' ≤ 16.0'	OK
Check L:	20' ≤ 50.3' ≤ 240'	OK
Check N_b:	6 ≥ 4	OK

Moment LLDF Correction for Skew (Table 4.6.2.2.2e-1)

Check θ: 30° ≤ 60° ≤ 60° 60°

Corr. = 1 - c_1(tan θ)^1.5

= 1 - 0.142(tan 60)^1.5

Corr. = 0.676

where: $$c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$$

c_1 = 0.142

INTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3a-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 1.227

mg = 0.750 * 1.227 = 0.920

Equation

g = $0.36 + \left(\frac{S}{25}\right)$

g = 0.36 + (8 / 25) = 0.680

Modify for Skew:

skew correction = 1.227

g = 0.680 * 1.227 = 0.834

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 50.3' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int1} = 0.834

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 1.227

mg = 0.875 * 1.227 = 1.074

Equation

g = $0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0}$

g = 0.2 + (8 / 12) - (8 / 35)^2.0 = 0.814

Modify for Skew:

skew correction = 1.227

g = 0.814 * 1.227 = 0.999

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int2+} = 0.999

TxDOT Policy states $gV_{Interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gV_{interior}$ is the Maximum of: gV_{int1} and $m \cdot N_L \div N_b$.

>> TxDOT Policy states that if W ≥ 20ft, $gV_{interior}$ is the Maximum of: gV_{int1}, gV_{int2+}, $m \cdot N_L \div N_b$.

$gV_{interior}$ = 0.999

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INTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 0.676

mg = 0.750 * 0.676 = 0.507

Equation

$$g = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

g = 0.06 + (8/14)^0.4 * (8/50.3)^0.3 * (1,271,611/(12*50.3*8^3))^0.1 = 0.591

Modify for Skew:

skew correction = 0.676

g = 0.591 * 0.676 = 0.400

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 50.3' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Check K_g: 10,000 ≤ 1,271,611 ≤ 7,000,000 OK

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int1} = 0.400

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 0.676

mg = 0.875 * 0.676 = 0.592

Equation

$$g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

g = 0.075 + (8/9.5)^0.6 * (8/50.3)^0.2 * (1,271,611/(12*50.3*8^3))^0.1 = 0.795

Modify for Skew:

skew correction = 0.676

g = 0.795 * 0.676 = 0.537

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int2+} = 0.537

TxDOT Policy states $gM_{Interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gM_{Interior}$ is the Maximum of: gM_{int1} and $m \cdot N_L \div N_b$.

>> TxDOT Policy states that if W ≥ 20ft, $gM_{Interior}$ is the Maximum of: gM_{int1}, gM_{int2+}, $m \cdot N_L \div N_b$.

$gM_{Interior}$ = 0.537

EXTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.0 = 0.625 TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 1.227

mg = 0.625 * 1.227 = 0.767

Use Lever Rule, as per AASHTO LRFD Table 4.6.2.2.3b-1.

gV_{ext1} = 0.767

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 1.227

mg = 0.625 * 1.227 = 0.767

Equation

d_e = dist. b/w CL web to curb

d_e = OH - Rail Width

d_e = 3ft - 1ft = 2.0 ft

$$e = 0.6 + \left(\frac{d_e}{10}\right)$$

e = 0.6 + (2.0/10) = 0.800

$g = e * gV_{int2+Eq}$

g = 0.800 * 0.999 = 0.799

Skew Correction is included in gV(interior).

Range of Applicability (ROA) Checks — Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: $-1.0' \le 2.0' \le 5.5'$ OK

Check N_b: $6 \ne 3$ OK

Use Equation from Table 4.6.2.2.3b-1 because all criteria is OK.

gV_{ext2+} = 0.799

TxDOT Policy states $gV_{Exterior}$ must be $\ge gV_{Interior}$

$gV_{interior}$ = 0.999

TxDOT Policy states $gV_{Exterior}$ must be $\ge m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

>> TxDOT Policy states that if OH ≤ S/2, $gV_{Exterior}$ is $gV_{Interior}$.

TxDOT Policy states that if OH > S/2 and W < 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, $gV_{Interior}$, and $m \cdot N_L \div N_b$.

TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, gV_{ext2+}, $gV_{Interior}$, and $m \cdot N_L \div N_b$.

$gV_{exterior}$ = 0.999

EXTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2d-1):

One Lane Loaded

Lever Rule

mg = 0.625 * 1.0 = 0.625 — TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 0.676

mg = 0.625 * 0.676 = 0.423

Use Lever Rule as per AASHTO LRFD Table 4.6.2.2.2d-1.

gM_{ext1} = 0.423

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 0.676

mg = 0.625 * 0.676 = 0.423

Equation

$$e = 0.77 + \left(\frac{d_e}{9.1}\right)$$

e = 0.77 + (2.0/9.1) = 0.990

$g = e^{*}gM_{int2+Eq}$

g = 0.99 * 0.537 = 0.532

Skew Correction included in gM(interior).

Range of Applicability (ROA) Checks — Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: -1.0' ≤ 2.0' ≤ 5.5' OK

Check N_b: 6 ≠ 3 OK

Use Equation from Table 4.6.2.2.2d-1 because all criteria is OK.

gM_{ext2+} = 0.532

TxDOT Policy states $gM_{Exterior}$ must be ≥ $gM_{Interior}$

$gM_{Interior}$ = 0.537

TxDOT Policy states $gM_{Exterior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

>> TxDOT Policy states that if OH ≤ S/2, $gM_{Exterior}$ is $gM_{Interior}$.

TxDOT Policy states that if OH > S/2 and W < 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, $gM_{Interior}$, and $m \cdot N_L \div N_b$.

TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, gM_{ext2+}, $gM_{Interior}$, and $m \cdot N_L \div N_b$.

$gM_{exterior}$ = 0.537

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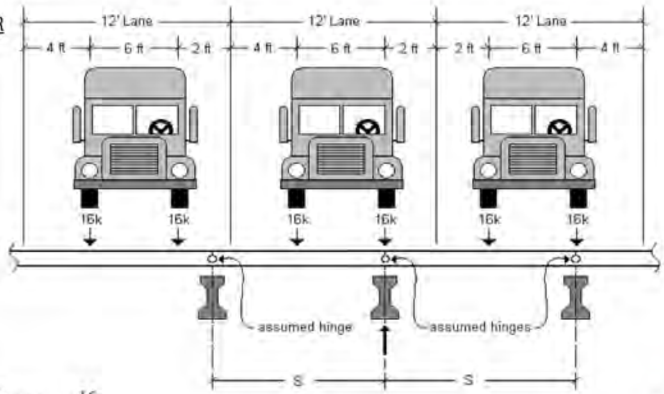
LEVER RULE S = 8.0 ft

INTERIOR



For S < 4:

One Lane = $\frac{16}{32}$ = 0.500

For 4 ≤ S < 6:

One Lane = $\frac{16}{32}$ = 0.500

Two Lanes = $\frac{16}{32}\left(1+\frac{S-4}{S}\right)$ = 0.750

>: **For 6 ≤ S < 10:**

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}\right)$ = 0.875

For 10 ≤ S < 12:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

For 12 ≤ S < 16:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

For 16 ≤ S < 18:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-16}{S}\right)$ = 0.000

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LEVER RULE S = 8.0 ft

INTERIOR (con't)

For 18 ≤ S < 22:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}\right)$ = -0.625

For 22 ≤ S ≤ 24:

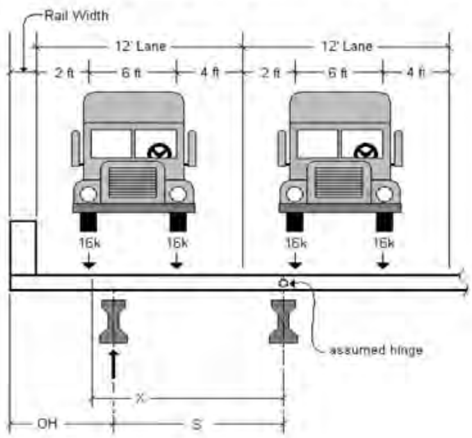
One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}+\frac{S-22}{S}\right)$ = -1.500

EXTERIOR



S = 8.0 ft
OH = 3.0 ft
Rail Width = RW = 1.0 ft
X = S+OH-RW-2ft = 8.0 ft

For X < 6:

One Lane = $\frac{16}{32}\left(\frac{X}{S}\right)$ = 0.500

>: **For 6 ≤ X < 12:**

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

For 12 ≤ X < 18:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}\right)$ = 0.375

LEVER RULE

EXTERIOR (con't) S = 8.0 ft OH = 3.0 ft
RW = 1.0 ft X = S+OH-RW-2ft = 8.0 ft

For 18 ≤ X < 24:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

For 24 ≤ X < 30:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}\right)$ = -1.250

For 30 ≤ X < 36:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

For 36 ≤ X < 42:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}\right)$ = -4.375

For 42 ≤ X ≤ 48:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}+\frac{X-42}{S}\right)$ = -6.500

INTERIOR			EXTERIOR		
One Lane Loaded	=	0.625	One Lane Loaded	=	0.625
Two Lanes Loaded	=	0.875	Two Lanes Loaded	=	0.625
Three Lanes Loaded	=	0.875	Three Lanes Loaded	=	0.625
Four Lanes Loaded	=	0.875	Four Lanes Loaded	=	0.625

4.5.15.4.2 Span 2

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
BRIDGE	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:		Rev. 10/18 - (No Interim)
DIVISION	Descrip:	ITBC Design Example 4, Span 2			File:	Ex4_Span2_distribution_factors.xl		Sheet:	1 of 8

LRFD Live Load Distribution Factors*

Live Load Distribution Factors are calculated according to AASHTO LRFD Bridge Design Specifications, 8th Edition (2017 with no interim revisions) as prescribed by TxDOT policies (LRFD Design Manual July 2018) and practices. The Lever Rule is used when outside the Range of Applicability. The Range of Applicability for the Skew Correction Factors is ignored.

INPUT:

Beam Type =	Tx54	
No. Beams, N_b =	6	
CL_{brg} to CL_{brg}, L =	106.50	ft
Beam Spacing, S =	8.00	ft
Avg. Skew Angle, θ =	60.00	deg
Slab Thickness, t_s =	8.00	in
Slab Overhang, OH =	3	ft
Rail Width, RW =	1	ft
Roadway Width, W =	44	ft
Number of Lanes, N_L =	3	

Deck Slab		
Conc wt =	0.145	k/ft^3
f'_c =	4.0	ksi
E_{slab} =	3644	ksi

Beam		
weight =	0.145	k/ft^3
f'_c =	8.5	ksi
E_{beam} =	5312	ksi
y_t =	30.49	in
A =	817.0	in^2
I =	299740	in^4

Longitudinal Stiffness Parameter: (4.6.2.2.1-1)

e_g (in) = 34.49 (dist. b/w cog of bm & deck)

n = 1.000

$K_g = n(I+Ae_g^2)$ = 1271611 in^4

*For typical cross sections (a,e,i,j & k). Table 4.6.2.2.1-1

RESULTS:

	Final LLDF
Interior Shear LLDF, $gV_{interior}$	**1.045**
Interior Moment LLDF, $gM_{interior}$	**0.529**
Exterior Shear LLDF, $gV_{exterior}$	**1.045**
Exterior Moment LLDF, $gM_{exterior}$	**0.529**

The Final LLDF may be modified according to the following TxDOT policies:

- Exterior beams use the interior LLDF when OH ≤ S/2.
- When OH > S/2 the exterior beam LLDF is determined by the lever rule for a single lane with a multiple presence factor of 1.0.
- In no case shall the LLDF for the exterior beams be less than the LLDFs for the interior beams.
- When the Roadway width is less than 20ft, all beams are designed for one lane loaded only.
- In no case shall the LLDF be less than $m \cdot N_L \div N_b$.

CALCULATIONS:

Shear LLDF Correction for Skew (Table 4.6.2.2.3c-1)

$$\text{Corr.} = 1.0 + 0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}\tan\theta$$

= 1.0 + 0.20 * [(12.0*106.5*8^3)/(1,271,611)]^0.3 * tan(60)

Corr. = 1.284

Check θ:	0° ≤ 60° ≤ 60°	OK
Check S:	3.5' ≤ 8.0' ≤ 16.0'	OK
Check L:	20' ≤ 106.5' ≤ 240'	OK
Check N_b:	6 ≥ 4	OK

Moment LLDF Correction for Skew (Table 4.6.2.2.2e-1)

Check θ: 30° ≤ 60° ≤ 60° 60°

Corr. = 1 - c_1(tan θ)^1.5

= 1 - 0.081(tan 60)^1.5

Corr. = 0.815

where: $$c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$$

c_1 = 0.081

TXDOT BRIDGE DIVISION	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:		Rev. 10/18 - (No Interim)
	Descrip:	ITBC Design Example 4, Span 2			File:	Ex4_Span2_distribution_factors.xl		Sheet:	2 of 8

INTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3a-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 1.284

mg = 0.750 * 1.284 = 0.963

Equation

$$g = 0.36 + \left(\frac{S}{25}\right)$$

g = 0.36 + (8 / 25) = 0.680

Modify for Skew:

skew correction = 1.284

g = 0.680 * 1.284 = 0.873

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 106.5' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int1} = 0.873

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 1.284

mg = 0.875 * 1.284 = 1.124

Equation

$$g = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0}$$

g = 0.2 + (8 / 12) - (8 / 35)^2.0 = 0.814

Modify for Skew:

skew correction = 1.284

g = 0.814 * 1.284 = 1.045

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.3a-1 because all criteria is OK.

gV_{int2+} = 1.045

TxDOT Policy states $gV_{Interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gV_{Interior}$ is the Maximum of: gV_{int1} and $m \cdot N_L \div N_b$.

>> TxDOT Policy states that if W ≥ 20ft, $gV_{Interior}$ is the Maximum of: gV_{int1}, gV_{int2+}, $m \cdot N_L \div N_b$.

$gV_{interior}$ = 1.045

INTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.2 = 0.750

Modify for Skew:

skew correction = 0.815

mg = 0.750 * 0.815 = 0.611

Equation

$$g = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^{\,3}}\right)^{0.1}$$

g = 0.06 + (8/14)^0.4 * (8/106.5)^0.3 * (1,271,611/(12*106.5*8^3))^0.1 = 0.453

Modify for Skew:

skew correction = 0.815

g = 0.453 * 0.815 = 0.369

Range of Applicability (ROA) Checks

Check S: 3.5' ≤ 8.0' ≤ 16.0' OK

Check t_s: 4.5" ≤ 8.0" ≤ 12.0" OK

Check L: 20' ≤ 106.5' ≤ 240' OK

Check N_b: 6 ≥ 4 OK

Check K_g: 10,000 ≤ 1,271,611 ≤ 7,000,000 OK

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int1} = 0.369

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875

Modify for Skew:

skew correction = 0.815

mg = 0.875 * 0.815 = 0.713

Equation

$$g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^{\,3}}\right)^{0.1}$$

g = 0.075 + (8/9.5)^0.6 * (8/106.5)^0.2 * (1,271,611/(12*106.5*8^3))^0.1 = 0.649

Modify for Skew:

skew correction = 0.815

g = 0.649 * 0.815 = 0.529

Range of Applicability (ROA) Checks (same as for one lane loaded)

Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.

gM_{int2+} = 0.529

TxDOT Policy states $gM_{Interior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is W ≥ 20ft ? Yes

TxDOT Policy states that if W < 20ft, $gM_{Interior}$ is the Maximum of: gM_{int1} and $m \cdot N_L \div N_b$.

>> TxDOT Policy states that if W ≥ 20ft, $gM_{Interior}$ is the Maximum of: gM_{int1}, gM_{int2+}, $m \cdot N_L \div N_b$.

$gM_{Interior}$ = 0.529

EXTERIOR BEAM:

Shear LL Distribution Per Lane (Table 4.6.2.2.3b-1):

One Lane Loaded

Lever Rule (Table 3.6.1.1.2)

mg = 0.625 * 1.0 = 0.625

TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 1.284

mg = 0.625 * 1.284 = 0.803

Use Lever Rule, as per AASHTO LRFD Table 4.6.2.2.3b-1.

gV_{ext1} = 0.803

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 1.284

mg = 0.625 * 1.284 = 0.803

Equation

d_e = dist. b/w CL web to curb

d_e = OH - Rail Width

d_e = 3ft - 1ft = 2.0 ft

$$e = 0.6 + \left(\frac{d_e}{10}\right)$$

e = 0.6 + (2.0/10) = 0.800

$g = e{*}gV_{int2+Eq}$

g = 0.800 * 1.045 = 0.836

Skew Correction is included in gV(interior).

Range of Applicability (ROA) Checks

Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: $-1.0' \le 2.0' \le 5.5'$ OK

Check N_b: $6 \ne 3$ OK

Use Equation from Table 4.6.2.2.3b-1 because all criteria is OK.

gV_{ext2+} = 0.836

TxDOT Policy states $gV_{Exterior}$ must be $\ge gV_{Interior}$

$gV_{interior}$ = 1.045

TxDOT Policy states $gV_{Exterior}$ must be $\ge m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

>> TxDOT Policy states that if OH ≤ S/2, $gV_{Exterior}$ is $gV_{Interior}$.

TxDOT Policy states that if OH > S/2 and W < 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, $gV_{Interior}$, and $m \cdot N_L \div N_b$.

TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gV_{Exterior}$ is the Maximum of: gV_{ext1}, gV_{ext2+}, $gV_{Interior}$, and $m \cdot N_L \div N_b$.

$gV_{exterior}$ = 1.045

EXTERIOR BEAM:

Moment LL Distribution Per Lane (Table 4.6.2.2.2d-1):

One Lane Loaded

Lever Rule

mg = 0.625 * 1.0 = 0.625 — TxDOT uses a multiple presence factor of 1.0 for one lane loaded on the exterior beam.

Modify for Skew:

skew correction = 0.815

mg = 0.625 * 0.815 = 0.509

Use Lever Rule as per AASHTO LRFD Table 4.6.2.2.2d-1.

gM_{ext1} = 0.509

Two or More Lanes Loaded

Lever Rule (Table 3.6.1.1.2)

mg = Max(0.625 * 1.0, 0.625 * 0.85, 0.625 * 0.65) = 0.625

Modify for Skew:

skew correction = 0.815

mg = 0.625 * 0.815 = 0.509

Equation

$$e = 0.77 + \left(\frac{d_e}{9.1}\right)$$

e = 0.77 + (2.0/9.1) = 0.990

$g = e^{*}gM_{int2+Eq}$

g = 0.99 * 0.529 = 0.524

Skew Correction included in gM(interior).

Range of Applicability (ROA) Checks — Interior ROA is implicitly applied to the exterior beam.

Check Interior Beam ROA: OK

Check d_e: -1.0' ≤ 2.0' ≤ 5.5' OK

Check N_b: 6 ≠ 3 OK

Use Equation from Table 4.6.2.2.2d-1 because all criteria is OK.

gM_{ext2+} = 0.524

TxDOT Policy states $gM_{Exterior}$ must be ≥ $gM_{Interior}$

$gM_{interior}$ = 0.529

TxDOT Policy states $gM_{Exterior}$ must be ≥ $m \cdot N_L \div N_b$

$m \cdot N_L \div N_b$ = 0.85 * 3 / 6 = 0.425

Is OH ≤ S/2 ? Yes

Is W ≥ 20ft ? Yes

>> TxDOT Policy states that if OH ≤ S/2, $gM_{Exterior}$ is $gM_{Interior}$.

TxDOT Policy states that if OH > S/2 and W < 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, $gM_{Interior}$, and $m \cdot N_L \div N_b$.

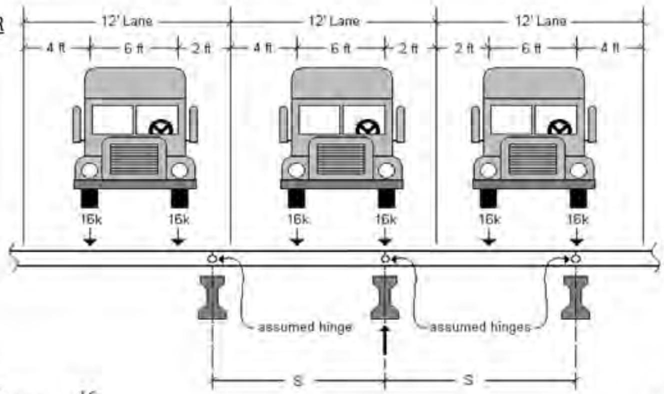
TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, $gM_{Exterior}$ is the Maximum of: gM_{ext1}, gM_{ext2+}, $gM_{Interior}$, and $m \cdot N_L \div N_b$.

$gM_{exterior}$ = 0.529

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
BRIDGE	C-S-J:	XXX-XX-XXXX	ID #:	XXXX	Ck Dsn:		Date:		Rev. 10/18 - (No Interim)
DIVISION	Descrip:	ITBC Design Example 4, Span 2			File:	Ex4_Span2_distribution_factors.xl			Sheet: 6 of 8

LEVER RULE S = 8.0 ft

INTERIOR



For S < 4:

One Lane = $\frac{16}{32}$ = 0.500

For 4 ≤ S < 6:

One Lane = $\frac{16}{32}$ = 0.500

Two Lanes = $\frac{16}{32}\left(1+\frac{S-4}{S}\right)$ = 0.750

>: **For 6 ≤ S < 10:**

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}\right)$ = 0.875

For 10 ≤ S < 12:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

For 12 ≤ S < 16:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

For 16 ≤ S < 18:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}\right)$ = 0.500

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-16}{S}\right)$ = 0.000

LEVER RULE S = 8.0 ft

INTERIOR (con't)

For 18 ≤ S < 22:

One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}\right)$ = -0.625

For 22 ≤ S ≤ 24:

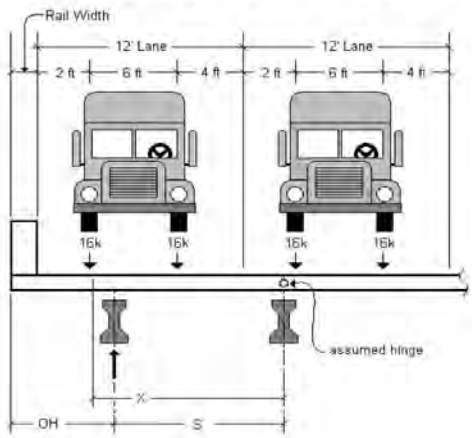
One Lane = $\frac{16}{32}\left(1+\frac{S-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}\right)$ = 0.750

Three Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}\right)$ = -0.125

Four Lanes = $\frac{16}{32}\left(1+\frac{S-6}{S}+\frac{S-4}{S}+\frac{S-10}{S}+\frac{S-12}{S}+\frac{S-18}{S}+\frac{S-16}{S}+\frac{S-22}{S}\right)$ = -1.500

EXTERIOR



S = 8.0 ft
OH = 3.0 ft
Rail Width = RW = 1.0 ft
X = S+OH-RW-2ft = 8.0 ft

For X < 6:

One Lane = $\frac{16}{32}\left(\frac{X}{S}\right)$ = 0.500

>: **For 6 ≤ X < 12:**

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

For 12 ≤ X < 18:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}\right)$ = 0.375

LEVER RULE

EXTERIOR (con't) S = 8.0 ft OH = 3.0 ft
RW = 1.0 ft X = S+OH-RW-2ft = 8.0 ft

For 18 ≤ X < 24:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

For 24 ≤ X < 30:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}\right)$ = -1.250

For 30 ≤ X < 36:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

For 36 ≤ X < 42:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}\right)$ = -4.375

For 42 ≤ X ≤ 48:

One Lane = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}\right)$ = 0.625

Two Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}\right)$ = -0.250

Three Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}\right)$ = -2.625

Four Lanes = $\frac{16}{32}\left(\frac{X}{S}+\frac{X-6}{S}+\frac{X-12}{S}+\frac{X-18}{S}+\frac{X-24}{S}+\frac{X-30}{S}+\frac{X-36}{S}+\frac{X-42}{S}\right)$ = -6.500

INTERIOR			EXTERIOR		
One Lane Loaded	=	0.625	One Lane Loaded	=	0.625
Two Lanes Loaded	=	0.875	Two Lanes Loaded	=	0.625
Three Lanes Loaded	=	0.875	Three Lanes Loaded	=	0.625
Four Lanes Loaded	=	0.875	Four Lanes Loaded	=	0.625

4.5.15.5 Concrete Section Shear Capacity Spreadsheet

Texas Department of Transportation

County:	ANY	Descrip:	ITBC Design Example 4- Bent 2
Highway:	ANY		
C-S-J:	XXXXXXX	Design:	BRG
		Ck Dsn:	BRG
Bridge Division	Rev: 09/26/08	Date:	Aug-20

CONCRETE SECTION SHEAR CAPACITY BY AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, FOURTH EDITION, 2007

Resistance Factors:

ϕ_V =	0.9
ϕ_M =	0.9
ϕ_N =	0.75

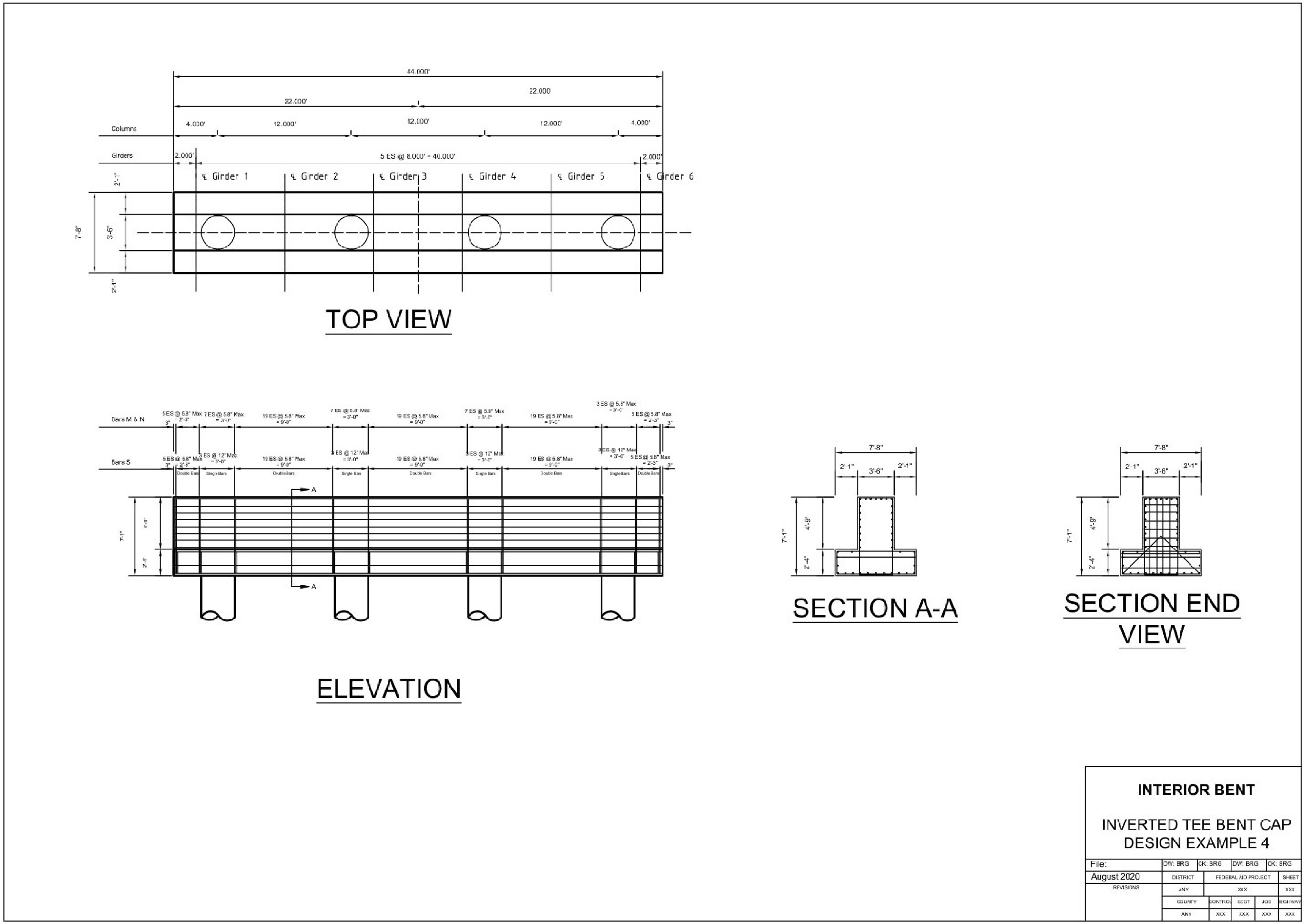
Units: US

Concrete:		Mild Steel:		Prestressed Steel:	
f'c =	5 ksi	fy =	60 ksi	fpu =	270 ksi
Ec =	4070 ksi	Es =	29000 ksi	Ep =	28500 ksi

		SECTIONS							
	Units	8	12	32	36	56	60	80	84
Input Data									
Bending moment, **Mu**	kip-ft	918.9	1278.8	1108.4	769.1	769.1	1108.4	1278.8	919
Shear force, **Vu**	kip	255	270.8	165.7	481.8	264.6	283.7	165	440
Axial force, **Nu (+ if tensile)**	kip	0	0	0	0	0	0	0	0
Web width, **bv**	in	42.00	42.00	42.00	42.00	42.00	42.00	42.00	42.00
Shear depth, **dv**	in	80.59	80.59	80.59	80.59	80.59	80.59	80.59	80.59
Mild steel reinf. area, **As**	in^2	10.92	10.92	10.92	10.92	10.92	10.92	10.92	10.92
Conc area on tension side, **Ac**	in^2	1785	1785	1785	1785	1785	1785	1785	1785
Area of stirrups, **Av**	in^2	1.76	1.76	1.76	1.76	1.76	1.76	1.76	1.76
Stirrup spacing, **s**	in	6.8	6.8	6.8	6.8	6.8	6.8	6.8	6.8
Prestressed steel area, **Aps**	in^2	0	0	0	0	0	0	0	0
Prestress shear, **Vp**	kip	0	0	0	0	0	0	0	0
Average prestress, **fps**	ksi	0	0	0	0	0	0	0	0
Torsional moment, **Tu**	kip-ft	830	415	415	830	830	415	415	830
Shear flow area, **Ao**	in^2	3493.5	3493.5	3493.5	3493.5	3493.5	3493.5	3493.5	3493.5
Area of one leg of stirrup, **At**	in^2	0.44	0.44	0.44	0.44	0.44	0.44	0.44	0.44
Perimeter of stirrup, **Ph**	in	334	334	334	334	334	334	334	334
Calculated Values									
Vc	kip	576.4	569.2	624.2	533.3	571.6	564.4	614.7	533.3
Vs	kip	1784.9	1812.7	2077.9	1569.9	1763.2	1791.6	2039.4	1569.9
ϕVn	kip	**2125**	**2144**	**2432**	**1893**	**2101**	**2120**	**2389**	**1893**
ε_x		7.10E-04	7.48E-04	4.86E-04	1.00E-03	7.33E-04	7.80E-04	5.20E-04	1.00E-03
θ	deg	33.20	33.70	30.30	36.40	33.50	34.00	30.75	36.40
β		2.410	2.380	2.610	2.230	2.390	2.360	2.570	2.230
Req'd Shear reinf. **Av/S**	in^2/in	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Req'd Torsion reinf. **At/S**	in^2/in	0.017	0.009	0.008	0.019	0.017	0.009	0.008	0.019
Maximum stirrup spacing, **Smax**	in	24.0	24.0	24.0	22.5	24.0	24.0	24.0	22.6
Conclusion									
Shear Reinforcing		OK	OK	OK	OK	OK	OK	OK	OK
Longitudinal Reinforcing		OK	OK	OK	OK	OK	OK	OK	OK

Note: Longitudinal Reinforcing check can be ignored for typical multi-column bent caps. For straddle bents with no overhangs, this check must be considered. Refer to LRFD 5.8.3.5 for further information.
If torsion is not being considered, leave last five rows of input data blank.

4.5.15.6 Bent Cap Details



CHAPTER 5: SUMMARY AND CONCLUSIONS

5.1 SUMMARY OF THE RESEARCH WORK

The summary of the test and analytical results on inverted-T bent cap specimens under the scope of this project work is presented below.

1. Bent 2, Bent 6, and Bent 7 of a seven-span bridge, which are under construction on Donigan Road over IH 10 near Brookshire in Waller County, are selected. These bent caps have skew angles of 43°, 33°, and 33°, respectively.
2. The preliminary finite element (FE) analysis of the selected skew ITBCs is performed using ABAQUS to better understand the overall structural behavior of skew reinforcement in actual ITBCs and to determine critical loading patterns during the load tests and crucial strain gauge locations.
3. Stresses in skew transverse reinforcement at the service load and at the ultimate state are obtained according to the finite element results. The displacement and principal tensile strains of the bent caps are studied to understand the structural behavior of actual ITBCs designed with skew transverse reinforcement.
4. To investigate the structural performance of skew ITBCs with traditional transverse reinforcement and with skew transverse reinforcement, a total of ninety-six large-scale specimens are modeled in ABAQUS.
5. Design parameters are the skew angle (43° or 33°), detailing of transverse reinforcements (skew transverse reinforcement or traditional transverse reinforcement), end bars (with or without U1 Bars, U2 Bars, U3 Bars, and G Bars), size of S Bars (minimum, current design, 20% more or 40% more than current design), size of G Bars (No. 3 to No. 7 bars), and concrete strength (5 or 7 ksi). Based on these parameters, the displacement and the stiffness at the service load, the principal tensile strain of concrete and crack widths at the service load, and the ultimate capacities of the bent caps are investigated.
6. Cost-benefit analyses of ninety-six specimens are conducted considering the design and construction costs of ITBCs.
7. According to the parametric analysis results, a set of design recommendations for skew ITBCs is presented.
8. Following AASHTO LRFD Bridge Design Specifications, 8th Ed. (2017) and TxDOT Bridge Manual - LRFD (January 2020), four ITBC design examples with different skew angles (0°, 30°, 45°, and 60°) are presented with the step by step procedures.

5.2 CONCLUSIONS

After performing the FE analysis on the actual ITBC structures, the conclusions are presented below.

1. For the selected skew ITBCs in this research, it is observed that the critical locations to paste the strain gauges and attach LVDTs are the cantilever end faces of the bent caps.

2. It is also observed that all the bent caps with skew transverse reinforcing are safe under service and ultimate state loading.
3. According to the cost-benefit analysis results, the skew transverse reinforcement (Case 1) provides better structural performance, reduced number of cracks and reduced crack width compared to the traditional transverse reinforcement (Case 2 and Case 3) with notably reduced construction cost. Therefore, the skew transverse reinforcement can well be used for the design of skewed ITBCs.
4. The increase of the S Bar area notably enhances the stiffness and ultimate strength. In addition, the increase of the S Bar area also reduces the crack width. The increase of the S Bar area will contribute notably to the construction cost. Based on the parametric simulation results, the current design of the S bar area is adequate for structural safety and crack resistance.
5. Having end bars (U1 Bars, U2 Bars, U3 Bars, and G Bars) significantly decreases the crack width on skew ITBCs.
6. The increase of the G Bar area notably reduces the maximum crack width with a negligible influence on the stiffness, ultimate strength, and construction cost. The current design of the G Bar (No. 7 Bars) is adequate for crack control.
7. When the concrete strength increases from 5 ksi to 7 ksi, the ultimate strength and the stiffness of ITBCs increase with reduced crack width. In addition, the influence of concrete strength on the construction cost is negligible.
8. Based on the research results, the RT completed four design examples of skewed ITBCs with various skew angles (0°, 30°, 45°, and 60°).

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APPENDIX 1

Updates from AASHTO LRFD 2010 to AASHTO LRFD 2017

This document shows the revisions from AASHTO LRFD Bridge Design Specifications, 5th Ed. (2010) to AASHTO LRFD Bridge Design Specifications, 8th Ed. (2017) for the sections, equations, and tables that are used in the design of the inverted Tee bent cap. "NR" denotes no revision.

Table A1.1 Comparison between AASHTO (2010) and AASHTO (2017)

AASHTO LRFD 2010		AASHTO LRFD 2017	
Section Number	**Title or Content**	**Section Number**	**Title or Content**
Eq. 1.3.2.1-1	$\sum \eta_i \gamma_i Q_i \leq \Phi R_n = R_r$	NR	NR
3.4.1	Load Factors and Combinations	NR	NR
3.6.1.1.2	Multiple Presence of Live Load	NR	
Table 3.6.1.1.2-1	Multiple Presence Factors, m	NR	NR
3.6.1.2.1	Design Vehicular Live Load - General	NR	NR
3.6.1.2.2	Design Vehicular Live Load - Design Truck	NR	NR
3.6.1.2.4	Design Vehicular Live Load - Design Lane Load	NR	NR
3.6.1.3	Design Vehicular Live Load - Application of Design Vehicular Live Loads	NR	NR
Table 3.6.2.1-1	Dynamic Load Allowance, IM	NR	NR
Table 4.6.2.2.1-1	Common Deck Superstructures	NR	NR
Eq. 4.6.2.2.1-1	$K_g = n(I + Ae_g^2)$	NR	NR
Table 4.6.2.2.2e-1	Reduction of Load Distribution Factors for Moment in Longitudinal Beams on Skewed Supports	NR	NR
Table 4.6.2.2.3a-1	Distribution of Live Load for Shear in Interior Beams	NR	NR
Table 4.6.2.2.3b-1	Distribution of Live Load for Shear in Exterior Beams	NR	NR
Table 4.6.2.2.3c-1	Correction Factors for Load Distribution Factors for Support Shear of the Obtuse Corner	NR	NR
Eq. 5.4.2.4-1	$E_c = 33000 K_1 w_c^{1.5} \sqrt{f_c}$	NR	$E_c = 120000 K_1 w_c^{2.0} f_c^{0.33}$

AASHTO LRFD 2010		AASHTO LRFD 2017	
Section Number	**Title or Content**	**Section Number**	**Title or Content**
5.5.4.2.1	Resistance Factors	5.5.4.2	Some revisons for lightweight concrete
5.7.2.1	Assumptions for Strength and Extreme Event Limit States - General	5.6.2.1	NR
5.7.2.2	Assumptions for Strength and Extreme Event Limit States – Rectangular Stress Distribution	5.6.2.2	α_1 to the description of the compression zone
Eq. 5.7.3.1.2-3	$c = \frac{A_{ps}f_{ps} + A_s f_s - A'_s f'_s - 0.85f_c(b - b_w)h_f}{0.85f_c\beta_1 b_w}$	Eq. 5.6.3.1.2-3	$c = \frac{A_{ps}f_{ps}+A_s f_s - A'_s f'_s - \alpha_1 f_c(b-b_w)h_f}{\alpha_1 f_c \beta_1 b_w}$
Eq. 5.7.3.1.2-4	$c = \frac{A_{ps}f_{ps}+A_s f_s - A'_s f'_s}{0.85 f_c \beta_1 b}$	Eq. 5.6.3.1.2-4	$c = \frac{A_{ps}f_{ps}+A_s f_s - A'_s f'_s}{\alpha_1 f_c \beta_1 b}$
Eq. 5.7.3.2.1-1	$M_r = \Phi M_n$	Eq. 5.6.3.2.1-1	NR
Eq. 5.7.3.2.2-1	$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right) - A'_s f'_s\left(d'_s - \frac{a}{2}\right) + 0.85f_c(b - b_w)h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$	Eq. 5.6.3.2.2-1	$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right) - A'_s f'_s\left(d'_s - \frac{a}{2}\right) + \alpha_1 f_c(b - b_w)h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$
Eq. 5.7.3.3.2-1	$M_{cr} = \gamma_3\left[(\gamma_1 f_r + \gamma_2 f_{cpe})S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)\right]$	Eq. 5.6.3.3-1	NR
5.7.3.4	Control of Cracking by Distribution of Reinforcement	5.6.7	NR
Eq. 5.7.3.4-1	$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$	Eq. 5.6.7-1	NR
Eq. 5.7.3.4-2	$A_{sk} \geq 0.012(d_l - 30) \leq \frac{A_s + A_{ps}}{4}$	Eq. 5.6.7-3	NR
5.7.5	Bearing	5.6.5	NR
Eq. 5.7.5-1	$P_r = \Phi P_n$	Eq. 5.6.5-1	NR
Eq. 5.7.5-2	$P_n = 0.85f_c A_1 m$	Eq. 5.6.5-2	NR
Eq. 5.7.5-3	$m = \sqrt{\frac{A_2}{A_1}} \leq 2.0$	Eq. 5.6.5-3	NR
5.8.2.1	Shear and Torsion – General Requirements – General	5.7.2.1	NR
Eq. 5.8.2.1-6	$V_{u\_eq} = \sqrt{V_u^2 + \left(\frac{0.9p_h T_u}{2A_o}\right)^2}$	Eq. B5.2-1	"Equivalent factored shear force" is placed into Appendix B5 as "effective shear force" with no revision in the equations
5.8.2.5	Shear and Torsion – Minimum Transverse Reinforcement	5.7.2.5	NR

AASHTO LRFD 2010		AASHTO LRFD 2017	
Section Number	**Title or Content**	**Section Number**	**Title or Content**
Eq. 5.8.2.5-1	$A_v \geq 0.0316\sqrt{f_c}\frac{b_v s}{f_y}$	Eq. 5.7.2.5-1	$A_v \geq 0.0316\lambda\sqrt{f_c}\frac{b_v s}{f_y}$
5.8.2.7	Shear and Torsion – Minimum Spacing of Transverse Reinforcement	5.7.2.6	NR
Eq. 5.8.2.7-1	$s_{max} = 0.8d_v \leq 24.0in$	Eq. 5.7.2.6-1	NR
Eq. 5.8.2.7-2	$s_{max} = 0.4d_v \leq 12.0in$	Eq. 5.7.2.6-2	NR
5.8.2.9	Shear and Torsion – Shear Stress on Concrete	5.7.2.8	NR
Eq. 5.8.2.9-2	$d_e = \frac{A_{ps}f_{ps}d_p + A_s f_y d_s}{A_{ps}f_{ps} + A_s f_y}$	Eq. 5.7.2.8-2	NR
Eq. 5.8.3.3-1	$V_n = V_c + V_s + V_p$	Eq. 5.7.3.3-1	NR
Eq. 5.8.3.3-2	$V_n = 0.25f_c b_v d_v + V_p$	Eq. 5.7.3.3-2	NR
Eq. 5.8.3.3-3	$V_c = 0.0316\beta\sqrt{f_c}b_v d_v$	Eq. 5.7.3.3-3	$V_c = 0.0316\beta\lambda\sqrt{f_c}b_v d_v$
Eq. 5.8.3.3-4	$V_s = \frac{A_v f_y d_v(cot\theta + cot\alpha)sin\alpha}{s}$	Eq. 5.7.3.3-4	NR
5.8.3.4.2	Shear and Torsion – Procedures for Determining Shear Resistance – General Procedure	5.7.3.4.2	Procedures for Determining Shear Resistance Parameter β and Θ - General Procedure
Eq. 5.8.3.4.2-1	$\beta = \frac{4.8}{(1+750\varepsilon_s)}$	Eq. 5.7.3.4.2-1	NR
Eq. 5.8.3.4.2-3	$\theta = 29 + 3500\varepsilon_s$	Eq. 5.7.3.4.2-3	NR
Eq. 5.8.3.4.2-4	$\varepsilon_s = \frac{\frac{\lvert M_u \rvert}{d_v} + 0.5N_u + \lvert V_u - V_p \rvert - A_{ps}f_{po}}{E_s A_s + E_p A_{ps}}$	Eq. 5.7.3.4.2-4	NR
Eq. 5.8.3.6.2-1	$T_n = \frac{2A_0 A_t f_y cot\theta}{s}$	Eq. 5.7.3.6.2-1	NR
5.8.4.1	Interface Shear Transfer – Shear Friction - General	5.7.4.1	NR
Eq. 5.8.4.1-1	$V_{ri} = \Phi V_{ni}$	Eq. 5.7.4.3-1	NR
Eq. 5.8.4.1-2	$V_{ri} \geq \Phi V_{ul}$	Eq. 5.7.4.3-2	NR
Eq. 5.8.4.1-3	$V_{ni} = cA_v + \mu(A_{vf}f_y + P_c)$	Eq. 5.7.4.3-3	NR
5.8.4.3	Cohesion and Friction Factors	5.7.4.4	NR

AASHTO LRFD 2010		AASHTO LRFD 2017	
Section Number	**Title or Content**	**Section Number**	**Title or Content**
Eq. 5.8.4.4-1	$A_{vf} \geq \frac{0.05A_{cv}}{f_y}$	Eq. 5.7.4.2-1	NR
5.11.2.4.2	Standard Hooks in Tension – Modification Factors	5.10.8.2.4b	NR
Eq. 5.11.2.4.1	$l_{hb} = \frac{38.0d_b}{\sqrt{f_c}}$	Eq. 5.10.8.2.4a-2	$l_{hb} = \frac{38.0d_b}{60.0}\left(\frac{f_y}{\sqrt{f_c}}\right)$
5.11.2.4.2	Standard Hooks in Tension – Modification Factors	5.10.8.2.4b	NR
5.13.2.4	Brackets and Corbels	5.8.4.2	NR
5.13.2.4.1	Brackets and Corbels – General	5.8.4.2.1	NR
Eq. 5.13.2.4.1-1	$M_u = V_u a_v + N_{uc}(h - d)$	Eq. 5.8.4.2.1-1	NR
5.13.2.4.2	Brackets and Corbels – Alternative to Strut-and-Tie Model	5.8.4.2.2	NR
Eq. 5.13.2.4.2-1	$V_n = 0.2f_c b_w d_e$	Eq. 5.8.4.2.2-1	NR
Eq. 5.13.2.4.2-2	$V_n = 0.8b_w d_e$	Eq. 5.8.4.2.2-2	NR
Eq. 5.13.2.4.2-5	$A_s \geq \frac{2A_{vf}}{3} + A_n$	Eq. 5.8.4.2.2-5	NR
Eq. 5.13.2.4.2-6	$A_h \geq 0.5(A_s - A_n)$	Eq. 5.8.4.2.2-6	NR
5.13.2.5.2	Beam Ledges – Design for Shear	5.8.4.3.2	NR
5.13.2.5.3	Beam Ledges – Design for Flexure and Horizontal Force	5.8.4.3.3	NR

AASHTO LRFD 2010		AASHTO LRFD 2017	
Section Number	**Title or Content**	**Section Number**	**Title or Content**
5.13.2.5.4	Beam Ledges – Design for Punching Shear $\Phi V_n = \Phi 0.125\sqrt{f_c}(W + 2L + 2d_f) * d_f$ $\Phi V_n = \Phi \min(0.125\sqrt{f_c}\left(\frac{1}{2}W + L + d_f + c\right) d_f, 0.125\sqrt{f_c}\,(W + 2L + 2d_f) \cdot d_f)$	5.8.4.3.4	$\Phi V_n = \Phi \cdot \lambda \cdot 0.125 \cdot \sqrt{f_c} \cdot (W + 2L + 2d_f) \cdot d_f$ $\Phi V_n = \Phi \cdot \lambda \cdot \min(0.125 \cdot \sqrt{f_c} \cdot \left(\frac{1}{2}W + L + d_f + c\right) \cdot d_f, 0.125 \cdot \sqrt{f_c} \cdot (W + 2L + 2d_f) \cdot d_f)$
5.13.2.5.5	Beam Ledges – Design of Hanger Reinforcement	5.8.4.3.5	NR
Eq. 5.13.2.5.5-1	$V_n = \frac{A_{hr}(0.5f_y)}{s}(W + 3a_v)$	Eq. 5.8.4.3.5-1	The equation has not changed. However, there is a limitation which $(W + 3a_v) < \min(S, 2c)$
Eq. 5.13.2.5.5-2	$V_n = \frac{A_{hr}f_y}{s}S$	Eq. 5.8.4.3.5-2	The equation has not changed. However, there is a limitation which $S < 2c$
Eq. 5.13.2.5.5-3	$V_n = (0.063\sqrt{f_c}b_f d_f) + \frac{A_{hr}f_y}{s}(W + 2d_f)$	Eq. 5.8.4.3.5-3	$V_n = (0.063\lambda\sqrt{f_c}b_f d_f) + \frac{A_{hr}f_y}{s}(W + 2d_f)$
Appendix B5	General Procedure for Shear Design with Tables	NR	NR
Eq. B5.2-1	$\varepsilon_x = \frac{\frac{\lvert M_u \rvert}{d_v} + 0.5N_u + 0.5\lvert V_u - V_p \rvert cot\theta - A_{ps}f_{po}}{2(E_s A_s + E_p A_{ps})}$	Eq. B5.2-3	NR
Eq. B5.2-3	$\varepsilon_x = \frac{\frac{\lvert M_u \rvert}{d_v} + 0.5N_u + 0.5\lvert V_u - V_p \rvert cot\theta - A_{ps}f_{po}}{2(E_c A_c + E_s A_s + E_p A_{ps})}$	Eq. B5.2-5	NR
Table B5.2-1	Values of Θ and β for Sections with Transverse Reinforcement	NR	NR
-	This section is not included in AASHTO LRFD 2010	5.4.2.8	Concrete Density Modification Factor
-	The equation for the elastic modulus of concrete in AASHTO LRFD 2010 is placed into commentary	Eq. C5.4.2.4-2	$E_c = 33000K_1 w_c^{1.5}\sqrt{f_c}$

AASHTO LRFD 2010		AASHTO LRFD 2017	
Section Number	**Title or Content**	**Section Number**	**Title or Content**
-	The equation for the elastic modulus of concrete in AASHTO LRFD 2010 is placed into commentary	Eq. C5.4.2.4-3	$E_c = 1820\sqrt{f_c}$