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

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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.

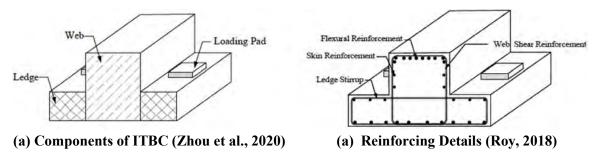


Figure 1.1 Design Detail of ITBC

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	430	330	330
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

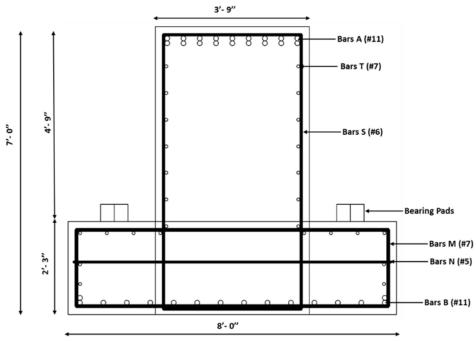


(b) Existing old skewed bridge

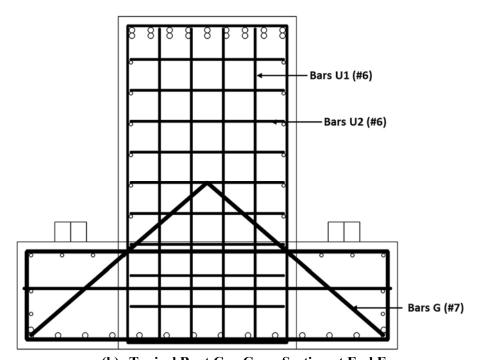
Figure 2.1 Proposed bridge on Donigan Road over IH 10 near Brookshire in Waller County

2.2 FINITE ELEMENT MODELING OF BENT CAPS IN ABAQUS

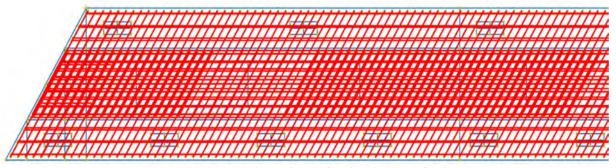
The finite element models of the actual ITBCs were developed using ABAQUS (2014). Figure 2.2(a) and Figure 2.2(b) show the typical cross-sectional view with reinforcing details of all the bent caps at the inner and end face locations, respectively. A partial plan view of the bent caps showing the transverse 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

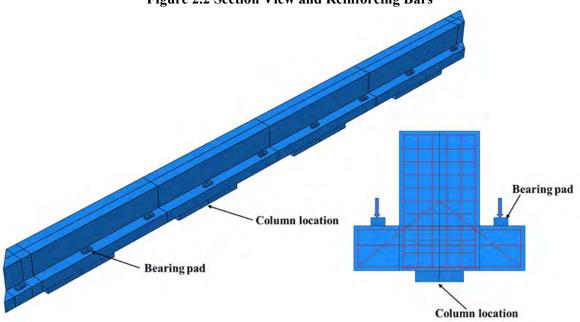


(b) Typical Bent Cap Cross Section at End Face

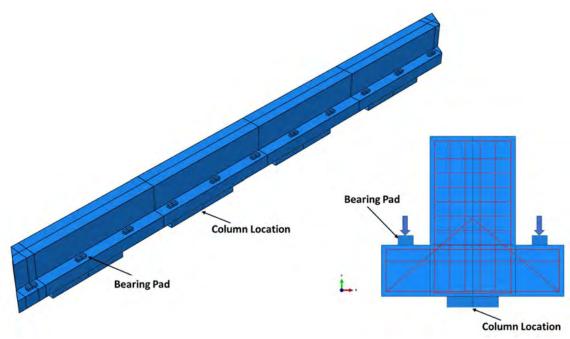


(c) Plan View of Reinforcing in ABAQUS

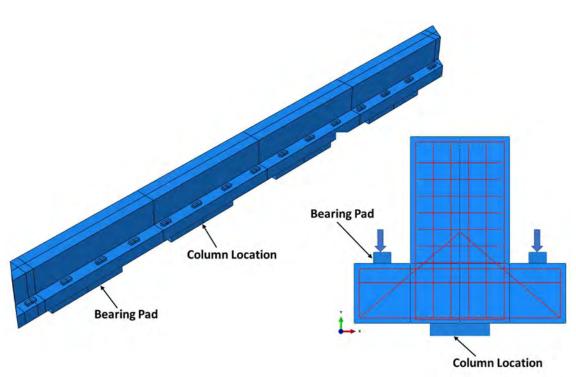
Figure 2.2 Section View and Reinforcing Bars



(a) Finite Element Model of Bent Cap 2 with Skew Angle 430



(b) Finite Element Model of Bent Cap 6 with Skew Angle 330



(c) Finite Element Model of Bent Cap 7 with Skew Angle 33⁰ Figure 2.3 3D FE Model of Bent Caps in ABAQUS

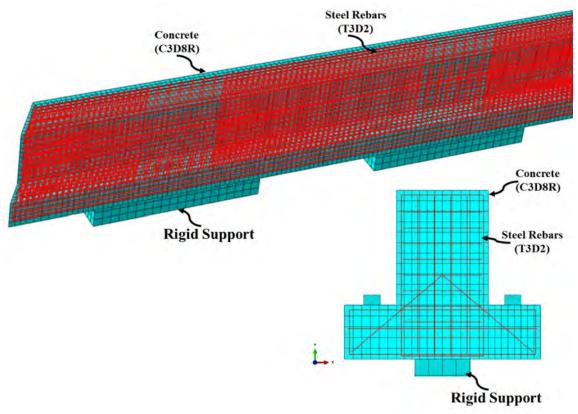


Figure 2.4 Partial 3D Finite Element Mesh of a Bent Cap (C3D8R Solid Element for Concrete and T3D2 Truss Element for Reinforcements)

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.

$$\sigma_{c} = f_{c}^{'} \left[\frac{2\varepsilon_{c}}{\varepsilon_{0}} - \left(\frac{\varepsilon_{c}}{\varepsilon_{0}} \right)^{2} \right]$$
(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 v. The initial elastic modulus E_c can be calculated using the AASHTO empirical equation (AASHTO 2014):

$$E_c = 57000 \sqrt{f_c'} \text{ (psi)}$$
 (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 v = 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:

$$\sigma_{c} = E_{c} \varepsilon_{c} \varepsilon_{c} \le \varepsilon_{cr} \tag{Eq. 2-3}$$

Descending branch:

$$\sigma_{c} = f_{cr} \left(\frac{\varepsilon_{cr}}{\varepsilon_{c}}\right)^{0.4} \varepsilon_{c} > \varepsilon_{cr}$$
 (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 0.00008 E_c .

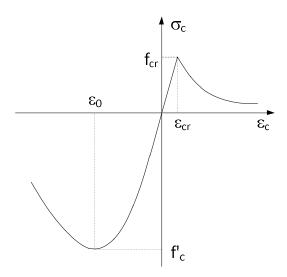


Figure 2.5 Stress-Strain Curves of Concrete in Tension and Compression

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:

$$f_s = E_s \varepsilon_s \ \varepsilon_s \le \varepsilon_v$$
 (Eq. 2-5)

Plastic branch:

$$f_s = f_v \varepsilon_s > \varepsilon_v$$
 (Eq. 2-6)

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

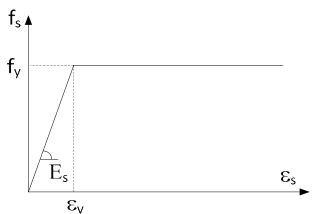


Figure 2.6 Stress-Strain Curve of Mild Steel

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

Specimen Young's Poisson's Compressive Tensile Dilation Flow K designation modulus ratio strength (ksi) strength angle (°) potential (ksi) eccentricity (ksi) 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

Table 2.1 Material Parameters for the Concrete Damaged Plasticity Model

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

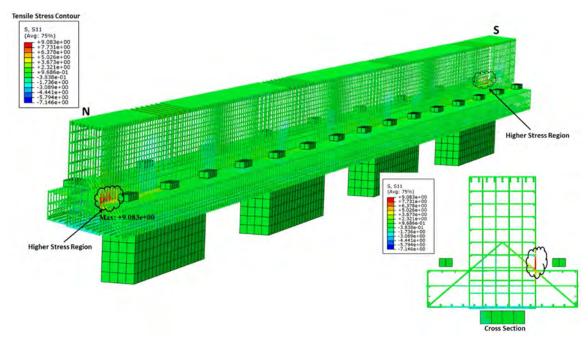


Figure 2.7 Tensile Stress Contour at Service Load of Bent Cap 2

[S11 = Tensile stresses in ksi in Rebars]

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

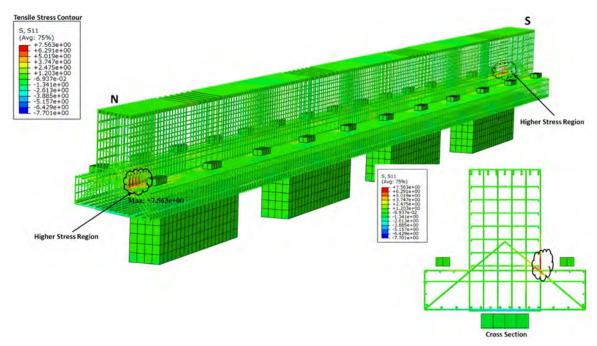


Figure 2.8 Tensile Stress Contour at Service Load of Bent Cap 6

[S11 = Tensile stresses in ksi in Rebars]

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

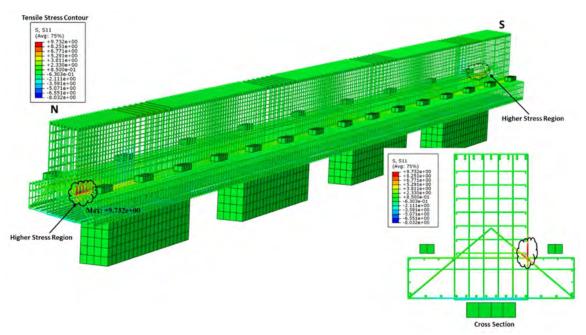


Figure 2.9 Tensile Stress Contour at Service Load of Bent Cap 7

[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.

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

Table 2.3 Strength Limit State Loading for Bent Caps

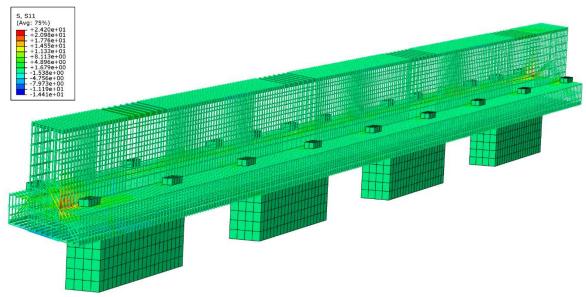


Figure 2.10 Tensile Stress Contour at Strength Limit State of Bent Cap 2

[S11 = Tensile stresses in ksi in Rebars]

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

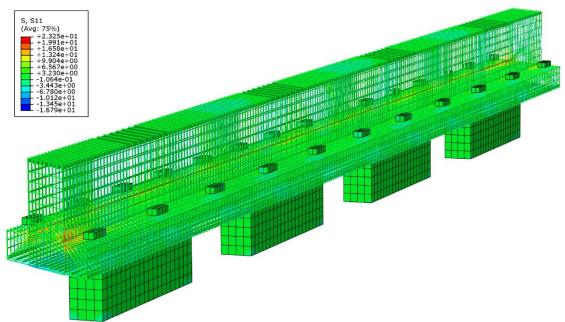


Figure 2.11 Tensile Stress Contour at Strength Limit State of Bent Cap 6

[S11 = Tensile stresses in ksi in Rebars]

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

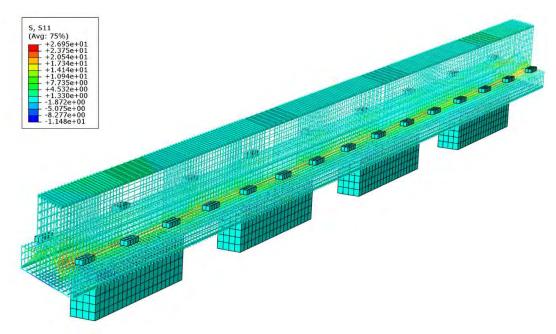


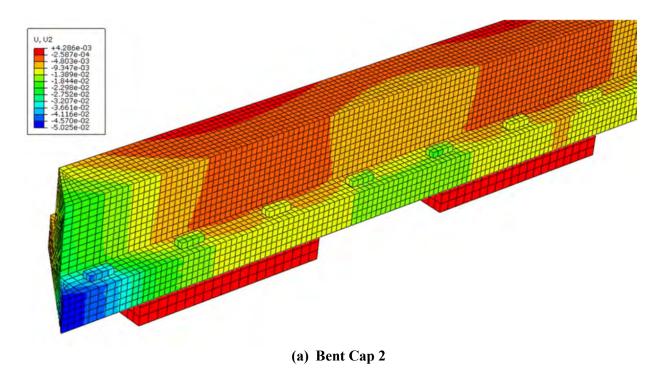
Figure 2.12 Tensile Stress Contour at Strength Limit State of Bent Cap 7

[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.



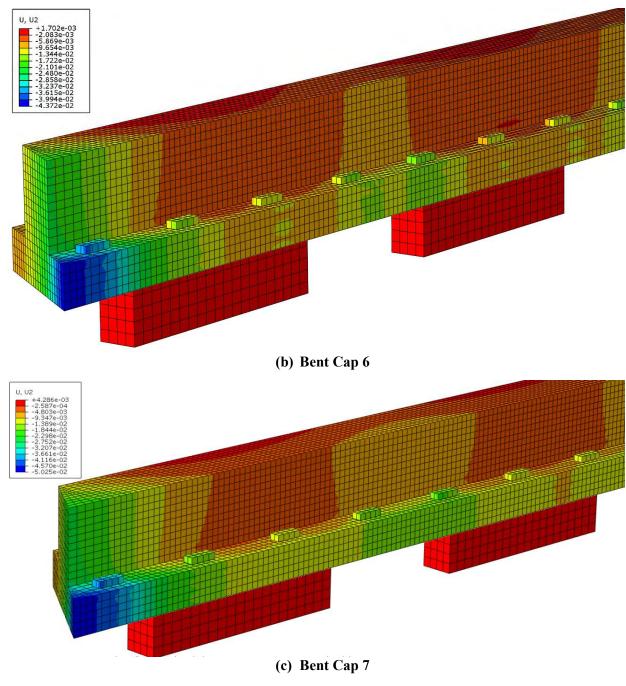
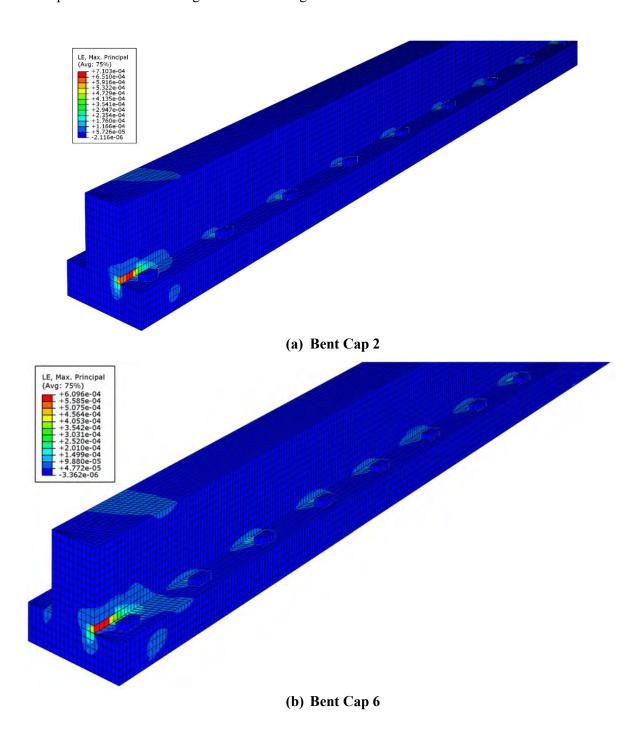


Figure 2.13 Displacement at Service Load for Bent Caps

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.



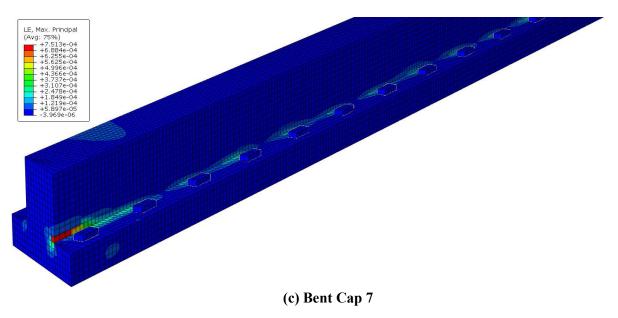


Figure 2.14 Comparison of Principal Tensile Strain at Service load

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.

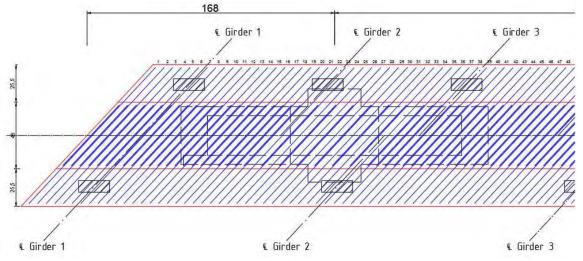


Figure 3.1 Case 1 for Bent 2 (Current Design of Skew Reinforcement, unit: inch)

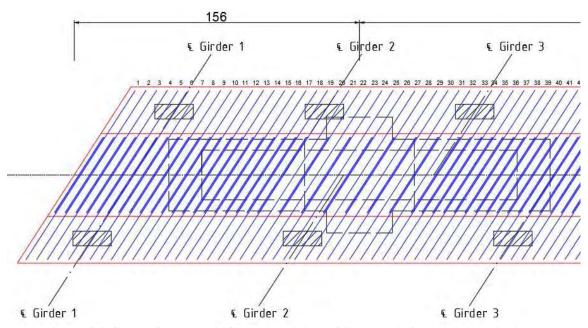


Figure 3.2 Case 1 for Bent 6 (Current Design of Skew Reinforcement, unit: inch)

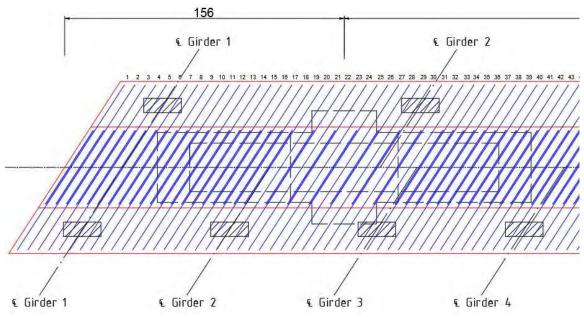


Figure 3.3 Case 1 for Bent 7 (Current Design of Skew Reinforcement, unit: inch)

(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.

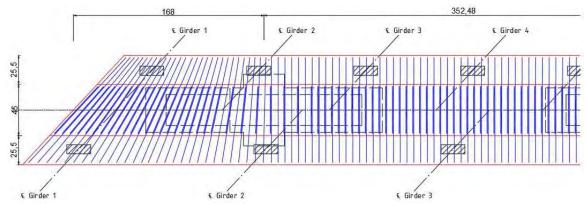


Figure 3.4 Case 2 for Bent 2 (Traditional Detailing of Reinforcement without End Bars, unit: inch)

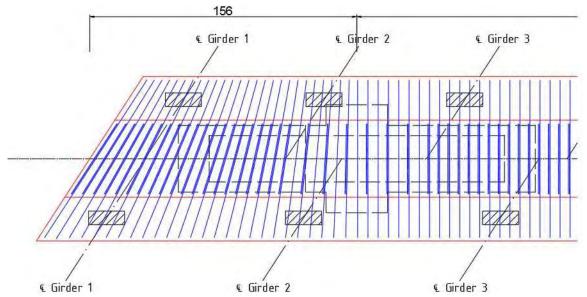


Figure 3.5 Case 2 for Bent 6 (Traditional Detailing of Reinforcement without End Bars, unit: inch)

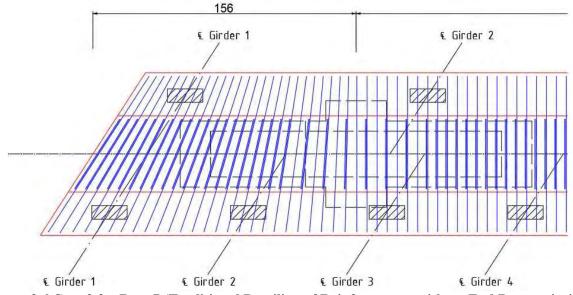


Figure 3.6 Case 2 for Bent 7 (Traditional Detailing of Reinforcement without End Bars, unit: inch)

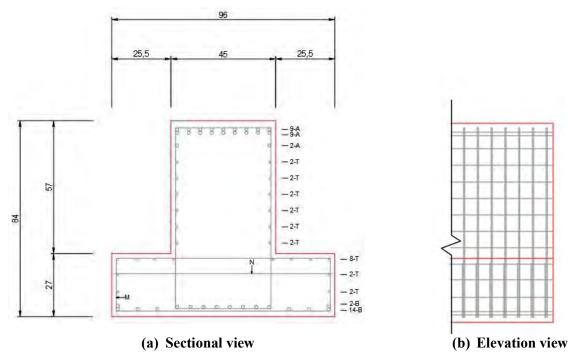


Figure 3.7 Bent 2-End View of Traditional Design Without End Bars in Case 2 (unit: inch)

(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.

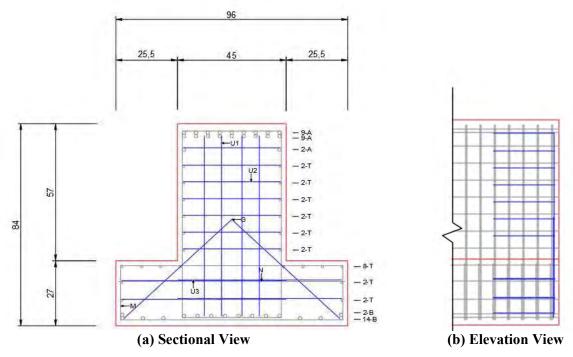


Figure 3.8 Bent 2-End View of Traditional Design with End Bars in Case 3 (unit: inch).

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

			I	Bent Ca	р	Cone Stree	ngth	Transv	erse Reinforce	ment Detailing	Amou	Amount of Transverse Rebar				G Ba	ır S	ize
No.	Name	Case	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	higher than current design	40% higher than current design	#3	#4 7	¥5 1	#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			

			F	Bent Ca	p		crete ngth si)	Transv	verse Reinforce	ment Detailing	Amount of Transverse Rebar			oar		G B	ar S	Size	
No.	Name	Case	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						

			F	Bent Ca	р	Cond Stren	ngth	Transv	rerse Reinforce	ment Detailing	Amou	nt of Trans	verse Rel	oar		G B	ar S	ize	
No.	Name	Case	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	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			

			I	Bent Ca	p		crete ngth si)	Transv	erse Reinforce	ment Detailing	Amoui	mount of Transvers		oar	(G B	ar S	Size
No.	Name	Case	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	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.

Concrete grade	Young's modulus (ksi)	Poisson's ratio	Tensile strength (ksi)	Density (<u>lb/ft³</u>)	Dilation angle (°)	Flow potential eccentricity	K
5 ksi	4031	0.2	0.325	150	31	0.1	0.67

150

31

0.1

0.67

0.382

Table 3.2 Material Parameters for the Concrete Damaged Plasticity Model

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

0.2

4770

7 ksi

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.

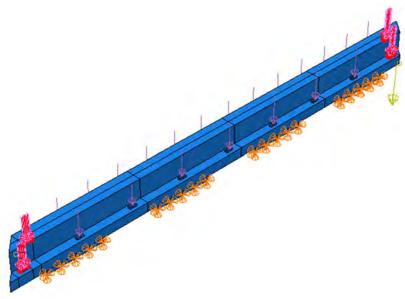
 Bent
 Service Load at Interior Bearing Pads (kips)
 Service Load at Exterior Bearing Pads (kips)

 Bent 2
 222.48
 240.19

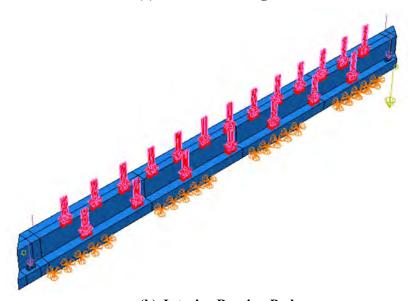
 Bent 6
 226.64
 238.86

 Bent 7
 244.52
 258.00

Table 3.3. Service Load for Bent Caps



(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.

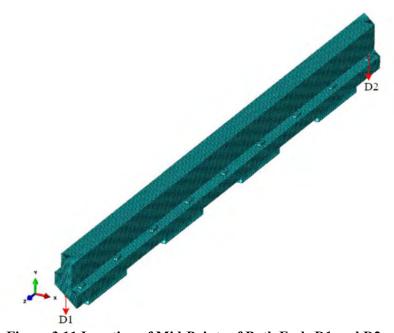


Figure 3.11 Location of Mid-Points of Both Ends D1 and D2

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

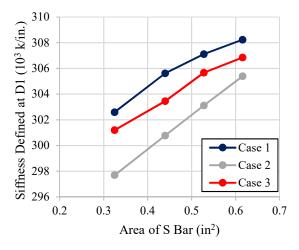
The total vertical load is the summation of the service load on the interior and exterior bearing pads and is calculated as 5413 lb, 5950 lb, and 5920 lb for Bent Cap 2, Bent Cap 6, and Bent Cap 7, respectively. The stiffness is calculated by the following equation.

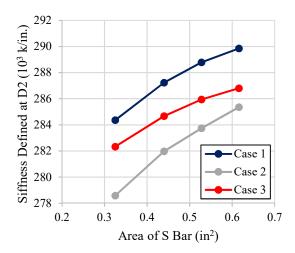
$$k = \frac{F}{\Delta}$$
 (Eq. 3-1)

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

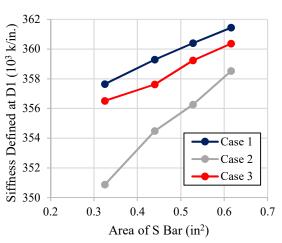
Figure 3.12 shows the comparison of stiffness values of the specimens for each bent cap at points D1 and D2. Based on the FE analysis results, the stiffness slightly increases with increasing the S Bar area because the S Bars reduce the tensile strain of the bent caps. In addition, increasing the concrete 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.

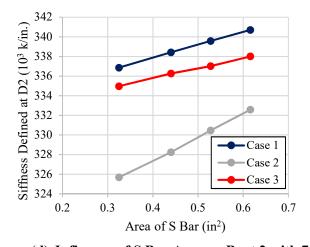




(a) Influence of S Bar Area on Bent 2 with 5 ksi Concrete at D1

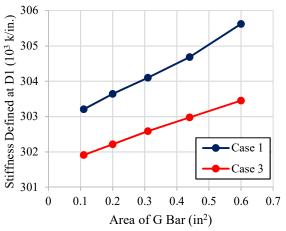


(b) Influence of S Bar Area on Bent 2 with 5 ksi Concrete at D2

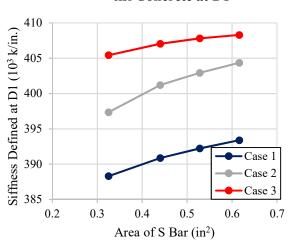


(c) Influence of S Bar Area on Bent 2 with 7 ksi Concrete at D1

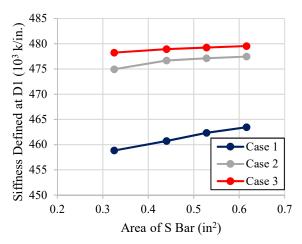
(d) Influence of S Bar Area on Bent 2 with 7 ksi Concrete at D2



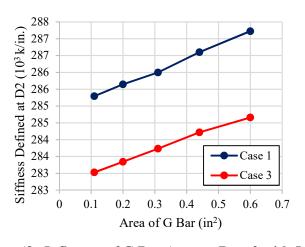
(e) Influence of G Bar Area on Bent 2 with 5 ksi Concrete at D1



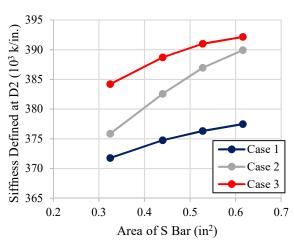
(g) Influence of S Bar Area on Bent 6 with 5 ksi Concrete at D1



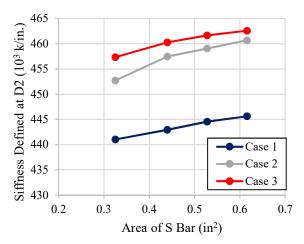
(i) Influence of S Bar Area on Bent 6 with 7 ksi Concrete at D1



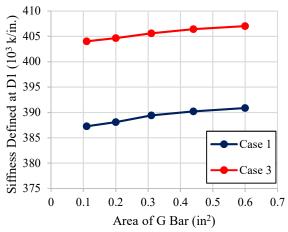
(f) Influence of G Bar Area on Bent 2 with 5 ksi Concrete at D2

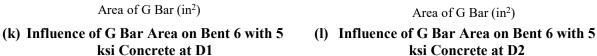


(h) Influence of S Bar Area on Bent 6 with 5 ksi Concrete at D2



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





0.1

0.2

0.3

390

385

380

375

370

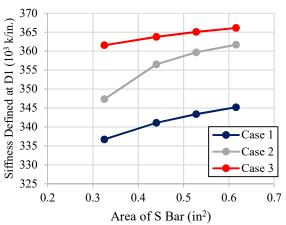
365

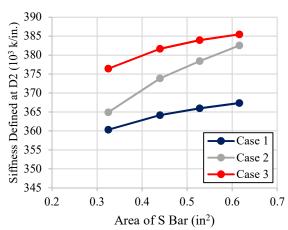
360

355

0

Siffness Defined at D2 (10³ k/in.)





0.4

0.5

Case 1

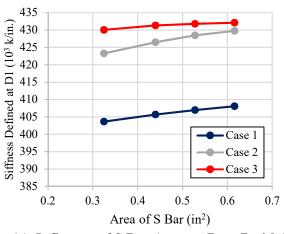
Case 3

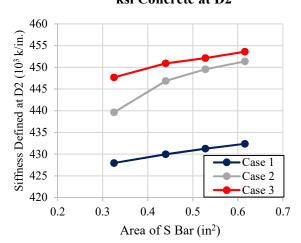
0.7

0.6

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

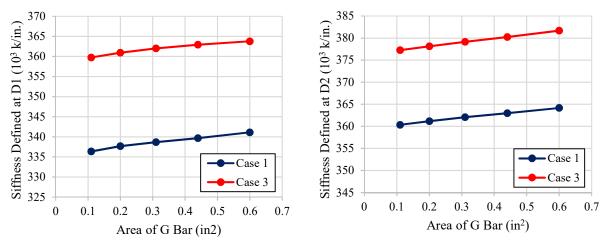
(n) Influence of S Bar Area on Bent 7 with 5 ksi Concrete at D2





(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

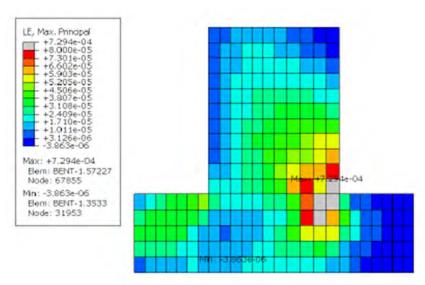


- (q) Influence of G Bar Area on Bent 7 with 5 ksi Concrete at D1
- (r) Influence of G Bar Area on Bent 7 with 5 ksi Concrete at D2

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

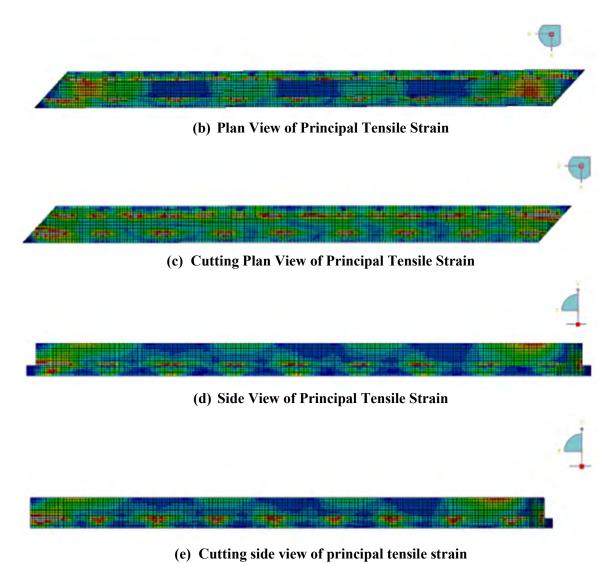


Figure 3.13 Principal Tensile Strains in Current Design of Bent 2 at the Service Load (Specimen C3B2C5S0)

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

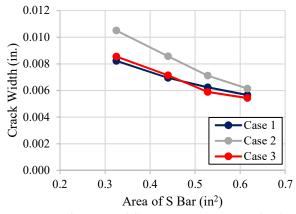
the increasing G Bar area. Increasing the S Bar area and the compressive strength of concrete notably decreases the crack width.

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

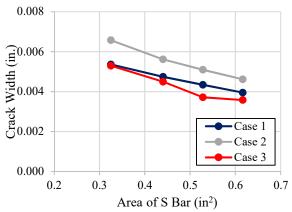
No.	Name	Maximum Tensile Strain	Maximum Crack Width (in.)
1	C1B2C5Smin	0.000833	0.0082
2	C1B2C5S0	0.000711	0.0069
3	C1B2C5S20	0.000644	0.0062
4	C1B2C5S40	0.000589	0.0057
5	C1B2C7Smin	0.000571	0.0054
6	C1B2C7S0	0.000511	0.0047
7	C1B2C7S20	0.000473	0.0043
8	C1B2C7S40	0.000436	0.0040
9	C1B6C5Smin	0.000700	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

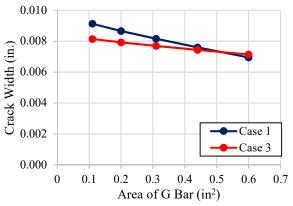
No.	Name	Maximum Tensile Strain	Maximum Crack Width (in.)
77	C3B7C5Smin	0.000866	0.0086
78	C3B7C5S0	0.000713	0.0070
79	C3B7C5S20	0.000677	0.0066
80	C3B7C5S40	0.000659	0.0064
81	C3B7C7Smin	0.000588	0.0055
82	C3B7C7S0	0.000523	0.0049
83	C3B7C7S20	0.000516	0.0048
84	C3B7C7S40	0.000507	0.0047
85	C3B2C5G3	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



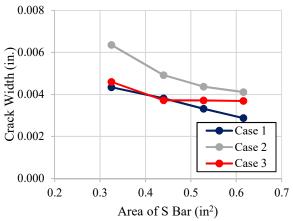
(a) Influence of S Bar Area on Bent 2 with 5 ksi Concrete



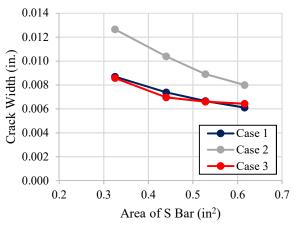
(b) Influence of S Bar Area on Bent 2 with 7 ksi Concrete



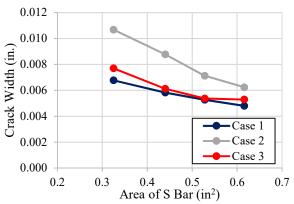
(c) Influence of G Bar Area on Bent 2 with 5 ksi Concrete



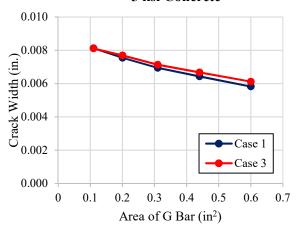
(e) Influence of S Bar area on Bent 6 with 7 ksi Concrete



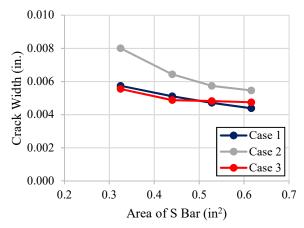
(g) Influence of S Bar area on Bent 7 with 5 ksi Concrete



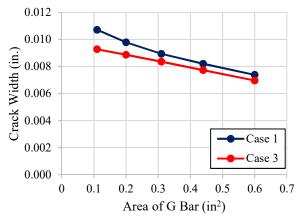
(d) Influence of S Bar Area on Bent 6 with 5 ksi Concrete



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



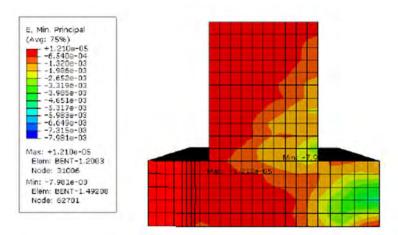
(h) Influence of S Bar area on Bent 7 with 7 ksi Concrete



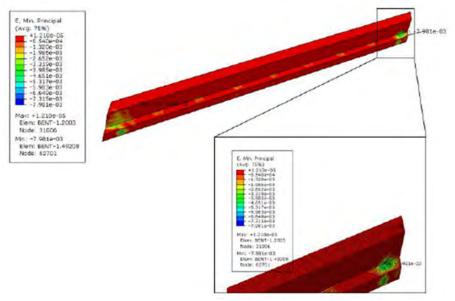
(i) Influence of G Bar area on Bent 7 with 5 ksi Concrete Figure 3.14 Comparison of Crack Width at the Service Load

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.



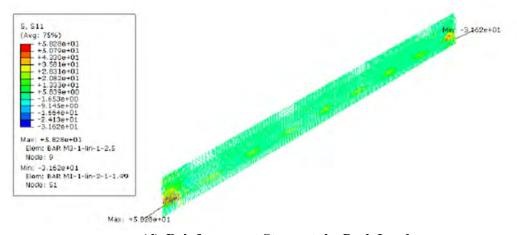
(a) Sectional View of Principal Compressive Strain of Concrete



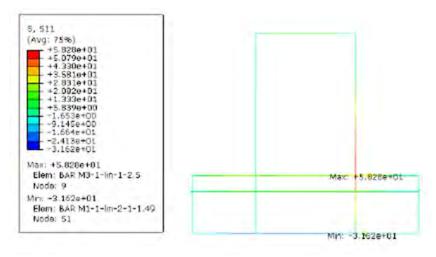
(b) Local View of Principal Compressive Strain of Concrete



(c) Plan View of Principal Compressive Strain of Concrete



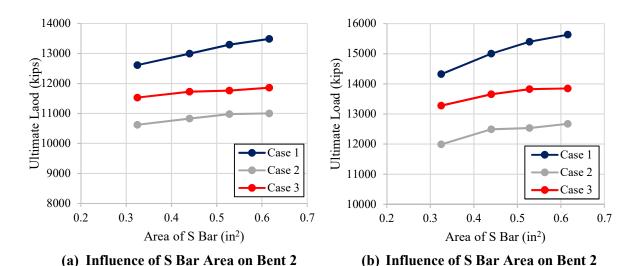
(d) Reinforcement Stress at the Peak Load



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

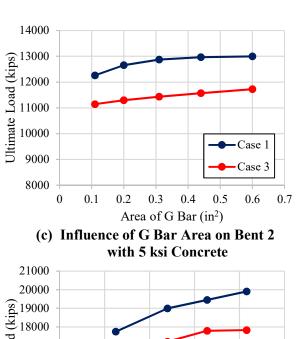
Figure 3.15 Stress and Strain Contours in Specimen C3B2C5S0 at the Ultimate Load

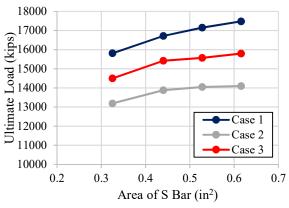
The ultimate capacity of specimens is compared in Figure 3.16. The ultimate capacity of specimens notably increases with the increase of the S Bar area and concrete compressive strength. In addition, the capacity of Case 2 and Case 3 are notably lower than Case 1, which indicates the rebar detailing 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.

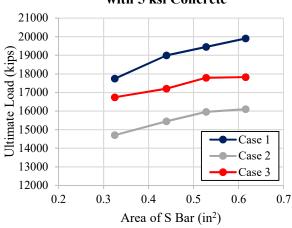


with 7 ksi Concrete

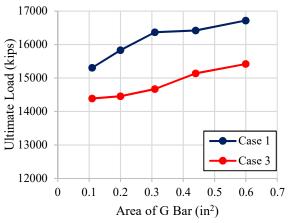
with 5 ksi Concrete



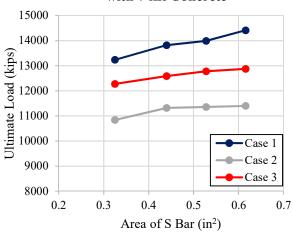




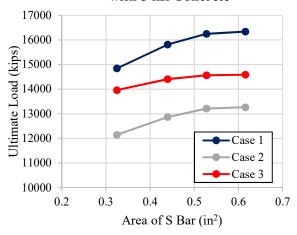
(d) Influence of S Bar Area on Bent 6 with 5 ksi Concrete



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

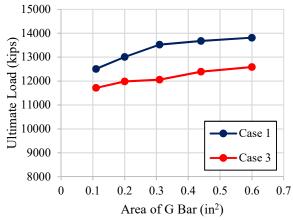


(f) Influence of G Bar Area on Bent 6 with 5 ksi Concrete



(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						
В	16	# 11	1.56	1389	9863						
Т	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	i)	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	
В	16	# 11	1.56	1389	9863	
Т	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							
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					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	
В	16	# 11	1.56	1389	9863	
Т	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				Volume (cy)		
Class "F" Concrete (Cap)			5000	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		Unit		
Item	Skewed	Traditional	Unit	
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 L	abors	Concrete Labors		
Item	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		Unit	
Item	Skewed	Traditional	Unit
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	

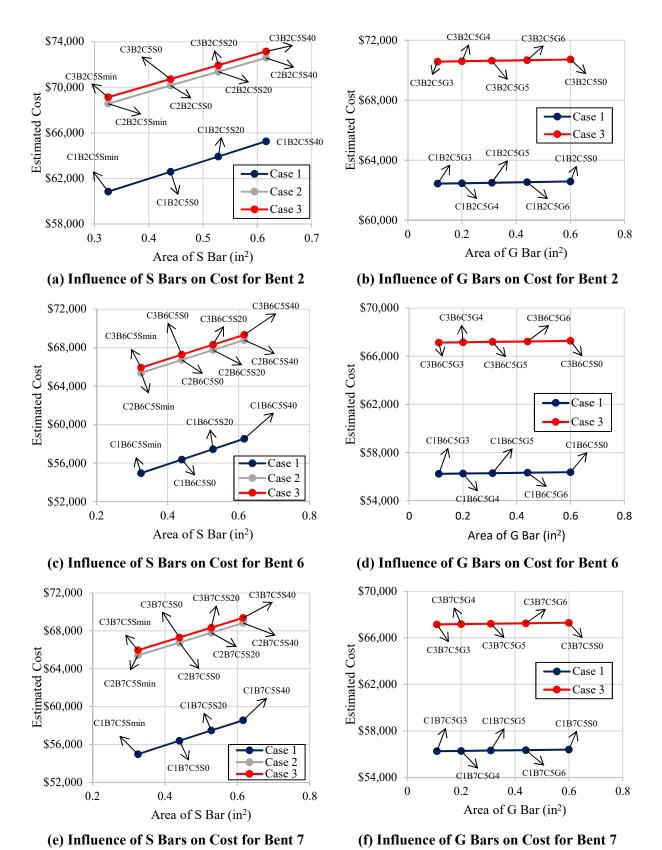


Figure 3.17 Comparison of Estimated Cost for Case 1, Case 2, and Case 3

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.

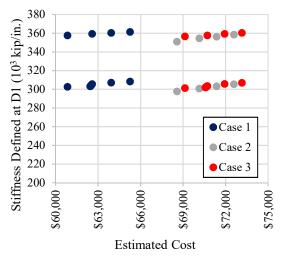
Table 3.14 Cost-Benefit Analysis Results

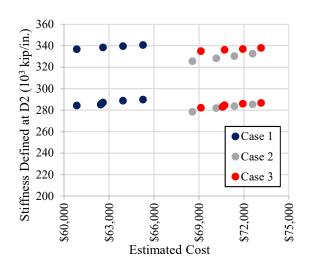
No.	Name	Cost	Stiffness defined at D1 (10 ³ kip/in.)	Stiffness defined at D2 (10 ³ kip/in.)	Crack Width (in.)	Ultimate Load (kips)
1	C1B2C5Smin	\$60,839	302.6	284.4	0.0082	12613
2	C1B2C5S0	\$62,585	305.6	287.2	0.0069	12997
3	C1B2C5S20	\$63,921	307.1	288.8	0.0062	13293
4	C1B2C5S40	\$65,257	308.2	289.8	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	0.0057	13266
61	C3B2C5Smin	\$69,129	301.2	282.3	0.0086	11530
62	C3B2C5SIIIII	\$70,717	303.5	284.7	0.0071	11725
63	C3B2C5S20	\$70,717	305.7	285.9	0.0071	11764
64	C3B2C5S40	\$73,148	306.9	286.8	0.0054	11859
65	C3B2C7Smin	\$69,129	356.5	335.0	0.0053	13277
66	C3B2C7SIIIII	\$70,717	357.6	336.3	0.0033	13653
67	C3B2C7S20	\$70,717	359.2	337.0	0.0043	13823
68	C3B2C7S40	\$73,148	360.4	338.0	0.0037	13846
69	C3B6C5Smin	\$65,927	405.4	384.2	0.0030	14496
70	C3B6C5SIIIII	\$67,275	407.0	388.7	0.0077	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	0.0033	16743
74	C3B6C7SIIIII	\$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	0.0076	12782
80	C3B7C5S40	\$69,366	366.1	385.5	0.0064	12874
81	C3B7C7Smin	\$65,952	430.1	447.7	0.0055	13962
82	C3B7C7SIIIII	\$67,301	431.4	450.9	0.0049	14408
83	C3B7C7S20	\$68,333	431.8	452.1	0.0049	14566
84	C3B7C7S40	\$69,366	432.1	453.6	0.0048	14589
85	C3B2C5G3	\$70,573	301.9	283.0	0.0047	11144
86	C3B2C5G4	\$70,573	302.2	283.3	0.0079	11294
87	C3B2C5G5	\$70,632	302.6	283.7	0.0079	11432
88	C3B2C5G6	\$70,632	303.0	284.2	0.0074	11568
89	C3B6C5G3	\$67,137	404.0	384.7	0.0074	14389
90	C3B6C5G4	\$67,162	404.7	385.4	0.0081	14458
91	C3B6C5G5	\$67,193	405.6	386.7	0.0077	14669
92	C3B6C5G6	\$67,230	406.4	387.6	0.0071	15141
93	C3B7C5G3	\$67,163	359.7	377.3	0.0093	11717
73	CODICOUS	\$07,103	339.1	311.3	0.0093	11/1/

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

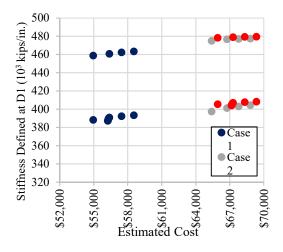
Figure 3.18 shows the cost and stiffness comparison of the specimens. In Figure 3.18, each point stands for the result of a specimen in the parametric analysis. Case 1 is marked by blue, Case 2 is marked by gray, and Case 3 is marked by red. For Bent Cap 2, the stiffness value of Case 1 is slightly higher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

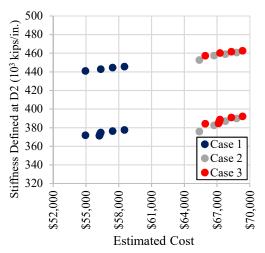




(a) Cost versus stiffness of Bent 2 defined at D1







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

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

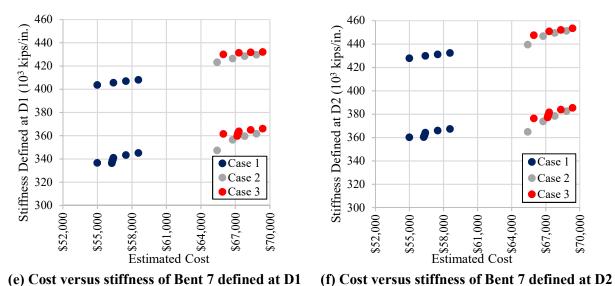
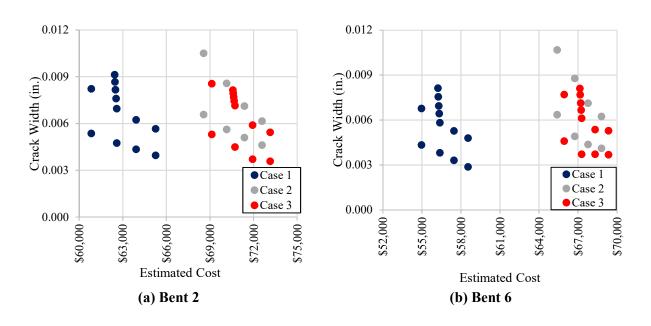


Figure 3.18 Cost and Stiffness Comparison of Bent 2, Bent 6, and Bent 7

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.



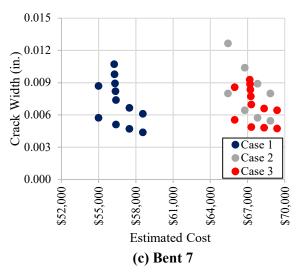
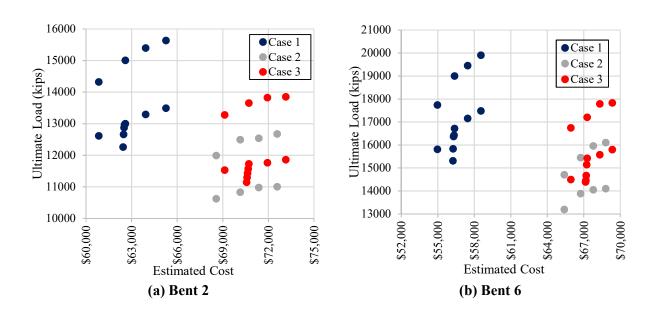


Figure 3.19 Cost and Crack Width Comparisons of Bent 2, Bent 6, and Bent 7

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.



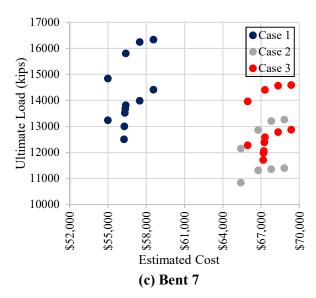
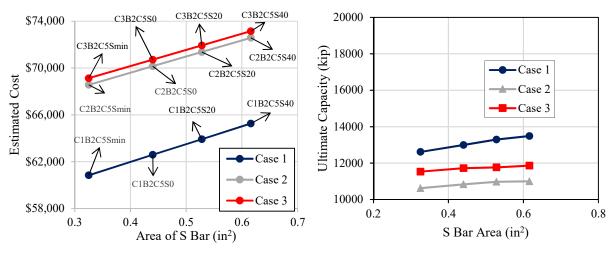


Figure 3.20 Cost and Ultimate Load Comparisons of Bent 2, Bent 6, and Bent 7

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

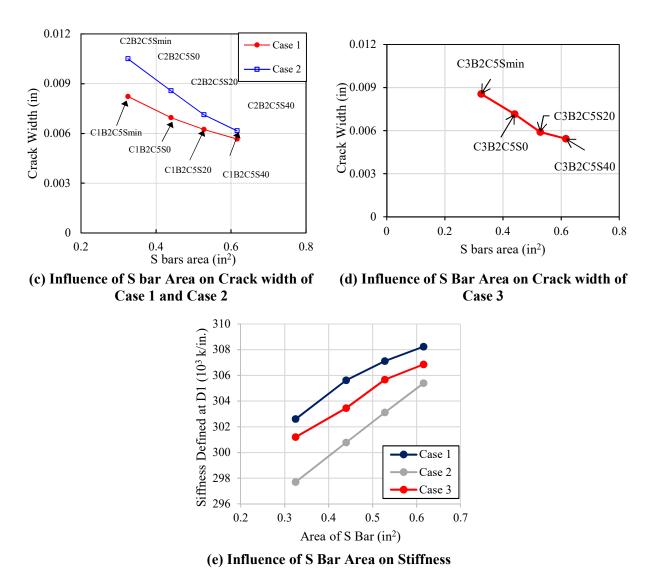


Figure 3.21 Influence of S Bar Area on Cost and Performance of Bent 2 with 5 ksi concrete

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.

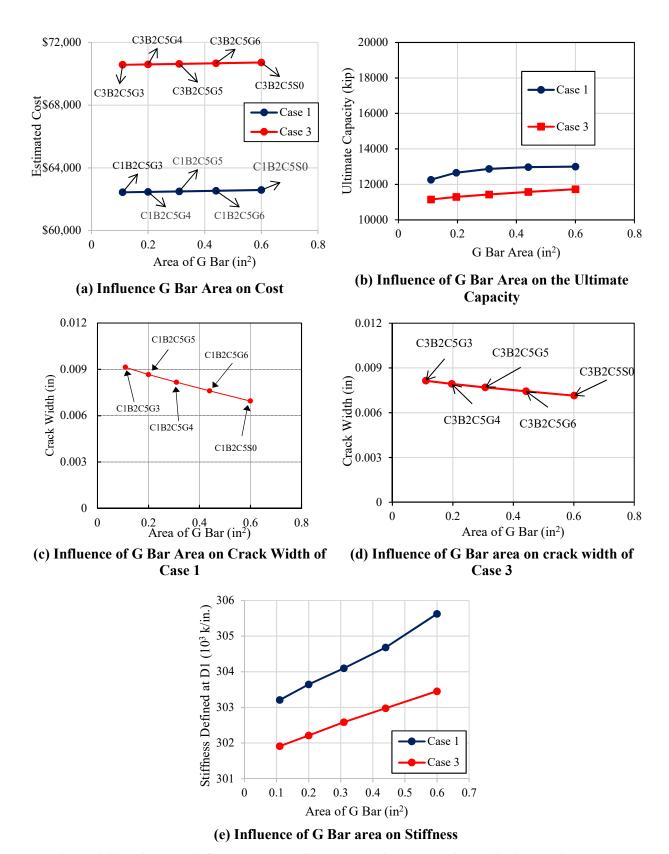


Figure 3.22 Influence of G Bar Area on Cost and Performance of Bent 2 with 5 ksi concrete

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:

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.

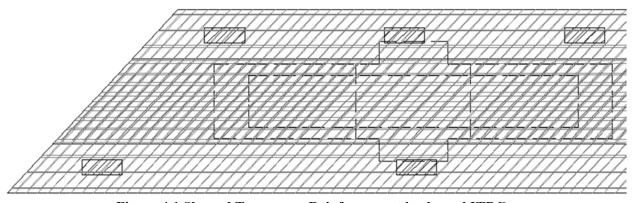


Figure 4.1 Skewed Transverse Reinforcement in skewed ITBCs

- 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.

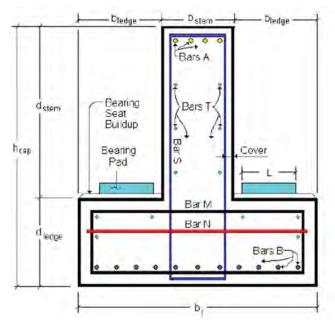


Figure 4.2 Typical Section View of ITBCs

- 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

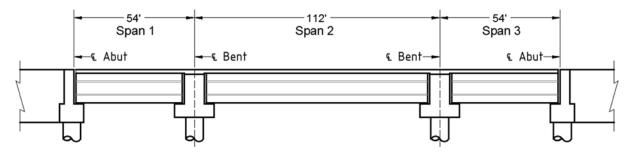


Figure 4.3 Spans of the Bridge with 0 Degree Skewed ITBC

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

 $f_c' = 5 \text{ ksi}$

 $w_c = 150 \text{ pcf (for weight)}$

 $w_c = 145 \text{ pcf (for Modulus of Elasticity calculation)}$

"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

$$F_y = 60 \text{ ksi}$$

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

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
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 = 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 = 0.372 klf		Weight of Rail
		Unit Weight of Concrete for Loads
$w_c = 0.150 kcf$		Unit Weigh of Overlay
$w_{Olay} = 0.140 \text{kcf}$		
Bents		Concrete Strength
$f_c = 5ksi$		Unit Weight of Concrete for E_c
$w_{cE} = 0.145kcf$		Modulus of Elasticity of Conquete
$E_{c} = 33000 \cdot w_{cE}^{1.5} \cdot $	$\overline{f_c}$ $E_c = 4074 \text{ ksi}$	Modulus of Elasticity of Concrete (AASHTO LRFD Eq. C5.4.2.4-2)
$f_y = 60$ ksi		Yield Strength of Reinforcement
$E_s = 29000ksi$		Modulus of Elasticity of Steel
$D_{column} = 36in$		Diameter of Columns

IM = 33%

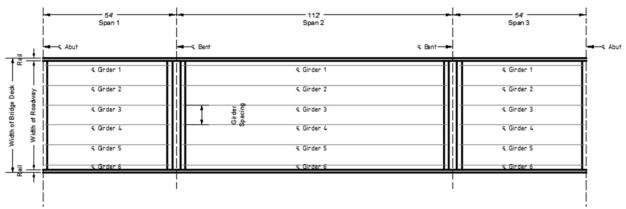


Figure 4.4 Top View of the 0 Degree Skewed ITBC with Spans and Girders

4.2.2 Determine Cap Dimensions

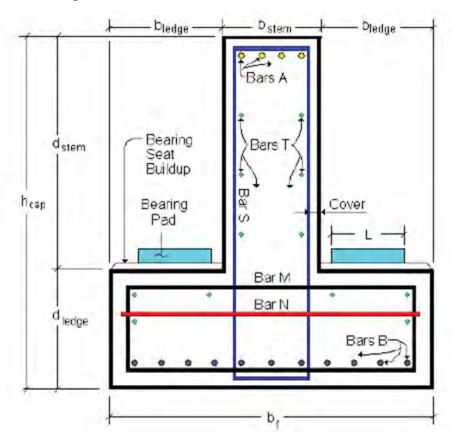


Figure 4.5 Section View of 0 Degree Skewed ITBC

4.2.2.1 Stem Width

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

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

4.2.2.2 Stem Height

Distance from Top of Slab to Top of Ledge:

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)

Haunch2 is the larger of the two haunches.

 $D_{Slab \ to \ Ledge} = SlabThk + Haunch2 + GirderD + BrgPad + BrgSeat$

 $D_{Slab_to_Ledge} = 70.00 \text{ in}$

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.

$$d_{stem} = D_{Slab_to_Ledge} - SlabThk - StemHaunch - 0.5in$$

$$d_{stem} = 57.75 in$$

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

4.2.2.3 Ledge Width

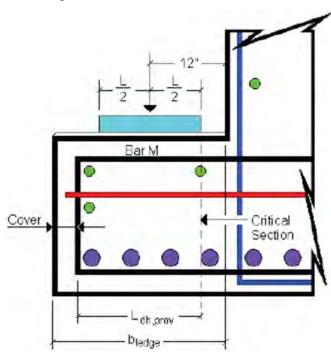


Figure 4.6 Ledge Section of 0 Degree ITBC

cover = 2.5 in

L = 8 in

Determine the Required Development Length of Bar M:

Try # 6 Bar for Bar M.

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

$$A_{bar\ M} = 0.44 \text{ in}^2$$

Basic Development Length

$$L_{dh} = \frac{^{38.0 \cdot d_{bar_M}}}{^{60}} \cdot \left(\frac{f_y}{\sqrt{f_c}}\right) \qquad \qquad L_{dh} = 12.75 \text{ in}$$

(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"?

The stem must accommodate 1/2" of bituminous fiber.

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

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

" $L_{dh,prov}$ " 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 $8" \times 21"$ as shown in the IGEB standard.

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

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

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, $\lambda = 1.0$

"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.

(AASHTO LRFD 5.4.2.8)

(AASHTO LRFD 5.10.8.2.4a)

The Required Development Length:

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

Therefore,

$$L_{dh_req} = 12.75 \text{ in}$$

$$b_{ledge_min} = L_{dh_req} + cover + 12in - \frac{L}{2} \qquad \quad b_{ledge_min} = 23.25 \ in$$

Use:

$$b_{ledge} = 24 in$$

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

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.

Width of Bottom Flange:

$$b_f = 2 \cdot b_{ledge} + b_{stem}$$

$$b_f = 87 in$$

4.2.2.4 <u>Ledge Depth</u>

Use a Ledge Depth of 28".

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

Total Depth of Cap:

$$h_{cap} = d_{stem} + d_{ledge}$$

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

4.2.2.5 Summary of Cross-Sectional Dimensions

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

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

 $b_{ledge} = 24 in$

 $d_{ledge} = 28 in$

 $h_{cap} = 85 in$

4.2.2.6 Length of Cap

First define Girder Spacing and End Distance:

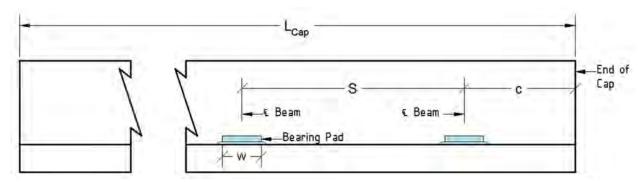


Figure 4.7 Elevation View of 0 Degree ITBC

$$S=8 \text{ ft} \\ c=2 \text{ ft} \\ \text{"c" is the distance from the Center} \\ \text{Line of the Exterior Girder to the} \\ \text{Edge of the Cap measured along} \\ \text{the Cap.} \\ L_{Cap}=S\cdot (\text{GdrNo1}-1)+2c \\ L_{Cap}=44 \text{ ft} \\ \text{Length of Cap} \\ \text{Length of C$$

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:

L = 8 in

W = 21 in

(IGEB standard)

Length of Bearing Pad

Width of Bearing Pad

4.2.3 Cross Sectional Properties of Cap

$$\begin{split} A_g &= d_{ledge} \cdot b_f + d_{stem} \cdot b_{stem} & A_g = 4659 in^2 \\ ybar &= \frac{d_{ledge} \cdot b_f \cdot \left(\frac{1}{2} d_{ledge}\right) + d_{stem} \cdot b_{stem} \cdot \left(d_{ledge} + \frac{1}{2} d_{stem}\right)}{A_g} & ybar = 34.3 \ in \\ I_g &= \frac{b_f \cdot d_{ledge}^3}{12} + b_f \cdot d_{ledge} \cdot \left(ybar - \frac{1}{2} d_{ledge}\right)^2 + \frac{b_{stem} \cdot d_{stem}^3}{12} + \cdots \end{split}$$

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

Distance from bottom of the cap to the center of gravity of the cap

4.2.4 Cap Analysis

4.2.4.1 <u>Cap Model</u>

Assume:

4 Columns Spaced @ 12'-0"

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

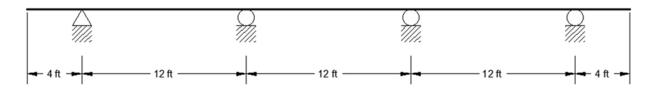


Figure 4.8 Continuous Beam Model for 0 Degree ITBC

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

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

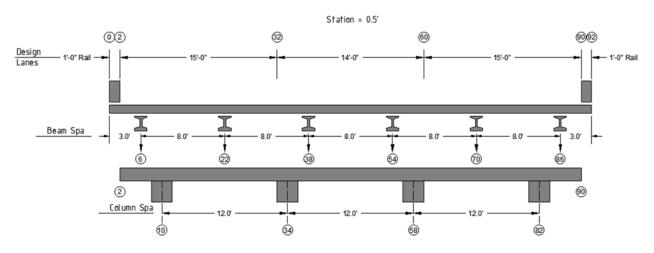


Figure 4.9 Cap 18 Model of 0 Degree ITBC

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.

$$station = 0.5 ft$$

Station increment for CAP 18

Recall:

$$\begin{split} E_c &= 4074 \, \mathrm{ksi} & I_g = 2.86 \times 10^6 \, \mathrm{in^4} \\ E_c I_g &= 1.165 \times 10^{10} \, \mathrm{kip \cdot in^2/} \, \left(12 \frac{\mathrm{in}}{\mathrm{ft}}\right)^2 & E_c I_g = 8.09 \times 10^7 \mathrm{kip \cdot ft^2} \end{split}$$

4.2.4.1.1 *Dead Load*

SPAN 1

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

$$Rail1 = \frac{2 \cdot RailWt \cdot \frac{Span1}{2}}{\min(GdrNo1,6)}$$

$$Rail1 = 3.44 \frac{kip}{girder}$$

$$Rail Weight is distributed$$

$$evenly among stringers, up to$$

$$3 stringers per rail (TxSP).$$

$$Slab1 = w_c \cdot GdrSpa1 \cdot SlabThk \cdot \frac{Span1}{2} \cdot 1.10 \qquad \qquad Slab1 = 23.76 \frac{kip}{girder} \quad \begin{array}{l} \textit{Increase slab DL by 10\% to} \\ \textit{account for haunch and} \\ \textit{thickened slab ends}. \end{array}$$

$$Girder1 = GdrWt1 \cdot \frac{Span1}{2}$$

$$Girder1 = 22.98 \frac{kip}{girder}$$

$$DLRxn1 = (Rail1 + Slab1 + Girder1) \qquad DLRxn1 = 50.17 \frac{kip}{girder} \qquad \begin{array}{l} \textit{Overlay is calculated} \\ \textit{separetely, because it has} \\ \textit{different load factor than} \\ \textit{the rest of the dead loads.} \\ \textit{ay1} = w_{Olay} \cdot GdrSpa1 \cdot OverlayThk \cdot \frac{Span1}{2} \qquad Overlay1 = 5.04 \frac{kip}{girder} \qquad \textit{Design for future overlay.} \\ \end{array}$$

$$Overlay1 = w_{Olay} \cdot GdrSpa1 \cdot OverlayThk \cdot \frac{Span1}{2}$$

SPAN 2

$$Rail2 = \frac{2 \cdot RailWt \cdot \frac{Span2}{2}}{\min(GdrNo2,6)}$$

$$Rail2 = 7.13 \frac{kip}{girder}$$

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

Girder2 = GdrWt1 ·
$$\frac{\text{Span2}}{2}$$
 Girder2 = 47.66 $\frac{\text{kip}}{\text{girder}}$

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

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

CAP

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

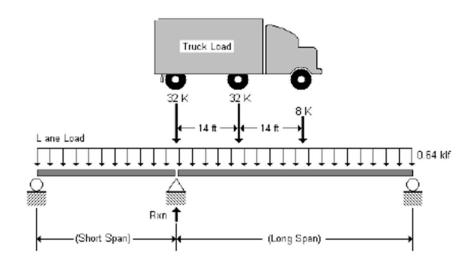


Figure 4.10 Live Load Model of 0 Degree ITBC

LongSpan = max(Span1, Span2)

LongSpan = 112 ft

ShortSpan = min(Span1, Span2)

ShortSpan = 54 ft

IM = 0.33

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

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

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

 $8\text{kip} \cdot \left(\frac{\text{LongSpan} - 2}{\text{LongSpan}}\right)$

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

LLRxn = Lane + Truck
$$\cdot$$
 (1 + IM)
LLRxn = 140.90 $\frac{\text{kip}}{\text{lane}}$

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.

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)

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

$$P = 21.28 \text{ kip}$$

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

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

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

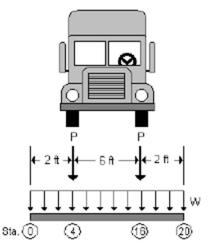


Figure 4.11 Live Load Model of 0 Degree Skewed ITBC for CAP18

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

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

<u>Limit States</u> (AASHTO LRFD 3.4.1)

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

Service I

Live Load and Dynamic Load Allowance LL+IM = 1.00

Dead Load and Wearing Surface DC & DW = 1.00

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.

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)

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

TxDOT allows the Overlay

4.2.4.1.4 Cap 18 Output

	Max + M	Max -M	These loads are the maximum loads from the CAP 18 Output
Dead Load:	$M_{posDL} = 249.2 \text{ kip} \cdot \text{ft}$	$M_{\text{negDL}} = -378.5 \text{ kip} \cdot \text{ft}$	File Located in the
Service Load:	$M_{posServ} = 491.6 \text{ kip} \cdot \text{ft}$	$M_{\text{negServ}} = -590.0 \text{ kip} \cdot \text{ft}$	Appendices.
Factored Load:	$M_{\text{posUlt}} = 740.6 \text{ kip} \cdot \text{ft}$	$M_{\text{negUlt}} = -851.0 \text{ kip} \cdot \text{ft}$	

4.2.4.2 Girder Reactions on Ledge

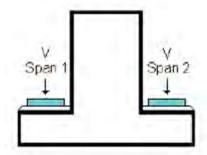


Figure 4.12 Girder Reactions on the Ledge of 0 Degree Skewed ITBC

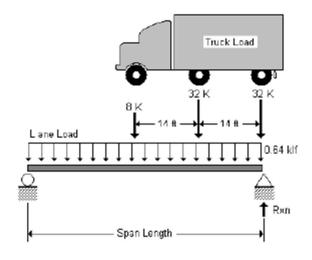
Dead Load

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

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

$$\begin{split} LaneSpan1 &= 0.64klf \cdot \left(\frac{Span1}{2}\right) & LaneSpan1 &= 17.28 \frac{kip}{lane} \\ LaneSpan2 &= 0.64klf \cdot \left(\frac{Span2}{2}\right) & LaneSpan2 &= 35.84 \frac{kip}{lane} \\ TruckSpan1 &= 32kip + 32kip \cdot \left(\frac{Span1 - 14ft}{Span1}\right) + 8kip \cdot \left(\frac{Span1 - 28ft}{Span1}\right) \end{split}$$

TruckSpan1 =
$$59.56 \frac{\text{kip}}{\text{lane}}$$

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

TruckSpan2 = $66.00 \frac{\text{kip}}{\text{lane}}$

IM = 0.33

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

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

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

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

 $gV_{Span1 Int} = 0.814$

 $gV_{Span1 Ext} = 0.814$

 $gV_{Span2 Int} = 0.814$

 $gV_{Snan2 Ext} = 0.814$

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.

LLSpan1Int = $gV_{Span1_Int} \cdot LLRxnSpan1$ LLSpan1Int = $78.54 \frac{kip}{girder}$

 $LLSpan1Ext = gV_{Span1_Ext} \cdot LLRxnSpan1 \qquad LLSpan1Ext = 78.54 \frac{kip}{girder}$

LLSpan2Int = $gV_{Span2_Int} \cdot LLRxnSpan2$ LLSpan2Int = $100.63 \frac{kip}{girder}$

 $LLSpan2Ext = gV_{Span2_Ext} \cdot LLRxnSpan2 \qquad LLSpan2Ext = 100.63 \frac{kip}{girder}$

Span 1

Interior Girder

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

$$V_{s Span1Int} = DLSpan1 + Overlay1 + LLSpan1Int$$

$$V_{s Span 1 Int} = 134 \text{ kip}$$

$$V_{u_Span1Int} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 + 1.75 \cdot LLSpan1Int$$

$$V_{u_Span1Int} = 208 \text{ kip}$$

Exterior Girder

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

$$V_{s \text{ Span1Ext}} = DLSpan1 + Overlay1 + LLSpan1Ext$$

$$V_{s Span1Ext} = 134 \text{ kip}$$

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)

$$V_{s_Span2Int} = DLSpan2 + Overlay2 + LLSpan2Int$$

$$V_{s \text{ Span2Int}} = 215 \text{ kip}$$

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

$$V_{u_Span2Int} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot LLSpan2Int$$

$$V_{u \text{ Span2Int}} = 322 \text{ kip}$$

Exterior Girder

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

$$V_{s \text{ Span2Ext}} = DLSpan2 + Overlay2 + LLSpan2Ext$$

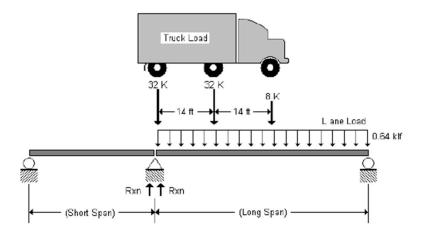
$$V_{s \text{ Snan2Ext}} = 215 \text{ kip}$$

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

$$V_{u \text{ Span2Ext}} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot LLSpan2Ext$$

$$V_{u \text{ Span}_2\text{Ext}} = 322 \text{ kip}$$

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

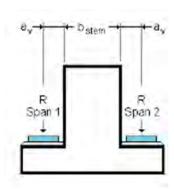


Figure 4.15 Loads on the Ledge of 0 Degree Skewed ITBC for Torsion

$$a_v = 12 \text{ in}$$

" 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.

$$b_{stem} = 39 in$$

LeverArm =
$$a_v + \frac{1}{2}b_{stem}$$

LeverArm
$$= 31.5$$
 in

Interior Girders

Girder Reactions

$$\begin{split} R_{u_Span1} &= 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 \\ R_{u_Span1} &= 70 \text{ kip} \\ R_{u_Span2} &= 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2_Int} \\ & \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)] \\ R_{u_Span2} &= 322 \text{ kip} \end{split}$$

Torsional Load

$$T_{u_Int} = \left| R_{u_Span1} - R_{u_Span2} \right| \cdot LeverArm$$

$$T_{u_Int} = 660 \; kip \cdot ft$$

Exterior Girders

Girder Reactions

$$\begin{split} R_{u_Span1} &= 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 \\ R_{u_Span1} &= 70 \text{ kip} \\ \\ R_{u_Span2} &= 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2_Ext} \\ & \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)] \\ \\ R_{u_Span2} &= 322 \text{ kip} \end{split}$$

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

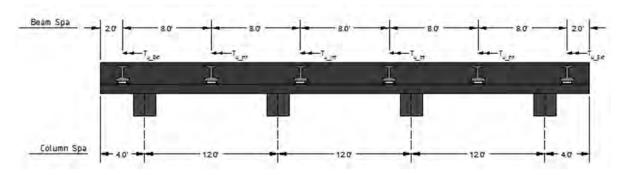


Figure 4.16 Elevation View of 0 Degree ITBC with Torsion Loads

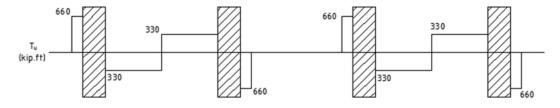


Figure 4.17 Torsion Diagram of 0 Degree ITBC

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

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

$$V_{u \text{ Int}} = \max(V_{u \text{ Span1Int}}, V_{u \text{ Span2Int}})$$
 $V_{u \text{ Int}} = 321.86 \text{ kip}$

Exterior Girder

Service Load

$$V_{s_Ext} = max(V_{s_Span1Ext}, V_{s_Span2Ext})$$
 $V_{s_Ext} = 215.15 \text{ kip}$

Factored Load

$$V_{u \text{ Ext}} = \max(V_{u \text{ Span1Ext}}, V_{u \text{ Span2Ext}})$$
 $V_{u \text{ Ext}} = 321.86 \text{ kip}$

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)

 $T_u = 660 \text{ kip} \cdot \text{ft}$ Located two stations away from

centerline of column.

 $V_{\mathrm{u}}=447.4~\mathrm{kip}$ V_{u} and M_{u} values are from

 $M_u = 334.5 \text{ kip} \cdot \text{ft}$ CAP18

4.2.5 Locate and Describe Reinforcing

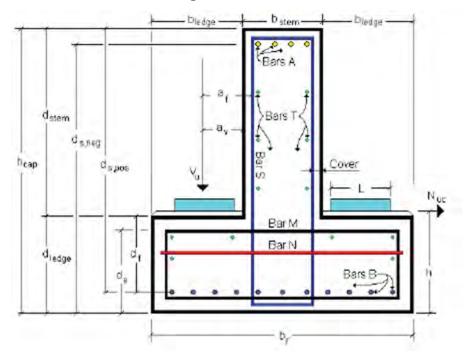


Figure 4.18 Section View of 0 Degree Skewed ITBC

Recall:

 $b_{stem}=39\ in$

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

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

 $d_{ledge} = 28 \ in$

 $b_f = 87 \ in$

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

cover = 2.5 in

Measured from Center of bar

4.2.5.1 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

$$A_{bar_B} = 1.56 \ in^2 \qquad \qquad d_{bar_B} = 1.410 \ in$$

Use # 6 bars for Bar M

$$A_{bar_M} = 0.44 \, in^2$$
 $d_{bar_M} = 0.75 \, in$ Bar M was considered. Bar M must be # 6 or smaller to allow it

In the calculation of b_{ledge} , # 6

same bar size for Bar N as Bar

fully develop.

M.

Use # 6 bars for Bar N

$$A_{bar_N} = 0.44 \text{ in}^2$$
 $d_{bar_N} = 0.75 \text{ in}$ To prevent confusion, use the

Use # 6 bars for Bar S

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

Use # 6 bars for Bar T

$$A_{bar_T} = 0.44 \text{ in}^2$$
 $d_{bar_T} = 0.75 \text{ in}$

4.2.5.2 <u>Calculate Dimensions</u>

$$d_{s_neg} = h_{cap} - cover - \frac{1}{2}d_{bar_s} - \frac{1}{2}d_{bar_A}$$
 $d_{s_neg} = 81.42 in$

$$d_{s_{pos}} = h_{cap} - cover - \frac{1}{2} max(d_{bar_S}, d_{bar_M}) - \frac{1}{2} d_{bar_B}$$
 $d_{s_{pos}} = 81.42 in$

$$a_v = 12 in$$

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

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

$$d_f = d_{ledge} - cover - \frac{1}{2}d_{bar_M} - \frac{1}{2}d_{bar_B} \qquad \qquad d_f = 24.42 \text{ in}$$

$$h = d_{ledge} + BrgSeat$$
 $h = 29.50 in$

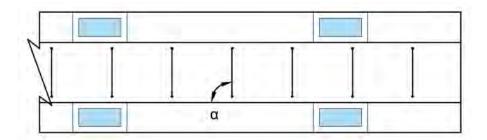


Figure 4.19 Plan View of 0 Degree Skewed ITBC

$$\alpha = 90 \text{ deg}$$

Angle of Bars S

Recall:

$$L = 8 in$$

$$W = 21 in$$

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.

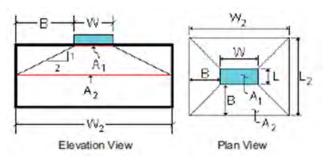


Figure 4.20 Bearing Check for 0-degree Skew Angle

Resistance Factor (ϕ) = 0.7

(AASHTO LRFD 5.5.4.2)

$$A_1 = L \cdot W$$

$$A_1 = 168 \text{ in}^2$$

Area under Bearing Pad

Interior Girders

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

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

B = 8 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

$$\phi V_n = \phi \quad 0.85 \quad f_c \quad A_1 \quad m$$

$$\phi V_n = 999.6 \text{ kips}$$

AASHTO LRFD Eqs. 5.6.5-1

$$V_{u \text{ Int}} = 321.86 < \phi V_{n}$$

$$V_{u_int}$$
 from "4.2.4.4Load Summary".

Exterior Girders

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

B= 8 in.

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

$$L_2 = L + 2 B$$
 $W_2 = W + 2 B$
 $A_2 = L_2 W_2$

$$L_2 = 24.00 \text{ in}$$
 $W_2 = 37.00 \text{ in}$
 $A_2 = 888 \text{ in}^2$

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

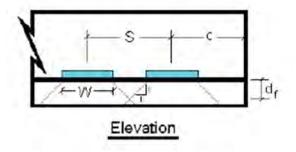
$$\varphi V_n = \varphi \ 0.85 \ f_c \ A_1 \ m$$

$$\phi V_n = 999.6 \text{ kips}$$

$$V_{u \text{ ext}} = 321.86 \text{ kips} < \Phi V_n$$

$$V_{u_ext}$$
 from "4.2.4.4 Load Summary".

4.2.7 Check Punching Shear



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

Figure 4.21 Punching Shear Check for 0degree Skew Angle

Resistance Factor (ϕ) = 0.90

AASHTO LRFD 5.5.4.2.

Determine if the Shear Cones Intersect

Is
$$\frac{1}{2}S - \frac{1}{2}W \ge d_f$$
?
 $\frac{1}{2}S - \frac{1}{2}W = 37.5 \text{ in}$

 $d_f = 24.42 \text{ in}$

$$\operatorname{Is} \frac{1}{2} b_{\text{stem}} + a_{\text{v}} - \frac{1}{2} L \ge d_{\text{f}}?$$

 $\frac{1}{2}b_{\text{stem}} + a_{\text{v}} - \frac{1}{2}L = 27.5 \text{ in}$

$$d_f = 24.42 \text{ in}$$

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

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

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

Interior Girders

Exterior Girders

$$\begin{array}{lll} V_n = & \min[(0.125 \cdot \sqrt{f_c} \cdot \left(\frac{1}{2}W + L + d_f + \right. & V_n = 545.15 \, \text{kips} & \textit{AASHTO LRFD} \\ c \left) * d_f, 0.125 \cdot \sqrt{f_c} \cdot \left(W + 2L + 2d_f\right) * d_f)] & 5.8.4.3.4-5 \end{array}$$

$$\phi V_n = 411.09 \text{ kips}$$

$$V_{u \text{ ext}} = 321.86 \text{ kips} < \phi V_{n}$$

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

Interior Girders

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

 $= \min (69 \text{ in}, 96 \text{ in})$

$$b_{s_Int} = 69 in$$

$$A_{cv} = b_{s Int} \cdot d_{e}$$

 $A_{cv} = 1759.5 \text{ in } 2$

Exterior Girders

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

"S" is the girder spacing.

"S" is the girder spacing.

$$= \min [69, 96, 48]$$

=48 in

$$A_{cv} = b_{s \text{ ext}} \cdot d_{e}$$

$$A_{cv} = 1224 \text{ in } 2$$

Interior Girders

$$V_n \ = min(0.2 \cdot f_c \cdot A_{cv}, 0.8 \cdot A_{cv}) \quad \ V_n \ = \ 1408 \ kips \label{eq:vn}$$

AASHTO LRFD 5.8.4.2.2-1 and

$$\phi V_n = 1267 \text{ kips}$$

 $= \min(1759.5, 1408)$

$$V_{u \text{ Int}} = 321.86 \text{ kips} < \phi V_{n}$$

ShearFrictionChk= "OK!"

 $V_{u,int}$ from "4.2.4.4 Load"

Summary".

Exterior Girders

$$V_n = min(0.2 \cdot f_c \cdot A_{cv}, 0.8 \cdot A_{cv})$$
 $V_n = 979.2 \text{ kips}$

AASHTO LRFD 5.8.4.2.2-1 and

5.8.4.2.2-2

$$\phi V_n = 881 \text{ kips}$$

 $= \min(1224, 979.2)$

$$V_{u \text{ ext}} = 321.86 \text{ kips} < \phi V_n$$

ShearFrictionChk= "OK!"

V_{u ext} from "4.2.4.4 Load

Summary".

4.2.9 Flexural Reinforcement for Negative Bending (Bars A)

$$M_{dl} = \left| M_{negDL} \right|$$
 $M_{dl} = 378.5 \text{ kip} \cdot \text{ft}$ 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 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
$$Distance \ to \ the \ Center \ of \ Gravity of \ the \ Cap \ from \ the \ bottom \ of \ the \ Cap$$
 Modulus of Rupture (BDM-

$$f_r = 0.24 \sqrt{f_c}$$
 $f_r = 0.537 \text{ ksi}$ $LRFD, Ch. 4, Sect. 5, Design Criteria)$

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

$$S = \frac{I_g}{y_t} \qquad \qquad S = 5.64 \times 10^4 \text{ in}^3 \qquad \qquad \begin{array}{l} \textit{Section Modulus for the extreme} \\ \textit{tension fiber} \end{array}$$

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

Design the lesser of
$$1.2M_{cr}$$
 or $1.2M_{cr} = 3028.7 \text{ kip} \cdot \text{ft}$ $1.33M_u$ 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

BarANo = 6

 $d_{bar A} = 1.410 in$

 $A_{bar A} = 1.56 in^2$

 $A_s = BarANo \cdot A_{bar A}$

 $d_{stirrup} = d_{bar S}$

 $d = d_{s \text{ neg}}$

 $b = b_f$

 $f_c = 5.0 \text{ ksi}$

 $f_v = 60 \text{ ksi}$

 $\beta_1 = 0.85 - 0.05(f_c - 4ksi)$

Bounded by: $0.65 \le \beta_1 \le 0.85$

 $c = \frac{A_s f_y}{0.85 f_c \beta_1 b}$

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

 $A_s = 9.36 \text{ in}^2$

d = 81.42 in

b = 87 in

 $\beta_1 = 0.80$

c = 1.90 in

 $d_{stirrup} = 0.75 in$

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

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) \cdot \frac{1ft}{12in}$$

 $M_n = 3774.9 \text{ kip} \cdot \text{ft}$

$$\epsilon_s = 0.003 \cdot \frac{d-c}{c}$$

 $\varepsilon_s = 0.126$

FlexureBehavior = "Tension Controlled"

 $\Phi_{M} = 0.90$

 $\varepsilon_{\rm s} > 0.005$

 $M_r = \Phi_M M_n$

 $M_r = 3397.4 \text{ kip} \cdot \text{ft}$

 $M_f = 1131.8 \text{ kip} \cdot \text{ft} < M_r$

 $M_u = 851.0 \text{ kip} \cdot \text{ft} < M_r$

MinReinfChk = "OK!"

UltimateMom = "OK!"

Number of bars in tension

Diameter of main reinforcing

bars

Area of main reinforcing bars

Area of steel in tension

Diameter of shear reinforcing

bars

Compressive Strength of Concrete

Yield Strength of Rebar

(AASHTO LRFD 5.6.2.2)

Depth of Cross Section under Compression under Ultimate Load (AASHTO LRFD Eq. 5.6.3.1.2-4)

Depth of Equivalent Stress Block (AASHTO LRFD 5.6.2.2)

Nominal Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.2-1)

Strain in Reinforcing at Ultimate

(AASHTO LRFD 5.6.2.1)

(AASHTO LRFD 5.5.4.2)

Factored Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.1-1)

4.2.9.3 Check Serviceability

To find s_{max} :

Modular Ratio:

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

Tension Reinforcement Ratio:

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

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

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

Therefore, the compression force acts over a rectangular area.

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

$$f_a = 0.6f_v$$
 $f_a = 36.00 \text{ ksi}$

$$f_{ss} < f_a$$
 ServiceStress = "OK!"
 $d_c = \text{cover} + \frac{1}{2} d_{stirrup} + \frac{1}{2} d_{bar A}$ $d_c = 3.58 \text{ in}$

Exposure Condition Factor:

$$\gamma_{\rm e} = 1.00$$

$$\beta_{\rm s} = 1 + \frac{d_{\rm c}}{0.7(h_{\rm can} - d_{\rm c})}$$
 $\beta_{\rm s} = 1.06$

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

$$s_{Actual} = \frac{b_{stem} - 2d_c}{Bar A No - 1}$$
 $s_{Actual} = 6.37 \text{ in}$

$$s_{Actual} < s_{max}$$
 ServiceabilityCheck = "OK!" the bent. (TxSP)

4.2.9.4 Check Dead Load

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

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

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

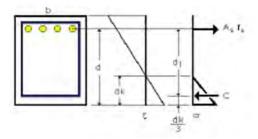


Figure 4.22 Stresses on the Cross Section for Service Loads of 0 Degree Skewed ITBC

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

Service Load Bending Stress in outer layer of the reinforcing.

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

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)

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

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

Allowable Dead Load Moment

$$M_{dl} = 378.5 \text{ kip} \cdot \text{ft} < M_a$$

DeadLoadMom = "OK!"

4.2.10 Flexural Reinforcement for Positive Bending (Bars B)

$M_{dl} = M_{posDL}$	$M_{dl} = 249.2 \text{ kip} \cdot \text{ft}$
$M_s = M_{posServ}$	$M_s = 491.6 \text{ kip} \cdot \text{ft}$
$M_{\rm u} = M_{\rm posUlt}$	$M_u = 740.6 \text{ kip} \cdot \text{ft}$

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).

$I_g = 2.86 \times 10^6 \text{ in}^4$		Gross Moment of Inertia	
$y_t = ybar$	$y_t = 34.3 \text{ in}$	Distance to the Center of Gravity of the Cap from the top of the Cap	
$f_{\rm r} = 0.24\sqrt{f_{\rm c}}$	$f_r = 0.537 \text{ ksi}$	Modulus of Rupture (BDM- LRFD, Ch. 4, Sect. 5, Design Criteria)	
$S = \frac{I_g}{y_t}$	$S = 8.34 \times 10^4 \text{ in}^3$	Section Modulus for the extreme tension fiber	
$M_{cr} = S \cdot f_r \cdot \frac{1ft}{12in}$	$M_{cr} = 3732.2 \text{ kip} \cdot \text{ft}$	Cracking Moment (AASHTO LRFD Eq. 5.6.3.3-1)	
$M_f = minimum of:$		Design the lesser of 1.2M _{cr} or	
$1.2M_{\rm cr} = 4478.6 \text{ kip} \cdot \text{ft}$		$1.33M_u$ when determining	
$1.33M_u = 985.0 \text{ kip} \cdot \text{ft}$		mininum area of steel required.	

Thus, M_r must be greater than $M_f = 985.0 \; \mathrm{kip} \cdot \mathrm{ft}$

4.2.10.2 Moment Capacity Design

Try,
$$11 \sim #11$$
's Bottom

BarBNo = 11

 $d_{bar B} = 1.41 in$

 $A_{bar B} = 1.56 in^2$

 $A_s = BarBNo \cdot A_{bar\ B}$

 $d = d_{s \text{ nos}}$

 $b = b_{stem}$

 $f_c = 5.0 \text{ ksi}$

 $f_v = 60 \text{ ksi}$

 $\beta_1 = 0.85 - 0.05(f_c - 4ksi)$

Bounded by: $0.65 \le \beta_1 \le 0.85$

 $c = \frac{A_s f_y}{0.85 \ c \beta_1 b}$

Number of bars in tension

Diameter of main reinforcing

bars

Area of main reinforcing bars

Area of steel in tension

Compressive Strength of Concrete

Yield Strength of Rebar

(AASHTO LRFD 5.6.2.2)

Depth of Cross Section under

Compression under Ultimate Load

(AASHTO LRFD Eq. 5.6.3.1.2-4)

Depth of Equivalent Stress Block

(AASHTO LRFD 5.6.2.2)

Nominal Flexural Resistance

(AASHTO LRFD Eq. 5.6.3.2.2-1)

Strain in Reinforcing at Ultimate

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.

$$a = c \cdot \beta_1$$

$$a = 6.21 in$$

 $A_s = 17.16 \text{ in}^2$

d = 81.42 in

b = 39 in

 $\beta_1 = 0.80$

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.

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) \cdot \frac{1ft}{12in}$$

$$M_n = 6719.4 \: kip \cdot ft$$

$$\varepsilon_{\rm S} = 0.003 \cdot \frac{\rm d-c}{\rm c}$$

$$\varepsilon_{\rm s} = 0.028$$

$$\Phi_{\rm M} = 0.90$$

 $\varepsilon_{\rm s} > 0.005$

$$M_r = \Phi_M \cdot M_n$$

$$M_r = 6047.5 \text{ kip} \cdot \text{ft}$$

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

$$M_u = 740.6 \text{ kip} \cdot \text{ft} < M_r$$

4.2.10.3 Check Serviceability

To find s_{max} :

Modular Ratio:

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

Tension Reinforcement Ratio:

$$\rho = \frac{A_s}{b_1 d} \qquad \qquad \rho = 0.0054$$

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

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

Therefore, the compression force acts over a rectangular area

$$f_{ss} = \frac{M_s}{A_c \cdot j \cdot d} \cdot \frac{12 i n}{1 f t} \qquad \qquad f_{ss} = 4.59 \text{ ksi}$$

$$f_a = 0.6f_v$$
 $f_a = 36.00 \text{ ksi}$

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

$$d_c = cover + \frac{1}{2}d_{stirrup} + \frac{1}{2}d_{bar_B}$$
 $d_c = 3.58 in$

Exposure Condition Factor:

$$\gamma_e = 1.00$$

$$\beta_s = 1 + \frac{d_c}{0.7(h_{can} - d_c)}$$

$$\beta_s = 1.06$$

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

Bars Inside Stirrup Bar S

Try: BarBInsideSNo = 5

$$s_{Actual} = \frac{b_{stem} - 2 \left(cover + \frac{1}{2} d_{bar_S} + \frac{1}{2} d_{bar_B} \right)}{BarBInsideSNo -}$$

$$s_{Actual} < s_{max}$$

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

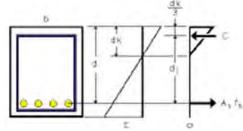


Figure 4.23 Stresses on the Cross Section for Bars B for Service Loads of 0 Degree Skewed ITBC

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

Service Load Bending Stress in outer layer of the reinforcing.

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

"cover" is measured to center of shear reinforcement.

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)

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

Number of Bars B that are inside Stirrup Bar S.

$$s_{Actual} = 7.96 in$$

ServiceabilityCheck = "OK

Bars Outside Stirrup Bar S

BarBOutsideSNo = 11 - BarBInsideSNo

Stirrup Bar S.

BarBOutsideSNo = 6

$$s_{Actual} = \frac{2b_{ledge} + 2\left(cove \quad \frac{1}{2}d_{bar_S} + \frac{1}{2}d_{bar_B} - cove \quad \frac{1}{2}d_{bar_M} - \frac{1}{2}d_{bar_B}\right)}{BarBOutsideSNo}$$

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

ServiceabilityCheck = "OK

4.2.10.4 Check Dead Load

Check allowable M_{dl}:

$$f_{dl} = 22 \text{ ksi}$$

$$M_a = A_s \cdot d \cdot j \cdot f_{dl} \cdot \frac{_{1ft}}{_{12in}}$$

$$M_a = 2354.00 \text{ kip} \cdot \text{ft}$$

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

DeadLoadMom = "OK!"

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

Number of Bars B that are inside

Allowable Dead Load Moment

Flexural Steel Summary:

Use $6 \sim #11$ 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.

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

$$s_{bar \ N} = 4.90 in$$

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 <u>Determine Distribution Widths</u>

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

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

$$b_{s Int} = 69.00 in$$

Exterior Girders

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

$$b_{s Ext} = 48.00 in$$

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

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

$$b_{m Int} = 93.50 in$$

Exterior Girders

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

$$b_{m Ext} = 48.00 in$$

4.2.11.2 Reinforcing Required for Shear Friction

AASHTO LRFD 5.7.4.1

$$\phi = 0.90$$

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

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

(AASHTO LRFD 5.5.4)

"u" 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)

Minimum Reinforcing (AASHTO LRFD Eq. 5.7.4.2-1)

$$A_{vf_min} = \frac{_{0.05\;ksi\cdot A_{cv}}}{_{f_y}}$$

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

" P_c " is zero as there is no axial

compression.

$$a_{vf_min} = \frac{0.05ksi \cdot d_e}{f_y}$$

$$a_{\text{vf_min}} = 0.26 \frac{\text{in}^2}{\text{ft}}$$

 $a_{vf_min} = 0.26 \frac{in^2}{ft}$ Minimum Reinforcing required for Shear Friction

Interior Girders

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

 $V_{u,Int} = 322 \text{ kip}$ $V_n = c_1 A_{cv} + \mu (A_{vf} f_v + P_c)$ $\Phi V_n \geq V_n$ $\phi \cdot \left[c_1 A_{cv} + \mu (A_{vf} f_v + P_c)\right] \ge V_{u}$ From "4.2.4.4 Load Summaryry". (AASHTO LRFD Eq. 5.7.4.3-3)

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

$$A_{vf} = \frac{\frac{v_{u_Int}}{\Phi} - c_1 A_{cv}}{\frac{\mu}{f_y}} - P_c$$

$$A_{vf} = 4.26 \text{ in}^2$$

$$a_{vf_Int} = \frac{A_{vf}}{b_{s_Int}}$$

$$a_{\text{vf_Int}} = 0.74 \frac{\text{in}^2}{\text{ft}}$$

 $a_{vf_Int} = 0.74 \frac{in^2}{ft}$ Required Reinforcing for Shear Friction per foot length of cap

Exterior Girders

$$\begin{split} A_{cv} &= d_e \cdot b_{s_Ext} & A_{cv} = 1224 \text{ in}^2 \\ V_{u_Ext} &= 322 \text{ kip} & \textit{From "Load Summary"}. \\ V_n &= c_1 A_{cv} + \mu (A_{vf} f_y + P_c) & (\textit{AASHTO LRFD Eq. 5.7.4.3-3}) \\ \Phi V_n &\geq V_u & (\textit{AASHTO LRFD Eq. 5.7.4.3-1 & AASHTO LRFD Eq. 5.7.4.3-2}) \\ \Phi \cdot \left[c_1 A_{cv} + \mu (A_{vf} f_y + P_c) \right] \geq V_u & A_{vf} &= \frac{\frac{V_{u_Ext}}{\Phi} - c_1 A_{cv}}{f_y} - P_c}{f_y} & A_{vf} &= 4.26 \text{ in}^2 & \textit{Required Reinforcing for Shear Friction} \\ a_{vf_Ext} &= \frac{A_{vf}}{b_{s_Ext}} & a_{vf_Ext} &= 1.06 \frac{\text{in}^2}{\text{ft}} & \textit{Required Reinforcing for Shear Friction per foot length of cap} \end{split}$$

4.2.11.3 Reinforcing Required for Flexure

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

 $N_{uc\ Int} = 0.2 \cdot V_{u\ Int}$

AASHTO LRFD 5.8.4.2.1

Recall: h = 29.50 in $d_e = 25.50 \text{ in}$ $a_v = 12 \text{ in}$

From "4.2.5.2 Calculate Dimensions"

(AASHTO LRFD 5.8.4.2.1)

per foot length of cap

From "4.2.4.4 Load Summary".

Interior Girders

$$\begin{split} &M_{u_Int} = V_{u_Int} \cdot a_v + N_{uc_Int}(h - d_e) \quad M_{u_Int} = 343.5 \text{ kip} \cdot \text{ft} \quad (\textit{AASHTO LRFD Eq. 5.8.4.2.1-1}) \\ &\text{Use the following equations to solve for A_f:} \\ &\Phi M_n \geq M_{u_Int} \qquad \qquad (\textit{AASHTO LRFD Eq. 1.3.2.1-1}) \\ &M_n = A_f f_y \left(d_e - \frac{a}{2} \right) \qquad \qquad (\textit{AASHTO LRFD Eq. 5.6.3.2.2-1}) \\ &c = \frac{A_f f_y}{\alpha_1 f_c \beta_1 b_{m_Int}} \qquad \qquad (\textit{AASHTO LRFD Eq. 5.6.3.1.2-4}) \\ &\alpha_1 = 0.85 \\ &\beta_1 = 0.80 \qquad \qquad (\textit{AASHTO LRFD 5.6.2.2}) \\ &a = c \beta_1 \\ &0.75 \leq \Phi = 0.65 + 0.15 \left(\frac{d_e}{c} - 1 \right) \leq 0.90 \qquad (\textit{AASHTO LRFD 5.5.4.2}) \\ &\text{Solve for A_f:} \qquad \qquad A_f = 3.02 \text{ in}^2 \qquad \textit{Required Reinforcing for Flexure} \\ &a_{f_Int} = \frac{A_f}{b_{m_Int}} \qquad \qquad a_{f_Int} = 0.39 \frac{\text{in}^2}{\text{ft}} \qquad \textit{Required Reinforcing for Flexure} \end{split}$$

 $N_{uc\ Int} = 64.4 \text{ kip}$

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)

$$M_{u_Ext} = V_{u_Ext} \cdot a_v + N_{uc_Ext}(h - d_e) \quad M_{u_Ext} = 343.5 \text{ kip} \cdot \text{ft} \quad (AASHTO \ LRFD \ Eq. \ 5.8.4.2.1-1)$$

$$M_{u Ext} = 343.5 \text{ kip} \cdot \text{f}$$

Use the following equations to solve for A_f:

$$\Phi M_n \ge M_{u Ext}$$

(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)

$$a = c\beta_1$$

$$0.75 \le \Phi = 0.65 + 0.15 \left(\frac{d_e}{c} - 1\right) \le 0.90$$

(AASHTO LRFD 5.5.4.2)

$$A_f = 3.05 \text{ in}^2$$

Required Reinforcing for Flexure

$$a_{f_Ext} = \frac{A_f}{b_{m_Ext}}$$

$$a_{f_Ext} = 0.76 \frac{in^2}{ft}$$

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:

$$N_{uc_Int} = 0.2 V_{u_Int}$$

$$N_{uc\ Int} = 64.4 \text{ kip}$$

$$A_n = \frac{N_{uc_Int}}{\Phi f_v}$$

$$A_n = 1.19 \text{ in}^2$$

Required Reinforcing for Axial

Tension

$$a_{n_Int} = \frac{A_n}{b_{m_Int}}$$

$$a_{n_{\perp}Int} = 0.15 \frac{in^2}{ft}$$

 $a_{n_Int} = 0.15 \frac{in^2}{ft}$ Required Reinforcing for Axial Tension per foot length of cap

Exterior Girders:

$$N_{uc Ext} = 0.2 V_{u Int}$$

$$N_{\text{uc Ext}} = 64.4 \text{ kip}$$

$$A_n = \frac{N_{uc_Ext}}{\Phi f_v}$$

$$A_n = 1.19 \text{ in}^2$$

Required Reinforcing for Axial

Tension

$$a_{n_{-}Ext} =$$

 $a_{n_Ext} = 0.30 \frac{in^2}{ft}$ Required Reinforcing for Axial Tension per foot length of cap

(AASHTO LRFD 5.8.4.2.1)

4.2.11.5 Minimum Reinforcing

$$a_{s_min} = 0.04 \frac{f_c}{f_y} \; d_e$$

$$a_{s_min} = 1.02 \frac{in^2}{ft}$$

 $a_{s_min} = 1.02 \frac{in^2}{fr}$ Minimum Required Reinforcing

4.2.11.6 Check Required Reinforcing

Actual Reinforcing:

$$a_{s} = \frac{A_{bar_M}}{s_{bar_M}}$$

$$a_{h} = \frac{A_{bar_N}}{s_{bar_N}}$$

$$a_s = 1.08 \frac{in^2}{ft}$$

Primary Ledge Reinforcing Provided

$$a_h = 1.08 \frac{in^2}{ft}$$

Auxiliary Ledge Reinforcing

Provided

(AASHTO LRFD 5.8.4.2.1)

(AASHTO LRFD 5.8.4.2.2)

(AASHTO LRFD Eq. 5.8.4.2.2-5)

(AASHTO LRFD Eq. 5.8.4.2.2-6)

$A_s \ge A_f + A_n$

 $\underline{\text{Checks:}} A_s \ge A_{s \text{ min}}$

$$A_{s} \ge \frac{2A_{vf}}{3} + A_{n}$$

$$A_h \ge 0.5(A_s - A_n)$$

Check Interior Girders:

Bar M:

Check if:
$$a_s \ge a_{s_min}$$

(AASHTO LRFD 5.8.4.2.1)

$$a_s \geq a_{f_Int} + a_{n_Int}$$

(AASHTO LRFD 5.8.4.2.2)

$$a_s \ge \frac{2a_{vf_Int}}{3} + a_{n_Int}$$

(AASHTO LRFD Eq. 5.8.4.2.2-5)

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

$$a_{s_{-}min} = 1.02 \frac{in^2}{ft} < a_s$$

$$a_{f_{-}Int} + a_{n_{-}Int} = 0.54 \frac{in^2}{ft} < a_{s}$$

$$\frac{2a_{vf_Int}}{3} + a_{n_Int} = 0.64 \frac{in^2}{ft} < a_s$$

BarMCheck = "OK!"

Bar N:

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

$$a_{\rm h} \ge 0.5 \cdot \left(a_{\rm s} - a_{\rm n_Int}\right)$$

(AASHTO LRFD Eq. 5.8.4.2.2-6)

" a_s " in this equation is the steel required for Bar M, based on the

 a_s = The maximum of:

$$a_{f Int} + a_{n Int}$$

$$\frac{2a_{\text{vf_Int}}}{3} + a_{\text{n_Int}}$$

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:
$$a_{s} \ge a_{s_min}$$
 (AASHTO LRFD 5.8.4.2.1)
$$a_{s} \ge a_{f_Ext} + a_{n_Ext}$$
 (AASHTO LRFD 5.8.4.2.2)
$$a_{s} \ge \frac{2a_{vf_Ext}}{3} + a_{n_Ext}$$
 (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:
$$a_h \geq 0.5 \cdot \left(a_s - a_{n_Ext}\right) \qquad (AASHTO \ LRFD \ Eq. \ 5.8.4.2.2-6)$$

$$a_s = \text{The maximum of:} \qquad "a_s" \ in \ this \ equation \ is \ the \ steel \ required$$

$$for \ Bar \ M, \ based \ on \ the \ requirements \ for \ Bar \ M \ in \ AASHTO \ LRFD \ 5.8.4.2.2. \ This \ is \ derived \ from \ the \ suggestion \ that \ Ah$$

$$a_s = 1.06 \frac{\text{in}^2}{\text{ft}} \qquad should \ not \ be \ less \ than \ A_f/2 \ nor \ less \ than \ A_v/3 \ (Furlong \ \& \ Mirza \ pg. \ 73 \ \& \ 74)$$

$$0.5 \cdot \left(a_s - a_{n_Ext}\right) = 0.38 \frac{\text{in}^2}{\text{ft}} < a_h$$

$$BarNCheck = "OK!"$$

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.

$$s_{bar_S} = 7.80 \text{ in}$$

$$A_{hr} = 2stirrups \cdot A_{bar_S}$$

$$A_v = 2legs \cdot A_{hr}$$

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

$A_{hr} = 0.88 \, in^2$

$$A_v = 1.76 \text{ in}^2$$

4.2.12.1 Check Minimum Transverse Reinforcement

$$b_v = b_{stem} \\$$

$$A_{v_min} = 0.0316\lambda \sqrt{f_c} \frac{b_v \cdot s_{bar_S}}{f_v}$$

 $\lambda = 1.0$ for normal weight concrete

$$b_v = 39 \text{ in}$$

(AASHTO LRFD Eq. 5.7.2.5-1) (AASHTO LRFD 5.4.2.8)

$$A_{v_{-}min} = 0.36 \text{ in}^2$$

 $A_v > A_{v \text{ min}}$

MinimumSteelCheck = "OK!"

4.2.12.2 Check Service Limit State

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

Interior Girders

$$V_{all}$$
 = minimum of:

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

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)

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

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

(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)

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

$$V_{s_{\perp}Int} = 215 \text{ kip} < V_{all}$$

ServiceCheck = "OK!"

Exterior Girders

 V_{all} = minimum of:

V_{all} for the Interior Girder

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

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

(BDM-LRFD Ch.4, Sect. 5, Design Criteria

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

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

 $V_{\rm all} = 217 \, \rm kip$

$$V_{s Ext} = 215 \text{ kip} < V_{all}$$

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.)

ServiceCheck = "OK!"

(AASHTO LRFD 5.8.4.3.5)

4.2.12.3 Check Strength Limit State

$$\Phi = 0.90$$

(AASHTO LRFD Eq. 5.5.4.2)

Interior Girders:

 $V_n = minimum of:$

$$\frac{A_{hr} \cdot f_y}{s_{har S}} \cdot S = 650 \text{ kip}$$

(AASHTO LRFD Eq. 5.8.4.3.5-2)

$$\left(0.063\sqrt{f_c}\cdot b_f\cdot d_f\right) + \frac{A_{hr}\cdot f_y}{s_{bar_S}}(W+2d_f) = 772kip$$

(AASHTO LRFD Eq. 5.8.4.3.5-3)

 $V_n = 650 \text{ kip}$

 $\Phi V_n = 585 \text{ kip}$

$$V_{u_Int} = 322 \text{ kip } < \Phi V_n$$

UltimateCheck = "OK!"

Exterior Girders:

 $V_n = minimum of:$

V_n for the Interior Girder

$$\frac{A_{\text{hr}} \cdot f_{y}}{s_{\text{bar S}}} \cdot \left(\frac{S}{2} + c\right) = 487 \text{ kip}$$

(AASHTO LRFD Eq. 5.8.4.3.5-2)

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

 $V_n = 487 \text{ kip}$

 $\Phi V_n = 438 \text{ kip}$

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

$$V_{u_{\text{Ext}}} = 322 \text{ kip } < \Phi V_{\text{n}}$$

UltimateCheck = "OK!"

4.2.12.4 Check Combined Shear and Torsion

 $d_v = 80.66 \text{ in}$

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.

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 :

$$\Phi_{V} = 0.90$$

$$v_{u} = \frac{|v_{u} - (\Phi_{V} \cdot v_{p})|}{\Phi_{V} \cdot b_{v} \cdot d_{v}}$$

 $v_u = 0.16 \, \mathrm{ksi}$

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

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

$$\theta = 36.4 \text{ deg}$$
 and $\beta = 2.23$

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

where $|M_u|=334.5 kip\cdot ft$ must be $>\left|V_u-V_p\right|d_v=3012.12 kip\cdot ft$ $\epsilon_x=1.38\times 10^{-3}~>~1.00\times 10^{-3}$

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

$$V_p = 0 \ \mathrm{kip}$$

$$A_c = b_{stem} \cdot \frac{h_{cap}}{2}$$

$$s = s_{bar_S}$$

 $A_c = 1657.5 \text{ in}^2$

$$s = 7.80 in$$

(AASHTO LRFD Eq. 5.5.4.2)

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

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.

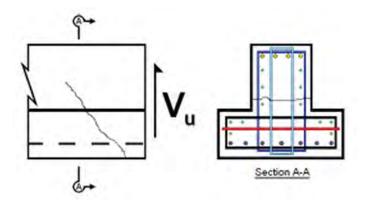
Strain halfway between the compressive and tensile resultants (AASHTO LRFD Eq. B5.2-3) If $\varepsilon_x < 0$, then use equation B5.2-5 and re-solve for ε_x .

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.

" V_p " is zero as there is no prestressing.

(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.



" 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.

The transverse reinforcement,

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

$$\begin{split} A_v &= 2 \text{legs} \cdot 2 \text{stirrups} \cdot A_{bar_S} & A_v &= 1.76 \text{ in}^2 \\ A_t &= 1 \text{leg} \cdot A_{bar_S} & A_t &= 0.44 \text{ in}^2 \\ A_{oh} &= (d_{stem}) \cdot (b_{stem} - 2 \text{cover}) + (d_{ledge} - 2 \text{cover}) \cdot (b_f - 2 \text{cover}) \\ & A_{oh} &= 3496 \text{ in}^2 \\ A_0 &= 0.85 A_{oh} & A_0 &= 2971.6 \text{in}^2 \\ p_h &= (b_{stem} - 2 \text{cover}) + 2 \big(b_{ledge}\big) + (b_f - 2 \text{cover}) + 2 \big(h_{cap} - 2 \text{cover}\big) \\ p_h &= 324 \text{ in} \end{split}$$

Equivalent Shear Force

$$V_{u_{-}Eq} = \sqrt{V_{u}^{2} + \left(\frac{0.9p_{h}T_{u}}{2A_{0}}\right)^{2}}$$
 $V_{u_{-}Eq} = 592.6 \text{ kip } (AASHTO LRFD Eq. B.5.2-1)$

Shear Steel Required

$$V_n$$
 = the lesser of:

$$V_c + V_s + V_p$$
 (AASHTO LRFD Eq. 5.7.3.3-1)
0.25 · f_c · b_v · d_v + V_p (AASHTO LRFD Eq. 5.7.3.3-2)

Check maximum ΦV_n for section:

$$\Phi V_{n_max} = \Phi \cdot \left(0.25 \cdot f_c \cdot b_v \cdot d_v + V_p\right)$$

$$\Phi V_{n_max} = 3539 \text{ kip}$$

$$V_u = 447.4 \text{ kip} < \Phi V_{n_max}$$

$$MaxShearCheck = "OK!"$$

Calculate required shear steel:

$$\begin{split} &V_{u} < \Phi V_{n} & (\textit{AASHTO LRFD Eq. 1.3.2.1-1}) \\ &V_{c} = 0.0316 \cdot \beta \cdot \sqrt{f_{c}} \cdot b_{v} \cdot d_{v} & V_{c} = 496 \text{ kip} & (\textit{AASHTO LRFD Eq. 5.7.3.3-3}) \\ &V_{u} < \Phi_{V} \cdot \left(V_{c} + V_{s} + V_{p}\right) & (\textit{AASHTO LRFD Eq. 5.7.3.3-4}) \\ &V_{s} = \frac{A_{v} \cdot f_{y} \cdot d_{v} \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha}{s_{req}} & (\textit{AASHTO LRFD Eq. 5.7.3.3-4}) \\ &a_{v_req} = \frac{\frac{V_{u}}{\Phi_{V}} \cdot V_{c} - V_{p}}{f_{v} \cdot d_{v} \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha} & a_{v_req} = 0.002 \frac{\sin^{2}}{f_{t}} \end{split}$$

Torsional Steel Required

$$\begin{split} \Phi_T &= 0.9 & (\textit{AASHTO LRFD 5.5.4.2}) \\ T_u &\leq \Phi_T T_n & (\textit{AASHTO LRFD Eq. 1.3.2.1-1}) \\ T_n &= \frac{2A_o A_t f_y \cot \theta}{s_{bar_S}} & (\textit{AASHTO LRFD Eq. 5.7.3.6.2-1}) \\ a_{t_req} &= \frac{T_u}{\Phi_T 2A_o f_y \cot \theta} & a_{t_req} &= 0.22 \frac{in^2}{ft} \end{split}$$

Total Required Transverse Steel

$$a_{req} = a_{v_req} + 2 sides \cdot a_{t_req} \qquad a_{req} = 0.44 \frac{in^2}{ft} \qquad \begin{array}{l} \textit{designed for the side of the section} \\ \textit{where the effects of shear and torsion} \\ a_{prov} = \frac{A_v}{s_{bar_S}} \qquad a_{prov} = 2.71 \frac{in^2}{ft} \qquad \begin{array}{l} \textit{are additive. (AASHTO LRFD} \\ \textit{C5.7.3.6.1)} \\ \end{array}$$

The transverse reinforcement is

Longitudinal Reinforcement

$$\begin{split} A_{ps}f_{ps} + A_{s}f_{y} &\geq \frac{|M_{u}|}{\Phi d_{v}} + \frac{0.5N_{u}}{\Phi} + \cdots \\ &\qquad \qquad (\textit{AASHTO LRFD Eq. 5.7.3.6.3-1}) \\ &\qquad \qquad \cot\theta \sqrt{\left(\left|\frac{V_{u}}{\Phi} - V_{p}\right| - 0.5V_{s}\right)^{2} + \left(\frac{0.45}{2A_{0}\Phi}\right)^{2}} \\ V_{s} &= a_{t_req} \cdot f_{y} \cdot d_{v} \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha \\ &\qquad \qquad (\textit{AASHTO LRFD Eq. 5.7.3.3-4}) \\ &\qquad \qquad \text{Bounded By: } V_{s} < \frac{V_{u}}{\Phi_{v}} \qquad \qquad V_{s} = 497.1 \text{ kip} \qquad (\textit{AASHTO LRFD Eq. 5.7.3.5-1}) \\ &\qquad \qquad \frac{|M_{u}|}{\Phi_{f}d_{v}} + \frac{0.5N_{u}}{\Phi_{c}} + \cot\theta \sqrt{\left(\left|\frac{V_{u}}{\Phi_{v}} - V_{p}\right| - 0.5V_{s}\right)^{2} + \left(\frac{0.45}{2A_{0}\Phi_{T}}\right)^{2}} = 502 \text{ kip} \\ &\qquad \qquad \text{Provided Force:} \end{split}$$

$$A_s f_v = 561.6 \text{ kip} > 502 \text{ kip}$$

LongitudinalReinfChk = "OK!"

4.2.12.5 Maximum Spacing of Transverse Reinforcement

(AASHTO LRFD 5.7.2.6)

Shear Stress

$$v_u = \frac{|v_u - \Phi_V v_p|}{\Phi_V b_v d_v} \qquad \qquad v_u = 0.158 \text{ ksi} \qquad \textit{(AASHTO LRFD Eq. 5.7.2.8-1)}$$

$$0.125 \cdot f_c = 0.625 \text{ ksi}$$

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

$$s_{\text{max}} = \min(0.8d_{\text{v}}, 24\text{in})$$

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

$$s_{\text{max}} = \min(0.4d_{\text{v}}, 12\text{in})$$

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

TxDOT limits the maximum transverse reinforcement spacing to 12".

$$s_{max} = 12.00 \text{ in}$$

 $s_{bar S} = 7.80 \text{ in} < s_{max}$

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.

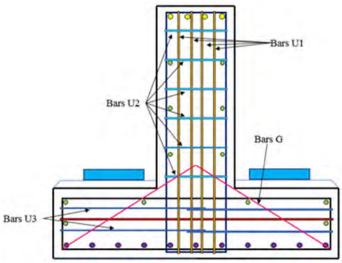


Figure 4.25 End Face Section View of 0 Degree ITBC

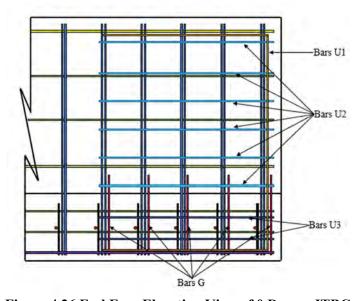


Figure 4.26 End Face Elevation View of 0 Degree ITBC

4.2.14 Skin Reinforcement (Bars T)

Try $7 \sim \# 6$ bars in Stem and $3 \sim \# 6$ 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)

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

TxDOT typically uses: a = 6 in

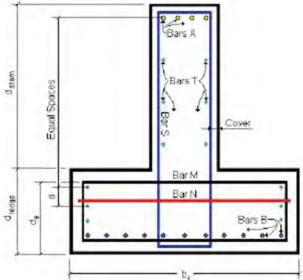


Figure 4.27 Section View for T Bars of 0 Degree Skewed ITBC

(AASHTO LRFD 5.6.7)

4.2.14.1 Required Area of Skin Reinforcement

$$A_{-1} = 0.012 \cdot (d - 30)$$

4.2.14.1 Required Area of Skin Reinforcement
$$A_{sk_Req} = 0.012 \cdot (d - 30)$$

$$A_{sk_Req} = 0.62 \frac{in^2}{ft}$$
(AASHTO LRFD Eq. 5.6.7-3)

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)

 $s_{req} = minimum of:$

$$\frac{A_{bar_T}}{A_{skReq}} = 8.52 \text{ in}$$

$$\frac{d_{s_neg}}{6} = 13.57 \text{ in}$$

$$\frac{d_{s_pos}}{6} = 13.57 \text{ in}$$

& 12 in

$$s_{reg} = 8.52 in$$

4.2.14.3 Actual Spacing of Skin Reinforcement

Check T Bars spacing in Stem:

$$\begin{split} h_{top} &= d_{stem} - \left(cover + \frac{d_{bar_S}}{2} + \frac{d_{bar_A}}{2}\right) + \left(cover + \frac{d_{bar_M}}{2} + \frac{d_{bar_T}}{2}\right) \\ h_{top} &= 56.67 \text{ in} \end{split}$$

$$s_{skStem} = \frac{h_{top}}{NoTBarsStem+1}$$

$$s_{skStem} = 7.08 in$$

$$s_{skStem} < s_{req}$$

SkinSpacing = "OK!"

Check T Bars spacing in Ledge:

$$\begin{split} h_{bot} = d_{ledge} - \left(cover + \frac{d_{bar_M}}{2} + \frac{d_{bar_T}}{2}\right) - \left(cover + \frac{d_{bar_S}}{2} + \frac{d_{bar_B}}{2}\right) \\ h_{bot} = 21.17 \text{ in} \end{split}$$

$$s_{skLedge} = \frac{h_{bot} - a}{NoTBarsLedge -}$$

$$s_{skLedge} = 7.59 in$$

$$s_{skLedge} < s_{req}$$

SkinSpacing = "OK!"

Check if "a" is less than s_{req}

$$a = 6 in < s_{rea}$$

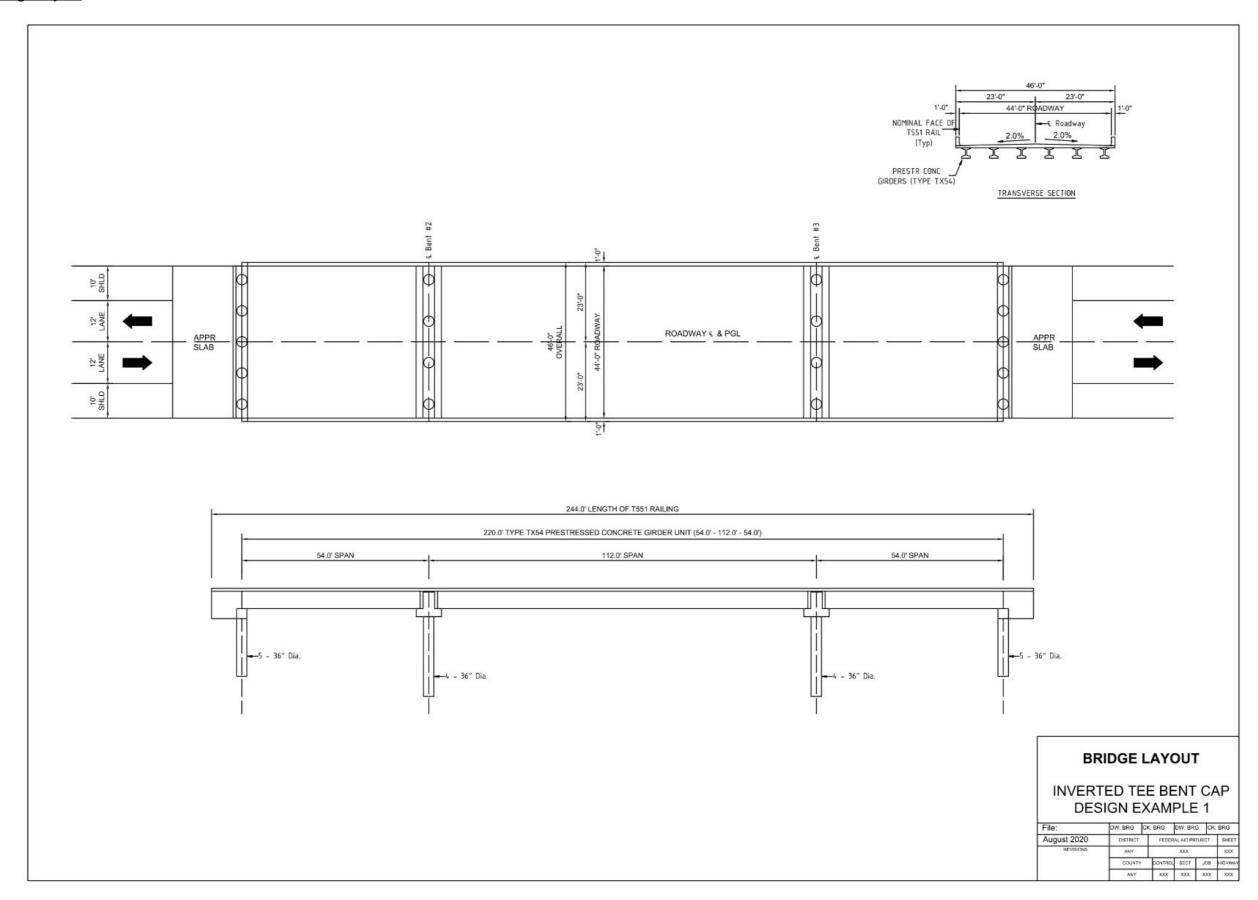
SkinSpacing = "OK!"

Skin Reinforcement Summary:

Use $7 \sim \# 6$ bars in Stem and $3 \sim \# 6$ 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

```
CAP18 Version 6.00 ITBC Design Example 1, Skew = 0.00
$Problem Card -----
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:
Fry Tab2 Tab3 Tab4 on Table 4 Envelope Pr
            Env Tab2 Tab3 Tab4
            Env Tab2 Tab3 Tab4 on Table 4 Envelope Print Skew Angle
X X X X X XX XXXXXXXXXX
16 0.0
                                 16
                                               0.0
STABLE 2 - CONSTANTS -----
                       Anly Opt (1=Working, |-Movable Load Data--| 2=Load Factor,3=Both)
  TABLE 2a
           Num Increment | Num Start Stop Step|Anly| Load Factors:
                     S
           Inc
               Length
           XX XXXXXXXXX
$
            92 0.5
 TABLE 2b
          Max # | ------Live Load Reduction Factors------
   Overlay
Str - Stringers, Sup - Supports

MCP - Moment Control Points

VCP - Shear Control Points
  Number of input values for
          Lane Str Sup MCP VCP
 XX XX XX XX XX XX (Num Inputs) 3 6 4 11 8
 Left Lane Boundary Stations
 Right Lane Boundary Stations
Ŝ
 $
  Station of Stringers (two rows max, may be at tenths of stations, XX.X)
 Station of Supports (two rows max)
         Moment Control Point Stations (two rows max)
        6 10 22 34 38 46
                                54
                                   58
                                       7.0
 (Mom CP)
           86
  Shear Control Point Stations (two rows max)
 56 60
$TABLE 4 - STIFFNESS AND LOAD DATA -----
                     Bending Sidewalk, Cap &
           Station 1 if Stiffness Slab Stringer Moving
                                                 Overlav
          From To Cont'd of Cap Loads
$Comments
                                   Loads Loads
                                                 Loads, DW
              $XXXXXXXXXXXXXX XXX
           2
(CAP EI & DL)
               90
                    8.09E+07
                                   -2.427
(DL Span1, Bm1)
                                   -50.17
                6
(DL Span1, Bm2)
           22
                                   -50.17
                                                 -5.04
(DL Span1, Bm3)
            38
               38
                                   -50.17
                                                 -5.04
                                   -50.17
(DL Span1, Bm4)
            54
               54
                                                 -5.04
(DL Span1, Bm5)
            70
               70
                                   -50.17
                                                 -5.04
(DL Span1, Bm6)
            86
               86
                                   -50.17
                                                 -5.04
(DL Span2, Bm1)
                                   -104.1
                                                 -10.5
                                                 -10.5
(DL Span2, Bm2)
            22
               22
                                   -104.1
(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
           0
                                          -4.92
(Dist. Lane Ld)
               20
(Conc. Lane Ld)
                                          -21.3
(Conc. Lane Ld) 16 16
                                          -21.3
```

4.2.15.3 CAP 18 Output File

AUG 06, 2020 TEXAS DEPARTMENT OF TRANSPORTATION (TXDOT) PAGE 1 CAP18 BENT CAP ANALYSIS Ver. 6.2 (Jul, 2011) PSF HIGHWAY PD- CONTROL- CODED COUNTY NO IPE SECTION-JOB BY DATE NO 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 CARDS INPUT THIS PROBLEM 16 OPTION TO CLEAR ENVELOPES BEFORE LANE LOADINGS (1=YES) 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): 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 ANALYSIS OPTION (1=WORKING STRESS, 2=LOAD FACTOR, 3=BOTH) 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 LIST OF LOAD COEFFICIENTS CORRESPONDING TO NUMBER OF LANES LOADED 5 1.000 1.200 0.850

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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 NUM OF NUM OF NUM MOM NUM SHEAR LANES STRINGERS SUPPORTS 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

FIXED-OR-MOVABLE ------ FIXED-POSITION DATA ----- MOVABLE-STA STA CONTD CAP BENDING SIDEWALK, STRINGER, OVERLAY POSITION FROM TO IF=1 STIFFNESS SLAB LOADS CAP LOADS LOADS SLAB LOADS

(K-FT*FT) (K) (K) (K)

2 90 0 80900000.000 0.000 -2.427 0.000 0.000 6 6 0 0.000 22 22 0 0.000 0.000 -50.170 -5.040 38 38 0 0.000 0.000 -50.170 -5.040 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 0.000 0.000 -50.170 -5.040 86 86 0 0.000 0.000 -104.100 -10.500 0.000 0.000 -104.100 -10.500 0.000 6 6 0 0.000 0.000 22 22 0 38 38 0 0.000 0.000 -104.100 -10.500 0.000 0.000 -104.100 -10.500 54 54 0 0.000 0.000 70 70 0 0.000 0.000 -104.100 -10.500 0.000 0.000 -104.100 -10.500 0.000 86 86 0 0.000 0 20 0 0.000 0.000 0.000 0.000 -4.920 0.000 -21.300 0.000 0.000 0.000 4 4 0 16 16 0 0.000 0.000 0.000 0.000 -21.300

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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) DEFLECTIO	N (FT) MO	MENT (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 30	14.50 15.00	-0.000004 -0.000003	-59.7 -108.7	-96.8 -99.2	
31	15.50	-0.000003	-158.9	-101.6	
32	16.00	0.000000	-210.3	-101.6	
33	16.50	0.000000	-262.9	-104.5	
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	

TABLE 4A. DEAD LOAD RESULTS (WORKING STRESS)

STA	DIST X (FT)	DEFLECTION	(FT) MOI		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 80.5	-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	71.4 -23.8 -120.2	-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

STA	EFFECT	ORDER	MAXIMUM	LANE STA	ORDER MAXIMUM	LANE
6	-9.7 0 1 2 3 0*	0.0 0.0 0.0 0.0	0 0 1 0 2 0 3 0	.0		
10	-378.5 0 1 2 3 0*	0.0 0.0 0.0 0.0	0 -17 1 -17 2 0 3 0 0*	76.2 1 2 76.2 1 2 .0	2	
22	249.2 0 1 2 3 0*	202.0 201.2 9.3 0.0	0 13 0 1 12 1 3 62 2 3 0	-33.4 2 -33.4 2 0.0	36 36	
34	-316.7 0 1 2 3 0*	18.7 18.7 0.0 0.0	3 62 0 3 62 1 2 -84 3 0 2*	-136.3 0 -116.6 1 4.7 2 32	18 12 2	
38	71.4 0 1 2 3 0*	83.6 83.6 3.2 0.0	2 32 0 2 32 1 3 62 2 3 0	-58.8 1 -58.8 1 0.0	9	
46	110.2 0 1 2 3 0*	69.4 69.4 0.0 0.0	2 36 0 2 36 1 2 -27 3 0 2*	7.8 3 63	9 9 3	
54	71.4 0 1 2 3 0*	83.6 83.6 3.2 0.0		-58.8 3 -58.8 3 0.0	63 63	
58	-316.7 0 1 2 3 0*	18.7 18.7 0.0 0.0	1 9 0 - 1 9 1 - 2 -84 3 0 2*	4.7 2 40	54 60)	

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

70 249.2 202.0 0 59 0 -33.4 2 36 201.2 3 60 1 -33.4 2 36 9.3 1 9 2 0.0 0 2 3 3 0.0 0.0 82 -378.5 0.0 0 -176.3 3 70 0 0.0 1 -176.3 3 70 1 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

SIA	EFFECT	ORDE	R MAXIMUM LANE STA ORDER MAXIMUN
8	1	0.0 0.0 0.0 0.0	0 -88.1 1 2 1 -88.1 1 2 2 0.0 3 0.0 0*
12	114.3 0 1 2 3 0*	44.8 44.8 1.6 0.0	1 6 0 -5.6 2 36 1 6 1 -5.6 2 36 3 62 2 0.0 3 0.0 0*
32	-104.0 0 1 2 3 0*	1.6 1.6 0.0 0.0	3 62 0 -54.6 0 15 3 62 1 -53.0 1 12 2 -11.2 2 32 3 0.0 0*
36	194.1 0 1 2 3 2*	87.6 84.1 30.7 0.0	0 28 0 -7.8 3 63 2 32 1 -7.8 3 63 1 12 2 0.0 3 0.0 0*
56	-194.1 0 1 2 3 0*	7.8 7.8 0.0 0.0	1 9 0 -87.6 0 44 1 9 1 -84.1 2 40 2 -30.7 3 60 3 0.0 2*
60	104.0 0 1 2 3 0*	54.6 53.0 11.2 0.0	0 57 0 -1.6 1 9 3 60 1 -1.6 1 9 2 40 2 0.0 3 0.0 0*
80	-114.3 0 1 2 3 0*	5.6 5.6 0.0 0.0	2 36 0 -44.8 3 66 2 36 1 -44.8 3 66 2 -1.6 1 9 3 0.0 0*
84	184.4 0 1 2 3 0*	88.1 88.1 0.0 0.0	3 70 0 0.0 3 70 1 0.0 2 0.0 3 0.0 0*

REACTION (K)

AT DEAD LD LANE POSITIVE LOAD AT LANE NEGATIVE LOAD AT STA EFFECT ORDER MAXIMUM LANE STA ORDER MAXIMUM LANE STA

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA	DIST X	MAX +	MOM MA	 AX - MOM	 MAX + SHEAR	MAX - SHEAR
	(FT)	(FT-K)	(FT-K)	(K) (
-1	-0.50	0.0	0.0	0.0	0.0	
0	0.00	0.0 0.0 0.0 -0.6	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 -2.4	-0.6	
3	1.50	-0.6	-0.6	-2.4	-2.4	
4 5	2.00 2.50	-2.4	-2.4 -5.5	-4.9	-4.9	
6	2.50	-5.5	-5.5	-7.5	-7.5 147 5	
7	3.00	-9.7	-9.7 -152.9 -297.4 -443.1	-181 0	-147.5 -287.7	
8	4.00	-191.7	-297.4	-1844	-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	-590.0 -507.1	170.5	110.1	
	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	-507.1 -425.5 -345.1 -265.9 -187.9 -111.2	160.8	100.4	
16	8.00	63.6	-111.2	158.4	97.9	
17	8.50	136.3	-111.2 -35.6 32.7 78.6 123.4 166.9 209.2 165.0 119.4 72.4 24.0	156.0	95.5	
	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	-14/./	
24 25	12.00	344.9	72.4	-82.7	-150.2	
26	12.50	104 5	24.0	-85.Z 97.6	152.0	
27	13.00	1194.5	-26.0	-07.0	-155.0	
28	14.00	47.3	-20.0	-90.0	-157.5	
29	14.50	-23.4	-26.0 -77.1 -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	-162.3 -164.7 -167.2 -169.6 -172.0 27.4 187.2 184.8 182.4 95.0 7.7 5.3	
40	20.00	183.1	27.1 38.4	23.9	5.3	
41 42	20.50	187.3	38.4 44.9	21.4	2.8	
42	21.00	190./	44.9	19.0	0.4	

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA (DIST X	MAX + I FT-K) (MOM MA	(K) (F	 MAX + SHEAR ()	MAX - SHEAR
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	
/9	39.50	-155.9	-345.1	-105.2	-165./	
80	40.00	-230.8	-425.5	-107.6	-168.1	
81	40.50	-306.4	-507.1	-110.1	-1/0.5	
82	41.00	-3/8.5	-590.0	64.1 292.5	18.1	
83	41.50	-284.4	-443.1	292.5	186.8	
84	42.00	-191.7	-297.4	290.1 287.7	184.4	
85	42.50	-100.1	-152.9	287.7	181.9	
86	43.00	-9.7	-9./	147.5	94.6	

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TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA	A DIST	X MAX	+ MOM +	MAX - MO	DM MAX	+ SHEAR	MAX - SHEAR
	(FT)	(FT-K)	(FT-K)	(K)	(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		

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TABLE 7. MAXIMUM SUPPORT REACTIONS (WORKING STRESS)

STA	DIST X	MAX+	REACT	MAX - REACT
(FT)	(K)	(K)	
				-
10	5.00	461.8	301	.7
34	17.00	486.7	29	6.7
58	29.00	486.7	29	6.7
82	41.00	461.8	30	1.7

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

STA	EFFECT	ORDER	MAXIMUM	LANE STA	ORDER	MAXIMUM	
6	-12.1 0 1 2 3 0*	0.0 0.0 0.0 0.0	0 0. 1 0. 2 0. 3 0. 0*	.0 .0			
10	-480.8 0 1 2 3 0*	0.0 0.0 0.0 0.0	0 -30 1 -30 2 0. 3 0. 0*	8.4 1 2 8.4 1 2 0			
22	316.4 0 1 2 3 0*	353.5 352.1 16.3 0.0	0 13 0 1 12 1 3 62 2 3 0. 0*	-58.4 2 -58.4 2 0.0	36 36		
34	-401.8 0 1 2 3 0*	32.7 32.7 0.0 0.0	3 62 0 - 3 62 1 - 2 -14 3 0. 2*	238.5 0 204.0 1 8.2 2 32 0	18 12 2		
38	91.2 0 1 2 3 0*	146.3 146.3 5.6 0.0	2 32 0 2 32 1 3 62 2 3 0. 0*	-102.9 1 -102.9 1 0.0	9 9		
46	139.7 0 1 2 3 0*	121.4 121.4 0.0 0.0	2 36 0 2 36 1 2 -48 3 0. 2*	-48.7 1 -48.7 1 3.7 3 63 0	9		
54	91.2 0 1 2 3 0*	146.3 146.3 5.6 1 0.0	2 40 0 - 2 40 1 - 10 2 0 3 0.0	102.9 3 102.9 3 0.0	63 63		
58	-401.8 0 1 2 3 0*	32.7 32.7 0.0 0.0	1 9 0 -2 1 9 1 -2 2 -148 3 0.0	38.5 0 5 04.0 3 6 3.2 2 40	54 50		

MOMENT (FT-K)

AT DEAD LD LANE POSITIVE LOAD AT LANE NEGATIVE LOAD AT STA EFFECT ORDER MAXIMUM LANE STA ORDER MAXIMUM LANE STA

70 316.4 353.5 0 59 0 -58.4 2 36 352.1 3 60 1 -58.4 2 36 16.3 1 9 2 0.0 1 2 3 0.0 3 0.0 0* 0* 82 -480.8 0.0 0 -308.4 3 70 0 1 0.0 -308.4 3 70 2 0.0 2 0.0 3 3 0.0 0.0 0* 86 -12.1 0.0 0 0.0 0 0.0 0.0 1 1 0.0 0.0 0.0 3 3 0.0 0* 0*

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 1 2 3 0*	0.0 0 -154.2 1 2 0.0 1 -154.2 1 2 0.0 2 0.0 0.0 3 0.0 0*
12	145.0 0 1 2 3 0*	78.4
32	-131.8 0 1 2 3 0*	2.7 3 62 0 -95.6 0 15 2.7 3 62 1 -92.7 1 12 0.0 2 -19.5 2 32 0.0 3 0.0 0*
36	246.5 0 1 2 3 2*	153.2 0 28 0 -13.6 3 63 147.2 2 32 1 -13.6 3 63 53.7 1 12 2 0.0 0.0 3 0.0 0*
56	-246.5 0 1 2 3 0*	13.6
60	131.8 0 1 2 3 0*	95.6 0 57 0 -2.7 1 9 92.7 3 60 1 -2.7 1 9 19.5 2 40 2 0.0 0.0 3 0.0 0*
80	-145.0 0 1 2 3 0*	9.7 2 36 0 -78.4 3 66 9.7 2 36 1 -78.4 3 66 0.0 2 -2.7 1 9 0.0 3 0.0 0*
84	234.3 0 1 2 3 0*	154.2 3 70 0 0.0 154.2 3 70 1 0.0 0.0 2 0.0 0.0 3 0.0 0*

REACTION (K)

AT DEAD LD LANE POSITIVE LOAD AT LANE NEGATIVE LOAD AT STA EFFECT ORDER MAXIMUM LANE STA ORDER MAXIMUM LANE STA

.....

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA	DIST)	 (MAX +	MOM N	 1AX - MON	 M MAX + SHEA	R MAX - SHEAR
(FT)		(FT-K)	(K)	(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		-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 10	4.50 5.00	-361.4 -480.8	-639.0 -851.0	-237.4	-422.4 - 95.5	
11	5.50	-383.1	-734.4	-15.1 242.2	136.4	
12	6.00	-278.8	-619.5	239.1	130.4	
13	6.50	-174.8	-506.0			
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		
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		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			-219.2	
24	12.00		128.6			
25	12.50					
26	13.00		3.3	-110.3		
27	13.50		-62.3	-113.4		
28	14.00		-129.5	-116.4		
29	14.50		-198.1	-119.5		
30	15.00					
31	15.50				5 -243.5	
32	16.00	-230.9				
33 34	16.50 17.00					
35	17.50		-528.0			
36	18.00					
37	18.50					
38	19.00					
39	19.50			37.5	5.0	
40	20.00		5.2	34.5	1.9	
41	20.50		21.7	31.4	-1.1	
42	21.00		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

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TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

					MAX + SHEAR	MAX - SHEAR
	(FI) (FT-K) (FI-K)	(K) (I	·····	
49	24.50	287.1	35.5	7.2	-25.4	
50	25.00	285.4	30.2	4.1	-28.4	
51 52	25.50	282.2 278.1	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 59.0	-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 -362.5	-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 -167.4	-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	-105.4 -12.5 88.3	-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	190.1 301.6 413.0	3.3	228.3	110.3	
67	33.50	413.0	66.9	225.3	107.3	
68	34.00	523.1 632.4 740.6	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	640.4 539.8 438.2	194.0	-106.0	-211.8	
72	36.00	539.8	140.3	-109.1	-214.9	
/3	36.50	438.2	85.0	-112.1	-217.9	
74	37.00	336.3 233.6 131.2	28.2	-115.1	-220.9	
75	37.50	233.6	-67.2	-118.2	-224.0	
/6	38.00	131.2	-1/4.6	-121.2	-227.0	
//	38.50	29.0	-283.6	-124.2	-230.0	
/8 70	39.00	29.0 -72.4 -174.8	-394.0	127.3	-233.1	
/9	40.00	-174.8	-506.0	130.3	-230.1	
01	40.00	-278.8 -383.1 -480.8	7244	126.4	-239.1	
02	40.50	-383.1 400.0	-/34.4	-136.4	-242.2	
02	41.00	-48U.8 261.4	-851.0	422.4	15.1	
0.0	41.50	-301.4	429 E	422.4	237.4	
04 05	42.00	-361.4 -243.5 -127.0	210.5	419.4	234.3	
86	42.50	-127.0	-213.0	212.7	120.2	
27	43.00	-12.1 -6.8 -3.0	-6.8	0.1	0.1	
88	44.00	-3.0	-3.0	6.1	6.1	
89	44.50	-0.8	-0.8	3.0	3.0	
09	44.50	-0.0	-0.6	3.0	5.0	

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

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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)

.....

ST	A DIST	X MAX	+ MOM +	MAX - M	OM MAX	+ SHEAR	MAX - SHEAR
	(FT)	(FT-K)	(FT-K)	(K)	(K)		
93	46.50	0.0	0.0	0.0	0.0		

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TABLE 7. MAXIMUM SUPPORT REACTIONS (LOAD FACTOR)

				-
STA	DIST X	MAX+	REACT	MAX - REACT
(FT)	(K)	(K)	
				-
10	5.00	660.0	379	.8
34	17.00	703.5	370	0.9
58	29.00	703.5	370	0.9
82	41.00	660.0	379	9.8

4.2.15.4 <u>Live Load Distribution Factor Spreadsheet</u>

4.2.15.4.1 *Spans 1 & 3*

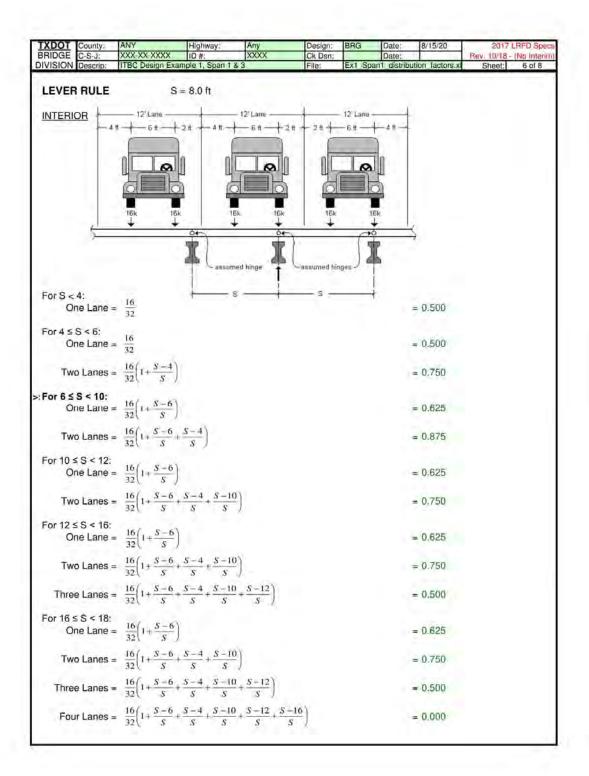
Division Descrip: TBC Design Example 1, Span 1 & 3 File: Ext Span1 distribution ractors.x Sheet: 1 c	TXDOT County: BRIDGE C-S-J:	ANY XXX-XX	K-XXXX	Highway: ID #:	Any XXXX	Design: Ck Dsn:	BRG	Date:	8/15/20	2017 LRFD Spa Rev. 10/18 - (No Interi
Live Load Distribution Factors are calculated according to 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 _o = 6 Clang to Clay, L = 50.38 ft Avg. Skew Angle, θ = 0.000 deg Slab Thickness, θ = 8.00 in Rail Width, RW = 1 ft Roadway With, W = 44 ft Number of Lanes, N _c = 3 Longitudinal Stiffness Parameter: (4.6.2.2.1-1) θ = (n) = 34.49 (dist. blw cog of bm & deck) θ = 1.000 θ = 1.000 θ = 1.271611 in θ *For typical cross sections (a.e.l.) 5 k), Table 4.6.2.2.1-1 RESULTS: Interior Shear LLDF, θ = 1271611 in θ *For typical cross sections (a.e.l.) 5 k), Table 4.6.2.2.1-1 Exterior Moment LLDF, θ = 1.0.794 Exterior Moment LLDF, θ = 1.0.794 Exterior Moment LLDF, θ = 1.0.0.1 (1.2.0 Lr.) (1.2							Ex1_Spa		ution_factors.>	
(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 _o = 6 Cl _{big} to Cl _{big} to Elegancy = 8.00 it Avg. Skew Angle, θ = 0.000 deg Slab Thickness, t_s = 8.00 in Rail Width, RW = 1 it Roadway With, W = 44 it Number of Lanes, N _t = 3 Longitudinal Stiffness Parameter: (4.6.2.2.1-1) θ_0 (in) = 34.49 (dist. b/w cog of bm & deck) θ_0 = 1.000 θ_0 = 1.000 θ_0 = 1.000 θ_0 = 1.000 θ_0 = 1.001				LRFD L	ive Load D	istributio	n Facto	rs*		
$e_{o} (in) = 34.49 \text{ (dist. b/w cog of bm \& deck)} \\ n = 1.000 \\ K_{o} = n(I + Ae_{o}^{2}) = 1271611 \text{ in}^{4}$ *For typical cross sections (a.e.l.) & k). Table 4.6.2.2.1-1 RESULTS: Interior Shear LLDF, gV _{interior} Interior Moment LLDF, gM _{interior} Exterior Shear LLDF, gV _{exterior} Exterior Shear LLDF, gV _{exterior} Exterior Moment LLDF, gM _{exterior} O.794 CALCULATIONS: Shear LLDF Correction for Skew (Table 4.6.2.2.3c-1) Check θ : $0^{\circ} \le 0^{\circ} \le 60^{\circ}$ Check S: $3.5^{\circ} \le 8.0^{\circ} \le 10^{\circ}$ Check C: $20^{\circ} \le 50.4^{\circ} \le 240^{\circ}$ Check N: $20^{\circ} \le 50$	(2017 with no in The Lever Rule Factors is Ignore INPUT: Beam No. Beam CL _{brg} to CL Beam Spacion Avg. Skew And Slab Thickne Slab Overhang Rail Width Roadway Width Roadway Width	terim revision seed of the se	Tx54 6 50.38 8.00 0.00 8.00 3 1 44	ft ft deg in ft ft	by TxDOT p	Deck S Conc wt =	Range of Bab = 0.14	gn Manu of Applic 5 k/ft ³ 0 ksi	Beam weight = f'c = Ebeam = Yt = A =	3) and practices. 30 and practices. 4
	*For typical cross RESULTS: Interior Sh Exterior Sh	Ae _o ²) = sections near LLDient LL	127161: (a.e.i.j & I F, gV _{interic} F, gM _{interic} F, gV _{exteric}	Final LLDF 0.814 0.794 0.814		TxDOT policy Exterior When O lever rule for In no cas the LLDFs I When the designed for	cies: beams us H > S/2 th r a single e shall the or the inte Roadwa r one lane	se the inte ne exterio lane with e LLDF fo erior bear y width is e loaded o	erior LLDF who have been been LLDF as multiple proof the exterior ns. Hess than 20 pnly.	nen OH ≤ S/2. is determined by the resence factor of 1.0. beams be less than off, all beams are
Corr. = 1 - $c_1(\tan \theta)^{-1.5}$ = 1 - $0(\tan \theta)^{-1.5}$ where: $c_1 = 0.25 \left(\frac{K_g}{12.0Lr_s^{-3}}\right)^{-1.5} \left(\frac{S}{L}\right)^{0.5}$	Shear LLDF Co	Corr. = Correction	1.0 + 0.2 1.0 + 0.2 1.000 ion for S	$co \left(\frac{12.0 L t_s^3}{K_g} \right)$ 0 * [(12.0*50.	$\int_{0.3}^{0.3} \tan \theta$ 4*8^3)/(1,271)	C C C (,611)]^0,3	heck S: heck L: heck N _b * tan(0) Check I	3.5' ≤ 20' ≤ 6 ≥ 4	8.0' ≤ 16.0' 50.4' ≤ 240' 80°	OK OK
Corr. = 1.000 $c_1 = 0.000$ because $\Theta < 30^{\circ}$				14101 h				- (L.	C 4 3 0.5

TXDOT	County:	ANY		Any	Design:	BRG	Date:	8/15/20		LRFD Spec
BRIDGE DIVISION	C-S-J: Descrip:	ITBC Design Exa	ID #: ample 1, Span 1 & 3	XXXX	Ck Dsn: File:	Ex1 Span	Date:	tion factors.xl	Rev. 10/18 - Sheet:	(No Interim 2 of 8
INTER	IOR BE									
Shear I	LL Distrib	ution Per Lane	(Table 4.6.2.2.3a	a-1):						
	One La	ine Loaded								
		Lever Rule	(Table 3,6,1.1	.2)						
		mg = 0.0	625 * 1.2 =	0.750						
		Modify f	or Skew:							
			skew correction	on =	1,000					
			mg = 0.750 *	1.000 =	0.750					
		Equation	(5)							
		g = 0.3	$6 + \left(\frac{S}{25}\right)$							
			- 3	0.680						
			or Skew:							
			skew correction	on =	1.000					
			g = 0.680 * 1.	000 =	0.680					
		Range of App	olicability (ROA)	Checks						
			$3.5' \le 8.0' \le 1$		OK					
		Check t	s: 4.5" ≤ 8.0" ≤	12.0"	OK					
			20' ≤ 50.4' ≤	240'	OK					
		Check N	N _b : 6≥4		OK					
			from Table 4.6.	2.2.3a-1	because all	criteria is i	OK.			
		gV _{int1} =	0.88.0							
	Two or	More Lanes Lo	oaded							
		Lever Rule	(Table 3.6.1.1							
			ax(0.875 * 1.0, 0	.875 * 0.8	35, 0.875 * 0	.65) =	0.875			
		Modify f	or Skew:							
			skew correction		1.000					
		-	mg = 0.875 *	1.000 =	0.875					
		Equation	(5) (5)	2.0						
		g = 0.2	$+(12)^{-}(35)$)						
		g = 0.2	+ (8 / 12) - (8 / 3	5)^2.0 =	0,814					
		Modify f	or Skew:							
			skew correction		1,000					
		Section Commission	g = 0.814 * 1.		0.814					
			olicability (ROA)		(same as f			d)		
			from Table 4.6.	2.2.3a-1	because all	criteria is (OK.			
		gV _{int2+} =	0.814							
	TXDOT		V _{Interior} must be ≥	m·N _L ÷N _t						
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6 =		0.425					
		20ft ? Yes	-1 11 14/ 2011 -1	V 154	ha Masslanda	-111	اندادد	N - NI		
			at if W < 20ft, g\							
>>			at if W ≥ 20ft, g\	Interior IS T	ne waximum	or. gvinti	g v int2+1	HEINL +IND		
	gV _{inte}	erior = 0.814								

TXDOT	County:	ANY	Highway:	Any XXXX	Design:	BRG	Date:	8/15/20	A SHARE WAS ARREST	LRFD Spe
RIDGE	C-S-J: Descrip:	ITBC Design Exam	ID #: ple 1, Span 1 & 3	XXXX	Ck Dsn:	Ex1 Span	Date:	ion factors.xl	Rev. 10/18 - Sheet:	3 of 8
INTER	IOR BE				1					
		ibution Per Lane	(Table 4622	2h-11:						
Women		ne Loaded	(Table 4.0.2.2	.20-1).						
	One La	Lever Rule	(Table 3,6,1,	1 2)						
		mg = 0.62		0.750						
		Modify for		0.750						
		Widding for	skew correct	ion –	1.000					
			mg = 0.750 *		0.750					
		Caustian	ilig = 0.750	1.000 =	0.7.00					
		$\frac{\text{Equation}}{\text{g} = 0.06}$	$+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.4}$	$\int_{0.3}^{0.3} \left(\frac{K_g}{12Lt} \right)^{0.3}$	3					
			(8/14)^0.4 * (A No	* /	11/(12*50	.4*8^3))	0.1 =	0.590	
		Modify for		22:00	y contract	,				
		17200	skew correct	on =	1,000					
			g = 0.590 * 1	.000 =	0.590					
		Range of Appli	cability (ROA)	Checks	-					
			3.5' ≤ 8.0' ≤			ОК				
			4.5" ≤ 8.0" ≤			ОК				
		Check L:	20' ≤ 50.4' ≤	240'		ОК				
		Check N _b				OK				
		Check Ka	10,000 ≤ 1,2	71,611 ≤ 7	.000,000	ОК				
		Use Equation f	rom Table 4.6.	2.2.2b-1 be	ecause all c	riteria is (OK.			
		gM _{iort} =	0.590				47.10			
	Two or	More Lanes Loa	dod							
	1 WO OI	Lever Rule	(Table 3.6.1.	1.2\						
			(0.875 * 1.0, 0	C217-1-17	0.875 * 0	651 -	0.875			
		Modify for		1.073 0.00	, 0.073 0.	03) =	0.075			
		Widding To	skew correcti	00 -	1.000					
			mg = 0.875 *		0.875					
		Equation			- 0.1					
		g = 0.07:	$5 + \left(\frac{S}{9.5}\right)^{0.6}$	$\left(\frac{S}{L}\right)^{0.2} \left(\frac{I}{12}\right)^{0.2}$	$\left(\frac{X_g}{L_f^3}\right)^{0.1}$					
			+ (8/9.5)^0.6			611/(12*5	0.4*8^3))^0.1 =	0.794	
		Modify for								
			skew correct	on =	1,000					
			g = 0.794 * 1	.000 =	0.794					
		Range of Appli	cability (ROA)	Checks	(same as fo	or one lar	ne loade	d)		
		Use Equation f	rom Table 4.6	2.2.2b-1 be	ecause all c	riteria is (OK.			
		gM _{int2+} =	0.794			V.VE.V.9.19.1				
	TYDOT			m/N + M						
	INDUI	Policy states gM ₁ m·N _L ÷N _b =			0.425					
	In IA/ > C		0.85 * 3 / 6 =		0.420					
		20ft ? Yes	11 W - 2011 A	M. in th	a Maximus	Ma to	and m	N = N		
		Policy states that								
>>	-	Policy states that	1 VV = 2011, g	Winterior IS (I)	e waxiiiuii	Dir giving	ALVINIS+	THEAT STAPE		
	gM _{inte}	rior= 0.794								

	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
DIVISION	C-S-J: Descrip:	ITBC Design Exa	ID #: ample 1. Span 1	XXXX & 3	Ck Dsn: File:	Ex1 Sr	Date:	ution factors.x	Rev. 10/18 - (No Interim) Sheet: 4 of 8
	IOR BE		- Post 1, Spail 1		1, 110,	12			Total
		ution Per Lane	(Table 4.6.2.2	2.3b-1):					
		ine Loaded							
		Lever Rule	(Table 3.6	.1.1.2)					
		mg = 0.0	625 * 1.0 =	0.625	TxDOT us	es a mu	Itiple pre	sence facto	r of 1,0 for one
		Modify f	or Skew:		lane loade				
			skew corre	ection =	1,000				
			mg = 0.62	5 * 1.000 =	0.625				
		Use Lever Ru	le, as per AA	SHTO LRF	Table 4.6.2	2.2.3b-1	1		
		gV _{ext1} =	0.625						
100	Two or	More Lanes L	oaded						
	e and e	Lever Rule	(Table 3.6	.1.1.2)					
			ax(0.625 * 1.0		35, 0.625 * 0	.65) =	0.625		
			or Skew:						
			skew corre	ection =	1,000				
			mg = 0.62	5 * 1.000 =	0.625				
		Equation							
		$d_e = dist$	t. b/w CL web	to curb					
		$d_e = OH$	- Rail Width						
		$d_e =$	3ft - 1ft =	2.0	ti				
		0.0	(d)						
		e = 0.0	$+\left(\frac{d_e}{10}\right)$						
		e = 0.6	+ (2.0/10) =	0.800					
		g = e*g\	V _{int24} Eq						
			00 * 0.814 =	0.651					
			orrection is in		/(interior).				
		Range of App				ROA is	implicitly	applied to t	he exterior beam.
			nterior Beam	Transfer of the second	OK				
		Check o	d _e : -1.0' ≤ 2.0	0' ≤ 5.5'	OK				
		Check N	N _b : 6 ≠ 3		OK				
		Use Equation	from Table 4	1.6.2.2.3b-1	because all	criteria i	s OK.		
		gV _{ext2+} =	0.651						
	TXDOT	Policy states g	Vestorer must b	e ≥ aV					
		gV _{interior} =	0.814	G minimum					
	TXDOT	Policy states g		e ≥ m·N _L ÷N	h				
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6		0.425				
	Is OH ≤	S/2 ? Yes			-				
		20ft ? Yes							
>>	TXDOT	Policy states th	at if OH ≤ S/2	2, gV _{Exterior} is	gV _{interior} .				
1	TXDOT	Policy states th	at if OH > S/2	2 and W < 2	Oft, gV _{Exterior}	is the M	aximum o	of: gV _{ext1} , g\	Interior, and
		$m \cdot N_L \div N_b$.							
	TXDOT	Policy states th	at if OH > S/2	2 ans W ≥ 20	Off, gV _{Exterior} i	s the M	aximum c	of: gV _{ext1} , gV	ext2+- gVinterior
1 4 4		and m·N _L ÷N _b							
4= - 1	gV _{ext}	erior = 0.814							

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Spec
BRIDGE	C-S-J: Descrip:	ITBC Design Exa	ID#:	XXXX	Ck Dsn:	Eut Co	Date:	ution factors.x	Rev. 10/18 -	- (No Interim
1101011	RIOR BE		imple 1, Opan 1 c	x 3	Triid.	EAT OF	arri_ustrib	JUNI IGCIOIS.A	a Sheet.	3010
			o /Table 4 6 3	22411						
viomen		ribution Per Lan ne Loaded	e (Table 4.0.2	2.20-1).						
	One La	Lever Rule								
			625 * 1.0 =	0.625	T-DOT		Wels assure	and the factor		
		1.00	or Skew:	0.023	lane loade				r of 1,0 for a	ne
		Widdily I	skew corre	ction -	1.000		· ontonio	o o o o o		
			mg = 0.625		0.625					
		Use Lever Ru	-		-	9.94:1				
		gM _{ext1} =	0.625	INTO LAFE	1 1 2018 4.0.2	E.EU-).				
		-	-							
	Two or	More Lanes Lo		Total						
		Lever Rule	(Table 3.6.		and the second		The second			
		760.50	ax(0.625 * 1.0	, 0.625 * 0.8	35, 0.625 * 0.	65) =	0.625			
		Modify f	or Skew:	2 - 1 -	2 4/4/4					
			skew corre		1,000					
		04.000	mg = 0.625	1.000 =	0.625					
		Equation	123							
		e = 0.7	$7 + \left(\frac{d_v}{9.1}\right)$							
		e = 0.77	+ (2.0/9.1) =		0.990					
		$g = e^*gN$	A _{Int2+Eq}							
		g = 0.99	* 0.794 =	0.786						
		Skew C	orrection inclu	ded in gM(i	nterior).					
		Range of App	licability (RO	A) Checks	Interior	ROA is	implicitly	applied to t	he exterior b	beam.
		Check In	nterior Beam I	ROA:	OK					
		Check d	e: -1.0' ≤ 2.0'	≤ 5.5'	OK					
		Check N	l _b : 6 ≠ 3		OK					
		Use Equation	from Table 4	6.2.2.2d-1	because all o	riteria is	s OK			
		gM _{ext2+} =	0.786							
	TXDOT	Policy states gN	Mestarias must b	e ≥ aM _{interess}						
	30531	gM _{interior} =	0.794	- S. Millettor						
	TXDOT	Policy states gN	- Contraction	e≥m·N _i ÷N	l.					
			0.85 * 3 / 6		0.425					
	Is OH ≤	S/2 ? Yes			-					
		20ft ? Yes								
>>		Policy states th	at if OH ≤ S/2	, gM _{Exterior} is	gM _{interior}					
	TxDOT	Policy states the	at if OH > S/2	and W < 20	Oft, gM _{Exterior} i	s the M	aximum o	of: gM _{ext1} , gl	M _{interior} , and	
	TXDOT	Policy states th	at if OH > S/2	ans W ≥ or	off, aM-	s the M	aximum c	of: aM al	M aM	
	7,700 50 1	and m·N _L +N _b			A. Citetion	C 4110 1VII	and the control of	a see all a	THE PARTY OF THE	HOLL
	[aM									
	gM _{exte}	enor - U.194								



TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Spe
BRIDGE		XXX-XX-XXXX	ID #: ample 1, Span 1 &	XXXX	Ck Dsn: File:	Evil Con	Date:	ion factors.xl	Rev. 10/18 - Sheet:	7 of 8
IVISIOIV	Descrip.	ITDQ Design Ex	ample 1, Span 1 &	· J	Trile.	LAT OPA	III UISUIUUI	ion idulois.a	Sileet.	7 01 0
LEVER	RULE	S	6 = 8.0 ft							
INTERI	OR (con't)								
For 18 5	S < 22: ne Lane =	$= \frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$)					0.625		
Tw	Lanes =	$= \frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$)			03	- 0.750		
Three	e Lanes =	$= \frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S-12}{S}$	-18 S			-0.125		
Fou	r Lanes =	$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S-12}{S}$	$\left(\frac{-18}{S} + \frac{S-16}{S}\right)$			0.625		
For 22 s	S ≤ 24; ne Lane =	$= \frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$)				174	0.625		
Twe	Lanes =	$= \frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$)				0.750		
Three	e Lanes =	$= \frac{16}{32} \left(1 + \frac{S-6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S}{S}$	$\left(\frac{-18}{S}\right)$		10	-0.125		
Fou	r Lanes =	$= \frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S}{S}$	$\frac{-18}{S} + \frac{S - 16}{S}$	$+\frac{S-22}{S}$	l lu	-1.500		
		16k	4 ft = 2 ft	15k	t bings				S = OH =	8.0 f
		он — — х-	_ s	- assumer	i hinge			Rail Widti X = S+OH-	n = RW =	1.0 ft 8.0 ft
For X <	6: ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} \right)$					1.9	0.500		
For 6 ≤ Or	X < 12; ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					0.625		
For 12 s	X < 18; ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	-6)					0.625		
4	Lanan	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	6 X -12)					0.375		

BRIDGE	County:	ANY XXX-XX-XXXX	Highway:	Any XXXX	Design: Ck Dsn:	_	Date:	8/15/20	2017 LRFD Sper Rev. 10/18 - (No Interir
IVISION			ample 1, Span 1		File:			on factors.xl	Sheet: 8 of 8
Cont	San W								
LEVER	RULE								
EXTER	IOR (con'	t) S	= 8.0 f	t.	OH =	3.0 ft			
	3	RW	= 1.0 f	t X = S+0	OH-RW-2ft =	8.0 ft			
For 18	X < 24:								
O	ne Lane =	$=\frac{16}{32}\left(\frac{X}{S}+\frac{X}{S}\right)$	-6				=	0.625	
		22/10/ 10	5 2000	C=18\					
Tw	o Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	5 + 2 + 2	S			=	-0.250	
For 24 s	< X < 30:	16/4 4	65						
O	ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	-0)				-	0.625	
-	4	16 (X X -	6 X-12 X	(-18)				0.000	
IW	o Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - S}{S} \right)$	S	5			=	-0.250	
Three	e Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	$-6 + \frac{X-12}{2} + \frac{3}{2}$	$\frac{x-18}{9} + \frac{x-2}{9}$	4)		=	-1.250	
		32 (3)	3	2 3	5.				
For 30 s	≤ X < 36: ne Lane =	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X}{S}\right)$	-6)					0.625	
		32 (3	1	3 068				0.020	
Tw	o Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{-6}{S} + \frac{X - 12}{S} + \frac{3}{2}$	$\left(\frac{c-18}{S}\right)$			=	-0.250	
		State of the		COVE A L	4 X = 30				
Thre	e Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - S}{S} \right)$	\$	s s	S		-	-2.625	
For 36 5	≤ X < 42:	16/X X-	-6)					11.07.00	
O	ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$					=	0.625	
Tw	o Lanes =	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X}{S}\right)$	$\frac{6}{4} + \frac{X - 12}{4} + \frac{3}{2}$	C = LR			1.2	-0.250	
		2200		A . X	To A rate			Cinco	
Thre	e Lanes =	$=\frac{16}{32}\left(\frac{X}{S}+\frac{X}{S}\right)$	$\frac{6}{S} + \frac{X-12}{S} + \frac{2}{S}$	$\frac{C-18}{S} + \frac{X-2}{S}$	$\frac{4}{4} + \frac{X - 30}{S}$			-2.625	
			4 2 2 2			X -36			
Fou	r Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	5	S + S	-+S	5	=	-4.375	
For 42	≤ X ≤ 48:	16/Y Y-	6)						
O	ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$					=	0.625	
Two	o Lange -	$= \frac{16}{32} \left(\frac{X}{8} + \frac{X - X}{8} \right)$	6 X-12 J	(-18)				-0.250	
1.44	o Laries -	32 8 8	2	S)			-	-0.230	
Three	e Lanes =	$=\frac{16}{32}\left(\frac{X}{5} + \frac{X-7}{5}\right)$	$\frac{-6}{e} + \frac{x-12}{e} + \frac{x}{2}$	$\frac{x-18}{5} + \frac{x-2}{5}$	$\frac{4}{e} + \frac{X - 30}{e}$		=	-2.625	
		0 × 5 0		0 0		X = 36 Y	-421		
Fou	r Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	S + S + S	8 + 8	+ x 50 +	5 + 3	<u>s</u> =	-6.500	
INTERN	00				EVICO	IOR			
One La	<u>OR</u> ne Loade	d	- 0.625		<u>EXTER</u>	<u>IOH</u> ne Loaded			0.625
	nes Loade		= 0.625 = 0.875			nes Loaded		=	0.625
	anes Load		= 0.875			anes Loade		-	0.625
	nes Load		= 0.875			nes Loade		_	0.625
. 551 60			5.075		. our ca	LOUGE			-104-0

4.2.15.4.2 Span 2

BRIDGE C-S-J: IVISION Descrip:	ITRC Design Fa		Any	Design: Ck Dsn:	BRG	Date:	8/14/20	2017 Rev. 10/18	
	Troo boolgir L	ID #: ample 1, Span 2	20000	File:	Ex1_Sp		ution_factors.xl	Sheet:	1 of 8
		I PED I	ive Load I	Dietribution	Facto	ret			
Live Load Distribe (2017 with no inte The Lever Rule is Factors is ignored INPUT: Beam T No. Beams CL _{brg} to CL _{br} Beam Spacing Avg. Skew Angl	erim revisions is used when odd. $ype = Tx54, N_b = 6$ $r_g, L = 106.7$ $g, S = 8.00$ $e, \theta = 0.00$	are calculated) as prescribed outside the Ran f ft deg	by TxDOT	AASHTO L	RFD Br D Desi Range	idge Desi gn Manua	Beam weight = beam f'c = beam yt =	0.145 (8.5) 5312 (30.49)	ces, rection k/ft ³ ksi ksi in
Slab Thicknes: Slab Overhang, Rail Width, Roadway Width Number of Lanes	OH = 3 RW = 1 , W = 44	in ft ft ft					A = 1 =	817.0 i 299740 i	
$K_0 = n(I+A)$	e_a^2) = 12716	in"							
*For typical cross s	ections (a,e,i,j	& k). Table 4.6.2		The Final II	DE may	he modifie	ed according t	a the followi	ng

Check L:

Check N_b: 6≥4

Check θ: 0° < 30°

20' ≤ 106.8' ≤ 240'

 $c_1 = 0.000$ because $\Theta < 30^\circ$

OK

OK

Set 0 = 0°

= 1.0 + 0.20 * [(12.0*106.8*8^3)/(1,271,611)]^0.3 * tan(0) Corr. = 1.000

Moment LLDF Correction for Skew (Table 4.6.2.2.2e-1) Corr. = 1 - c_1 (tan θ)^1.5 = 1 - 0(tan θ)^1.5

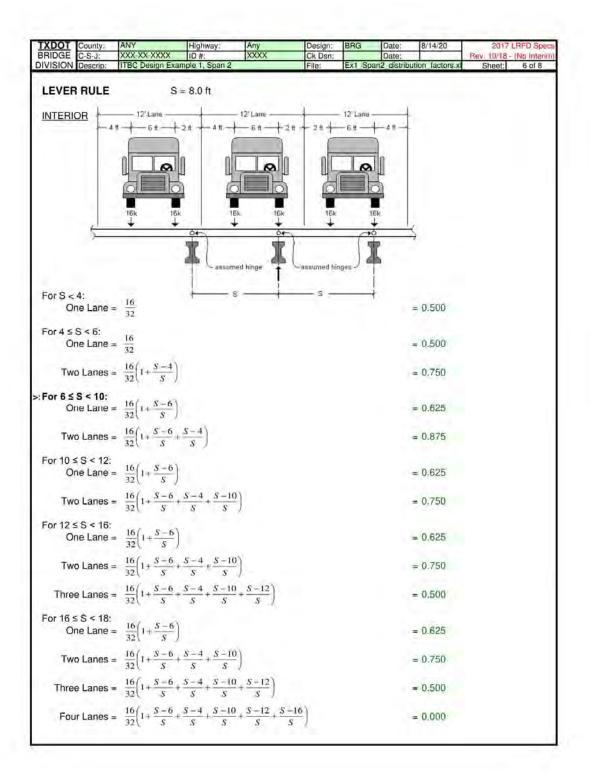
Corr. = 1.000

XDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/14/20		LRFD Spe
BRIDGE IVISION	C-S-J: Descrip:	ITBC Design Exa	ID #: mple 1, Span 2	XXXX	Ck Dsn: File:	Ex1 Spar	Date: n2 distribu	tion factors.xl	Rev. 10/18 - Sheet:	2 of 8
	IOR BE									
Shear L	L Distrib	ution Per Lane	Table 4.6.2.2	.3a-1):						
		ne Loaded	.,							
		Lever Rule	(Table 3.6.)	1.1.2)						
		mg = 0.6	625 * 1.2 =	0.750						
		Modify fo	or Skew:							
			skew corre	ction =	1,000					
			mg = 0.750	* 1.000 =	0.750					
		Equation	100							
		g = 0.36	$6 + \left(\frac{S}{25}\right)$							
		g = 0.36	+ (8 / 25) =	0.680						
		Modify to	or Skew:							
			skew corre	ction =	1.000					
			g = 0.680 *	1.000 =	0.680					
		Range of App	licability (ROA	A) 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	3' ≤ 240'	OK					
		Check N	l _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 Lo	paded							
		Lever Rule	(Table 3.6.)	1.1.2)						
		mg = Ma	ax(0.875 * 1.0	0.875 * 0.8	35, 0.875 * 0	.65) =	0.875			
		Modify fo	or Skew:							
			skew corre	ction =	1.000					
			mg = 0.875	* 1.000 =	0.875					
		Equation	(0) (2.0						
		g = 0.2	$+\left(\frac{3}{12}\right) - \left(\frac{3}{2}\right)$	5						
			(8/12) - (8/	35\\\20 -	0.814					
		7:0-20	or Skew:	33) 2.0 =	0,014					
		widany it	skew corre	ction -	1,000					
			g = 0.814 *		0.814					
		Range of App			(same as f	or one la	ne loade	ed)		
		Use Equation						,		
		gV _{int2+} =	0.814	0.2.2.00	bedause and	antena is	Cis.			
	TUDOT			Va. 10 es c						
	IXDOI	Policy states g\ m·N _L ÷N _b =	0.85 * 3 / 6							
	1-14/-		0.05 3/6	=	0.425					
		20ft ? Yes Policy states the	at II W = 20#	oV in	he Mavimum	of: aV	and m.l	N. s.N.		
		Policy states the								
>>			1	9 Vinlerior 15 1	ing maximum	or. gvint	9 V int2+1	HUME STAP		
	gV _{inte}	erior = 0.814								

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/14/20	20171	LRFD Specs
BRIDGE	C-S-J: Descrip:	ITBC Design Exam	ID#:	XXXX	Ck Dsn: File:	Evi So	Date:	ition factors.xl	Rev. 10/18 - Sheet:	(No Interim) 3 of 8
	IOR BE		ipie 1, Opail 2		Triid.	LAT OP	arie distribu	non lactore.si	. Sileet,	3010
		ibution Per Lane	/Table 4.6.2	2.2h 11						
Momen		ne Loaded	(Table 4.0.2	.E.EU-1).						
	One La	Lever Rule	(Table 3.6,	1 1 2)						
			25 * 1.2 =	0.750						
		Modify for		0.750						
		Widdiny for	skew corre	ction =	1,000					
			mg = 0.750		0.750					
		Equation			- (t.t.					
		g = 0.06	$+\left(\frac{S}{14}\right)^{0.4}$	$\left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12Lt_s}\right)^{0.3}$	3					
		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 corre	ction =	1.000					
			g = 0.453*	1,000 =	0.453					
		Range of Appli	cability (RO)	A) Checks						
		Check S:	3.5' ≤ 8.0'	≤ 16.0'		OK				
		Check ts:	4.5" ≤ 8.0"	≤ 12.0"		OK				
		Check L:	20' ≤ 106.8	3' ≤ 240'		OK				
		Check N _b	: 6≥4			OK				
		Check Kg	: 10,000 ≤ 1	,271,611 ≤ 7,	000,000	OK				
		Use Equation f	rom Table 4	6.2.2.2b-1 be	cause all	criteria is	OK.			
		gM _{int1} =	0.453							
	Two or	More Lanes Loa	aded							
		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 corre	ction =	1.000					
			mg = 0.875	* 1.000 =	0.875					
		Equation	(= > 0.0	(a) 02/ P	70.1					
				$\left(\frac{S}{L}\right)^{0.2} \left(\frac{K}{12L}\right)^{0.2}$						
				8 * (8/106.8)^(0.2 * (1,27	1,611/(1	2*106.8*8	3^3))^0.1 =	0.649	
		Modify for		and the	west 200					
			skew corre		1,000					
			g = 0.649 *		0.649					
		Range of Appli	cability (RO	A) Checks	(same as f	or one la	ane loade	ed)		
		Use Equation f	rom Table 4	.6.2.2.2b-1 be	cause all o	oriteria is	OK.			
		$gM_{int2+} =$	0.649							
	TXDOT	Policy states gM	Interior must be	$a \ge m \cdot N_L + N_b$						
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6	=	0.425					
	Is W ≥ 2	20ft ? Yes								
		Policy states that	t if W < 201t,	gMinternal is the	Maximun	n of: gM	and m	N _L ÷N _b -		
>>	TXDOT	Policy states that	t if W ≥ 20ft,	gM _{interior} is the	Maximum	n oli gM	nn gMinte	m·NL÷Nb		
	gM _{inte}	erior = 0.649								
			-							

	County:	ANY	Highway:	Алу	Design:	BRG	Date:	8/14/20	2017 LRFD Sp
DIVISION	C-S-J: Descrip:	ITBC Design Ex	ID #: ample 1. Span 2	XXXX	Ck Dsn: File:	Ex1 So	Date:	ution factors.x	Rev. 10/18 - (No Inter
	IOR BE		anple of open a		11.1101	14		Direct Identition	Oncott Toro
		ution Per Lane	(Table 4.6.2.2	2.3b-1):					
		ne Loaded							
		Lever Rule	(Table 3.6	.1.1.2)					
		mg = 0.	625 * 1.0 =	0.625	TxDOT us	es a mu	Itiple pre	sence facto	r of 1,0 for one
		Modify t	or Skew:		lane loade				
			skew corre	ection =	1,000				
			mg = 0.62	5 * 1.000 =	0.625				
		Use Lever Ru	ile, as per AA	SHTO LRF	D Table 4.6.2	2.2.3b-1	1		
		gV _{ext1} =	0.625						
	Two or	More Lanes L	oaded						
		Lever Rule	(Table 3.6	.1.1.2)					
		mg = M	ax(0.625 * 1.0	0, 0.625 * 0.	85, 0.625 * 0	.65) =	0.625		
		Modify I	or Skew:						
			skew corre	ection =	1,000				
			mg = 0.62	5 * 1.000 =	0.625				
		Equation							
		$d_e = dis$	t. b/w CL web	to curb					
			- Rail Width						
			3ft - 1ft =	2.0	tt				
		e = 0.6	$+\left(\frac{d_e}{10}\right)$						
		e = 0.6	+ (2.0/10) =	0.800					
		$g = e^*g^*$	V _{int24} Eq						
			0 * 0.814 =	0.651					
		Skew C	orrection is in	cluded in g	V(interior).				
		Range of App	olicability (RO	A) Checks	Interior	ROA is	implicitly	applied to t	he exterior beam.
		Check I	nterior Beam	ROA:	OK				
		Check of	d _e : -1.0' ≤ 2.0	0' ≤ 5.5'	OK				
		Check f	N _b : 6 ≠ 3		OK				
		Use Equation	from Table 4	4.6.2.2.3b-1	because all	criteria i	s OK.		
		$gV_{ext2+} =$	0.651						
	TXDOT	Policy states g	V _{Exterior} must b	e ≥ gV _{interior}					
		gV _{interior} =	0.814						
	TXDOT	Policy states g	V _{Exterior} must b	e ≥ m·N _L ÷N	l _b				
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6	S =	0.425				
		S/2 ? Yes							
		Oft? Yes			957				
>>		Policy states th						4 310 31	
		Policy states th m·N _L ÷N _b .							
	TXDOT	Policy states th	at if OH > S/2	2 ans W ≥ 2	Oft, gV _{Exterior} i	s the M	aximum c	of: gV _{ext1} , gV	ext2+- gV _{interior}
1 1		and $m \cdot N_L \div N_b$							
1	gV _{exte}	rior = 0.814	25						

XDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/14/20		RFD Spece
RIDGE	C-S-J: Descrip:	ITBC Design Ex	ID#:	XXXX	Ck Dsn: File:	Fy1 Sn	Date:	ution factors.xl	Rev. 10/18 - Sheet:	5 of 8
XTER	RIOR BE		ample 11 Opair 2		Ti iio,	JEAT OF	ane_distrib	DUDIT_IGUIDIO.SI	Sileot.	5 01 0
			ne (Table 4.6.2.	2 2d-1):						
iomen		ne Loaded	io (radio 4.o.e.	17.						
	.= ::	Lever Rule								
			625 * 1.0 =	0.625	TxDOT us	es a mu	Itiple pres	sence factor	of 1,0 for or	ne.
		Modify 1	for Skew:		lane loade				50,114,150,25	
			skew correc	tion =	1,000					
			mg = 0.625	* 1.000 =	0.625					
		Use Lever Ri	ule as per AAS	HTO LRFD	Table 4.6.2	2.2d-1.				
		gM _{ext1} =	0.625							
	Two or	More Lanes L	oaded							
	C. C. C.	Lever Rule	(Table 3.6.1	.1.2)						
		mg = M	ax(0.625 * 1.0,		35, 0.625 * 0	.65) =	0.625			
		760.70	for Skew:							
			skew correct	tion =	1,000					
			mg = 0.625	* 1.000 =	0.625					
		Equation								
		e = 0.7	$7 + \left(\frac{d_v}{9.1}\right)$							
			7 + (2.0/9.1) =		0.990					
		g = e*gl	M _{int2+Eq}							
			9 * 0.649 =	0.643						
		Skew C	orrection includ	ded in gM(i	nterior).					
		Range of Ap	olicability (ROA) Checks	Interior	ROA is	implicitly	applied to the	ne exterior b	eam.
		Check I	nterior Beam F	OA:	OK					
		Check	d_e : $-1.0' \le 2.0'$	≤ 5.5'	OK					
		Check I	N _b : 6 ≠ 3		OK					
		Use Equation	from Table 4.	6.2.2.2d-1	because all	criteria is	s OK			
		$gM_{ext2+} =$	0.643							
	TXDOT	Policy states g	M _{Exterior} must be	e ≥ gM _{interior}						
		gM _{interior} =	0.649							
	TXDOT	Policy states g	M _{Exterior} must be	$\geq m \cdot N_L + N$	Ь					
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6	=	0.425					
	Is OH ≤	S/2 ? Yes								
		20ft ? Yes								
>>			nat if OH ≤ S/2,							
	TxDOT	Policy states the m·N _L ÷N _b .	nat if OH > S/2	and W < 20	Oft, gM _{Exterior}	is the M	aximum (of: gM _{ext1} , gN	A _{interior} , and	
	TXDOT		nat if OH > S/2	ans W ≥ 20	oft, gM _{Extension}	is the M	aximum o	of: gM _{ext1} , gN	Aest2+1 gMmten	incr:
		and m·N _L ÷N _E			- miles					
	gM _{exte}									
	- SAN		_							



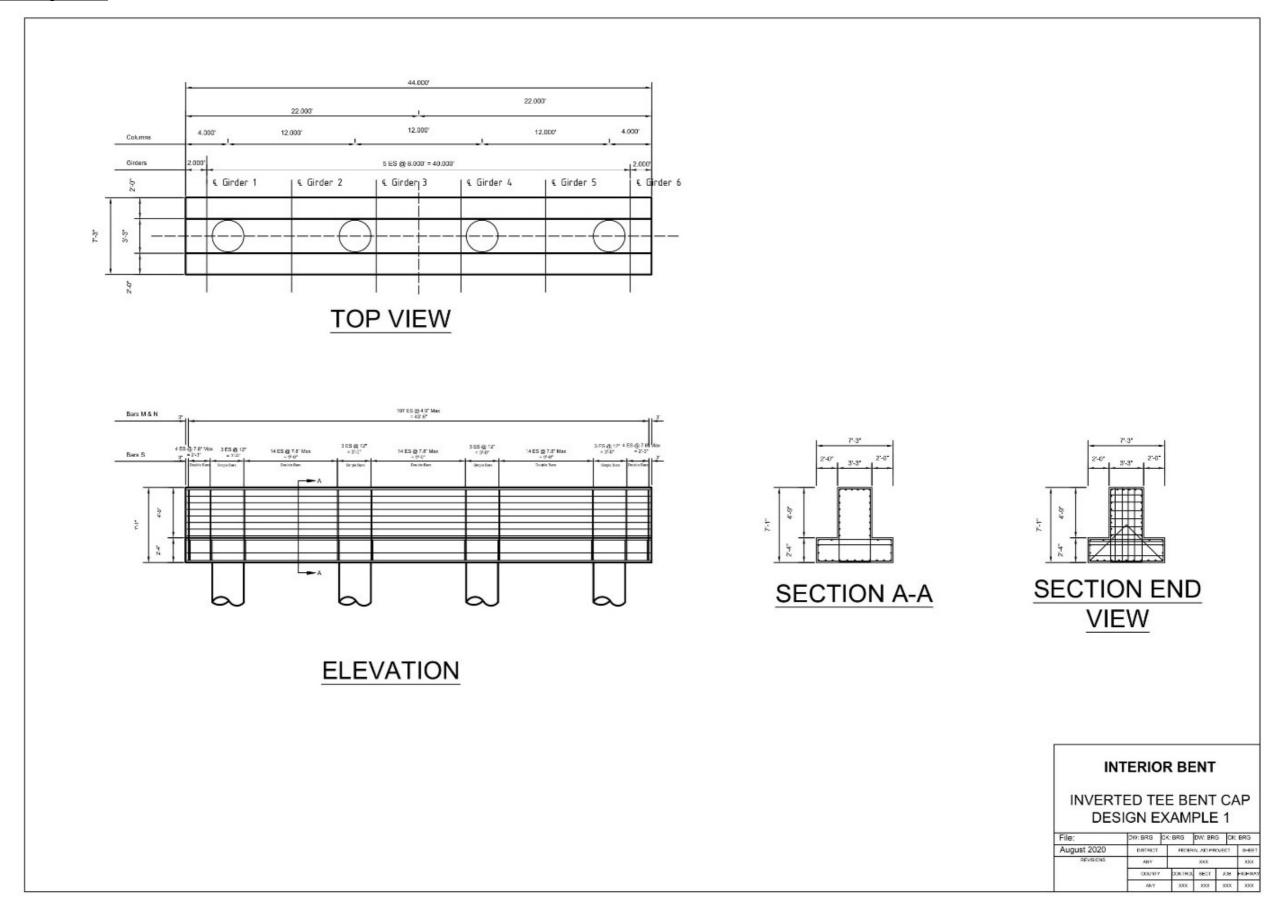
XDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/14/20		LRFD Spe
RIDGE		ITBC Design Exa	ID#:	XXXX	Ck Dsn: File:	Eut Con	Date:	ion factors.xl	Rev. 10/18 - Sheet:	7 of 8
VISION	Descrip.	I TOC Design Exa	imple 1, opan 2		Trite.	LAT OPA	ILE_UISUIDUI	UII_IGUIUIS:AI	Sileet	7 01 0
	RULE		= 8.0 ft							
	OR (con't)									
For 18 s	s S < 22: ne Lane =	$\frac{16}{32}\bigg(1+\frac{S-6}{S}\bigg)$					r a	0.625		
Tw	o Lanes =	$\frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$)			0,5	0.750		
		5-7 6	$+\frac{S-4}{S}+\frac{S-10}{S}$				-	-0.125		
		$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S-12}{S}$	$\left(\frac{-18}{S} + \frac{S-16}{S}\right)$		1	0.625		
For 22 s	≤ S ≤ 24; ne Lane =	$\frac{16}{32}\bigg(1+\frac{S-6}{S}\bigg)$					119	0.625		
Tw	o Lanes =	$\frac{16}{32}\bigg(1+\frac{S-6}{S}$	$+\frac{S-4}{S}+\frac{S-10}{S}$)				0.750		
Three	e Lanes =	$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S}{S}$	- 18 S		110	-0.125		
Fou	r Lanes =	$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S-12}{S}$	$\frac{-18}{S} + \frac{S - 16}{S}$	$+\frac{S-22}{S}$		-1.500		
		16k	4 ft - 2 ft	S ft	4 11				S=	8.0 f
		он — х-	_s	assumed	t hinge		,	Rail Widti X = S+OH-	OH = 1 = RW =	3.0 f 1.0 f 8.0 f
For X <	6: ne Lane =	$\frac{16}{32} \left(\frac{X}{S} \right)$					1.4	0.500		
For 6 ≤ Or	X < 12; ne Lane =	$\frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					0.625		
For 12 s	X < 18; ne Lane =	$\frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					0.625		
O		25/2 13								

RIDGE C-S-J:	XXX-XX-XXXX	Highway: ID #:	XXXX	Design: Ck Dsn:	BRG	Date:	8/14/20	Rev. 10/18 - (No Interin
IVISION Descrip:	ITBC Design Exa			File:	Ex1 Spar		ution factors.	
LEVER RULE								
EXTERIOR (con	rt) S RW	= 8.0 ft = 1.0 ft		OH = H-RW-2ft =				
For 18 ≤ X < 24: One Lane	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					= 0.625	
Two Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X - 12}{S} + \frac{X}{S}$	$\left(\frac{18}{S}\right)$				= -0.250	
For 24 ≤ X < 30: One Lane	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					= 0.625	
Two Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X - 12}{S} + \frac{X}{S}$	$\left(\frac{-18}{S}\right)$				= -0.250	
Three Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X - 12}{S} + \frac{A}{S}$	$\frac{x-18}{S} + \frac{x-24}{S}$	1)			= -1.250	
For 30 ≤ X < 36: One Lane	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					= 0.625	
Two Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X - 12}{S} + \frac{X}{S}$	$\left(\frac{-18}{S}\right)$				= -0.250	
Three Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{s} + \frac{x-12}{s} + \frac{x}{s}$	$\frac{C-18}{S} + \frac{X-24}{S}$	$+\frac{X-30}{S}$			= -2.625	
For 36 ≤ X < 42: One Lane	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					= 0.625	
Two Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X - 12}{S} + \frac{X}{S}$	$\left(\frac{S-18}{S}\right)$				= -0.250	
Three Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X-12}{S} + \frac{X}{S}$	$\frac{C-18}{S} + \frac{X-24}{S}$	$+\frac{X-30}{S}$			= -2.625	
Four Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - S}{S} \right)$	$\frac{6}{S} + \frac{X - 12}{S} + \frac{X}{S}$	$\frac{C-18}{S} + \frac{X-24}{S}$	$+\frac{X-30}{S}+$	$\left(\frac{X-36}{S}\right)$		= -4.375	
For 42 ≤ X ≤ 48: One Lane	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					= 0.625	
Two Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X - 12}{S} + \frac{X}{S}$	$\left(\frac{c-18}{S}\right)$				= -0.250	
	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$						= -2.625	
Four Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X-12}{S} + \frac{\lambda}{S}$	$\frac{x-18}{s} + \frac{x-24}{s}$	$\frac{4}{S} + \frac{X-30}{S} + \frac{1}{S}$	$\frac{X-36}{S}+\frac{2}{S}$	$\left(\frac{s-42}{s}\right)$	= -6.500	
INTERIOR				EXTER				25.5
One Lane Loade		= 0.625			ne Loade		1.3	0.625
Two Lanes Load		= 0.875		Two La	nes Load	ed	(=	0.625
Three Lanes Loa	aded	= 0.875		Three L	anes Loa	ded	100	0.625
Four Lanes Load	ded	= 0.875		Four La	nes Load	led		0.625

4.2.15.5 Concrete Section Shear Capacity Spreadsheet

=	Highway:	ANY							
Texas	C-S-J:	XXXXXX			Design:	BRG C	k Dsn:	BRG	
Department of Transportation	Bridge	Division	Re	ev: 09/26/08		τ	Date:	Aug-20	
CONCRETE SECTION SHEA	AR CAPA	ACITY BY A	ASHTO L	RFD BRID	GE DESIG	N SPECIFIC	ATIONS, FO	URTH EDIT	ON, 200
Resistance Factors:			Units:	US					
λ _V =	0.9								
b _M =	0.9								
þ _N =	0.75								
Concrete:			Mild Steel:			Prestressed	Steel:		
fc =	5	ksi	fy =	60	ksi	fpu =	270 k	si	
Ec =	4070	ksi	Es =	29000	ksi	Ep =	28500 k	si	
					SECTIONS				
	Units	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	42
Shear force, Vu	kip	234.3	239.1	128.6	447.4	230.2	246.5	133.3	419.
Axial force, Nu (+ if tensile)	kip	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	39.0
Shear depth, dv	in	80.79	80.79	80.79	80.79	80.79	80.79	80.79	80.7
Mild steel reinf. area, As	in^2	9.36	9.36	9.36	9.36	9.36	9.36	9.36	9.3
Conc area on tension side, Ac	in^2	1657.5	1657.5	1657.5	1657.5	1657.5	1657.5	1657.5	1657.
Area of stirrups, Av Stirrup spacing, s	in^2	1.76	1.76	1.76	1.76	1.76	1.76	1.76	1.7
and the second s	in	7,8	7.8	7.8	7.8	7.8	7.8	7.8	7,
Prestressed steel area, Aps Prestress shear, Vp	in^2	0	0	0	0	0	0	0	
Average prestress, fps	kip ksi	0	0	0	0	0	0	0	
Torsional moment, Tu	kip-ft	660	330	330	660	660	330	330	66
Shear flow area, Ao	in^2	2971.6	2971.6	2971.6	2971.6	2971.6	2971.6	2971.6	2971.
Area of one leg of stirrup, At	in^2	0.44	0.44	0,44	0.44	0.44	0.44	0.44	0.4
Perimeter of stirrup, Ph	in	324	324	324	324	324	324	324	32
Calculated Values									
Vc	kip	529.9	527.6	594.4	496.5	532.1	525.4	590.0	496.
Vs	kip	1517.9	1567.9	1865.6	1363.9	1526.6	1555.7	1842.3	1363.
φVn	kip	1843	1886	2214	1674	1853	1873	2189	167
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-0
9	deg	33.74	33.90	29.60	36.40	33.60	34.10	29.90	36.4
β Reg'd Shear reinf. Av/S	1-100-	2.380	2.370	2.670	2.230	2,390	2.360	2.650	2.23
Reg'd Torsion reinf. At/S	in^2/in in^2/in	0,000 0,016	0.000	0.000	0.000	0.000	0.000	0,000	0.00
Maximum stirrup spacing, Smax	in 2/in	24.0	24.0	24.0	24.0	24.0	24.0	24.0	24.
Conclusion		21.0	2,119	2.1.0	2110	2 110	41.9	27.0	E TV
7707-277-00	inforcing	ОК	OK	OK	OK	OK	OK	ОК	OK
Shear Reinforcing Longitudinal Reinforcing		ОК	OK	ОК	ОК	ОК	OK	OK	OK

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

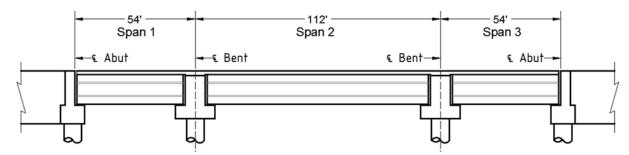


Figure 4.28 Spans of the Bridge with 30 Degree Skewed ITBC

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

 $f_c' = 5 \text{ ksi}$

 $w_c = 150 \text{ pcf (for weight)}$

 $w_c = 145 \text{ pcf (for Modulus of Elasticity calculation)}$

"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

 $f_y = 60 \text{ ksi}$

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

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 = 0.372 klf		Weight of Rail
$w_c = 0.150 kcf$		Unit Weight of Concrete for Loads
$w_{Olay} = 0.140 \text{kcf}$		Unit Weigh of Overlay
Bents		
$f_c = 5$ ksi		Concrete Strength
$w_{cE} = 0.145 kcf$		Unit Weight of Concrete for E_c
$E_{c} = 33000 \cdot w_{cE}^{1.5} \cdot $	$\overline{f_c}$ $E_c = 4074 \text{ ksi}$	Modulus of Elasticity of Concrete (AASHTO LRFD Eq. C5.4.2.4-2)
$f_y = 60$ ksi		Yield Strength of Reinforcement
$E_s = 29000ksi$		Modulus of Elasticity of Steel
$D_{column} = 36in$		Diameter of Columns

Other Variables

IM = 33%

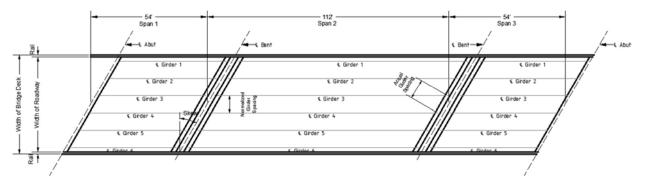


Figure 4.29 Top View of the 30 Degrees Skewed ITBC with Spans and Girders

4.3.2 Determine Cap Dimensions

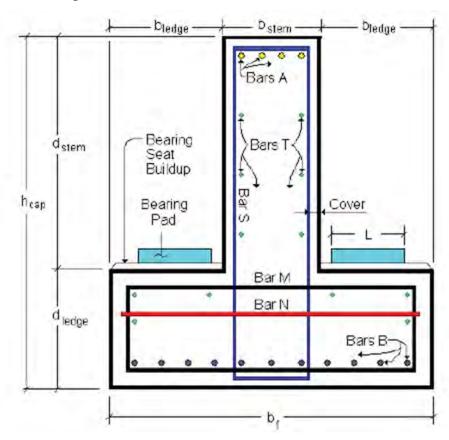


Figure 4.30 Section View of 30 Degrees Skewed ITBC

4.3.2.1 Stem Width

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

 $b_{stem} = 39 in$

4.3.2.2 Stem Height

Distance from Top of Slab to Top of Ledge:

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)

Haunch2 is the larger of the two

 $D_{Slab_to_Ledge} = SlabThk + Haunch2 + GirderD + BrgPad + BrgSeat$

 $D_{Slab_to_Ledge} = 70.00 \text{ in}$

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.

$$d_{stem} = D_{Slab_to_Ledge} - SlabThk - StemHaunch - 0.5in \\$$

 $d_{stem} = 57.75 in$

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

4.3.2.3 <u>Ledge Width</u>

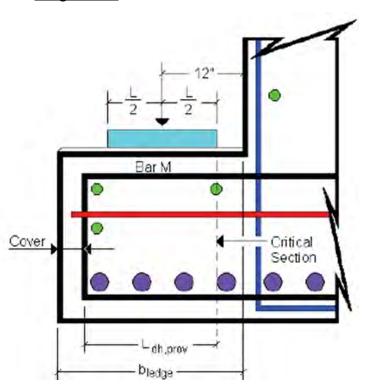


Figure 4.31 Ledge Section of 30 Degrees ITBC

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

$$L_{dh} = \frac{_{38.0 \cdot d_{bar_M}}}{_{60}} \cdot \left(\frac{f_y}{\sqrt{f_c}}\right) \qquad \qquad L_{dh} = 12.75 \text{ in}$$

Modification Factors for L_{dh}:

The stem must accommodate $\frac{1}{2}$ " of bituminous fiber.

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

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

" $L_{dh,prov}$ " 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.

(AASHTO LRFD Eq. 5.10.8.2.4a-2)

(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"?

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

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

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, $\lambda = 1.0$

"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.

(AASHTO LRFD 5.4.2.8)

(AASHTO LRFD 5.10.8.2.4a)

The Required Development Length:

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

Therefore,

$$L_{dh_req} = 12.75 \text{ in}$$

$$b_{ledge_min} = L_{dh_req} + cover + 12in - \frac{L}{2} \qquad \quad b_{ledge_min} = 23.25 \ in$$

Use:

$$b_{ledge} = 24 in$$

Width of Bottom Flange:

$$b_f = 2 \cdot b_{ledge} + b_{stem}$$

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

 $b_f = 87 in$

4.3.2.4 Ledge Depth

Use a Ledge Depth of 28".

$$d_{ledge} = 28 in$$

Total Depth of Cap:

$$h_{cap} = d_{stem} + d_{ledge}$$

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.

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

4.3.2.5 Summary of Cross Sectional Dimensions

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

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

$$b_{ledge} = 24 in$$

$$d_{ledge} = 28 in$$

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

4.3.2.6 Length of Cap

First define Girder Spacing and End Distance:

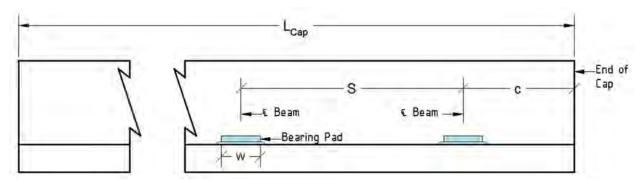


Figure 4.32 Elevation View of 30 Degrees Skewed ITBC

$$S=8 \text{ ft} \\ c=2 \text{ ft} \\ \text{"c" is the distance from the Center} \\ \text{Line of the Exterior Girder to the} \\ \text{Edge of the Cap measured along} \\ \text{the Cap.} \\ L_{Cap}=S\cdot (\text{GdrNo1}-1)+2c \\ L_{Cap}=44 \text{ ft} \\ \text{Length of Cap} \\ \text{Length of C$$

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 in W = 21 in Width of Bearing Pad

4.3.3 Cross Sectional Properties of Cap

$$\begin{split} A_g &= d_{ledge} \cdot b_f + d_{stem} \cdot b_{stem} & A_g = 4659 in^2 \\ ybar &= \frac{d_{ledge} \cdot b_f \cdot \left(\frac{1}{2} d_{ledge}\right) + d_{stem} \cdot b_{stem} \cdot \left(d_{ledge} + \frac{1}{2} d_{stem}\right)}{A_g} & ybar = 34.3 \text{ in } \frac{Distance from bottom of the cap to the center of gravity of the cap}{the center of gravity of the cap} \\ I_g &= \frac{b_f \cdot d_{ledge}^3}{12} + b_f \cdot d_{ledge} \cdot \left(ybar - \frac{1}{2} d_{ledge}\right)^2 + \frac{b_{stem} \cdot d_{stem}^3}{12} + \cdots \\ b_{stem} \cdot d_{stem} \cdot \left[ybar - \left(d_{ledge} + \frac{1}{2} d_{stem}\right)\right]^2 & I_g = 2.86 \times 10^6 \text{ in}^4 \end{split}$$

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.

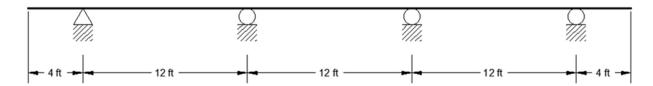


Figure 4.33 Continuous Beam Model for 30 Degrees Skewed ITBC

TxDOT does not consider frame action for typical multi-column bents (BDM-LRFD, Ch. 4, Sect. 5, Structural Analysis).

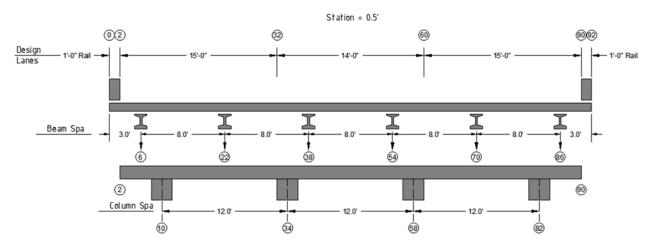


Figure 4.34 Cap 18 Model of 30 Degrees Skewed ITBC

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.

$$station = 0.5 ft$$

Station increment for CAP 18

Recall:

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

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

4.3.4.1.1 *Dead Load*

SPAN 1

$$Rail1 = \frac{2 \cdot RailWt \cdot \frac{Span1}{2}}{min(GdrNo1,6)}$$

$$Slab1 = w_c \cdot GdrSpa1 \cdot SlabThk \cdot \frac{Span1}{2} \cdot 1.10$$

$$Girder1 = GdrWt1 \cdot \frac{Span1}{2}$$

$$DLRxn1 = (Rail1 + Slab1 + Girder1)$$

Overlay1 = $w_{Olay} \cdot GdrSpa1 \cdot OverlayThk \cdot \frac{Span1}{2}$

Values used in the following equations can be found on "4.3.1 Design Parameters"

$$Rail1 = 3.44 \frac{kip}{girder}$$

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

$$Slab1 = 23.76 \frac{kip}{girder}$$

Slab1 = $23.76 \frac{\text{kip}}{\text{girder}}$ Increase slab DL by 10% to account for haunch and thickened slab ends.

$$Girder1 = 22.98 \frac{kip}{girder}$$

$$DLRxn1 = 50.17 \frac{kip}{girder}$$

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

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

Rail2 = $7.13 \frac{\text{kip}}{\text{girder}}$

 $Slab2 = 49.28 \frac{kip}{girder}$

 $Girder2 = 47.66 \frac{kip}{girder}$

Design for future overlay.

SPAN 2

$$Rail2 = \frac{2 \cdot RailWt \cdot \frac{Span2}{2}}{min(GdrNo2,6)}$$

$$Slab2 = w_c \cdot GdrSpa2 \cdot SlabThk \cdot \frac{Span2}{2} \cdot 1.10$$

$$Girder2 = GdrWt1 \cdot \frac{Span2}{2}$$

$$DLRxn2 = (Rail2 + Slab2 + Girder2)$$

$$Overlay2 = w_{Olay} \cdot GdrSpa2 \cdot OverlayThk \cdot \frac{Span2}{2}$$

CAP

$$Cap = w_c \cdot A_g = 4.853 \frac{kip}{ft} \cdot \frac{0.5ft}{station}$$

Overlay2 =
$$10.45 \frac{\text{kip}}{\text{girder}}$$

 $DLRxn2 = 104.07 \frac{kip}{girder}$

$$Cap = 2.427 \frac{kip}{station}$$

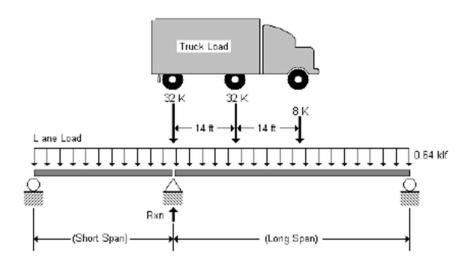


Figure 4.35 Live Load Model of 30 Degrees Skewed ITBC

LongSpan = max(Span1, Span2)

LongSpan = 112 ft

ShortSpan = min(Span1, Span2)

ShortSpan = 54 ft

IM = 0.33

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

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

$$Truck = 32kip + 32kip \cdot \left(\frac{LongSpan - 14ft}{LongSpan}\right) + 8kip \cdot \left(\frac{LongSpan - 28ft}{LongSpan}\right)$$

Truck =
$$66.00 \frac{\text{kip}}{\text{lane}}$$

LLRxn = Lane + Truck
$$\cdot$$
 (1 + IM)
LLRxn = 140.90 $\frac{\text{kip}}{\text{lane}}$

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 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)

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

$$P = 21.28 \text{ kip}$$

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

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

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

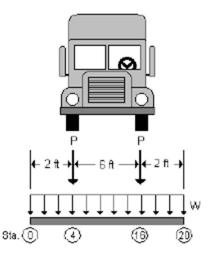


Figure 4.36 Live Load Model of 30 Degrees Skewed ITBC for CAP18

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

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
	I DED 2 4 1

<u>Limit States</u> (AASHTO LRFD 3.4.1)

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

Service I

Live Load and Dynamic Load Allowance LL+IM = 1.00

Dead Load and Wearing Surface DC & DW = 1.00

The cap design need only

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

Factors".

consider Strength I, Service I, and Service I with DL (TxSP).

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.

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: $M_{posDL} = 294.2 \; kip \cdot ft \qquad \qquad M_{negDL} = -443.9 \; kip \cdot ft$

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

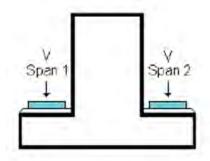


Figure 4.37 Girder Reactions on the Ledge of 30 Degrees Skewed ITBC

Dead Load

DLSpan1 = Rail1 + Slab1 + Girder1 DLSpan1 =
$$50.17 \frac{\text{kip}}{\text{girder}}$$

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

DLSpan2 = Rail2 + Slab2 + Girder2 DLSpan2 =
$$104.07 \frac{\text{kip}}{\text{girder}}$$

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.

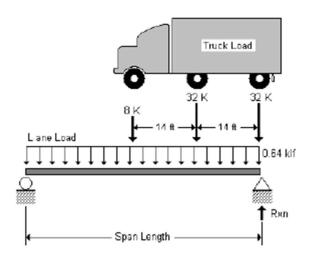


Figure 4.38 Live Load Model of 30 Degrees Skewed ITBC for Girder Reactions on Ledge

$$\begin{split} LaneSpan1 &= 0.64klf \cdot \left(\frac{Span1}{2}\right) & LaneSpan1 = 17.28 \frac{kip}{lane} \\ LaneSpan2 &= 0.64klf \cdot \left(\frac{Span2}{2}\right) & LaneSpan2 = 35.84 \frac{kip}{lane} \\ & TruckSpan1 = 32kip + 32kip \cdot \left(\frac{Span1-14ft}{Span1}\right) + 8kip \cdot \left(\frac{Span1-28ft}{Span1}\right) \\ & TruckSpan1 = 59.56 \frac{kip}{lane} \\ & TruckSpan2 = 32kip + 32kip \cdot \left(\frac{Span2-14ft}{Span2}\right) + 8kip \cdot \left(\frac{Span2-28ft}{Span2}\right) \\ & TruckSpan2 = 66.00 \frac{kip}{lane} \end{split}$$

$$IM = 0.33$$

LLRxnSpan1 = LaneSpan1 + TruckSpan1 * (1 + IM)

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

LLRxnSpan2 = LaneSpan2 + TruckSpan2 * (1 + IM)

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

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).

$$\begin{split} gV_{Span1_Int} &= 0.876 \\ gV_{Span1_Ext} &= 0.876 \\ gV_{Span2_Int} &= 0.891 \\ gV_{Span2_Ext} &= 0.891 \end{split}$$

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

LLSpan1Int =
$$gV_{Span1_Int} \cdot LLRxnSpan1$$
 LLSpan1Int = $84.53 \frac{kip}{girder}$

$$LLSpan1Ext = gV_{Span1_Ext} \cdot LLRxnSpan1 \qquad LLSpan1Ext = 84.53 \frac{kip}{girder}$$

$$LLSpan2Int = gV_{Span2_Int} \cdot LLRxnSpan2 \qquad \qquad LLSpan2Int = 110.15 \frac{kip}{girder}$$

$$LLSpan2Ext = gV_{Span2_Ext} \cdot LLRxnSpan2 \qquad LLSpan2Ext = 110.15 \frac{kip}{girder}$$

Span 1

Interior Girder

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

$$V_{s_Span1Int} = DLSpan1 + Overlay1 + LLSpan1Int$$

$$V_{s_Span1Int} = 140 \text{ kip}$$

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

$$V_{u_Span1Int} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 + 1.75 \cdot LLSpan1Int$$

$$V_{u \text{ Span1Int}} = 218 \text{ kip}$$

Exterior Girder

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

$$V_{s Span1Ext} = DLSpan1 + Overlay1 + LLSpan1Ext$$

$$V_{s Span1Ext} = 140 \text{ kip}$$

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

$$V_{u_Span1Ext} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 + 1.75 \cdot LLSpan1Ext$$

$$V_{u \text{ Span1Ext}} = 218 \text{ kip}$$

Span 2

Interior Girder

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

$$V_{s Span2Int} = DLSpan2 + Overlay2 + LLSpan2Int$$

$$V_{s_Span2Int} = 225 \text{ kip}$$

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

$$V_{u \text{ Span2Int}} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot LLSpan2Int$$

$$V_{u \text{ Snan2Int}} = 339 \text{ kip}$$

Exterior Girder

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

$$V_{s \ Span2Ext} = DLSpan2 + Overlay2 + LLSpan2Ext$$

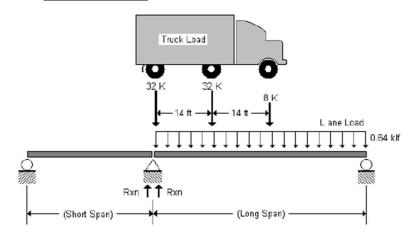
$$V_{s \; Span2Ext} = 225 \; kip$$

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

$$V_{u \; Span2Ext} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot LLSpan2Ext$$

$$V_{u,Snan2Ext} = 339 \, kip$$

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

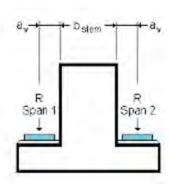


Figure 4.40 Loads on the Ledge of 30 Degrees Skewed ITBC for Torsion

$$a_v = 12 in$$

" 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.

$$b_{stem} = 39 in$$

$$LeverArm = a_v + \frac{1}{2}b_{stem}$$

LeverArm = 31.5 in

Interior Girders

Girder Reactions

$$\begin{split} R_{u_Span1} &= 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 \\ R_{u_Span1} &= 70 \text{ kip} \\ \\ R_{u_Span2} &= 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2_Int} \\ & \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)] \\ \\ R_{u_Span2} &= 339 \text{ kip} \end{split}$$

Torsional Load

$$T_{u_Int} = \left| R_{u_Span1} - R_{u_Span2} \right| \cdot LeverArm$$

$$T_{u_Int} = 706 \; kip \cdot ft$$

Exterior Girders

Girder Reactions

$$\begin{split} R_{u_Span1} &= 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 \\ R_{u_Span1} &= 70 \text{ kip} \\ R_{u_Span2} &= 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2_Ext} \\ & \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)] \\ R_{u_Span2} &= 339 \text{ kip} \end{split}$$

Torsional Load

$$T_{u_Ext} = \left| R_{u_Span1} - R_{u_Span2} \right| \cdot LeverArm$$

$$T_{u_Ext} = 706 \; kip \cdot ft$$

Torsion on Cap

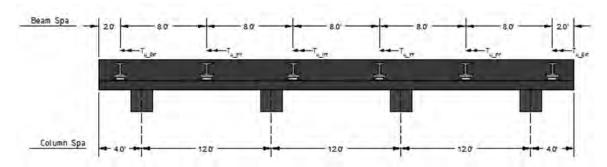


Figure 4.41 Elevation View of 30 Degrees Skewed ITBC with Torsion Loads

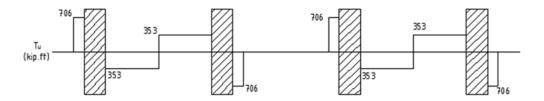


Figure 4.42 Torsion Diagram of 30 Degrees Skewed ITBC

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

$$T_u = 706 \: kip \cdot ft$$

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

$$V_{u \text{ Int}} = \max(V_{u \text{ Span1Int}}, V_{u \text{ Span2Int}})$$
 $V_{u \text{ Int}} = 338.53 \text{ kip}$

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)

$$T_u = 706 \: kip \cdot ft$$

 $V_u = 452.1 \mathrm{\,kip}$

$$M_u = 394.2 \text{ kip} \cdot \text{ft}$$

Located two stations away from centerline of column.

 V_u and M_u values are from CAP18

4.3.5 Locate and Describe Reinforcing

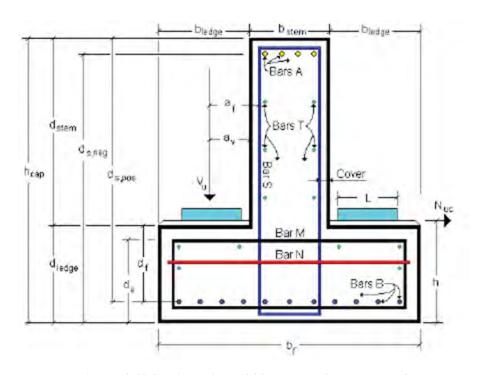


Figure 4.43 Section View of 30 Degrees Skewed ITBC

Recall:

$$b_{stem} = 39 in$$

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

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

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

$$b_f = 87 \text{ in}$$

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

cover = 2.5 in

4.3.5.1 <u>Describe Reinforcing Bars</u>

Use # 11 bars for Bar A

$$A_{bar_A} = 1.56 \text{ in}^2$$
 $d_{bar_A} = 1.410 \text{ in}$

Use # 11 bars for Bar B

$$A_{bar\ B} = 1.56\ in^2$$
 $d_{bar\ B} = 1.410\ in$

Use # 6 bars for Bar M

$$A_{bar\ M} = 0.44\ in^2$$
 $d_{bar\ M} = 0.75\ in$

Use # 6 bars for Bar N

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

Use # 6 bars for Bar S

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

Use # 6 bars for Bar T

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

4.3.5.2 Calculate Dimensions

$$d_{s_{neg}} = h_{cap} - cover - \frac{1}{2}d_{bar_{s}} - \frac{1}{2}d_{bar_{s}}$$
 $d_{s_{neg}} = 81.42 in$

In the calculation of b_{ledge} , # 6

Bar M was considered. Bar M must be # 6 or smaller to allow it

To prevent confusion, use the

same bar size for Bar N as Bar

fully develop.

M.

$$d_{s_pos} = h_{cap} - cover - \frac{1}{2} max(d_{bar_S}, d_{bar_M}) - \frac{1}{2} d_{bar_B}$$
 $d_{s_pos} = 81.42 in$

$$a_v = 12 in$$

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

$$d_e = d_{ledge} - cover$$
 $d_e = 25.50 in$

$$d_f = d_{ledge} - cover - \frac{1}{2} d_{bar_M} - \frac{1}{2} d_{bar_B} \qquad \qquad d_f = 24.42 \ in \label{eq:dedge}$$

$$h = d_{ledge} + BrgSeat$$
 $h = 29.50 in$

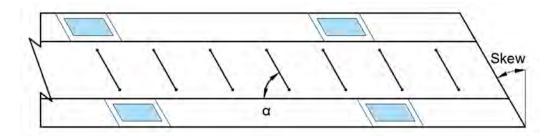


Figure 4.44 Plan View of 30 Degrees Skewed ITBC

$$\alpha = 60 \deg$$

Recall:

L = 8 in

W = 21 in

Angle of Bars S (Angle from the horizontal)

Dimension of Bearing Pad

4.3.6 Check Bearing

The load on the bearing pad propagates along a truncated pyramid whose top has the area A₁ and whose base has the area A₂. A₁ is the loaded area (the bearing pad area: L×W). A₂ is the area of the lowest rectangle contained wholly within the support (the Inverted Tee Cap). A₂ 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.

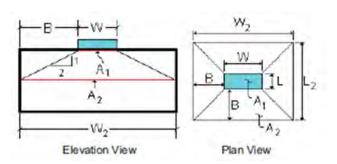


Figure 4.45. Bearing Check for 30 Degrees Skew Angle

Resistance Factor (ϕ) = 0.7

$$A_1 = L \cdot W$$

$$A_1 = 168 \text{ in}^2$$

(*AASHTO LRFD 5.5.4.2*)

Area under Bearing Pad

Interior Girders

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

B = 8 in.

$$L_2 = L + 2 \cdot B$$

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

$$L_2 = 24.00 \text{ in}$$

$$W_2 = W + 2 \cdot B$$

$$A_2 = L_2 \cdot W_2$$

$$W_2 = 37.00 \text{ in}$$

$$A_2 = 888 \text{ in}^2$$

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

$$\phi V_n = \phi \quad 0.85 \quad f_c \quad A_1 \quad m$$

$$\phi V_n = 999.6 \text{ kips}$$

AASHTO LRFD Eqs. 5.6.5-1 and 5.6.5-2.

$$V_{u \text{ Int}} = 338.53 < \phi V_{n}$$

 V_{u_int} from "4.3.4.4 Load Summary".

Exterior Girders

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

B= 8 in.

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

$$L_2 = L + 2 B$$

$$W_2 = W + 2 B$$

$$A_2 = L_2 W_2$$

$$L_2 = 24.00 \text{ in}$$

$$W_2 = 37.00 \text{ in}$$

$$A_2 = 888 \text{ in}^2$$

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

$$\phi V_n = \phi \ 0.85 \ f_c \ A_1 \ m$$

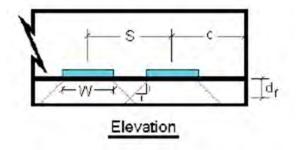
$$\phi V_n = 999.6 \text{ kips}$$

AASHTO LRFD Eqs. 5.6.5-1 and 5.6.5-2:

$$V_{u \text{ ext}} = 338.53 \text{ kips} < \Phi V_{n}$$

 V_{u_ext} from "4.3.4.4 Load Summary".

4.3.7 Check Punching Shear



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

Figure 4.46 Punching Shear Check for 30 Degrees Skew Angle

Resistance Factor (ϕ) = 0.90

AASHTO LRFD 5.5.4.2.

Determine if the Shear Cones Intersect

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

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

$$d_f = 24.42 \text{ in}$$

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

TxDOT uses "df" instead of "de" for Punching Shear (BDM-LRFD, Ch. 4, Sect. 5, Design Criteria). This is because "df" 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?

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

 $d_f = 24.42 \text{ in}$

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

Interior Girders

$$V_n = 0.125 \ \ 2 \ \lambda \sqrt{f_c'} \ b_o \ d_f$$

$$V_{\rm n} = 585.91 \, {\rm kips}$$

$$b_o = W + 2L + 2d_f$$

$$b_0 = 84.84 \text{ in}$$

$$\phi V_n = 527.32 \text{ kips}$$

$$V_{u_Int} = 338.53 \text{ kips} < \phi V_n$$

$$V_{u_int}$$
 from "4.3.4.4 Load Summary"

Exterior Girders

$$V_{n} = \min[(0.125 \cdot \sqrt{f_{c}} \cdot (\frac{1}{2}W + L + d_{f} + V) + d_{f}, 0.125 \cdot \sqrt{f_{c}} \cdot (W + 2L + 2d_{f}) * d_{f})]$$

$$V_n = 545.15 \text{ kips}$$
 AASHTO LRFD 5.8.4.3.4-3 and 5.8.4.3.4-5

$$\phi V_n = 411.09 \text{ kips}$$

$$V_{u_ext} = 338.53 \text{ kips} < \phi V_n$$

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)

$$b_{s Int} = 69 in$$

$$A_{cv} = b_{s, Int} \cdot d_e \qquad \qquad A_{cv} = 1759.5 \text{ in } 2$$

Exterior Girders

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

$$=48 \text{ in}$$

$$A_{cv} = b_{s_ext} \cdot d_e \qquad \qquad A_{cv} = 1224 \text{ in 2}$$

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

$$\varphi V_n \ = \ 1267 \ kips$$

$$V_{u_Int} = 338.53 \text{ kips} < \phi V_n$$
 ShearFrictionChk="OK!" V_{u_int} from "4.3.4.4 Load

Summary".

Exterior Girders

 $\begin{array}{lll} V_n = min(0.2 \cdot f_c \cdot A_{cv}, 0.8 \cdot A_{cv}) & V_n = 979.2 \ kips & \textit{AASHTO LRFD 5.8.4.2.2-1 and} \\ = min\,(1224, 979.2) & 5.8.4.2.2-2 & \\ \Phi V_n = 881 \ kips & & \\ V_{u_ext} = 338.53 \ kips < \Phi V_n & \textbf{ShearFrictionChk="OK!"} & \textit{Vu_ext from "4.3.4.4 Load Summary"}. \end{array}$

4.3.9 Flexural Reinforcement for Negative Bending (Bars A)

$$\begin{split} M_{dl} &= \left| M_{negDL} \right| & M_{dl} = 443.9 \text{ kip} \cdot \text{ft} \\ M_{s} &= \left| M_{negServ} \right| & M_{s} = 688.2 \text{ kip} \cdot \text{ft} \\ M_{u} &= \left| M_{negUlt} \right| & M_{u} = 991.3 \text{ kip} \cdot \text{ft} \end{split}$$

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).

$I_g = 2.86 \times 10^6 \text{ in}^4$		Gross Moment of Inertia
$h_{cap} = 85 \text{ in}$		Depth of Cap
ybar = 34.3 in		Distance to the Center of Gravity of the Cap from the bottom of the Cap
$f_{\rm r} = 0.24\sqrt{f_{\rm c}}$	$f_r = 0.537 \text{ ksi}$	Modulus of Rupture (BDM- LRFD, Ch. 4, Sect. 5, Design Criteria)
$y_t = h_{cap} - ybar$		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
$M_{cr} = S \cdot f_r \cdot \frac{1ft}{12in}$	$M_{cr} = 2523.9 \mathrm{kip} \cdot \mathrm{ft}$	Cracking Moment (AASHTO LRFD Eq. 5.6.3.3-1)
M_f = minimum of: $1.2M_{cr} = 3028.7 \text{ kip} \cdot \text{ft}$ $1.33M_u = 1318.4 \text{ kip} \cdot \text{ft}$		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 $M_{f}=1318.4\; kip \cdot ft$

4.3.9.2 Moment Capacity Design

Try,
$$7 \sim #11$$
's Top

BarANo = 7

 $d_{bar_A} = 1.410 \ in$

 $A_{bar A} = 1.56 in^2$

 $A_s = BarANo \cdot A_{bar_A}$

 $d_{stirrup} = d_{bar_S}$

 $d = d_{s_neg}$

 $b = b_f$

 $f_c = 5.0 \text{ ksi}$

 $f_v = 60 \text{ ksi}$

 $\beta_1 = 0.85 - 0.05(f_c - 4ksi)$

Bounded by: $0.65 \le \beta_1 \le 0.85$

 $c = \frac{A_s f_y}{0.85 c \beta_1 b}$

Number of bars in tension

Diameter of main reinforcing

bars

Area of main reinforcing bars

Area of steel in tension

Diameter of shear reinforcing

bars

Compressive Strength of Concrete

Yield Strength of Rebar

(AASHTO LRFD 5.6.2.2)

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 in

 $A_s = 10.92 \text{ in}^2$

d = 81.42 in

b = 87 in

 $\beta_1 = 0.80$

c = 2.22 in

 $d_{stirrup} = 0.75 in$

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.

 $M_n = A_s f_y \left(d - \frac{a}{2} \right) \cdot \frac{1ft}{12in}$

 $M_n = 4397 \text{ kip} \cdot \text{ft}$

 $\varepsilon_s = 0.107$

 $\varepsilon_{\rm s} = 0.003 \cdot \frac{\rm d-c}{\rm c}$

 $\varepsilon_{\rm s} > 0.005$

Depth of Equivalent Stress Block (AASHTO LRFD 5.6.2.2)

Nominal Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.2-1)

Strain in Reinforcing at Ultimate

FlexureBehavior = "Tension Controlled"

 $\Phi_{\rm M} = 0.90$

 $M_r = \Phi_M M_n$

 $M_r = 3957.3 \text{ kip} \cdot \text{ft}$

 $M_f = 1318.4 \text{ kip} \cdot \text{ft} < M_r$

MinReinfChk = "OK!"

(AASHTO LRFD 5.6.2.1)

(AASHTO LRFD 5.5.4.2)

Factored Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.1-1)

 $M_u = 991.3 \text{ kip} \cdot \text{ft} < M_r$ UltimateMom = "OK!"

4.3.9.3 Check Serviceability

To find s_{max}:

Modular Ratio:

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

Tension Reinforcement Ratio:

$$\rho = \frac{A_s}{b \cdot d} \qquad \qquad \rho = 0.0015$$

$$k = \sqrt{(2\rho n) + (\rho n)^2} - (\rho n)$$
 $k = 0.136$

$$d \cdot k = 11.07 \ \text{in} \ < \ d_{ledge} = 28 \ \text{in}$$

Therefore, the compression force acts over a rectangular

$$f_{ss} = \frac{M_s}{A_{c} \cdot i \cdot d} \cdot \frac{12in}{1ft} \qquad \qquad f_{ss} = 9.73 \text{ ksi}$$

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

$$f_{ss} < f_a$$
 ServiceStress $d_c = cover + \frac{1}{2}d_{stirrup} + \frac{1}{2}d_{bar_A}$ $d_c = 3.58 in$

Exposure Condition Factor:

$$y_e = 1.00$$

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

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

$$s_{Actual} = \frac{b_{stem} - 2d_c}{Bar A No - 1}$$
 $s_{Actual} = 5.31 \text{ in}$

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

4.3.9.4 Check Dead Load

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

$$M_a = A_s \cdot d \cdot j \cdot f_{dl} \cdot \frac{1 ft}{12 in}$$
 $M_a = 1556.7 \text{ kip} \cdot \text{ft}$

$$M_{dl} = 443.9 \text{ kip} \cdot \text{ft} < M_a$$
 DeadLoadMom = "OK!"

For service loads, the stress on the cross-section is located as shown in Figure 4.47.

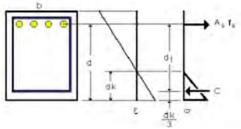


Figure 4.47 Stresses on the Cross Section for Service Loads of 30 Degrees Skewed **ITRC**

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

Service Load Bending Stress in outer layer of the reinforcing.

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

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)

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

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

Allowable Dead Load Moment

ServiceStress = "OK!"

4.3.10 Flexural Reinforcement for Positive Bending (Bars B)

$$\begin{split} M_{dl} &= M_{posDL} & M_{dl} &= 294.4 \text{ kip} \cdot \text{ft} \\ M_{s} &= M_{posServ} & M_{s} &= 574.3 \text{ kip} \cdot \text{ft} \\ M_{u} &= M_{posUlt} & M_{u} &= 863.4 \text{ kip} \cdot \text{ft} \end{split}$$

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).

$I_g = 2.86 \times 10^6 \text{ in}^4$		Gross Moment of Inertia
$y_t = ybar$	$y_t = 34.3 \text{ in}$	Distance to the Center of Gravity of the Cap from the top of the Cap
$f_{\rm r}=0.24\sqrt{f_{\rm c}}$	$f_r = 0.537 \text{ ksi}$	Modulus of Rupture (BDM- LRFD, Ch. 4, Sect. 5, Design
$S = \frac{I_g}{y_t}$	$S = 8.34 \times 10^4 \text{ in}^3$	Criteria) Section Modulus for the extreme tension fiber
$M_{cr} = S \cdot f_r \cdot \frac{1ft}{12in}$	$M_{cr} = 3732.2 \text{ kip} \cdot \text{ft}$	Cracking Moment (AASHTO LRFD Eq. 5.6.3.3-1)
$M_f = minimum of:$		Design the lesser of $1.2M_{cr}$ or
$1.2M_{\rm cr} = 4478.6 \rm kip \cdot ft$		$1.33M_u$ when determining
$1.33M_{u} = 1148.3 \text{ kip} \cdot \text{ft}$		mininum area of steel required.

Thus, $M_{\rm r}$ must be greater than $M_{\rm f}=1148.3~{\rm kip}\cdot{\rm ft}$

4.3.10.2 Moment Capacity Design

Try,
$$11 \sim #11$$
's Bottom

BarBNo = 11

 $d_{bar B} = 1.41 in$

 $A_{bar B} = 1.56 in^2$

 $A_s = BarBNo \cdot A_{bar_B}$

 $d = d_{s_pos}$

 $b = b_{stem}$

 $f_c = 5.0 \text{ ksi}$

 $f_y = 60 \text{ ksi}$

 $\beta_1 = 0.85 - 0.05(f_c - 4ksi)$

Bounded by: $0.65 \le \beta_1 \le 0.85$

 $c = \frac{A_s f_y}{0.85 f_c \beta_1 b}$

Number of bars in tension

Diameter of main reinforcing

bars

Area of main reinforcing bars

Area of steel in tension

Compressive Strength of Concrete

Yield Strength of Rebar

(AASHTO LRFD 5.6.2.2)

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

 $A_s = 17.16 \text{ in}^2$

d = 81.42 in

b = 39 in

 $\beta_1 = 0.80$

c = 7.76 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.

 $M_n = A_s f_y \left(d - \frac{a}{2} \right) \cdot \frac{1 ft}{12 in}$

 $\varepsilon_s = 0.003 \cdot \frac{d-c}{c}$

 $\varepsilon_{\rm s} > 0.005$

Depth of Cross Section under Compression under Ultimate Load (AASHTO LRFD Eq. 5.6.3.1.2-4)

Depth of Equivalent Stress Block (AASHTO LRFD 5.6.2.2)

 $M_n = 6719.4 \text{ kip} \cdot \text{ft}$

 $\varepsilon_{\rm s} = 0.028$

Nominal Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.2-1)

Strain in Reinforcing at Ultimate

FlexureBehavior = "Tension Controlled"

 $\Phi_{\rm M} = 0.90$

 $M_r = \Phi_M \cdot M_n$

 $M_r = 6047.5 \text{ kip} \cdot \text{ft}$

(AASHTO LRFD 5.6.2.1)

(AASHTO LRFD 5.5.4.2)

Factored Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.1-1)

 $M_f = 1148.3 \text{ kip} \cdot \text{ft} < M_r$ MinReinfChk = "OK!"

 $M_u = 863.4 \text{ kip} \cdot \text{ft} < M_r$ UltimateMom = "OK!"

4.3.10.3 Check Serviceability

To find s_{max} :

Modular Ratio:

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

Tension Reinforcement Ratio:

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

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

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

Therefore, the compression force acts over a rectangular

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

$$f_{ss} = \frac{M_s}{A_s \cdot j \cdot d} \cdot \frac{12in}{1ft} \qquad \qquad f_{ss} = 5.37 \text{ ksi}$$

$$f_a = 0.6f_v$$
 $f_a = 36.00 \text{ ksi}$

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

 $d_c = cover + \frac{1}{2}d_{stirrup} + \frac{1}{2}d_{bar_B} \qquad \quad d_c = 3.58 \ in$

Exposure Condition Factor:

$$\gamma_e = 1.00$$

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

$$\beta_s = 1.06$$

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

Bars Inside Stirrup Bar S

Try: BarBInsideSNo = 5

$$s_{Actual} = \frac{b_{stem} - 2\left(cover + \frac{1}{2}d_{bar_S} + \frac{1}{2}d_{bar_B}\right)}{BarBInsideSNo - 1}$$

$$s_{actual} < s_{max}$$

For service loads, the stress on the cross-section is located as shown in Figure 4.48.

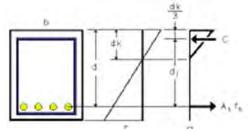


Figure 4.48 Stresses on the Cross Section for Bars B for Service Loads of 30 Degrees Skewed ITBC

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

Service Load Bending Stress in outer layer of the reinforcing.

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

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)

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

Number of Bars B that are inside Stirrup Bar S.

 $s_{Actual} = 7.96 in$

ServiceabilityCheck = "OK

Bars Outside Stirrup Bar S

BarBOutsideSNo = 11 - BarBInsideSNo

Number of Bars B that are inside Stirrup Bar S.

BarBOutsideSNo = 6

$$s_{Actual} = \frac{^{2b_{ledge}+2\left(cover+\frac{1}{2}d_{bar_S}+\frac{1}{2}d_{bar_B}-cove -\frac{1}{2}d_{bar_M}-\frac{1}{2}d_{bar_B}\right)}}{BarBOutsideSNo}$$

$$s_{actual} = 8.00 \text{ in } < s_{max}$$

ServiceabilityCheck = "OK

4.3.10.4 Check Dead Load

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

$$f_{dl} = 22 \text{ ksi}$$

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

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

$$M_{dl} = 294.4 \text{ kip} \cdot \text{ft} < M_{a}$$

$$M_a = 2354.00 \text{ kip}$$

 $M_a = 2354.00 \ kip$ Allowable Dead Load Moment

DeadLoadMom = "OK!"

Flexural Steel Summary:

Use $7 \sim #11$ 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.

$$s_{bar\ M} = 4.70 \text{ in}$$

$$s_{bar \ N} = 4.70 in$$

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

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

4.3.11.1 Determine Distribution Widths

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

<u>Distribution Width for Shear (AASHTO LRFD 5.8.4.3.2)</u>

Interior Girders

$$b_{s_Int} = min(W + 4a_v, S)$$

 $b_{s_Int} = 69.00 in$

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

$$b_{m_{\text{Ext}}} = \min(W + 5a_f, 2c, S)$$
$$b_{m_{\text{Ext}}} = 48.00 \text{ in}$$

4.3.11.2 Reinforcing Required for Shear Friction

AASHTO LRFD 5.7.4.1

$$\Phi = 0.90$$

$$u = 1.4$$

$$\mu = 1.4$$
 $c_1 = 0$ ksi $P_c = 0$ kip

Minimum Reinforcing (AASHTO LRFD Eq. 5.7.4.2-1)

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

Recall:
$$d_e = 25.50$$
 in

(AASHTO LRFD 5.5.4)

"u" 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.

$$a_{vf_min} = \frac{0.05ksi \cdot d_e}{f_y}$$

 $A_{vf_min} = \frac{0.05 \text{ ksi} \cdot A_{cv}}{f_{v}}$

$$a_{\text{vf_min}} = 0.26 \frac{\text{in}}{\text{ft}}$$

 $a_{vf_min} = 0.26 \frac{in^2}{ft}$ Minimum Reinforcing required for

Interior Girders

$$A_{cv} = d_e \cdot b_{s,Int}$$

 $a_{vf_Int} = \frac{A_{vf}}{b_{s_Int}}$

$$A_{cv} = 1759 \text{ in}^2$$

$$V_{u \text{ Int}} = 338.5 \text{ kip}$$

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

$$\Phi V_n \ge V_u$$

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

From "4.3.4.4 Load Summary".

(AASHTO LRFD Eq. 5.7.4.3-3)

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

$$A_{vf} = \frac{\frac{V_{u_Int}}{\Phi} - c_1 A_{cv}}{f_y} - P_c$$

$$A_{\rm vf} = 4.48 \, \rm in^2$$

$$8\frac{\mathrm{in}^2}{\mathrm{ft}}$$
 R

Required Reinforcing for Shear Friction

 $a_{vf_Int} = 0.78 \frac{in^2}{ft}$ Required Reinforcing for Shear Friction per foot length of cap

Exterior Girders

$$\begin{split} A_{cv} &= d_e \cdot b_{s_Ext} & A_{cv} = 1224 \text{ in}^2 \\ V_{u_Ext} &= 338.5 \text{ kip} & \textit{From "4.3.4.4 Load Summary".} \\ V_n &= c_1 A_{cv} + \mu (A_{vf} f_y + P_c) & (\textit{AASHTO LRFD Eq. 5.7.4.3-3}) \\ \Phi V_n &\geq V_u & (\textit{AASHTO LRFD Eq. 5.7.4.3-1 & AASHTO LRFD Eq. 5.7.4.3-2}) \\ \Phi \cdot \left[c_1 A_{cv} + \mu (A_{vf} f_y + P_c) \right] \geq V_u & \\ A_{vf} &= \frac{\frac{V_{u_Ext}}{\Phi} c_1 A_{cv}}{f_y} - P_c}{f_y} & A_{vf} &= 4.48 \text{ in}^2 & \textit{Required Reinforcing for Shear Friction} \\ a_{vf_Ext} &= \frac{A_{vf}}{b_{s_Ext}} & a_{vf_Ext} &= 1.12 \frac{\text{in}^2}{\text{ft}} & \textit{Required Reinforcing for Shear Friction per foot length of cap} \end{split}$$

4.3.11.3 Reinforcing Required for Flexure

Recall: $h=29.50\,\mbox{in}\quad d_e=25.50\,\mbox{in}\quad a_v=12\,\mbox{in}$

From "4.3.5.2 Calculate Dimensions"

per foot length of cap

AASHTO LRFD 5.8.4.2.1

Interior Girders

$$\begin{array}{llll} V_{u_Int} = 338.5 \text{ kip} & \textit{From "4.3.4.4 Load Summary".} \\ N_{uc_Int} = 0.2 \cdot V_{u_Int} & N_{uc_Int} = 67.7 \text{ kip} & \textit{(AASHTO LRFD 5.8.4.2.1)} \\ M_{u_Int} = V_{u_Int} \cdot a_v + N_{uc_Int} (h - d_e) & M_{u_Int} = 361.1 \text{ kip } \cdot \text{ft} & \textit{(AASHTO LRFD Eq. 5.8.4.2.1-1)} \\ \text{Use the following equations to solve for A_f:} \\ & \Phi M_n \geq M_{u_Int} & \textit{(AASHTO LRFD Eq. 1.3.2.1-1)} \\ & M_n = A_f f_y \left(d_e - \frac{a}{2} \right) & \textit{(AASHTO LRFD Eq. 5.6.3.2.2-1)} \\ & c = \frac{A_f f_y}{\alpha_1 f_c \beta_1 b_{m_Int}} & \textit{(AASHTO LRFD Eq. 5.6.3.1.2-4)} \\ & \alpha_1 = 0.85 & \textit{(AASHTO LRFD Eq. 5.6.3.1.2-4)} \\ & a_1 = 0.80 & \textit{(AASHTO LRFD 5.6.2.2)} \\ & a = c\beta_1 & \textit{(AASHTO LRFD 5.5.4.2)} \\ & Solve \text{ for A_f:} & A_f = 3.18 \text{ in}^2 & \textit{Required Reinforcing for Flexure} \\ & a_{f_Int} = \frac{A_f}{b_{m_Int}} & a_{f_Int} = 0.41 \frac{\text{in}^2}{\text{ft}} & \textit{Required Reinforcing for Flexure} \\ \end{array}$$

Exterior Girders

$$V_{u Ext} = 338.5 \text{ kip}$$

From "4.3.4.4 Load Summary".

$$N_{uc_Ext} = 0.2 \cdot V_{u_Ext}$$

$$N_{uc Ext} = 67.7 \text{ kip}$$

(AASHTO LRFD 5.8.4.2.1)

$$M_{u_Ext} = V_{u_Ext} \cdot a_v + N_{uc_Ext}(h - d_e) \quad M_{u_Ext} = 361.1 \text{ kip} \cdot \text{ft} \quad \textit{(AASHTO LRFD Eq. 5.8.4.2.1-1)}$$

Use the following equations to solve for A_f:

$$\Phi M_n \ge M_{u_Ext}$$

(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)

$$a = c\beta_1$$

$$0.75 \le \Phi = 0.65 + 0.15 \left(\frac{d_e}{c} - 1\right) \le 0.90$$

AASHTO LRFD 5.5.4.2

$$A_f = 3.21 \text{ in}^2$$

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:

$$N_{uc_Int} = 0.2 V_{u_Int}$$

$$N_{uc\ Int} = 67.7 \text{ kip}$$

$$A_n = \frac{N_{uc_Int}}{\Phi f_v}$$

$$A_n = 1.25 \text{ in}^2$$

Required Reinforcing for Axial Tension

$$a_{n_Int} = \frac{A_n}{b_{m_Int}}$$

$$a_{n_{\perp}Int} = 0.16 \frac{in^2}{ft}$$

 $a_{n_Int} = 0.16 \frac{in^2}{ft}$ 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

$$a_{n_Ext} = \frac{A_n}{b_{m Ext}}$$

$$a_{n_{\perp}Ext} = 0.31 \frac{in^2}{ft}$$

 $a_{n_Ext} = 0.31 \frac{in^2}{ft}$ Required Reinforcing for Axial Tension per foot length of cap

4.3.11.5 Minimum Reinforcing

$$a_{s_min} = 0.04 \frac{f_c}{f_y} d_e$$

$$a_{s_min} = 1.02 \frac{in^2}{ft}$$
 Minimum Required Reinforcing

4.3.11.6 Check Required Reinforcing

Actual Reinforcing:

$$a_{s} = \frac{A_{bar_M}}{s_{bar_M}} \qquad a_{s} = 1.12 \frac{in^{2}}{ft} \qquad \begin{array}{l} \textit{Primary Ledge Reinforcing} \\ \textit{Provided} \end{array}$$

$$a_{h} = \frac{A_{bar_N}}{s_{bar_N}} \qquad a_{h} = 1.12 \frac{in^{2}}{ft} \qquad \begin{array}{l} \textit{Auxiliary Ledge Reinforcing} \\ \textit{Provided} \end{array}$$

$$\underbrace{Checks:}_{S} A_{s} \geq A_{s_min} \qquad (\textit{AASHTO LRFD 5.8.4.2.1}) \qquad (\textit{AASHTO LRFD 5.8.4.2.2}) \qquad (\textit{AASHTO LRFD Eq. 5.8.4.2.2-5}) \\ A_{s} \geq \frac{2A_{vf}}{3} + A_{n} \qquad (\textit{AASHTO LRFD Eq. 5.8.4.2.2-5}) \qquad (\textit{AASHTO LRFD Eq. 5.8.4.2.2-6}) \end{array}$$

$$A_{h} \geq 0.5(A_{s} - A_{n}) \qquad (\textit{AASHTO LRFD Eq. 5.8.4.2.2-6})$$

Check Interior Girders:

Bar M:

Check if:
$$a_{s} \ge a_{s_min}$$
 (AASHTO LRFD 5.8.4.2.1)
$$a_{s} \ge a_{f_Int} + a_{n_Int}$$
 (AASHTO LRFD 5.8.4.2.2)
$$a_{s} \ge \frac{2a_{vf_Int}}{3} + a_{n_Int}$$
 (AASHTO LRFD Eq. 5.8.4.2.2-5)
$$a_{s} = 1.12 \frac{in^{2}}{ft}$$

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

$$a_{f_Int} + a_{n_Int} = 0.57 \frac{in^{2}}{ft} < a_{s}$$

$$\frac{2a_{vf_Int}}{3} + a_{n_Int} = 0.68 \frac{in^{2}}{ft} < a_{s}$$

BarMCheck = "OK!"

Bar N:

Check if:
$$a_h \ge 0.5 \cdot (a_s - a_{n_Int})$$
 (AASHTO LRFD Eq. 5.8.4.2.2-6)
$$a_s = \text{The maximum of:} \qquad \text{"a_s" in this equation is the steel} \\ a_{f_Int} + a_{n_Int} \qquad \text{required for Bar M, based on the} \\ \frac{2a_{vf_Int}}{3} + a_{n_Int} \qquad \text{AASHTO LRFD 5.8.4.2.2. This is} \\ a_s = 0.68 \frac{in^2}{ft} \qquad \text{Ah should not be less than A_{vf}} 2 \text{ nor less than A_{vf}} 3 \text{ (Furlong \& Mirza)}$$

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:
$$a_{s} \ge a_{s_min}$$
 (AASHTO LRFD 5.8.4.2.1)
$$a_{s} \ge a_{f_Ext} + a_{n_Ext}$$
 (AASHTO LRFD 5.8.4.2.2)
$$a_{s} \ge \frac{2a_{vf_Ext}}{3} + a_{n_Ext}$$
 (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:
$$a_h \geq 0.5 \cdot \left(a_s - a_{n_Ext}\right) \qquad (AASHTO\ LRFD\ Eq.\ 5.8.4.2.2-6)$$

$$a_s = \text{The maximum of:} \qquad "a_s" \text{ in this equation is the steel required}$$

$$a_{f_Ext} + a_{n_Ext} \qquad for\ Bar\ M,\ based\ on\ the\ requirements\ for\ Bar\ M\ in\ AASHTO\ LRFD\ 5.8.4.2.2.\ This is derived from\ the\ suggestion\ that\ Ah$$

$$a_s = 1.11\frac{\text{in}^2}{\text{ft}} \qquad should\ not\ be\ less\ than\ A_f/2\ nor\ less\ than\ A_v/3\ (Furlong\ \&\ Mirza\ pg.\ 73\ \&\ 74)$$

$$0.5 \cdot \left(a_s - a_{n_Ext}\right) = 0.40\frac{\text{in}^2}{\text{ft}} < a_h$$

$$\text{BarNCheck} = \text{"OK!"}$$

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.

$$s_{bar S} = 7.40 in$$

$$A_{hr} = 2stirrups \cdot A_{bar S}$$

$$A_v = 2legs \cdot A_{hr}$$

$$A_{hr}=0.88\,in^2$$

$$A_v=1.76\ in^2$$

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.

4.3.12.1 Check Minimum Transverse Reinforcement

$$b_v = b_{stem}$$

$$A_{v_min} = 0.0316\lambda \sqrt{f_c} \frac{b_v \cdot s_{bar_S}}{f_v}$$

 $\lambda = 1.0$ for normal weight concrete

$$b_v = 39 \text{ in}$$

(AASHTO LRFD Eq. 5.7.2.5-1) (AASHTO LRFD 5.4.2.8)

$$A_{v_min} = 0.34 \ in^2$$

$$A_v > A_{v \text{ min}}$$

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

$$V_{all} = minimum of:$$

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{bar_S}} \cdot (W + 3a_v) = 228 \text{ kip}$$

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)

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

$$\frac{A_{hr}\cdot\left(\frac{2}{3}f_y\right)}{s_{bar_S}}\cdot S = 457 \text{ kip}$$

 $V_{all} = 228 \text{ kip}$

 $V_{s Int} = 225 \text{ kip} < V_{all}$

(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)

ServiceCheck = "OK!"

Exterior Girders

 V_{all} = minimum of:

V_{all} for the Interior Girder

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{har} \cdot s} \cdot \left(\frac{W + 3a_v}{2} + c\right) = 228 \text{ kip}$$

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

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

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{bar,S}} \cdot \left(\frac{S}{2} + c\right) = 342 \text{ kip}$$

 $V_{all} = 228 \text{ kip}$

$$V_{s Ext} = 225 \text{ kip } < V_{all}$$

(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.)

ServiceCheck = "OK!"

(AASHTO LRFD 5.8.4.3.5)

4.3.12.3 Check Strength Limit State

$$\Phi = 0.90$$

(AASHTO LRFD Eq. 5.5.4.2)

Interior Girders:

 $V_n = minimum of:$

$$\frac{A_{hr} \cdot f_y}{s_{har S}} \cdot S = 685 \text{ kip}$$

(AASHTO LRFD Eq. 5.8.4.3.5-2)

$$(0.063\sqrt{f_c} \cdot b_f \cdot d_f) + \frac{A_{hr} \cdot f_y}{s_{bar_S}} (W + 2d_f) = 798 \text{ kip}$$
 (AASHTO LRFD Eq. 5.8.4.3.5-3)

 $V_n = 685 \text{ kip}$

 $\Phi V_n = 617 \text{ kip}$

$$V_{u,Int} = 339 \text{ kip } < \Phi V_n$$

UltimateCheck = "OK!"

Exterior Girders:

 $V_n = minimum of:$

V_n for the Interior Girder

$$\frac{A_{\text{hr}} \cdot f_y}{s_{\text{bar S}}} \cdot \left(\frac{S}{2} + c\right) = 514 \text{ kip}$$

(AASHTO LRFD Eq. 5.8.4.3.5-2)

$$\left(0.063\sqrt{f_c} \cdot b_f \cdot d_f\right) + \frac{A_{hr} \cdot f_y}{s_{bar} \ s} \left(\frac{W + 2d_f}{2} + c\right) = 720 \ kip \ \text{(AASHTO LRFD Eq. 5.8.4.3.5-3)}$$

 $V_n = 514 \text{ kip}$

 $\Phi V_n = 463 \text{ kip}$

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

$$V_{u Ext} = 339 \text{ kip } < \Phi V_{n}$$

UltimateCheck = "OK!"

4.3.12.4 Check Combined Shear and Torsion

 $d_v = 80.53 \text{ in}$

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.

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 :

$$\begin{split} &\Phi_V = 0.90 \\ &v_u = \frac{|v_u - (\Phi_V \cdot V_p)|}{\Phi_V \cdot b_V \cdot d_V} \\ &v_u = 0.16 \text{ ksi} \\ &\frac{v_u}{f_c} = 0.03 \end{split}$$

Using Table B5.2-1 with
$$\frac{v_u}{f_c}=0.03$$
 and $\epsilon_x=0.001$ $\theta=36.4$ deg and $\beta=2.23$

$$\begin{split} \epsilon_x &= \frac{\frac{|M_u|}{d_v} + 0.5 N_u + 0.5 |V_u - V_p| \cot \theta - A_{ps} f_{po}}{2 (E_s A_s + E_p A_{ps})} \\ \text{where } |M_u| &= 394.2 \text{ kip} \cdot \text{ft must be} > \big| V_u - V_p \big| d_v = 3034 \text{ kip} \cdot \text{ft} \\ \epsilon_x &= 1.20 \times 10^{-3} > 1.00 \times 10^{-3} \\ \text{use } \epsilon_x &= 1.00 \times 10^{-3}. \end{split}$$

$$V_p = 0 \text{ kip}$$

$$\begin{aligned} A_c &= b_{stem} \cdot \frac{h_{cap}}{2} & A_c &= 1657.5 \text{ in}^2 \\ s &= s_{bar_S} & s &= 7.40 \text{ in} \end{aligned}$$

(AASHTO LRFD Eq. 5.5.4.2)

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

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.

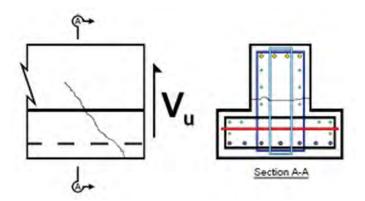
Strain halfway between the compressive and tensile resultants (AASHTO LRFD Eq. B5.2-3) If $\varepsilon_x < 0$, then use equation B5.2-5 and re-solve for ε_x .

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 " is zero as there is no prestressing.

(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

$$\begin{split} A_v &= 2 \text{legs} \cdot 2 \text{stirrups} \cdot A_{bar_S} & A_v &= 1.76 \text{ in}^2 \\ A_t &= 1 \text{leg} \cdot A_{bar_S} & A_t &= 0.44 \text{ in}^2 \\ A_{oh} &= (d_{stem}) \cdot (b_{stem} - 2 \text{cover}) + \left(d_{ledge} - 2 \text{cover}\right) \cdot (b_f - 2 \text{cover}) \\ & A_{oh} &= 3496 \text{ in}^2 \\ A_0 &= 0.85 A_{oh} & A_0 &= 2971.6 \text{in}^2 \\ p_h &= (b_{stem} - 2 \text{cover}) + 2 \left(b_{ledge}\right) + \left(b_f - 2 \text{cover}\right) + 2 \left(h_{cap} - 2 \text{cover}\right) \\ p_h &= 324 \text{ in} \end{split}$$

Equivalent Shear Force

$$V_{u_{-}Eq} = \sqrt{V_{u}^{2} + \left(\frac{0.9p_{h}T_{u}}{2A_{0}}\right)^{2}}$$
 $V_{u_{-}Eq} = 614.2 \text{ kip } (AASHTO LRFD Eq. B.5.2-1)$

Shear Steel Required

$$V_n$$
 = the lesser of:

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

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

Check maximum ΦV_n for section:

$$\Phi V_{n_{max}} = \Phi \cdot \left(0.25 \cdot f_{c} \cdot b_{v} \cdot d_{v} + V_{p}\right)$$

$$\Phi V_{n_{max}} = 3533 \text{ kip}$$

$$V_{u} = 452.1 \text{ kip} < \Phi V_{n_{max}}$$

$$MaxShearCheck = "OK!"$$

Calculate required shear steel:

$$\begin{split} &V_{u} < \Phi V_{n} & (\textit{AASHTO LRFD Eq. 1.3.2.1-1}) \\ &V_{c} = 0.0316 \cdot \beta \cdot \sqrt{f_{c}} \cdot b_{v} \cdot d_{v} & V_{c} = 495 \text{ kip} & (\textit{AASHTO LRFD Eq. 5.7.3.3-3}) \\ &V_{u} < \Phi_{V} \cdot \left(V_{c} + V_{s} + V_{p}\right) & (\textit{AASHTO LRFD Eq. 5.7.3.3-4}) \\ &V_{s} = \frac{A_{v} \cdot f_{y} \cdot d_{v} \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha}{s_{req}} & (\textit{AASHTO LRFD Eq. 5.7.3.3-4}) \\ &a_{v_req} = \frac{\frac{V_{u}}{\Phi_{V}} \cdot V_{c} - V_{p}}{f_{v} \cdot d_{v} \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha} & a_{v_req} = 0.011 \frac{\sin^{2}}{f_{t}} \end{split}$$

Torsional Steel Required

$$\begin{split} \Phi_T &= 0.9 & (\textit{AASHTO LRFD 5.5.4.2}) \\ T_u &\leq \Phi_T T_n & (\textit{AASHTO LRFD Eq. 1.3.2.1-1}) \\ T_n &= \frac{2A_0 A_t f_y \cot \theta}{s_{bar_S}} & (\textit{AASHTO LRFD Eq. 5.7.3.6.2-1}) \\ a_{t_req} &= \frac{T_u}{\Phi_T 2A_0 f_y \cot \theta} & a_{t_req} &= 0.23 \frac{in^2}{ft} \end{split}$$

Total Required Transverse Steel

$$a_{req} = a_{v_req} + 2 sides \cdot a_{t_req} \qquad a_{req} = 0.47 \frac{in^2}{ft} \qquad \begin{array}{l} \textit{designed for the side of the section} \\ \textit{where the effects of shear and torsion} \\ a_{prov} = \frac{A_v}{s_{bar_S}} \qquad a_{prov} = 2.85 \frac{in^2}{ft} \qquad \begin{array}{l} \textit{are additive. (AASHTO LRFD} \\ \textit{C5.7.3.6.1)} \\ \end{array}$$

The transverse reinforcement is

Longitudinal Reinforcement

 $A_s f_v = 655.2 \text{ kip} > 528 \text{ kip}$

$$\begin{split} A_{ps}f_{ps} + A_sf_y &\geq \frac{|\mathsf{M}_u|}{\Phi d_v} + \frac{0.5\mathsf{N}_u}{\Phi} + \cdots \\ & cot\theta \sqrt{\left(\left|\frac{\mathsf{V}_u}{\Phi} - \mathsf{V}_p\right| - 0.5\mathsf{V}_s\right)^2 + \left(\frac{0.45p_hT_u}{2A_0\Phi}\right)^2} \\ V_s &= a_{t_req} \cdot f_y \cdot d_v \cdot (cot\theta + cot\alpha) \cdot sin\alpha \\ & Bounded \ By: \ V_s < \frac{\mathsf{V}_u}{\Phi_v} \\ & V_s = 502.3 \ kip \\ & \frac{|\mathsf{M}_u|}{\Phi_f d_v} + \frac{0.5\mathsf{N}_u}{\Phi_c} + cot\theta \sqrt{\left(\left|\frac{\mathsf{V}_u}{\Phi_v} - \mathsf{V}_p\right| - 0.5\mathsf{V}_s\right)^2 + \left(\frac{0.45p_hT_u}{2A_0\Phi_T}\right)^2} = 528 \ kip \end{split}$$
 Provided Force:

LongitudinalReinfChk = "OK!"

4.3.12.5 Maximum Spacing of Transverse Reinforcement

(AASHTO LRFD 5.7.2.6)

Shear Stress

$$v_u = \frac{|v_u - \Phi_V v_p|}{\Phi_V b_v d_v}$$

$$v_u = 0.16 \text{ ksi} \qquad (AASHTO LRFD Eq. 5.7.2.8-1)$$

$$0.125 \cdot f_c = 0.625 \text{ ksi}$$

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

$$s_{\text{max}} = \min(0.8d_{\text{v}}, 24\text{in})$$

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

$$s_{\text{max}} = \min(0.4d_{\text{v}}, 12\text{in})$$

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

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

$$s_{max} = 12.00 \text{ in}$$
 Detailing)

$$s_{\text{bar S}} = 7.40 \text{ in } < s_{\text{max}}$$
 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.

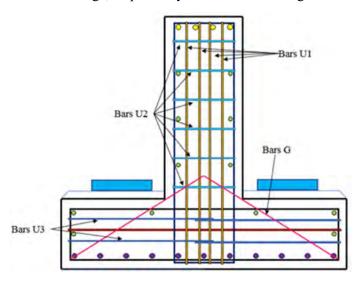


Figure 4.50 End Face Section View of 30 Degrees Skewed ITBC

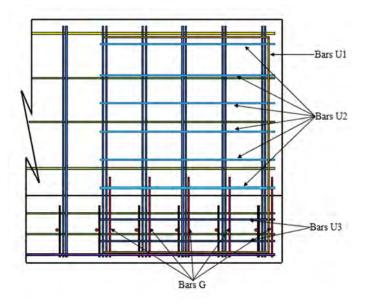


Figure 4.51 End Face Elevation View of 30 Degrees Skewed ITBC

4.3.14 Skin Reinforcement (Bars T)

Try $7 \sim \# 6$ bars in Stem and $3 \sim \# 6$ 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

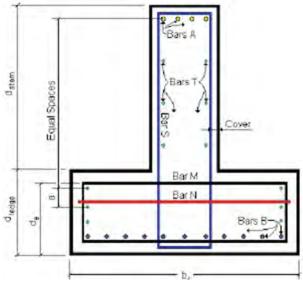


Figure 4.52 Section View for T Bars of 30 Degrees Skewed ITBC

(AASHTO LRFD 5.6.7)

4.3.14.1 Required Area of Skin Reinforcement

$$A_{\rm sk_Req} = 0.012 \cdot (d - 30)$$

$$A_{sk_Req} = 0.62 \frac{in^2}{ft}$$
 (AASHTO LRFD Eq. 5.6.7-3)

 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)

 $s_{req} = minimum of:$

$$\frac{A_{bar_T}}{A_{skReq}} = 8.52 \text{ in}$$

$$\frac{d_{s_neg}}{6} = 13.57 \text{ in}$$

$$\frac{d_{s_pos}}{6} = 13.57 \text{ in}$$
 & 12 in

$$s_{reg} = 8.52 in$$

4.3.14.3 Actual Spacing of Skin Reinforcement

Check T Bars spacing in Stem:

$$\begin{split} h_{top} = d_{stem} - \left(cover + \frac{d_{bar_S}}{2} + \frac{d_{bar_A}}{2}\right) + \left(cover + \frac{d_{bar_M}}{2} + \frac{d_{bar_T}}{2}\right) \\ h_{top} = 56.67 \text{ in} \end{split}$$

$$s_{skStem} = \frac{h_{top}}{NoTBarsSte}$$

$$s_{skStem} = 7.08 in$$

$$s_{skStem} < s_{req}$$

SkinSpacing = "OK!"

Check T Bars spacing in Ledge:

$$\begin{split} h_{bot} = d_{ledge} - \left(cover + \frac{d_{bar_M}}{2} + \frac{d_{bar_T}}{2}\right) - \left(cover + \frac{d_{bar_S}}{2} + \frac{d_{bar_B}}{2}\right) \\ h_{bot} = 21.17 \text{ in} \end{split}$$

$$s_{skLedge} = \frac{h_{bot} - a}{NoTBarsLedge}$$

$$s_{skLedge} = 7.59 in$$

$$s_{skLedge} < s_{req}$$

Check if "a" is less than s_{req}

$$a = 6 in < s_{req}$$

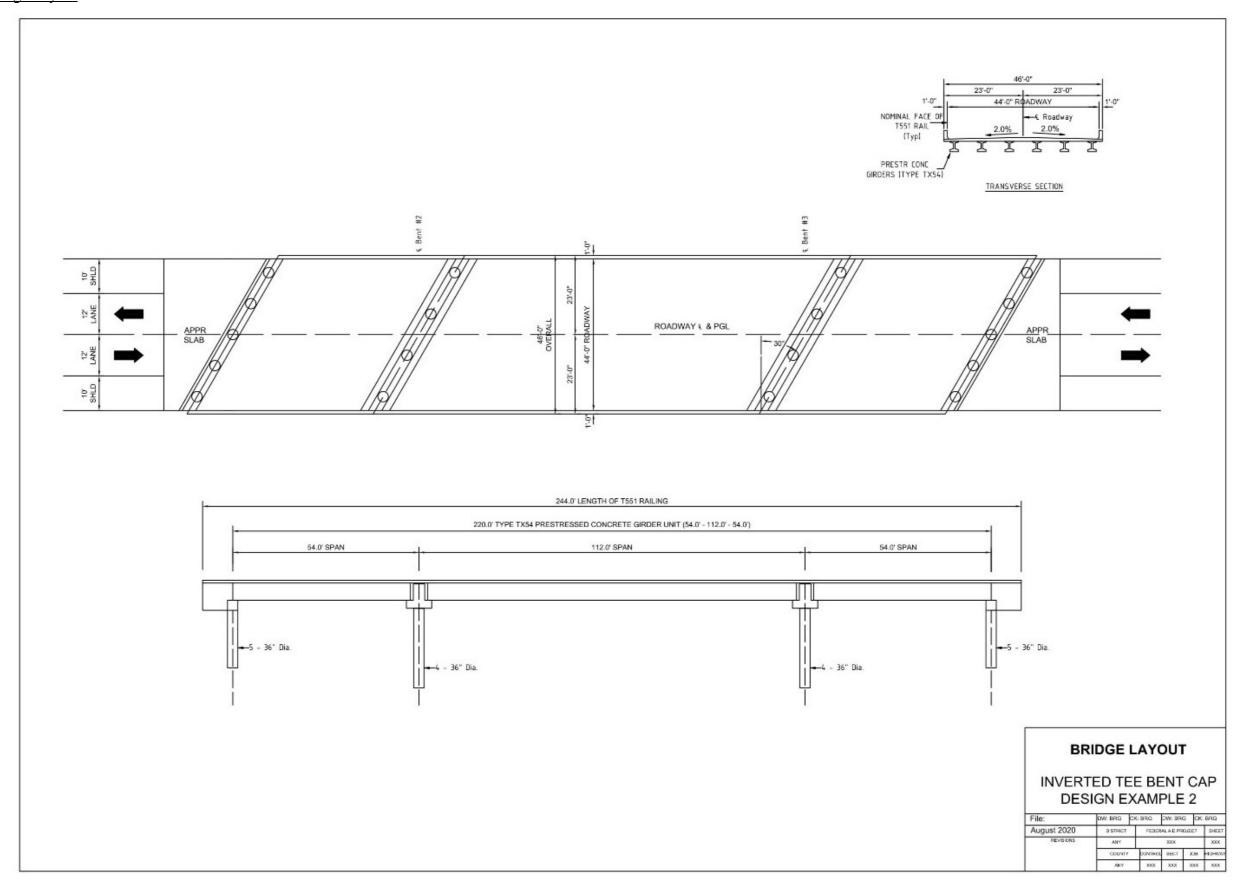
SkinSpacing = "OK!"

Skin Reinforcement Summary:

Use $7 \sim \# 6$ bars in Stem and $3 \sim \# 6$ 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

```
CAP18 Version 6.00 ITBC Design Example 2, Skew = 30.00
SProblem Card -----
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
Ŝ
               X X X X XX
                                            X XX XXXXXXXX
                                        16
$TABLE 2 - CONSTANTS -------
              Anly Opt (1=Working,

|-Movable Load Data--| 2=Load Factor,3=Both)

Num Increment |Num Start Stop Step|Anly| Load Factors:
   TABLE 2a

        Inc
        Length
        | Inc
        Sta
        Sta
        Size| Opt| Dead
        Live

        XX XXXXXXXXX
        XXX XXX XXX
        X X XXXXXXXXX
        XXXXXXXXXX

        92
        0.5
        20
        2
        70
        1
        3
        1.25
        1.75

$
Ś
  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 XX VCP - Shear Control Points
             XX XX XX XX XX
3 6 4 11 8
$
  (Num Inputs)
                               8
  Left Lane Boundary Stations
S
$
  Station of Stringers (two rows max, may be at tenths of stations, XX.X)
   Station of Supports (two rows max)
            XXX XXX XXX XXX XXX XXX XXX XXX XXX
Ś
 (Supports)
              10
                   34
                       58
                           82
  Moment Control Point Stations (two rows max)
          XXX XXX XXX XXX XXX XXX XXX XXX XXX
            6
86
                  10 22 34 38
                                   46 54
                                            58
                                                70
                                                     82
  (Mom CP)
  Shear Control Point Stations (two rows max)
  36 56 60 80 84
$TABLE 4 - STIFFNESS AND LOAD DATA -----
             Bending Sidewalk, Cap &
Station 1 if Stiffness Slab Stringer Moving
             From To Cont'd of Cap
$Comments
                                    Loads
                                            Loads
                                                     Loads
                                                             Loads, DW
                       $XXXXXXXXXXXXXX XXX
                  XXX
              2
                         8.09E+07
(CAP EI & DL)
                   90
                                           -2.427
(DL Span1, Bm1)
                                            -50.17
(DL Span1, Bm2)
               22
                   22
                                            -50.17
                                                             -5.04
(DL Span1, Bm3)
                   38
                                            -50.17
                                                             -5.04
               38
(DL Span1, Bm4)
               54
                   54
                                            -50.17
                                                             -5.04
(DL Span1, Bm5)
               70
                   70
                                            -50.17
                                                             -5.04
(DL Span1, Bm6)
                                            -50.17
(DL Span2, Bm1)
               6
                   6
                                            -104.1
                                                             -10.5
(DL Span2, Bm2)
                   22
                                            -104.1
                                                             -10.5
              38
(DL Span2, Bm3)
                   38
                                            -104.1
                                                             -10.5
(DL Span2, Bm4)
                                            -104.1
                                                             -10.5
(DL Span2, Bm5)
                                            -104.1
                                                             -10.5
(DL Span2, Bm6)
              86
                   86
                                            -104.1
                                                             -10.5
(Dist. Lane Ld)
              0
                   20
                                                    -4.92
(Conc. Lane Ld)
                                                    -21.3
(Conc. Lane Ld)
                                                    -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) **ENVELOPES TABLE NUMBER** OF MAXIMUMS 2 3 4 KEEP FROM PRECEDING PROBLEM (1=YES) 0 0 0 CARDS INPUT THIS PROBLEM 16 OPTION TO CLEAR ENVELOPES BEFORE LANE LOADINGS (1=YES) 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): 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 ANALYSIS OPTION (1=WORKING STRESS, 2=LOAD FACTOR, 3=BOTH) LOAD FACTOR FOR DEAD LOAD LOAD FACTOR FOR OVERLAY LOAD 1.50 LOAD FACTOR FOR LIVE LOAD MAXIMUM NUMBER OF LANES TO BE LOADED SIMULTANEOUSLY

LIST OF LOAD COEFFICIENTS CORRESPONDING TO NUMBER OF LANES LOADED

1.000

1.200

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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 NUM OF NUM OF NUM MOM NUM SHEAR LANES STRINGERS SUPPORTS 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

FIXED-OR-MOVABLE ------ FIXED-POSITION DATA ----- MOVABLESTA STA CONTD CAP BENDING SIDEWALK, STRINGER, OVERLAY POSITION
FROM TO IF=1 STIFFNESS SLAB LOADS CAP LOADS LOADS SLAB LOADS
(K-FT*FT) (K) (K) (K) (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 0.000 0.000 -50.170 -5.040 38 38 0 0.000 54 54 0 0.000 0.000 -50.170 -5.040 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 0.000 0.000 -104.100 -10.500 6 6 0 0.000 0.000 0.000 -104.100 -10.500 0.000 22 22 0 38 38 0 0.000 0.000 -104.100 -10.500 0.000 54 54 0 0.000 0.000 -104.100 -10.500 70 70 0 0.000 -104.100 -10.500 0.000 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

TABLE 4A. DEAD LOAD RESULTS (WORKING STRESS)

STA	DIST X (F	T) DEFLECTIO	N (FT) MO	MENT (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 41	23.09 23.67	-0.000020 -0.000023	102.7 111.6	16.8 14.0	
41	24.25	-0.000023	118.9	11.2	
42	24.25	-0.000025	124.6	8.4	
45	24.63	-0.000027	124.6	0.4	

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) MOI	 MENT (K-FT)	SHEAR (K)
					JITEAN (N)
44	25.40	-0.000028	128.6	5.6	
45	25.98	-0.000029	131.0	2.8	
46		-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 -12.90 0.0 0 0.0 1 0.0 1 0.0 2 0.0 2 0.0 3 3 0.0 0.0 0* 0* 10 -443.9 0.0 0 -203.5 1 2 0 -203.5 1 2 0.0 1 1 2 0.0 2 0.0 3 0.0 3 0.0 294.4 22 233.3 0 13 0 -38.5 2 36 232.3 1 12 1 -38.5 2 36 10.8 3 62 2 0.0 0 2 0.0 3 0.0 3 0* 0* -376.8 34 21.6 3 62 0 -157.4 0 18 0 3 62 1 -134.6 1 12 1 21.6 2 -97.8 2 32 2 0.0 3 0.0 3 0.0 0* 80.1 38 0 96.5 2 32 0 -67.9 1 9 2 32 1 -67.9 1 9 3 62 2 0.0 96.5 1 3.7 2 3 0.0 3 0.0 0* 0* 131.8 46 2 36 0 -32.1 1 9 2 36 1 -32.1 1 9 0 80.1 1 80.1 2 -32.1 3 63 0.0 2 3 3 0.0 0.0 0* 2* 80.1 0 96.5 2 40 0 -67.9 3 63 2 40 1 -67.9 3 63 1 10 2 0.0 96.5 1 3.7 2 3 0.0 3 0.0 0* 0* 58 -376.8 21.6 1 9 0 -157.4 0 54 0 21.6 1 9 1 -134.6 3 60 2 -97.8 2 40 3 0.0 2 0.0 3 0.0 0.0 3

2*

0*

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 1 2 3 0*	233.3 232.3 10.8 0.0	0 3 1	59 60 9 2 3 0*	1 -3	8.5 8.5 0		36 36
82	-443.9 0 1 2 3 0*	0.0 0.0 0.0 0.0		0 1 2 3 0*	-203.5 -203.5 0.0 0.0		70	
86	-12.9 0 1 2 3 0*	0.0 0.0 0.0 0.0		0 1 2 3 0*	0.0 0.0 0.0 0.0			

SHEAR (K)

AT DEAD LD LANE POSITIVE LOAD AT LANE NEGATIVE LOAD AT STA EFFECT ORDER MAXIMUM LANE STA ORDER MAXIMUM LANE STA

8	-186.6 0 1 2 3 0*	0.0 0.0 0.0 0.0	0 -88.1 1 2 1 -88.1 1 2 2 0.0 3 0.0 0*
12	117.8 0 1 2 3 0*	44.8 44.8 1.6 0.0	1 6 0 -5.6 2 36 1 6 1 -5.6 2 36 3 62 2 0.0 3 0.0 0*
32	-108.1 0 1 2 3 0*	1.6 1.6 0.0 0.0	3 62 0 -54.6 0 15 3 62 1 -53.0 1 12 2 -11.2 2 32 3 0.0 0*
36	197.8 0 1 2 3 2*	87.6 84.1 30.7 0.0	0 28 0 -7.8 3 63 2 32 1 -7.8 3 63 1 12 2 0.0 3 0.0 0*
56	-197.8 0 1 2 3 0*	7.8 7.8 0.0 0.0	1 9 0 -87.6 0 44 1 9 1 -84.1 2 40 2 -30.7 3 60 3 0.0 2*
60	108.1 0 1 2 3 0*	54.6 53.0 11.2 0.0	0 57 0 -1.6 1 9 3 60 1 -1.6 1 9 2 40 2 0.0 3 0.0 0*
80	-117.8 0 1 2 3 0*	5.6 5.6 0.0 0.0	2 36 0 -44.8 3 66 2 36 1 -44.8 3 66 2 -1.6 1 9 3 0.0 0*
84	186.6 0 1 2 3 0*	88.1 88.1 0.0 0.0	3 70 0 0.0 3 70 1 0.0 2 0.0 3 0.0 0*

REACTION (K)

AT DEAD LD LANE POSITIVE LOAD AT LANE NEGATIVE LOAD AT STA EFFECT ORDER MAXIMUM LANE STA ORDER MAXIMUM LANE STA

.....

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

		(MAX + (FT-K)				+ SHEAR	MAX - SHEAR
	-0.58	0.0	0.0 0.0 0.0 0.0	0.0 0.0 0.0 -0.7	0.0		
	0.00	0.0	0.0	0.0	0.0		
1	0.58	0.0	0.0	0.0	0.0		
	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	-0.8 -3.2 -7.3 -12.9	-/.3	-8.4	-8.4		
6 7	3.46	-12.9	-12.9	-96.1	-149.0)	
8	4.04	-118.3 -225.2	-1/9.3	-183.8	-285	9.6	
0	5.20	-225.2	-347.3 516.0	100.0	-292	2.4	
10	5.20	-333.0	-688.2	-109.4	-63	5	
11	6.35	-358.4	-590.2	174.4	11	3.9	
12	6.93	-269 N	-493.8	171.4	11	1 1	
13	7.51	-180.6	-399.1	168.8	10	8.3	
14	8.08	-225.2 -333.8 -443.9 -358.4 -269.0 -180.6 -93.8 -8.3	-306.0	166.0	105	5.5	
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	77.1 162.1	-36.4	157.6	97.	1	
18	10.39	246.6	43.3	154.7	94	.3	
19	10.97	246.6 330.0	97.0	151.9	91	.5	
20	11.55	330.0 412.6 493.9 574.3 489.9 404.1 317.1 228.7	149.0	149.1	88	3.7	
21	12.12	493.9	199.4	146.3	85	5.9	
22	12.70	574.3	248.1	20.8	-8.	3	
23	13.28	489.9	196.8	-81.0	-14	8.4	
24	13.86	404.1	143.7	-83.8	-15	1.2	
25	14.43	317.1	88.7	-86.6	-154	1.0	
26	15.01	228.7	31.9	-89.4	-156	0.8	
21	15.59	139.6 56.3	-26.9	-92.2	-159	3.6	
28 29	16.17	56.3	-87.4	-95.0	-162.	.4	
30	17.74	-27.0 -105.6	-149.4	-97.8	-103 16- 3	5.Z	
31	17.32	1645	200 5	100.0	1 1	70.0	
32	18.48	-164.5 -225.0	-407.1	-105.5	2 -17	73.6	
33	19.05	-223.0	-507.1	-100.2) -17	76.4	
34	19.63	-350.9	-609.2	88.7	27	.3	
35	20.21	-239.6	-426.3	315.4	1 19	91.3	
36	20.78	-130.0	-265.4	312.6	5 18	88.5	
37	21.36	-225.0 -287.2 -350.9 -239.6 -130.0 26.8	-129.9	309.8	185	5.7	
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	5	
41	23.67	195.9 204.6 212.0 218.1	46.2	23.3	4.7	7	
42	24.25	223.0	54.6	20.5	1.9)	

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

43 24.83 226.3 60.3 17.7 -0.9 44 25.40 227.9 66.8 12.1 -6.5 46 26.56 227.9 66.8 6.5 -12.1 48 27.71 227.9 66.8 6.5 -12.1 48 27.71 227.9 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 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	SHEAR
49 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	
49 28.29 223.0 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	
49 28.29 223.0 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	
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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	
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	
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	
64 30.95 50.3 -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 46.03 -38.3 -214.5 -102.7 -165.0	
64 30.95 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 46.03 -32.8 -206.0 -102.7 -163.2	
64 30.95 50.3 -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 46.03 -38.3 -214.5 -102.7 -165.0	
64 30.95 50.3 -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 46.03 -38.3 -214.5 -102.7 -165.0	
64 30.95 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 46.03 -32.8 -206.0 -102.7 -163.2	
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 41.3<	
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 40.77 358.4 -590.2 -113.9 -174.4	
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 -171.6 413.9	
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 -590.3 -113.9 -174.4	
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	
70	
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 -590.3 -113.9 -174.4	
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 -590.3 -113.9 -174.4	
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	
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	
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	
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	
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	
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	
80 46.19 -269.0 -493.8 -111.1 -171.6 81 46.77 358.4 590.2 113.9 174.4	
81 46 77 358 4 500 2 113 0 174 4	
61 40.77 -330.4 -330.2 -113.3 -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	

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TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

DIST 2	X MAX	+ MOM	MAX - MO	DM MAX	+ SHEAR	MAX - SHEAR
(FT)	(FT-K)	(FT-K)	(K)	(K)		
50.23	-7.3	-7.3	8.4	8.4		
50.81	-3.2	-3.2	5.6	5.6		
51.38	-0.8	-0.8	2.8	2.8		
51.96	0.0	0.0	0.7	0.7		
52.54	0.0	0.0	0.0	0.0		
53.12	0.0	0.0	0.0	0.0		
53.69	0.0	0.0	0.0	0.0		
	50.23 50.81 51.38 51.96 52.54 53.12	50.23 -7.3 50.81 -3.2 51.38 -0.8 51.96 0.0 52.54 0.0 53.12 0.0	50.23 -7.3 -7.3 50.81 -3.2 -3.2 51.38 -0.8 -0.8 51.96 0.0 0.0 52.54 0.0 0.0 53.12 0.0 0.0	50.23 -7.3 -7.3 8.4 50.81 -3.2 -3.2 5.6 51.38 -0.8 -0.8 2.8 51.96 0.0 0.0 0.7 52.54 0.0 0.0 0.0 53.12 0.0 0.0 0.0	50.23 -7.3 -7.3 8.4 8.4 50.81 -3.2 -3.2 5.6 5.6 51.38 -0.8 -0.8 2.8 2.8 51.96 0.0 0.0 0.7 0.7 52.54 0.0 0.0 0.0 0.0 53.12 0.0 0.0 0.0	50.23 -7.3 -7.3 8.4 8.4 50.81 -3.2 -3.2 5.6 5.6 51.38 -0.8 -0.8 2.8 2.8 51.96 0.0 0.0 0.7 0.7 52.54 0.0 0.0 0.0 0.0 53.12 0.0 0.0 0.0 0.0

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TABLE 7. MAXIMUM SUPPORT REACTIONS (WORKING STRESS)

				-
STA	DIST X	MAX+	REACT	MAX - REACT
(FT)	(K)	(K)	
				-
10	5.77	469.1	308	.9
34	19.63	496.0	306	5.0
58	33.49	496.0	306	5.0
82	47.34	469.1	308	3.9

CAP18

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 1 2 3 0*	0.0 0 0.0 0.0 1 0.0 0.0 2 0.0 0.0 3 0.0 0*
10	-563.9 0 1 2 3 0*	0.0 0 -356.2 1 2 0.0 1 -356.2 1 2 0.0 2 0.0 0.0 3 0.0 0*
22	373.6 0 1 2 3 0*	408.2 0 13 0 -67.4 2 36 406.5 1 12 1 -67.4 2 36 18.9 3 62 2 0.0 0.0 3 0.0
34	-477.8 0 1 2 3 0*	37.8 3 62 0 -275.4 0 18 37.8 3 62 1 -235.5 1 12 0.0 2 -171.1 2 32 0.0 3 0.0 2*
38	102.3 0 1 2 3 0*	168.9 2 32 0 -118.9 1 9 168.9 2 32 1 -118.9 1 9 6.5 3 62 2 0.0 0.0 3 0.0 0*
46	167.0 0 1 2 3 0*	140.1 2 36 0 -56.2 1 9 140.1 2 36 1 -56.2 1 9 0.0 2 -56.2 3 63 0.0 3 0.0 2*
54	102.3 0 1 2 3 0*	168.9
58	-477.8 0 1 2 3 0*	37.8

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
 408.2
 0
 59
 0
 -67.4
 2
 36

 406.5
 3
 60
 1
 -67.4
 2
 36

 18.9
 1
 9
 2
 0.0
 0 1 2 3 0.0 3 0.0 0* 0* 82 -563.9 0 0.0 0 -356.2 3 70 3 70 1 0.0 1 -356.2 0.0 2 0.0 2 3 0.0 3 0.0 0* 0* 86 -16.2 0.0 0.0 0 0.0 0.0 1 1 0.0 0.0 3 3 0.0 0.0 0* 0*

PROB 1 (Spans L=54'-112'-54', Type TX54 Girder @ 8.0', 8" Slab, 2" O'la (CONTINUED)

SHEAR (K)

2

0*

0.0

3

0*

0.0

AT DEAD LD LANE POSITIVE LOAD AT LANE NEGATIVE LOAD AT STA EFFECT ORDER MAXIMUM LANE STA ORDER MAXIMUM LANE STA 8 -237.2 0 -154.2 1 2 1 -154.2 1 2 0 0.0 1 0.0 2 0.0 2 0.0 3 3 0.0 0.0 0* 149.3 12 78.4 1 6 0 -9.7 2 36 78.4 1 6 1 -9.7 2.7 3 62 2 0.0 1 2 36 2 3 3 0.0 0.0 0* 0* 32 -136.9 2.7 3 62 0 -95.6 0 15 2.7 3 62 1 -92.7 1 12 0 1 2 -19.5 2 32 3 0.0 0.0 3 0.0 36 251.2 153.2 0 28 0 -13.6 3 63 0 1 3 0.0 3 0.0 0* 2* 56 -251.2 0 1 0.0 3 0.0 3 0* 2* 60 136.9 95.6 0 57 0 -2.7 1 9 92.7 3 60 1 -2.7 1 9 19.5 2 40 2 0.0 0 1 2 3 0.0 0.0 0* 80 -149.3 9.7 2 36 0 -78.4 3 66 9.7 2 36 1 -78.4 3 66 1 2 -2.7 1 9 2 0.0 3 3 0.0 0.0 0* 0* 84 237.2 154.2 3 70 0 0.0 154.2 3 70 1 0.0 0.0 2 0.0 0 1

REACTION (K)

AT DEAD LD LANE POSITIVE LOAD AT LANE NEGATIVE LOAD AT STA EFFECT ORDER MAXIMUM LANE STA ORDER MAXIMUM LANE STA

0*

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

TA (DIST X	(FT-K) (MOM N FT-K)	MAX - MON	MAX + SHEAR	MAX - SHEAF
-1	-0.58	0.0	0.0	0.0	0.0	
0	0.00 0.58	0.0	0.0	0.0	0.0	
1	0.58	0.0 0.0 0.0 -1.0 -4.0	0.0	0.0	0.0 0.0 -0.9	
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 -286.0	-256.9	-233.7	-418.7	
8	4.62	-286.0	-499.7	-237.2	-422.2	
9	5.20	-423.9 -563.9	-744.5	-240.7	-425.7	
10	5.77	-563.9	-991.3	-14.3	-94.7	
11	6.35	-448.1 -325.0	-853.8	247.0	141.1 137.6	
12	6.93	-325.0	-718.4	243.5	137.6	
13	7.51	-202.7 -82.3	-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	
1/	9.81	2/5./	-/1./	225.9	130.6 127.1 123.6 120.1 116.6 113.1 109.6 106.1 -16.8 -220.1 -223.6 -227.1 -230.6 -234.1 -241.1 -244.6 -244.6 -248.1 -251.6	
18	10.39	395.2	39.5	222.4	116.6	
20	11.57	513.7	170.8	218.9	113.1	
20	11.55	747.0	222.4	213.4	109.6	
21	12.12	962.4	202.4	2/1.9	160.1	
22	12.70	720 2	232.7	-102.1	-10.0	
23	12.20	/30.2 611.2	155.0	105.1	-220.1	
24	14.42	492.2	93.6	100.0	-223.0	
25	15.43	463.Z 252.5	0.0	1126	220.6	
27	15.01	223.3	-68.2	-116.1	-230.0	
28	16.17	104.0	-147.4	-110.1	-237.6	
29	16.74	-14.4	-228.7	-173.0	-241 1	
30	17.74	-123.9	-312 9	-125.1	-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	-251.6 -255.1	
34	19.63	-432.5	-884.4	133.8	26.4	
35	20.21	-432.5 -293.8	-620.4	455.6	26.4 238.4	
36	20.78	-157.1	-394.2	452.1	234.9	
37	21.36	-157.1 62.8	-211.4	448.6	231.4	
38	21.94	305.0 314.2	-40.3	238.4	119.8	
39	22.52	314.2	-15.8	40.8	8.2	
40	23.09	321.9 328.2	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	
	24.83	335.9	45.5	26.8	-5.8	
	25.40	336.8				

46 26.56 335.2 54.6 16	
40 20.30 333.2 34.0 10	.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

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TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA (DIST X (FT)		иом мо	AX - MOM (K) (F	MAX + SHEAR	MAX - SHEAR
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	328.2 321.9	27.3	-1.2	-33.8	
52	30.02	321.9	6.8	-4.7	-37.3	
	30.60	314.2	-15.8	-8.2	-40.8	
54	31.18	305.0 62.8	-40.3	-119.8	-238.4	
55	31.75	62.8	-211.4	-231.4	-448.6	
		-157.1				
57	32.91	-293.8 -432.5	-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 -198.0	-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	-14.4 104.0	-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 67	38.11	353.5 483.2	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	738.2 863.4	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	631.5 513.7	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	275.7 156.1	-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 -202.7	-453./	-130.6	-236.4	
79	45.61	-202.7	-585.0	-134.1	-240.0	
80	46.19	-325.0	-/18.4	-13/.6	-243.5	
81	46.77	-448.1 -563.9	-853.8	-141.1	-247.0	
82	47.34	-563.9	-991.3	94./	14.3	
83	47.92	-423.9	-/44.5	425.7	240.7	
84	48.50	-423.9 -286.0 -150.1	-499.7	422.2	237.2	
85	49.07	-150.1	-256.9	418./	233./	
07	49.05	-16.2 -9.1 -4.0	-10.2	10.5	122.1	
0/	50.23	-9.1	-9.1	7.0	7.0	
00	50.81	-4.0	1.0	7.0	7.0	
89	31.38	-1.0	-1.0	3.5	3.3	

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

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TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

				MAX - M (K)		+ SHEAR	MAX - SHEAR
93	53.69	0.0	0.0	0.0	0.0		

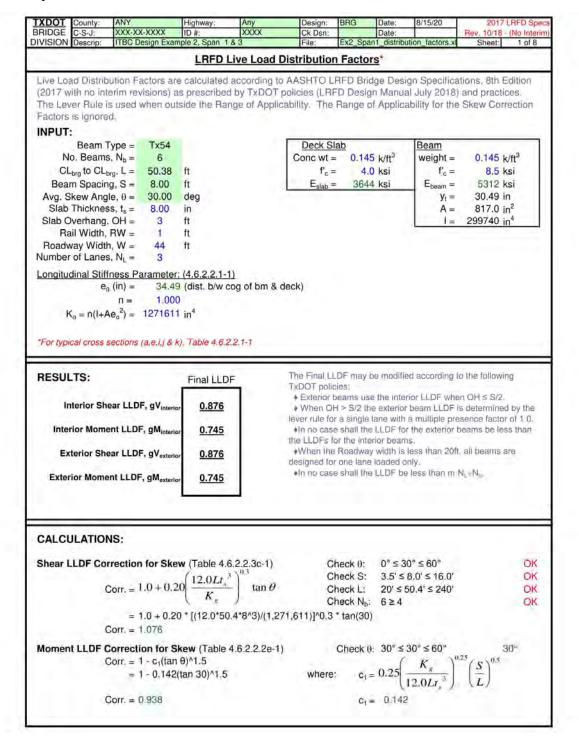
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TABLE 7. MAXIMUM SUPPORT REACTIONS (LOAD FACTOR)

				-
STA	DIST X	MAX+	REACT	MAX - REACT
(FT)	(K)	(K)	
				-
10	5.77	669.0	388	.8
34	19.63	715.2	382	2.5
58	33.49	715.2	382	2.5
82	47.34	669.0	388	3.8

4.3.15.4 <u>Live Load Distribution Factor Spreadsheet</u>

4.3.15.4.1 Spans 1 & 3

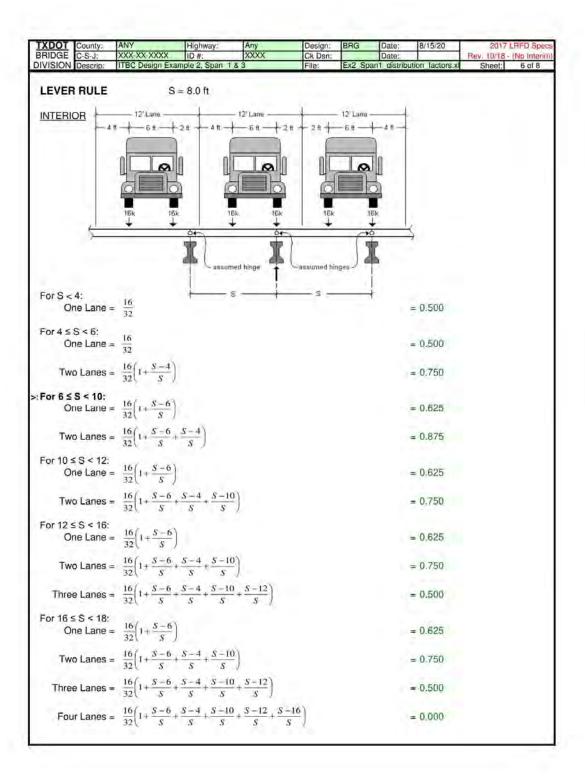


TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Spec
BRIDGE DIVISION	C-S-J: Descrip:	ITBC Design Exa	ID #: mple 2, Span 1	XXXX & 3	Ck Dsn: File:	Ex2 Span	Date: 1 distribu	tion factors.xl	Rev. 10/18 - Sheet:	2 of 8
INTER	IOR BE									
Shear L	L Distrib	ution Per Lane	Table 4.6.2.2	.3a-1):						
	One La	ine Loaded								
		Lever Rule	(Table 3.6.	1.1.2)						
		mg = 0.6	625 * 1.2 =	0.750						
		Modify fo	or Skew:							
			skew corre		1,076					
		41.1515	mg = 0.750	1.076 =	0.807					
		$\frac{\text{Equation}}{g = 0.36}$	$6+\left(\frac{S}{25}\right)$							
			+ (8 / 25) =	0.680						
			or Skew:							
			skew corre	ction =	1.076					
			g = 0.680 *	1.076 =	0.732					
		Range of App								
			3.5' ≤ 8.0'		OK					
			4.5" ≤ 8.0"		OK					
			20' ≤ 50.4'	≤ 240'	OK					
		Check N	77	55550	OK					
		Use Equation		.6.2.2.3a-1	because all	criteria is (OK.			
		gV _{int1} =	0.732							
	Two or	More Lanes Lo								
		Lever Rule	(Table 3.6.	The second	DE 0.07E * 0	CEV .	0.075			
		2000	ax(0.875 * 1.0 or Skew:			.65) =	0.875			
			skew corre		1.076					
			mg = 0.875	* 1.076 =	0.942					
		Equation $g = 0.2$	$+\left(\frac{S}{12}\right)-\left(\frac{S}{3}\right)$	$\left(\frac{3}{5}\right)^{2.0}$						
		g = 0.2 +	(8 / 12) - (8 /	35)^2.0 =	0.814					
			or Skew:							
			skew corre		1,076					
			g = 0.814 *	1.076 =	0.876					
		Range of App	licability (ROA	A) Checks	(same as f	or one lar	ne loade	ed)		
		Use Equation gV _{int2+} =	from Table 4 0.876	.6.2.2.3a-1	because all	criteria is (OK.			
	TXDOT	Policy states gV	loteror must be	≥ m·N; ÷N;						
		$m \cdot N_L \div N_b =$	0.85 * 3/6		0.425					
	Is W≥	20ft ? Yes								
	TXDOT	Policy states the	at if W < 20ft,	gV _{interior} is t	he Maximum	of: gV _{mti}	and m	NL+Nb		
>>	TXDOT	Policy states that	at if W ≥ 20ft,	gV _{Interior} is t	he Maximum	of: gV _{int1}	gV _{int2+} ,	$m\text{-}N_L\div N_b.$		
	gV _{inte}	erior = 0.876								

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017	LRFD Specs
BRIDGE	C-S-J: Descrip:	ITBC Design Exam	ID#:	XXXX	Ck Dsn: File:	Ev2 Son	Date:	tion factors.xl	Rev. 10/18 Sheet:	- (No Interim 3 of 8
	OR BE		ipie 2, Opan i	ŭ J	Trile.	LAE OPO	iii ustiibu	IIOII IGUIDIS.XI	Sileet	3010
		ribution Per Lane	(Table 4.6.2	2.2h-1):						
Moment		ne Loaded	114616 4.0.2	17.						
		Lever Rule	(Table 3.6,	1.1.2)						
			25 * 1.2 =	0.750						
		Modify fo	r Skew:							
			skew corre	ction =	0.938					
			mg = 0.750	* 0.938 =	0.704					
		Equation	(5)04	5 \0.3/ K	70.1					
			$+\left(\frac{S}{14}\right)^{0.4}$							
		JT0 100 100	+ (8/14)^0.4	* (8/50.4)^0.	3 + (1,271,6	11/(12*5	0.4*8^3))	0.1 =	0.590	
		Modify fo								
			skew corre		0.938					
		Service Co.	g = 0.590 *		0.553					
		Range of Appli								
			3.5' ≤ 8.0' 4.5" ≤ 8.0"			OK				
			20' ≤ 50.4'	300		OK OK				
		Check N		3 240		OK				
			: 10,000 ≤ 1	271 611 < 7	7 000 000	ОК				
		Use Equation I					OK			
		gM _{int1} =	0.553	reinieled (P	22422 411	7110110210	~ / / /			
	Two or	More Lanes Lo	aded							
	3.00.00	Lever Rule	(Table 3.6.	1.1.2)						
			(0.875 * 1.0	A 1217 - 12 - 13	5, 0.875 * 0.	.65) =	0.875			
		Modify fo	r Skew:							
			skew corre	ction =	0.938					
			mg = 0.875	* 0.938 =	0.821					
		Equation $q = 0.07$	$5 + \left(\frac{S}{9.5}\right)^{0.6}$	$\left(\frac{S}{s}\right)^{0.2}\left(\frac{S}{s}\right)^{0.2}$	$\left(K_{g}\right)^{0.1}$					
						P4 4 //4 P4	ED 4*0AD	\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\	0.704	
		g = 0.075 Modify fo	+ (8/9.5)^0. r Skew:	6 (8/50.4)	0.2 (1,2/1,	611/(12	50.4 873))^0.1 =	0.794	
			skew corre	ction =	0.938					
			g = 0.794 *	0.938 =	0.745					
		Range of Appli						d)		
		Use Equation I	rom Table 4	.6.2.2.2b-1 b	ecause all o	criteria is	OK.			
		$gM_{int2+} =$	0.745							
	TXDOT	Policy states gM	Interior must be	$e \ge m \cdot N_L \div N_b$						
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6	=	0.425					
		20ft ? Yes	6311		11/2			5. 6.		
		Policy states tha								
>>		Policy states tha	tifW ≥ 20ft,	gM _{interior} is U	ne Maximum	of gM	gM _{mi2+}	m·NL=No		
	gM _{inte}	erior = 0.745								

TXDOT		ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Specs
DIVISION	C-S-J: Descrip:	ITBC Design Exa	ID #: ample 2. Span 1	XXXX & 3	Ck Dsn: File:	Ex2 Sr	Date:	ution factors.x	Rev. 10/18 - (No Interim) Sheet: 4 of 8
	RIOR BE		- poe a, wpoint		1, 110,	, m. b	and the		Toron Toro
100000000000000000000000000000000000000		ution Per Lane	(Table 4.6.2.)	2.3b-1):					
511047		ine Loaded	Tracio Tronsi	17.					
		Lever Rule	(Table 3.6	.1.1.2)					
		mq = 0.	625 * 1.0 =		TxDOT us	es a mu	Itiple pre	sence facto	r of 1,0 for one
		Modify f	or Skew:		lane loade				
			skew corre	ection =	1,076				
			mg = 0.62	5 * 1.076 =	0.673				
		Use Lever Ru	ile, as per AA	SHTO LRF	Table 4.6.2	2.2.3b-1	1		
		gV _{ext1} =	0.673						
100	Two or	More Lanes L	oaded						
	0.02/3/15/	Lever Rule	(Table 3.6	.1.1.2)					
		mg = M	ax(0.625 * 1.0		35, 0.625 * 0	.65) =	0.625		
			or Skew:						
			skew corre	ection =	1,076				
			mg = 0.62	5 * 1.076 =	0.673				
		Equation							
		$d_e = dis$	t. b/w CL web	to curb					
		$d_e = OH$	- Rail Width						
		$d_e =$	3ft - 1ft =	2.0	tt				
		0.6	$+\left(\frac{d_{\kappa}}{10}\right)$						
		6 = 0.0	+(10)						
		e = 0.6	+ (2.0/10) =	0.800					
		$g = e^*g^{V}$	V _{int2+Eq}						
		g = 0.80	0 * 0.876 =	0.701					
		Skew C	orrection is in	cluded in gl	/(interior).				
		Range of App	olicability (RO	A) Checks	Interior	ROA is	implicitly	applied to t	he exterior beam.
		Check I	nterior Beam	ROA:	OK				
		Check of	d _e : -1.0' ≤ 2.0	0' ≤ 5.5'	OK				
		Check N	N _b : 6 ≠ 3		OK				
		Use Equation	from Table 4	4.6.2.2.3b-1	because all	criteria i	s OK.		
		$gV_{ext2+} =$	0.701						
	TXDOT	Policy states g	V _{Exterior} must b	e ≥ gV _{interior}					
		gV _{interior} =	0.876						
	TXDOT	Policy states g	V _{Exterior} must b	e ≥ m·N _L ÷N	b				
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6	3 =	0.425				
	Is OH ≤	S/2 ? Yes							
		20ft ? Yes							
>>		Policy states th							
1	TXDOT	Policy states th	at if OH > S/2	2 and W < 2	Oft, gV _{Exterior}	s the M	aximum o	of: gV _{ext1} , g\	interior, and
		m·N _L ÷N _b .	Over an area	Crown to a	G N			V 11	0.00
	TXDOT	Policy states th		2 ans W ≥ 20	oft, gV _{Exterior} i	s the M	aximum c	of: gV _{ext1} , gV	ext2+- gVinterior
1 4		and m·N _L +N _b							
10.0	gV _{ext}	erior = 0.876							

TXDOT	County:	ANY XXX-XX-XXXX	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Spece
BRIDGE	C-S-J: Descrip:	ITBC Design Exa	ID #: ample 2, Span 1		Ck Dsn: File:	Ex2 So	Date: an1 distrib	ution factors.x	Rev. 10/18 Sheet:	5 of 8
	RIOR BE				1, 1101				0,1000	
		ribution Per Lan	e (Table 4.6.2	2.2d-1):						
		ne Loaded								
		Lever Rule								
		mg = 0.6	625 * 1.0 =	0.625	TxDOT us	es a mu	Itiple pres	sence factor	r of 1,0 for o	nie
		Modify f	or Skew:		lane loade					
			skew corre	ction =	0.938					
			mg = 0.625	* 0.938 =	0.586					
		Use Lever Ru	ile as per AAS	HTO LRFD	Table 4.6.2	.2.2d-1.				
		gM _{ext1} =	0.586							
	Two or	More Lanes Lo	paded							
		Lever Rule	(Table 3.6.	1.1.2)						
		mg = Ma	ax(0.625 * 1.0	0.625 * 0.8	35, 0.625 * 0	.65) =	0.625			
		Modify f	or Skew:							
			skew corre	ction =	0.938					
			mg = 0.625	* 0.938 =	0.586					
		Equation								
		e = 0.7	$7 + \left(\frac{d_{_{\ell}}}{9.1}\right)$							
		e = 0.77	+ (2.0/9.1) =		0.990					
		$g = e^*gN$	A _{Int2+Eq}							
		g = 0.99	* 0.745 =	0.738						
		Skew Co	orrection inclu	ded in gM(i	nterior).					
		Range of App	licability (RO	A) Checks	Interior	ROA is	implicitly	applied to t	he exterior l	beam.
		Check In	nterior Beam I	ROA:	OK					
		Check d	l _e : -1.0' ≤ 2.0'	≤ 5.5'	OK					
		Check N	N _b : 6 ≠ 3		OK					
		Use Equation	from Table 4	6.2.2.2d-1	because all	criteria is	s OK			
		gM _{ext2+} =	0.738							
	TXDOT	Policy states gl	M _{Exterior} must b	e ≥ gM _{interior}						
		gM _{interior} =	0.745							
	TXDOT	Policy states gN	M _{Exterior} must b	e≥m·N _L ÷N	ь					
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6	=	0.425					
	Is OH ≤	S/2 ? Yes								
		20ft ? Yes								
>>		Policy states th		-	-					
	TXDOT	Policy states th	at if OH > S/2	and W < 20	Oft, gM _{Exterior}	is the M	aximum (of: gM _{ext1} , gl	M _{interior} , and	
	3000	$m \cdot N_L \div N_b$	Na Swe Sid							
	TXDOT	Policy states th		ans W ≥ 20	oft, gM _{Exterior}	is the M	aximum o	of: gM _{ext1} , gf	M _{ext2+} , gM _{mte}	morr
		and m·N _L +N _b								
	gM _{ext}	erior = 0.745								



XDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Spe
RIDGE		ITBC Design Exa	ID#:	XXXX	Ck Dsn: File:	Ev2 Con	Date:	ion factors.xl	Rev. 10/18 - Sheet:	7 of 8
VISION	Descrip:	IT BC Design Exa	inple 2, Span 1 &	3	[File;	Exc_opai	11 distribut	on_lactors.x	Sheet	7 01 0
	RULE		= 8.0 ft							
	OR (con't)									
For 18 ≤ Or	S < 22: ne Lane =	$\frac{16}{32} \left(1 + \frac{S-6}{S} \right)$					-	0.625		
Two	Lanes =	$\frac{16}{32}\bigg(1+\frac{S-6}{S}+$	$\frac{S-4}{S} + \frac{S-10}{S} \bigg)$				0.5	0.750		
		$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$					-	-0.125		
		$\frac{16}{32}\left(1+\frac{S-6}{S}+\right)$	$\frac{S-4}{S} + \frac{S-10}{S} +$	$\frac{S-12}{S}+\frac{S-12}{S}$	$\left(\frac{-18}{S} + \frac{S-16}{S}\right)$		1	0.625		
or 22 ≤ Or	S ≤ 24; ne Lane =	$\frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$					119	0.625		
Two	Lanes =	$\frac{16}{32}\bigg(1+\frac{S-6}{S}\bigg)$	$\frac{S-4}{S} + \frac{S-10}{S}$				-	0.750		
Three	e Lanes =	$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$\frac{S-4}{S} + \frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S}{S}$	$\left(\frac{-18}{S}\right)$		110	-0.125		
Fou	r Lanes =	$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$\frac{S-4}{S} + \frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S}{}$	$\frac{-18}{S} + \frac{S - 16}{S}$	$+\frac{S-22}{S}$	-	-1.500		
		16k	16k	5 ft	46-7				S=	8.0 f
	-	он ж	- s	assumer	1 hinge			Rail Widti X = S+OH-		3.0 f 1.0 f 8.0 f
For X < Or	6: ne Lane =	$-\frac{16}{32} \left(\frac{X}{S} \right)$						0.500		
For 6 ≤ Or	X < 12; ne Lane =	$\frac{16}{32} \left(\frac{X}{S} + \frac{X - 6}{S} \right)$	5)					0.625		
For 12 s	X < 18; ne Lane =	$\frac{16}{32} \left(\frac{X}{S} + \frac{X - S}{S} \right)$	5)					0.625		
Or		Contract of the contract of th								

BRIDGE	County:	ANY XXX-XX-XXXX	Highway:	XXXX	Design:		Date: 8/15/20	_	2017 LRFD Sper
IVISION		ITBC Design Exa			Ck Dsn: File:		distribution factor	ors.xl	Sheet: 8 of 8
Cart	i de la constante de la consta								
LEVER	RULE								
EXTERI	IOR (con') S	= 8.0 fr	É	OH =	3.0 ft			
	,,,,,,	RW			H-RW-2ft =				
For 19 e	X < 24:								
Or	ne Lane =	$\frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6				= 0.625		
		22/10 10	5 (100 - 00 - 10	7-101					
Two	b Lanes =	$\frac{16}{32}\left(\frac{X}{S} + \frac{X - X}{S}\right)$	$\frac{\alpha_+}{s}$	S			= -0.250	0	
For 24 ≤	X < 30:	107.00	13						
Or	ne Lane =	$\frac{16}{32}\left(\frac{X}{S} + \frac{X - X}{S}\right)$	0				= 0.625		
	4	16 (X X -	6 X-12 X	(-18)			5.500		
Two	Lanes =	$\frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	S	S			= -0.25)	
Three	e Lanes =	$\frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{4} + \frac{X - 12}{4} + \frac{X}{4} +$	x - 18 + x - 2	4)		= -1.250	Ò	
		32(8 8	S	8 5	7.				
For 30 ≤	X < 36:	$\frac{16}{32}\left(\frac{X}{S} + \frac{X - X}{S}\right)$	6)				= 0.625		
		32 (3	1	100			- 0.020		
Two	Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{5} + \frac{x-12}{5} + \frac{x}{2}$	(-18)			= -0.250	0	
		Coreta II			4 X - 30)				
Three	e Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	\$	S S	+		= -2.62	5	
For 36 ≤	X < 42:	16/ X X -	63						
Or	ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	-				= 0.625		
Torr	o Lange -	$\frac{16}{32}\left(\frac{X}{S} + \frac{X}{S}\right)$	$\frac{6}{4} \times -12 \times \frac{1}{4} $	(-18)			= -0.250	0	
1 440	o Lanes =	32 S	S	S)			= -0.25	J	
Three	e Lanes =	$\frac{16}{32}\left(\frac{X}{S} + \frac{X-S}{S}\right)$	$\frac{6}{e} + \frac{x-12}{e} + \frac{x}{2}$	$\frac{x-18}{6} + \frac{x-2}{6}$	$\frac{4}{4} + \frac{X - 30}{6}$		= -2.62	5	
		*****				V - 363			
Fou	r Lanes =	$=\frac{16}{32}\left(\frac{X}{S}+\frac{X-S}{S}\right)$	$\frac{0}{S} + \frac{x - 12}{S} + \frac{x}{S}$	$\frac{1}{S} + \frac{\lambda - 2}{S}$	+ 3 - 30 +	5	= -4.37	5	
For 42 ≤	≤ X ≤ 48:	12.00 in in	-1						
Or	ne Lane =	$\frac{16}{32}\left(\frac{X}{S} + \frac{X-X}{S}\right)$	6				= 0.625		
1.2		16 (X X -	6 X-12 X	(-18)			4144		
Two	o Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - C}{S} \right)$	- + - S	S			= -0.25	0	
Three	e Lanes =	$\frac{16}{32}\left(\frac{X}{5} + \frac{X-}{5}\right)$	$\frac{6}{4} + \frac{X - 12}{4} + \frac{X}{4}$	x - 18 + x - 2	$\frac{4}{4} + \frac{X - 30}{1}$		= -2.62	5	
		Server B							
Fou	r Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X-12}{S} + \frac{X}{S}$	$\frac{c-18}{s} + \frac{x-2}{s}$	$\frac{4}{S} + \frac{X - 30}{S} + \frac{1}{S}$	$\frac{X-36}{S} + \frac{X-36}{S}$	$\left(\frac{-42}{s}\right) = -6.500$	0	
INTERIO	85.				EXTER	7.01			
	ne Loade	d	= 0.625			ne Loaded		-	0.625
	nes Load		= 0.875			nes Loaded		=	0.625
	anes Loa		= 0.875			anes Load		-	0.625
	nes Load		= 0.875			nes Loade			0.625

4.3.15.4.2 *Span 2*

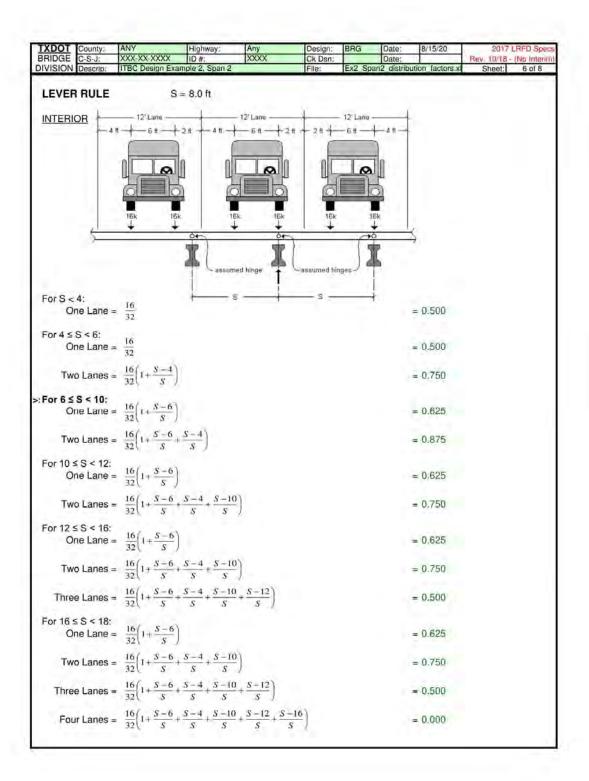
TXDOT County: BRIDGE C-S-J:	ANY XXX-XX	-XXXX	Highway:	Any	Design: Ck Dsn:	BRG	Date:	8/15/20		LRFD Specs (No Interim)
DIVISION Descrip:	ITBC De	sign Exan	ple 2, Span 2		File:	Ex2_Spa	an2 distrib	ution_factors.x	Sheet:	1 of 8
			LRFD	Live Load Di	stribution	Facto	rs*			
Live Load Distribe (2017 with no inte The Lever Rule is Factors is ignored	erim rev s used v	isions) a	s prescribe	d by TxDOT po	olicies (LRI	FD Desig	gn Manu	al July 2018) and practi	ces.
INPUT:										
Beam T No. Beams	, N _b =	Tx54 6			Deck Si Conc wt =	0.14	5 k/ft ³	Beam weight =		
CL _{brg} to CL _b			ft	- 4	f'c =		0 ksi	f'c≃		
Beam Spacing Avg. Skew Angl		8.00	ft deg	4	E _{slab} =	304	4 ksi	E _{beam} =		
Slab Thicknes		8.00	in					A =		35.0
Slab Overhang,	/ To	3	ft					1=		
Rail Width,		1	ft							
Roadway Width		44	ft							
Number of Lanes		3								
K _q = n(l+A *For typical cross s RESULTS: Interior She Interior Mome Exterior She Exterior Mome	ear LLDF ear LLDF ear LLDF	gV _{interio}	in ⁴ <i>Table 4.6.</i> Final LLC 0.891 0.626	DF	TxDOT polic Exterior I When Oil lever rule for In no case the LLDFs II When the designed for	cies: peams us H > S/2 ti r a single e shall th or the intel Roadwa r one lane	se the inte ne exterio lane with e LLDF to erior bean y width is e loaded o	less than 20	en OH ≤ S/2, Is determine esence facto beams be le ft, all beams :	d by the r of 1.0. ss than
Shear LLDF Cor	157	for Ska	M (Table 1	6 2 2 3c-1\	C	heck θ:	0° < 3	30° ≤ 60°		ОК
						heck S:	2	8.0' ≤ 16.0'		OK
C	orr. = 1	.0 + 0.2	$0\left(\frac{12.0Lt_s}{K_s}\right)$	$ \tan \theta$		heck L:		106.8' ≤ 240		OK
			K_g)			6≥4			OK
C	= 1 Corr. = 1		0 * [(12,0*1)	06.8*8^3)/(1,27						
Moment LLDF C	arreati	on for C	kow /Table	1622251		Chook	300-	30° ≤ 60°		30°
		- c _t (tan		4.0.2.2.28-1)		GHECK I	/ 50 5	00 ≥ 00°	0.25	
			tan 30)^1.5	P	where	: c _t	= 0.25	$\frac{K_g}{12.0Lt_s^3}$	$\left(\frac{S}{L}\right)^{-1}$	
C	corr. = 0	.964				C ₁	= 0.08	1		

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Spec
BRIDGE DIVISION	C-S-J: Descrip:	ITBC Design Exa	ID #: ample 2, Span 2	XXXX	Ck Dsn: File:	Ex2 Spa	Date: in2_distribu	ition_factors.xl	Rev. 10/18 - Sheet:	2 of 8
INTER	IOR BE	AM:								
Shear I	L Distrib	ution Per Lane	(Table 4.6.2.2	.3a-1):						
	One La	ine Loaded								
		Lever Rule	(Table 3.6.	1.1.2)						
		mg = 0.0	625 * 1.2 =	0.750						
		Modify f	or Skew:							
			skew corre		1,095					
		64.1545	mg = 0.750	1.095 =	0.821					
		Equation $g = 0.3$	$6+\left(\frac{S}{25}\right)$							
			5 + (8 / 25) =	0.680						
			or Skew:	0.000						
		wicelly i	skew corre	ction =	1.095					
			g = 0.680 *	23.04	0.745					
		Range of Apr	olicability (RO/							
			3.5' ≤ 8.0'		OK					
		Check t	s: 4.5" ≤ 8.0"	≤ 12.0"	OK					
		Check L	20' ≤ 106.8	3' ≤ 240'	OK					
		Check N	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 L	oaded							
		Lever Rule	(Table 3.6.	1.1.2)						
		2003000	ax(0.875 * 1.0 or Skew:	, 0.875 * 0.8	35, 0.875 * 0	.65) =	0.875			
			skew corre	ction =	1.095					
			mg = 0.875	* 1.095 =	0.958					
		Equation $g = 0.2$	$+\left(\frac{S}{S}\right)-\left(\frac{S}{S}\right)$	2.0						
			(12) (3	5)	0.044					
		7.7	+ (8 / 12) - (8 / for Skew:	35)-2.0 =	0.814					
		Widdily I	skew corre	ction =	1,095					
			g = 0.814 *		0.891					
		Range of Apr	olicability (RO		(same as t	for one la	ne loade	ed)		
			from Table 4							
		gV _{int2+} =	0.891							
	TXDOT	Policy states g								
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6	=	0.425					
		20ft ? Yes	-11110 000	-W 1154		-1	200-0	N . N .		
		Policy states th								
>>		Policy states th	at II VV 2 20ft,	g Vinlerior IS t	ne waximun	or. gv _{int}	1. 9 Vint2+1	H-NL÷N _b .		
	gV _{inte}	erior = 0.891								

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRF	D Spece
BRIDGE	C-S-J: Descrip:	ITBC Design Exar	ID#:	XXXX	Ck Dsn: File:	Ev2 So	Date:	ition factors.xl	Rev. 10/18 - (No Sheet: 3	Interim 3 of 8
	IOR BE		ipic a, opair a		Triid.	LAE OP	anz distribu	UDIT_IGUIDIS.SI	Sildet.	UI U
		ribution Per Lane	(Table 4.6.	2.2.2b-1):						
Monto		ne Loaded	114410 1101	17.						
		Lever Rule	(Table 3.6	,1.1.2)						
		mg = 0.6	25 * 1.2 =	0.750						
		Modify fo	r Skew:							
			skew corre	ection =	0.964					
			mg = 0.75	0 * 0.964 =	0.723					
		$\frac{\text{Equation}}{\text{g} = 0.06}$	$+\left(\frac{S}{14}\right)^{0.4}$	$\left(\frac{S}{L}\right)^{0.3} \left(\frac{K}{12L}\right)^{0.3}$	$\left(\frac{s}{t}\right)^{0.1}$					
				* (8/106.8)^(611/(12	106.8*8^	3))^0.1 =	0.453	
		Modify fo	r Skew:							
			skew corre	ection =	0.964					
			g = 0.453	* 0.964 =	0.437					
		Range of Appl	icability (RC	A) Checks						
			3.5' ≤ 8.0'			OK				
			4.5" ≤ 8.0			OK				
			20' ≤ 106.	.8' ≤ 240'		OK				
		Check N			7 000 000	OK				
				1,271,611 ≤		OK	60			
		Use Equation gM _{int1} =	0.437	4,6.2,2,26-11	oecause all	criteria is	S OK			
	Two or	More Lanes Lo	aded							
		Lever Rule	(Table 3.6	.1.1.2)						
		mg = Ma	x(0.875 * 1.0	0, 0.875 * 0.8	5, 0.875 * 0	.65) =	0.875			
		Modify fo	r Skew:							
			skew corre		0.964					
			mg = 0.87	5 * 0.964 =	0.844					
		$\frac{\text{Equation}}{\text{g} = 0.07}$	$5+\left(\frac{S}{9.5}\right)^6$	$\frac{16}{L}$ $\left(\frac{S}{L}\right)^{0.2} \left(\frac{1}{12}\right)^{0.2}$	$\left(\frac{K_g}{2Lt_s^3}\right)^{0.1}$					
		g = 0.075 Modify fo	+ (8/9.5)^0	.6 * (8/106.8)	^0.2 * (1,27	1,611/(1	2*106.8*8	3^3))^0.1 =	0.649	
			skew corre	ection =	0.964					
			g = 0.649	* 0.964 =	0.626					
		Range of Appl	icability (RC	A) Checks	(same as f	or one la	ane loade	ed)		
		Use Equation	from Table 4	4.6.2.2.2b-1 l	pecause all	oriteria is	OK.			
		gM _{int2+} =	0.626							
	TXDOT	Policy states gM	Interior must b	e ≥ m·N _L ÷N _k						
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6		0.425					
	Is W ≥	20ft ? Yes								
		Policy states tha	t if W < 2011	, gM _{triterior} is t	he Maximun	of: gM	nti and m	N _L ÷N _b -		
>>	TXDOT	Policy states that	t if W ≥ 20ft	gM _{interior} is t	he Maximun	oli gM	ma gMinte	+ m·NL÷Nb		
	gM _{inte}	erior = 0.626								
_			_							_

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		RFD Specs
BRIDGE	C-S-J: Descrip:	ITBC Design Exa	ID#:	XXXX	Ck Dsn: File:	Fx2 Sn	Date:	ution factors.x	Rev. 10/18 - Sheet:	(No Interim) 4 of 8
	RIOR BE		Imple E, Open E		It iid.	Lane Op	ant distrib	bioti_lubtore.s	. Onder.	7010
man 2 2 3 mm.		ution Per Lane	(Table 4.6.2.)	2.3b-1):						
311041		ne Loaded	Tradio Hones	17.						
1		Lever Rule	(Table 3.6	.1.1.2)						
			625 * 1.0 =		TxDOT us	es a mu	Itiple pre	sence factor	of 1,0 for an	ne
			or Skew:		lane loade				50 114 150 50	
			skew corre	ection =	1,095					
			mg = 0.62	5 * 1.095 =	0.684					
		Use Lever Ru	ile. as per AA	SHTO LRF	D Table 4.6.2	2.2.3b-1				
		gV _{ext1} =	0.684							
	Two or	More Lanes L	oaded							
		Lever Rule	(Table 3.6	.1.1.2)						
		mg = M	ax(0.625 * 1.0	0, 0.625 * 0.0	85, 0.625 * 0	.65) =	0.625			
			or Skew:							
			skew corre	ection =	1.095					
			mg = 0.62	5 * 1.095 =	0.684					
		Equation								
			t. b/w CL web	to curb						
			- Rail Width							
			3ft - 1ft =	2.0	tt					
		e = 0.6	$+\left(\frac{d_e}{10}\right)$							
		e = 0.6	+ (2.0/10) =	0.800						
		g = e*g\	V _{int2+Eq}							
			0 * 0.891 =	0.713						
		Skew C	orrection is in	cluded in g	/(interior).					
		Range of App	olicability (RC	A) Checks	Interior	ROA is	implicitly	applied to t	he exterior b	eam.
		Check I	nterior Beam	ROA:	OK					
		Check of	d _e : -1.0' ≤ 2.0	0' ≤ 5.5'	OK					
		Check N	N _b : 6 ≠ 3		OK					
		Use Equation	from Table 4	4.6.2.2.3b-1	because all	criteria i	s OK.			
		$gV_{ext2+} =$	0.713							
	TXDOT	Policy states g	V _{Eidenor} must b	oe ≥ gV _{interior}						
		gV _{interior} =	0.891							
	TXDOT	Policy states g	V _{Exterior} must b	oe ≥ m·N _L ÷N	ь					
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6	ô =	0.425					
		S/2 ? Yes								
		20ft ? Yes	-111011 - 011	. GU 16	-14					
>>		Policy states th				a tha AA	a tilipattai i	de all al	ned	
	IXDOI	Policy states th m·N _L ÷N _b .	at II OH > 5/	z and vv < z	UIT, GV Exterior	s trie ivi	aximum c	or. gv _{ext1} , gv	interior, and	
	TYDOT	Policy states th	at if OH > 9/	2 ans W > 2	Off aV-	s the M	aximum o	of aV aV	αV	
	TADOT	and m·N _L ÷N _b		alia VV = Z	9 Y Exterior	o tile Mi	aannum C	y extly 9 v	ext2+> 9 v interior	6.1
1. 7	gV _{exte}									
	∠ y v exte	nor - 0.091								

TXDOT	County:	ANY	Highway:	Апу	Design:	BRG	Date:	8/15/20		LRFD Spece
BRIDGE	C-S-J: Descrip:	ITBC Design Exa	ID#:	XXXX	Ck Dsn:	Ev2 Sn	Date:	ution factors.x	Rev. 10/18 Sheet:	- (No Interim
	RIOR BE		nipie z, opan z		Jrile;	Exz op	anz_usuioi	JUOIT_IBCIOIS.X	Sneet.	2010
			/Table 4.6.	2 24 11						
Momen		ibution Per Lane ne Loaded	1 (1 aule 4.0.2	2.2.20-1).						
	One Lai	Lever Rule								
			25 * 1.0 =	0.625	TypoTue	20 2 001	ltiple pros	ennen Facto	of 1,0 for a	200
		Modify fo		0.025	lane loade				di Na idi c	III IE
		(Modify 10	skew corre	ection =	0.964					
			G11510-5510-	5 * 0.964 =						
		Use Lever Ru				2.2d-1.				
		gM _{ext1} =	0.603	ry-serie	A STATE OF THE STATE OF					
	Tue or	More Lanes Lo	-							
	I WO OF	Lever Rule	(Table 3.6.	1121						
			x(0.625 * 1.0	The second section is	85 0 625 * 0	65) -	0.625			
		Modify fo		, 0.025 0.0	35, 0.025 0.	00) -	0,025			
		wiodily it	skew corre	ection =	0.964					
			mg = 0.625		0.603					
		Equation			2.000					
			(d.)							
		e = 0.77	$7 + \left(\frac{d_e}{9.1}\right)$							
		e = 0.77	+ (2.0/9.1) =		0.990					
		g = e*gN	lust Fe							
			* 0.626 =	0.620						
			rrection inclu		nterior).					
		Range of App				ROA is	implicitly	applied to t	he exterior l	beam.
		and the second second	terior Beam		OK					
		Check d	: -1.0' ≤ 2.0	' ≤ 5.5'	OK					
		Check N	b: 6≠3		ОК					
		Use Equation	from Table 4	.6.2.2.2d-1	because all o	riteria is	OK.			
		gM _{ext2+} =	0.620							
	TXDOT	Policy states gM	Messas must b	ne≥aM						
	The o	gM _{interior} =	0.626	o - g.minigrior						
	TXDOT	Policy states gN	- Contraction	e ≥ m·N _i ÷N	li.					
			0.85 * 3 / 6		0.425					
	Is OH ≤	S/2 ? Yes			-					
	Is W ≥ 2	Oft ? Yes								
>>	TXDOT	Policy states that	at if OH ≤ S/2	, gM _{Exterior} is	gM _{interior}					
	TXDOT	Policy states that	at if OH > S/2	and W < 20	Oft, gM _{Exterior} i	s the M	aximum o	of: gM _{ext1} , gl	Minterior, and	
		$m \cdot N_L \div N_b$								
	TXDOT	Policy states that	at if OH > S/2	ans W ≥ 20	oft, gM _{Exterior} i	s the Ma	aximum c	of: gM _{ext1} , gl	Mext2+r gMmje	anorr
		and $m \cdot N_L + N_b$.								
	gM _{exte}	erior = 0.626								



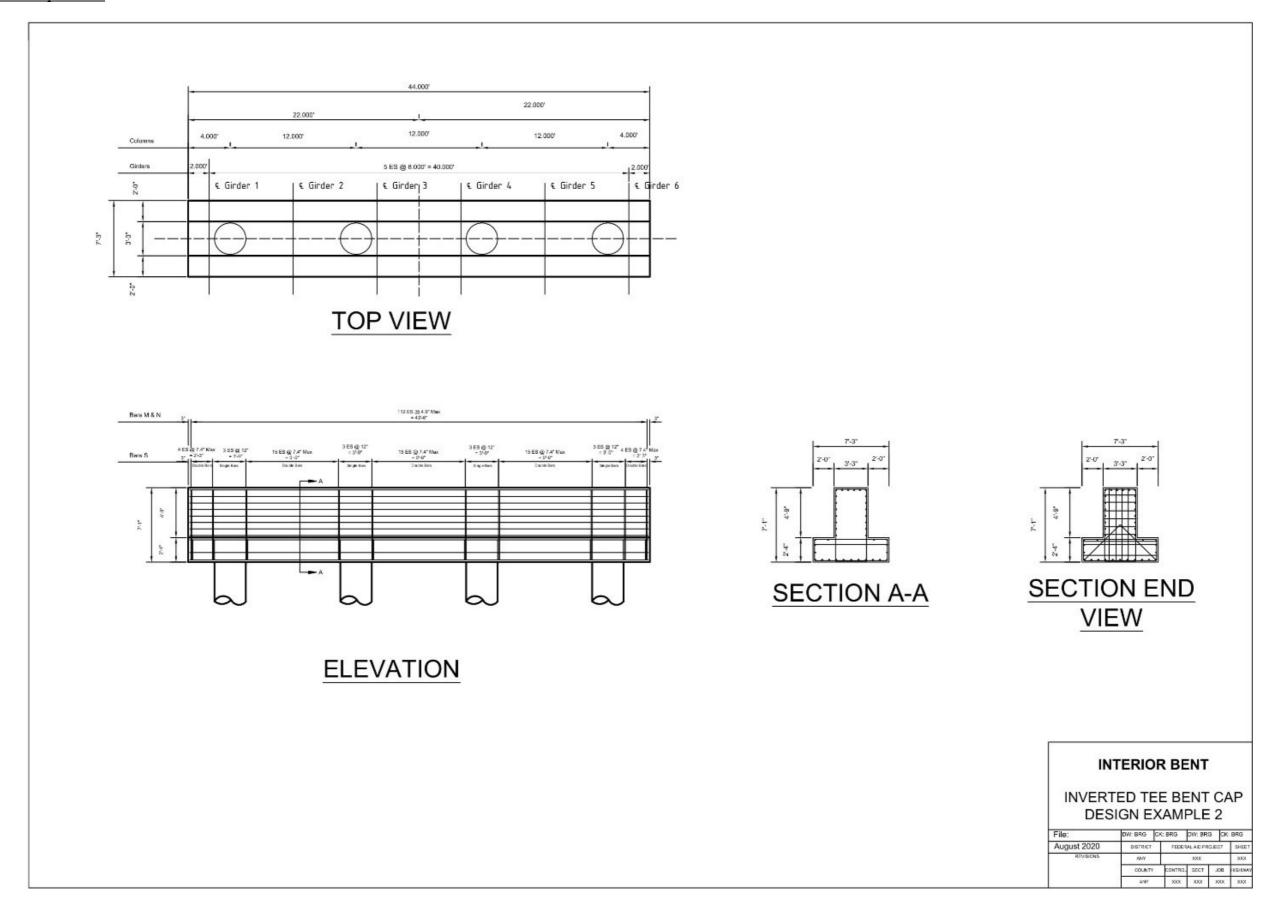
XDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Spe
RIDGE		ITBC Design Ex	ID#:	XXXX	Ck Dsn:	Eug Con	Date:	ion factors.xl	Rev. 10/18 - Sheet:	7 of 8
VISION	Descrip.	I TOO Dealgh Ext	imple 2, Opan 2		Trile,	LAE OPA	ie_ustribut	UII_IGUIUIS:AI	Sheet	7 01 0
	RULE		= 8.0 ft							
	OR (con't)									
For 18 s	s S < 22: ne Lane =	$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$					13	0.625		
Tw	o Lanes =	$\frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$)			0,5	0.750		
		5-7 6	$+\frac{S-4}{S}+\frac{S-10}{S}$				-	-0.125		
		$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S-12}{S}$	$\left(\frac{-18}{S} + \frac{S-16}{S}\right)$		1	0.625		
For 22 s	≤ S ≤ 24; ne Lane =	$\frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$					119	0.625		
Tw	o Lanes =	$\frac{16}{32}\bigg(1+\frac{S-6}{S}$	$+\frac{S-4}{S}+\frac{S-10}{S}$)				0.750		
Three	e Lanes =	$-\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S}{S}$	$\left(\frac{-18}{S}\right)$		110	-0.125		
Fou	r Lanes =	$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S}{}$	$\frac{-18}{S} + \frac{S - 16}{S}$	$+\frac{S-22}{S}$		-1.500		
		16k	4 h + 2 h		41-41-41-41-41-41-41-41-41-41-41-41-41-4				S=	8.0 f
	-	он —	_s	assumer	d hinge			Rail Widti X = S+OH-	OH = 1 = RW =	3.0 f 1.0 f 8.0 f
For X <	6: ne Lane =	$\frac{16}{32} \left(\frac{X}{S} \right)$					1.0	0.500		
For 6 ≤ Or	X < 12; ne Lane =	$\frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					0.625		
For 12 s	X < 18; ne Lane =	$\frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					0.625		
O		25/2 13								

RIDGE C-S-J:	ANY XXX-XX-XXXX	Highway: ID #:	XXXX	Design: Ck Dsn:	BRG	Date:	8/15/20	2017 LRFD Spec Rev. 10/18 - (No Interin
IVISION Descrip:	ITBC Design Exa			File:	Ex2 Span		ution factors.	
LEVER RULE								
EXTERIOR (con	n't) S RW			OH = H-RW-2ft =				
For 18 ≤ X < 24: One Lane	$= \frac{16}{32} \left(\frac{X}{S} \pm \frac{X - X}{S} \right)$	<u>6</u>)					= 0.625	
Two Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X - 12}{S} + \frac{X}{S}$	$\left(\frac{r-18}{S}\right)$				= -0.250	
For 24 ≤ X < 30: One Lane	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					= 0.625	
Two Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X - 12}{S} + \frac{X}{S}$	$\left(\frac{-18}{s}\right)$				= -0.250	
Three Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X - 12}{S} + \frac{X}{S}$	$\frac{X-18}{S} + \frac{X-2s}{S}$	1)			= -1.250	
For 30 ≤ X < 36: One Lane	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					= 0.625	
Two Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X - 12}{S} + \frac{X}{S}$	$\left(\frac{-18}{S}\right)$				= -0.250	
Three Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{s} + \frac{x-12}{s} + \frac{x}{s}$	$\frac{C-18}{S} + \frac{X-24}{S}$	$+\frac{X-30}{S}$			= -2.625	
For 36 ≤ X < 42: One Lane	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					= 0.625	
Two Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{s} + \frac{x - 12}{s} + \frac{x}{s}$	$\left(\frac{C-18}{S}\right)$				= -0.250	
Three Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X-12}{S} + \frac{X}{S}$	$\frac{C-18}{S} + \frac{X-24}{S}$	$+\frac{X-30}{S}$			= -2.625	
Four Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - S}{S} \right)$	$\frac{6}{S} + \frac{X - 12}{S} + \frac{X}{S}$	$\frac{C-18}{S} + \frac{X-24}{S}$	$\frac{4}{S} + \frac{X - 30}{S} + \frac{4}{S}$	$\left(\frac{X-36}{S}\right)$		= -4.375	
For 42 ≤ X ≤ 48: One Lane	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6					= 0.625	
Two Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X - 12}{S} + \frac{X}{S}$	$\left(\frac{C-18}{S}\right)$				= -0.250	
	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$						= -2.625	
Four Lanes	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{S} + \frac{X-12}{S} + \frac{\lambda}{S}$	$\frac{\ell-18}{S} + \frac{X-24}{S}$	$\frac{4}{S} + \frac{X - 30}{S} + \frac{3}{S}$	$\frac{X-36}{S} + \frac{X}{S}$	$\left(\frac{-42}{S}\right)$	= -6.500	
INTERIOR				EXTER	IOR			
One Lane Loade	ed	= 0.625		One La	ne Loade	d		0.625
Two Lanes Load	led	= 0.875		Two La	nes Loade	ed	(=	0.625
Three Lanes Loa	aded	= 0.875		Three L	anes Loa	ded		0.625
Four Lanes Load	ded	= 0.875		Four La	nes Load	ed	11.4	0.625

4.3.15.5 Concrete Section Shear Capacity Spreadsheet

	County: Highway:	ANY			Descrip:				
		XXXXXX			Design:	BRG C	k Dsn:	BRG	
Department of Transportation	Bridge I	Division	Re	ev: 09/26/08			Date:	Aug-20	
CONCRETE SECTION SHEA	R CAPA	CITY BY A	ASHTO L	RFD BRID	GE DESIG	N SPECIFIC	ATIONS, FO	URTH EDITI	ON, 200
Resistance Factors:			Units:	US					
l _V =	0.9		Olinta.	O.S					
om =	0.9								
The state of the s	0.75								
h _N =	0.75								
Concrete:			Mild Steel:			Prestressed	Steel:		
fc =[5	ksi	fy =	60	ksi	fpu =	270 k	si	
Ec =	4070	ksi	Es =	29000	ksi	Ep =	28500 k	si	
					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	50
Shear force, Vu	kip	237.2	243.5	133.6	452.1	234.9	251.6	137.6	422.
Axial force, Nu (+ if tensile)	kip	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	39.0
Shear depth, dv	in	80.53	80.53	80,53	80.53	80.53	80.53	80.53	80.5
Mild steel reinf. area, As	in^2	10.92	10.92	10.92	10.92	10.92	10.92	10.92	10.9
Conc area on tension side, Ac Area of stirrups, Av	in^2	1657.5	1657.5	1657.5	1657.5	1657.5	1657.5	1657.5	1657.
Stirrup spacing, s	in^2 in	1.76 7.4	1.76 7.4	1.76 7.4	1.76 7.4	1.76 7.4	1.76 7.4	1.76 7.4	1.7
Prestressed steel area, Aps	in^2	0	0	0	0	0	0	0	- 7,
Prestress shear, Vp	kip	0	0	0	0	0	0	0	
Average prestress, fps	ksi	0	0	0	0	0	0	0	
Torsional moment, Tu	kip-ft	706	353	353	706	706	353	353	70
Shear flow area, Ao	in^2	2971.6	2971.6	2971.6	2971.6	2971.6	2971.6	2971.6	2971.
Area of one leg of stirrup, At	in^2	0.44	0.44	0.44	0.44	0.44	0.44	0.44	0.4
Perimeter of stirrup, Ph	in	324	324	324	324	324	324	324	32
Calculated Values									
Vc	kip	543.7	541.5	605.8	494.9	543.7	537.0	601.4	494.
Vs	kip	1669.4	1719.4	2009.4	1431.2	1669.4	1699.1	2000.9	1431.
φVn	kip	1992	2035	2354	1733	1992	2013	2342	173
Ex	V.0	6.67E-04	6.83E-04	4.01E-04	1.00E-03	6.61E-04	7.02E-04	4.12E-04	1.00E-0
θ	deg	32.60	32.80	29.00	36.40	32.60	33.10	29.10	36.4
β Reg'd Shear reinf, Av/S	in^2/in	2.450 0.000	2.440 0.000	2.730 0.000	0.001	0.000	0.000	0.000	2.23
Reg'd Torsion reinf, At/S	in^2/in	0.000	0.000	0.000	0.001	0.000	0.000	0.000	0.00
Maximum stirrup spacing, Smax	in 2/in	24.0	24.0	24.0	22.3	24.0	24.0	24.0	22.
Conclusion									
Shear Re	inforcing	OK	OK	OK	OK	OK	OK	OK	OK
Longitudinal Reinforcing		OK	ОК	OK	OK	ОК	OK	OK	OK

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

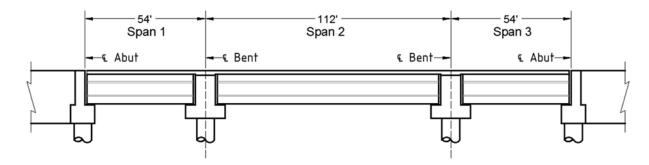


Figure 4.53 Spans of the Bridge with 45 Degrees Skewed ITBC

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

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

 $f_c' = 5 \text{ ksi}$

 $w_c = 150 \text{ pcf (for weight)}$

"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)

 $w_c = 145 \text{ pcf (for Modulus of Elasticity calculation)}$

Grade 60 Reinforcing

 $f_v = 60 \text{ ksi}$

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

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 = 0.372 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		
$f_c = 5ksi$		Concrete Strength
$w_{cE} = 0.145 \text{kcf}$		Unit Weight of Concrete for E_c
$W_{cE} = 0.145 \text{ KeV}$ $E_{c} = 33000 \cdot W_{cE}^{1.5} \cdot $	$\overline{f_c}$ $E_c = 4074 \text{ ksi}$	Modulus of Elasticity of Concrete (AASHTO LRFD Eq. C5.4.2.4-2)
$f_y = 60$ ksi		Yield Strength of Reinforcement
$\mathrm{E_{s}=29000ksi}$		Modulus of Elasticity of Steel
$D_{column} = 36in$		Diameter of Columns

Other Variables

IM = 33%

Dynamic Load Allowance (AASHTO LRFD Table 3.6.2.1-1)

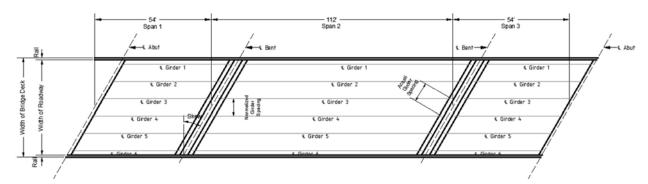


Figure 4.54Top View of the 45 Degrees Skewed ITBC with Spans and Girders

4.4.2 **Determine Cap Dimensions**

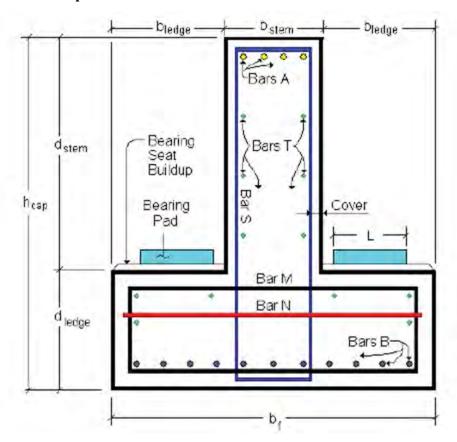


Figure 4.55 Section View of 45 Degrees Skewed ITBC

4.4.2.1 Stem Width

 $b_{stem} = at least D_{column} + 3in$

Use: $b_{stem} = 42 in$

4.4.2.2 Stem Height

Distance from Top of Slab to Top of Ledge:

 $D_{Slab to Ledge} = SlabThk + Haunch2 + GirderD + BrgPa$

 $D_{Slab to Ledge} = 70.00 in$

StemHaunch = 3.75 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)

Haunch2 is the larger of the two haunches.

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.

$$d_{stem} = D_{Slab_to_Ledge} - SlabThk - StemHaunch - 0.5in$$

$$d_{stem} = 57.75 in$$

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

4.4.2.3 Ledge Width

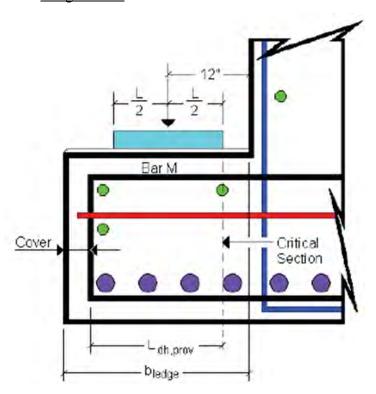


Figure 4.56 Ledge Section of 45 Degrees ITBC

cover = 2.5 in

L = 9 in

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"?

The stem must accommodate ½" of bituminous fiber.

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

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

" $L_{dh,prov}$ " 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 9" × 21" for 45° skewed beents, as shown in the IGEB standard.

SideCover = cover
$$-\frac{d_{bar_M}}{2}$$
 = 2.06 in

$$TopCover = cover - \frac{d_{bar_M}}{2} = 2.06 in$$

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 dimension "cover" is

M.

(AASHTO LRFD 5.10.8.2.4a)

"Side Cover" and "Top Cover" are the clear cover on the side

and top of the hook respectively.

measured from the center of Bar

The Required Development Length:

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

Therefore,

$$L_{dh_req} = 14.87 \text{ in}$$

$$b_{ledge_min} = L_{dh_req} + cover + 12in - \frac{L}{2} \qquad \quad b_{ledge_min} = 24.87 \ in$$

Use:

$$b_{ledge} = 25 in$$

Width of Bottom Flange:

$$b_f = 2 \cdot b_{ledge} + b_{stem}$$

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

4.4.2.4 Ledge Depth

Use a Ledge Depth of 28".

$$d_{ledge} = 28 in$$

Total Depth of Cap:

$$h_{cap} = d_{stem} + d_{ledge}$$

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.

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

 $b_f = 92 in$

4.4.2.5 Summary of Cross Sectional Dimensions

$$b_{stem} = 42 in$$

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

$$b_{ledge} = 25 in$$

$$d_{ledge} = 28 in$$

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

4.4.2.6 Length of Cap

First define Girder Spacing and End Distance:

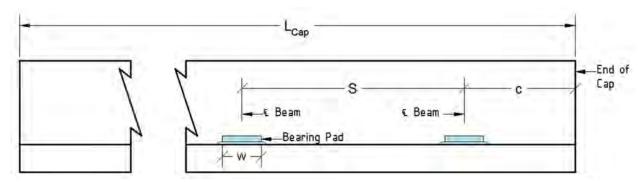


Figure 4.57 Elevation View of 45 Degrees Skewed ITBC

$$S=8 \text{ ft} \\ c=2 \text{ ft} \\ \text{"c" is the distance from the Center} \\ \text{Line of the Exterior Girder to the} \\ \text{Edge of the Cap measured along} \\ \text{the Cap.} \\ L_{Cap}=S\cdot (\text{GdrNo1}-1)+2c \\ L_{Cap}=44 \text{ ft} \\ \text{Length of Cap} \\ \text{Length of C$$

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 in W = 21 in Width of Bearing Pad Width of Bearing Pad

4.4.3 Cross Sectional Properties of Cap

$$\begin{split} A_g &= d_{ledge} \cdot b_f + d_{stem} \cdot b_{stem} & A_g = 4970 \text{ in}^2 \\ ybar &= \frac{d_{ledge} \cdot b_f \cdot \left(\frac{1}{2} d_{ledge}\right) + d_{stem} \cdot b_{stem} \cdot \left(d_{ledge} + \frac{1}{2} d_{stem}\right)}{A_g} & ybar = 34.5 \text{ in} & \textit{Distance from bottom of the cap to the center of gravity of the cap} \\ I_g &= \frac{b_f \cdot d_{ledge}^3}{12} + b_f \cdot d_{ledge} \cdot \left(ybar - \frac{1}{2} d_{ledge}\right)^2 + \frac{b_{stem} \cdot d_{stem}^3}{12} + \cdots \\ b_{stem} \cdot d_{stem} \cdot \left[ybar - \left(d_{ledge} + \frac{1}{2} d_{stem}\right)\right]^2 & I_g = 3.06 \times 10^6 \text{ in}^4 \end{split}$$

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.

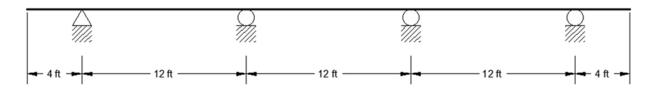


Figure 4.58 Continuous Beam Model for 45 Degrees Skewed ITBC

TxDOT does not consider frame action for typical multi-column bents (BDM-LRFD, Ch. 4, Sect. 5, Structural Analysis).

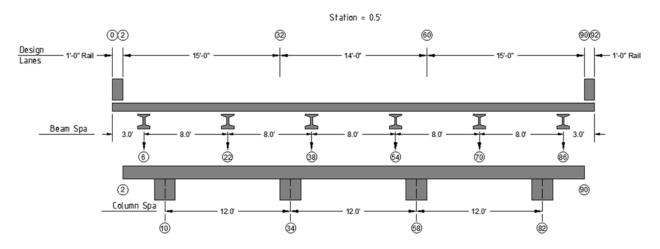


Figure 4.59 Cap 18 Model of 45 Degrees Skewed ITBC

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.

$$station = 0.5 ft$$

Station increment for CAP 18

Recall:

$$E_c = 4074 \; ksi \qquad \qquad I_g = 3.06 \times 10^6 \; in^4$$

$$E_c I_g = 1.25 \times 10^{10} \; \mathrm{kip} \cdot \mathrm{in}^2 / \; \left(12 \frac{\mathrm{in}}{\mathrm{ft}}\right)^2 \; E_c I_g = 8.66 \times 10^7 \mathrm{kip} \cdot \mathrm{ft}^2$$

4.4.4.1.1 *Dead Load*

SPAN 1

Values used in the following equations can be found on "4.4.1 Design Parameters"

$$Rail1 = \frac{2 \cdot RailWt \cdot \frac{Span1}{2}}{\min(GdrNo1,6)}$$

$$Rail1 = 3.44 \frac{kip}{girder}$$

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

$$Slab1 = w_c \cdot GdrSpa1 \cdot SlabThk \cdot \frac{Span1}{2} \cdot 1.10$$

$$Slab1 = 23.76 \frac{kip}{girder}$$

Slab1 = $23.76 \frac{\text{kip}}{\text{girder}}$ Increase slab DL by 10% to account for haunch and thickened slab ends.

$$Girder1 = GdrWt1 \cdot \frac{Span1}{2}$$

$$Girder1 = 22.98 \frac{kip}{girder}$$

$$DLRxn1 = (Rail1 + Slab1 + Girder1)$$

$$DLRxn1 = 50.17 \frac{kip}{girder}$$

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

$$Overlay1 = w_{Olay} \cdot GdrSpa1 \cdot OverlayThk \cdot \frac{Span1}{2}$$

Overlay1 =
$$5.04 \frac{\text{kip}}{\text{girder}}$$

Design for future overlay.

SPAN 2

$$Rail2 = \frac{2 \cdot RailWt \cdot \frac{Span2}{2}}{min(GdrNo2,6)}$$

Rail2 =
$$7.13 \frac{\text{kip}}{\text{girder}}$$

$$Slab2 = w_c \cdot GdrSpa2 \cdot SlabThk \cdot \frac{Span2}{2} \cdot 1.10$$

$$Slab2 = 49.28 \frac{kip}{girder}$$

$$Girder2 = GdrWt1 \cdot \frac{Span2}{2}$$

$$Girder2 = 47.66 \frac{kip}{girder}$$

$$DLRxn2 = (Rail2 + Slab2 + Girder2)$$

$$DLRxn2 = 104.07 \frac{kip}{girder}$$

$$Overlay2 = w_{Olay} \cdot GdrSpa2 \cdot OverlayThk \cdot \frac{Span2}{2}$$

Overlay2 =
$$10.45 \frac{\text{kip}}{\text{girder}}$$

CAP

Cap =
$$w_c \cdot A_g = 5.177 \frac{kip}{ft} \cdot \frac{0.5ft}{station}$$

$$Cap = 2.589 \frac{kip}{station}$$

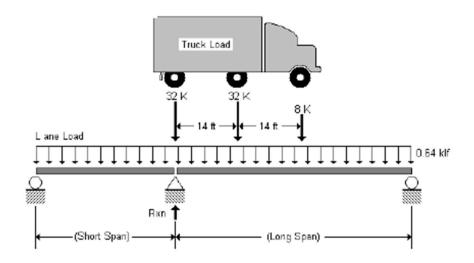


Figure 4.60 Live Load Model of 45 Degrees Skewed ITBC

LongSpan = max(Span1, Span2)

LongSpan = 112 ft

ShortSpan = min(Span1, Span2)

ShortSpan = 54 ft

IM = 0.33

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

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

$$Truck = 32kip + 32kip \cdot \left(\frac{LongSpan - 14ft}{LongSpan}\right) + 8kip \cdot \left(\frac{LongSpan - 28ft}{LongSpan}\right)$$

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

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

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

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.

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)

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

$$P = 21.28 \text{ kip}$$

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

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

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

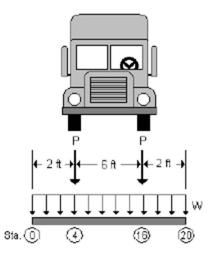


Figure 4.61 Live Load Model of 45 Degrees Skewed ITBC for CAP18

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)

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

Factors".

4.4.4.1.3 Cap 18 Data Input

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

<u>Limit States</u> (AASHTO LRFD 3.4.1)

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

Service I

Live Load and Dynamic Load Allowance LL+IM = 1.00

Dead Load and Wearing Surface DC & DW = 1.00

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

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.

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

	$\underline{\mathbf{Max} + \mathbf{M}}$	Max -M
Dead Load:	$M_{posDL} = 379.0 \text{ kip} \cdot \text{ft}$	$M_{\text{negDL}} = -563.1 \text{kip} \cdot \text{ft}$
Service Load:	$M_{posServ} = 721.8 kip \cdot ft$	$M_{negServ} = -862.2 kip \cdot ft$
Factored Load:	$M_{posUlt} = 1080.5 \text{ kip} \cdot \text{ft}$	$M_{\text{negUlt}} = -1238.4 \text{ kip} \cdot \text{ft}$

4.4.4.2 Girder Reactions on Ledge

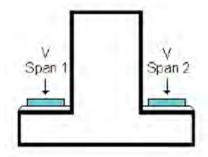


Figure 4.62 Girder Reactions on the Ledge of 45 Degrees Skewed ITBC

Dead Load

$$DLSpan1 = Rail1 + Slab1 + Girder1$$

$$DLSpan1 = 50.17 \frac{kip}{girder}$$

Overlay1 =
$$5.04 \frac{\text{kip}}{\text{girder}}$$

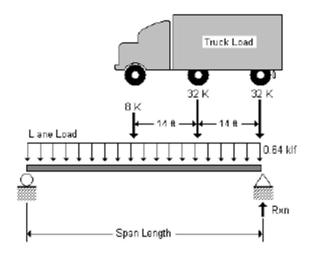
$$DLSpan2 = Rail2 + Slab2 + Girder2$$

$$DLSpan2 = 104.07 \frac{kip}{girder}$$

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

$$LaneSpan1 = 0.64klf \cdot \left(\frac{Span1}{2}\right)$$

$$LaneSpan1 = 17.28 \frac{kip}{lane}$$

$$LaneSpan2 = 0.64klf \cdot \left(\frac{Span2}{2}\right)$$

$$LaneSpan2 = 35.84 \frac{kip}{lane}$$

$$\begin{split} TruckSpan1 &= 32kip + 32kip \cdot \left(\frac{Span1 - 14ft}{Span1}\right) + 8kip \cdot \left(\frac{Span1 - 28ft}{Span1}\right) \\ TruckSpan1 &= 59.56\frac{kip}{lane} \end{split}$$

$$\begin{split} TruckSpan2 &= 32kip + 32kip \cdot \left(\frac{Span2 - 14ft}{Span2}\right) + 8kip \cdot \left(\frac{Span2 - 28ft}{Span2}\right) \\ TruckSpan2 &= 66.00 \frac{kip}{lane} \end{split}$$

$$IM = 0.33$$

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

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

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

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

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

LLSpan1Int =
$$gV_{Span1_Int} \cdot LLRxnSpan1$$
 LLSpan1Int = $88.87 \frac{kip}{girder}$

$$LLSpan1Ext = gV_{Span1_Ext} \cdot LLRxnSpan1 \qquad LLSpan1Ext = 88.87 \frac{kip}{girder}$$

$$LLSpan2Int = gV_{Span2_Int} \cdot LLRxnSpan2 \qquad LLSpan2Int = 117.07 \frac{kip}{girder}$$

LLSpan2Ext =
$$gV_{Span2_Ext} \cdot LLRxnSpan2$$
 LLSpan2Ext = $117.07 \frac{kip}{girder}$

Span 1

Interior Girder

$$V_{s Span1Int} = DLSpan1 + Overlay1 + LLSpan1Int$$

$$V_{s \text{ Span1Int}} = 144 \text{ kip}$$

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)

$$V_{u \text{ Span1Ext}} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 + 1.75 \cdot LLSpan1Ext$$

$$V_{u \text{ Span}_{1}\text{Ext}} = 226 \text{ kip}$$

Span 2

Interior Girder

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

$$V_{s \text{ Span2Int}} = DLSpan2 + Overlay2 + LLSpan2Int$$

$$V_{s_Span2Int} = 232 \text{ kip}$$

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

$$V_{u_Span2Int} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot LLSpan2Int$$

$$V_{u \text{ Snan2Int}} = 351 \text{ kip}$$

Exterior Girder

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

$$V_{s_Span2Ext} = DLSpan2 + Overlay2 + LLSpan2Ext$$

$$V_{s \text{ Span}2\text{Ext}} = 232 \text{ kip}$$

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

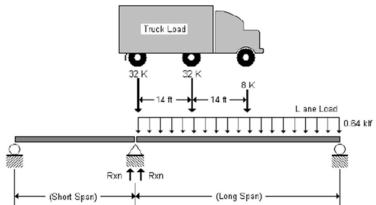
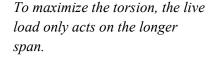


Figure 4.64 Live Load Model of 45 Degrees Skewed ITBC for

Torsional Loads



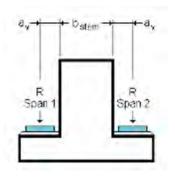


Figure 4.65. Loads on the Ledge of 45 Degrees Skewed ITBC for Torsion

$$a_v = 12 \text{ in}$$

" 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 =
$$a_v + \frac{1}{2}b_{stem}$$

$$LeverArm = 33 in$$

Interior Girders

Girder Reactions

$$\begin{split} R_{u_Span1} &= 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 \\ R_{u_Span1} &= 70 \text{ kip} \\ \\ R_{u_Span2} &= 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2_Int} \\ & \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)] \end{split}$$

$$R_{u_Span2} = 351 \, \text{kip}$$

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

$$\begin{split} R_{u_Span1} &= 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 \\ R_{u_Span1} &= 70 \text{ kip} \\ R_{u_Span2} &= 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2_Ext} \\ & \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)] \\ R_{u_Span2} &= 351 \text{ kip} \end{split}$$

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

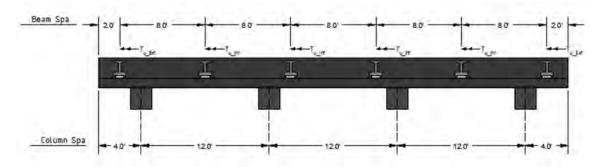


Figure 4.66 Elevation View of 45 Degrees Skewed ITBC with Torsion Loads

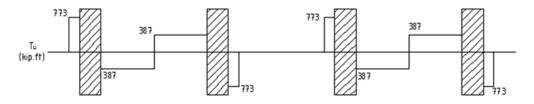


Figure 4.67 Torsion Diagram of 45 Degrees Skewed ITBC

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

$$T_u = 773 \text{ kip} \cdot \text{ft}$$
 Maximum Torsion on Cap

4.4.4.4 <u>Load Summary</u>

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

$$V_{u \text{ Int}} = \max(V_{u \text{ Span1Int}}, V_{u \text{ Span2Int}})$$
 $V_{u \text{ Int}} = 350.64 \text{ kip}$

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)

 $T_u = 773 \text{ kip} \cdot \text{ft}$ Located two stations away from

centerline of column.

 $V_{\rm u}=462.8~{
m kip}$ $V_{\rm u}$ and $M_{\rm u}$ values are from

 $M_u = 504.8 \text{ kip} \cdot \text{ft}$ CAP18

4.4.5 Locate and Describe Reinforcing

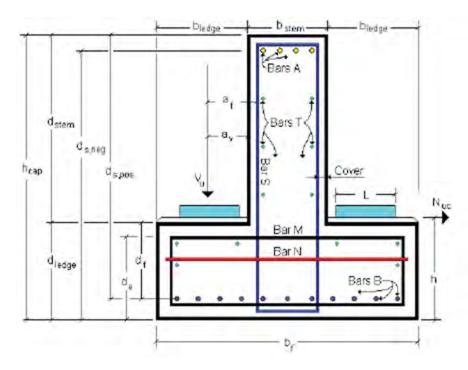


Figure 4.68 Section View of 45 Degrees Skewed ITBC

Recall:

 $b_{stem} = 42 in$

 $d_{stem} = 57 in$

 $b_{ledge} = 25 in$

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

 $b_f = 92 \text{ in}$

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

cover = 2.5 in

4.4.5.1 <u>Describe Reinforcing Bars</u>

Use # 11 bars for Bar A

$$A_{bar_A} = 1.56 \text{ in}^2$$
 $d_{bar_A} = 1.410 \text{ in}$

Use # 11 bars for Bar B

$$A_{bar_B} = 1.56 \ in^2 \qquad \qquad d_{bar_B} = 1.410 \ in$$

Use # 7 bars for Bar M

$$A_{bar_M} = 0.60 \text{ in}^2$$
 $d_{bar_M} = 0.875 \text{in}$ Bar M was considered. Bar M must be # 7 or smaller to allow it

In the calculation of b_{ledge} , # 7

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

fully develop.

M.

Use # 7 bars for Bar N

$$A_{bar \ N} = 0.60 \ in^2$$
 $d_{bar \ N} = 0.875 \ in$

 $A_{bar_N} = 0.875 \,\text{II}$

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

Use # 6 bars for Bar T

Use # 6 bars for Bar S

$$A_{bar_T} = 0.44 \ in^2 \qquad \qquad d_{bar_T} = 0.75 \ in$$

4.4.5.2 <u>Calculate Dimensions</u>

$$d_{s_neg} = h_{cap} - cover - \frac{1}{2}d_{bar_s} - \frac{1}{2}d_{bar_A}$$
 $d_{s_neg} = 81.42 in$

$$d_{s_pos} = h_{cap} - cover - \frac{1}{2} max(d_{bar_S}, d_{bar_M}) \ - \frac{1}{2} d_{bar_B} \qquad \quad d_{s_pos} = 81.36 \ in$$

$$a_v = 12 in$$

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

$$d_e = d_{ledge} - cover$$
 $d_e = 25.50 in$

$$d_f = d_{ledge} - cover - \frac{1}{2} d_{bar_M} - \frac{1}{2} d_{bar_B} \qquad \qquad d_f = 24.36 \text{ in}$$

$$h = d_{ledge} + BrgSeat$$
 $h = 29.50 in$

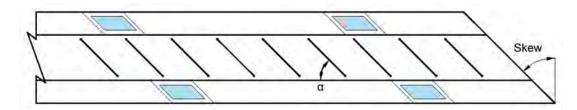


Figure 4.69 Plan View of 45 Degrees Skewed ITBC

$$\alpha = 45 \deg$$

Recall:

L = 9 in

W = 21 in

Angle of Bars S (Angle from the horizontal)

Dimension of Bearing Pad

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.

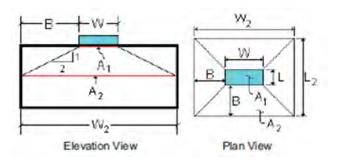


Figure 4.70 Bearing Check for 45 Degrees Skew Angle

Resistance Factor (ϕ) = 0.7

 $A_1 = L \cdot W$

 $A_1 = 189 \text{ in}^2$

(AASHTO LRFD 5.5.4.2)

Area under Bearing Pad

Interior Girders

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

B = 8.5 in.

$$L_2 = L + 2 \cdot B$$

$$W_2 = W + 2 \cdot B$$

$$A_2 = L_2 \cdot W_2$$

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

 $L_2 = 26.00 \text{ in}$

 $W_2 = 38.00 \text{ in}$

 $A_2 = 988 \text{ in}^2$

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

$$\phi V_n = \phi \quad 0.85 \quad f_c \quad A_1 \quad m$$

$$\phi V_{\rm n} = 1124.55 \, \text{kips}$$

AASHTO LRFD Eqs. 5.6.5-1 and 5.6.5-2.

$$V_{u \text{ Int}} = 350.64 < \phi V_{n}$$

 V_{u_int} from "4.4.4.4 Load Summary".

Exterior Girders

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

"B" is the distance from B= 8.5 in. perimeter of A_1 to the perimeter of A2 as seen in the above figure

$$L_2 = L + 2 B$$

$$W_2 = W + 2 B$$

$$A_2 = L_2 W_2$$

$$L_2 = 26.00 \text{ in}$$

$$W_2 = 38.00 \text{ in}$$

$$A_2 = 988 \text{ in}^2$$

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

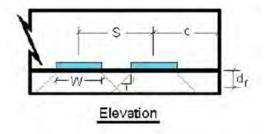
$$\varphi V_n = \varphi \ 0.85 \ f_c \ A_1 \ m$$

$$\phi V_n = 1124.55 \text{ kips}$$

$$V_{u_ext} = 350.64 \, kips < \Phi V_n$$

$$V_{u_ext}$$
 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 $(\phi) = 0.90$

AASHTO LRFD 5.5.4.2.

Determine if the Shear Cones Intersect

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

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

$$d_f = 24.36 \text{ in}$$

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

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

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

$$\frac{1}{2}b_{stem} + a_v - \frac{1}{2}L = 28.5 \text{ in}$$

 $d_f = 24.36 \text{ in}$

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

Interior Girders

$$V_n = 0.125 \ \text{?} \ \lambda \sqrt{f_c'} \ b_o \ d_f$$

$$V_n = 597.27 \,\mathrm{kips}$$

$$b_o = W + 2L + 2d_f$$

$$b_o = 87.72 \text{ in}$$

$$\varphi V_n = \ 537.54 \ kips$$

$$V_{u~Int} = 350.64 \, \text{kips} < \varphi V_n$$

$$V_{u_int}$$
 from "4.4.4.4 Load Summary".

Exterior Girders

$$V_{n} = \min[(0.125 \cdot \sqrt{f_{c}} \cdot (\frac{1}{2}W + L + d_{f} + c) * d_{f}, 0.125 \cdot \sqrt{f_{c}} \cdot (W + 2L + 2d_{f}) * d_{f})]$$

$$V_{\rm n} = 462.04 \, {\rm kips}$$
 AASHTO LRFD 5.8.4.3.4-3 and 5.8.4.3.4-5

$$\phi V_n = 415.84 \text{ kips}$$

$$V_{u_ext}~=~350.64~kips~<~\varphi V_n$$

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)

$$b_{s Int} = 69 in$$

$$A_{cv} = b_{s_Int} \cdot d_e \qquad \qquad A_{cv} = 1759.5 \text{ in 2}$$

Exterior Girders

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

$$=48 \text{ in}$$

$$A_{cv} = b_{s_ext} \cdot d_e \qquad \qquad A_{cv} = 1224 \text{ in 2}$$

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

$$\phi V_n = 1267 \text{ kips}$$

$$V_{u_Int} = 350.64 \text{ kips} < \phi V_n$$
 ShearFrictionChk= "OK!" V_{u_int} from "4.4.4.4 Load Summary".

Exterior Girders

$$V_{\rm n} = \min(0.2 \cdot f_{\rm c} \cdot A_{\rm cv}, 0.8 \cdot A_{\rm cv})$$
 $V_{\rm n} = 979.2 \ {\rm kips}$ AASHTO LRFD 5.8.4.2.2-1 and 5.8.4.2.2-2

$$\phi V_n = 881 \, \text{kips}$$

$$V_{u_{\text{ext}}} = 350.64 \text{ kips} < \phi V_{\text{n}}$$
 ShearFrictionChk= "OK!" $V_{u_{\text{ext}}}$ from "4.4.4.4 Load Summary".

4.4.9 Flexural Reinforcement for Negative Bending (Bars A)

$$\begin{split} M_{dl} &= \left| M_{negDL} \right| & M_{dl} = 563.1 \text{ kip} \cdot \text{ft} \\ M_{s} &= \left| M_{negServ} \right| & M_{s} = 862.2 \text{ kip} \cdot \text{ft} \\ M_{u} &= \left| M_{negUlt} \right| & M_{u} = 1238.4 \text{ kip} \cdot \text{ft} \end{split}$$

4.4.9.1 <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).

$I_g = 3.06 \times 10^6 \text{ in}^4$		Gross Moment of Inertia
$h_{cap} = 85 \text{ in}$		Depth of Cap
ybar = 34.5 in		Distance to the Center of Gravity of the Cap from the bottom of the Cap
$f_{\rm r} = 0.24\sqrt{f_{\rm c}}$	$f_r = 0.537 \text{ ksi}$	Modulus of Rupture (BDM- LRFD, Ch. 4, Sect. 5, Design Criteria)
$y_t = h_{cap} - ybar$	$y_t = 50.50 \text{ in}$	Distance from Center of Gravity to extreme tension fiber
$S = \frac{I_g}{y_t}$	$S = 6.06 \times 10^4 \text{ in}^3$	Section Modulus for the extreme tension fiber
$M_{cr} = S \cdot f_r \cdot \frac{_{1ft}}{_{12in}}$	$M_{cr} = 2711.8 \mathrm{kip} \cdot \mathrm{ft}$	Cracking Moment (AASHTO LRFD Eq. 5.6.3.3-1)
M_f = minimum of: $1.2 M_{cr} = 3254.2 \text{ kip} \cdot \text{ft}$ $1.33 M_u = 1647.1 \text{ kip} \cdot \text{ft}$		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 $M_{f}=1647.1\:\text{kip}\cdot\text{ft}$

4.4.9.2 Moment Capacity Design

Try,
$$7 \sim #11$$
's Top

BarANo = 7

 $d_{bar A} = 1.410 in$

 $A_{bar A} = 1.56 in^2$

 $A_s = BarANo \cdot A_{bar A}$

 $d_{stirrup} = d_{bar S}$

 $d = d_{s \text{ neg}}$

 $b = b_f$

 $f_c = 5.0 \text{ ksi}$

 $f_v = 60 \text{ ksi}$

 $\beta_1 = 0.85 - 0.05(f_c - 4ksi)$

Bounded by: $0.65 \le \beta_1 \le 0.85$

 $c = \frac{A_s f_y}{0.85 f_c \beta_1 b}$ c = 2.09 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.

$$a = c \cdot \beta_1$$

$$a = 1.67 in$$

 $A_s = 10.92 \text{ in}^2$

d = 81.42 in

b = 92 in

 $\beta_1 = 0.80$

 $d_{\text{stirrup}} = 0.75 \text{ in}$

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

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) \cdot \frac{1ft}{12in}$$

$$\epsilon_s = 0.003 \cdot \tfrac{d-c}{c}$$

$$\varepsilon_{\rm s} > 0.005$$

FlexureBehavior = "Tension Controlled"

 $\Phi_{\text{M}} = 0.90$

 $M_r = \Phi_M M_n$

 $M_f = 1647.1 \text{ kip} \cdot \text{ft} < M_r$

 $M_{\rm u} = 1238.4 \, \text{kip} \cdot \text{ft} < M_{\rm r}$ UltimateMom = "OK!"

 $\varepsilon_s = 0.114$

 $M_r = 3960 \text{ kip} \cdot \text{ft}$

 $M_n = 4400 \text{ kip} \cdot \text{ft}$

MinReinfChk = "OK!"

Number of bars in tension

Diameter of main reinforcing

Area of main reinforcing bars

Area of steel in tension

Diameter of shear reinforcing

bars

Compressive Strength of Concrete

Yield Strength of Rebar

(AASHTO LRFD 5.6.2.2)

Depth of Cross Section under Compression under Ultimate Load (AASHTO LRFD Eq. 5.6.3.1.2-4)

Depth of Equivalent Stress Block (AASHTO LRFD 5.6.2.2)

Nominal Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.2-1)

Strain in Reinforcing at Ultimate

(AASHTO LRFD 5.6.2.1)

(AASHTO LRFD 5.5.4.2)

Factored Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.1-1)

4.4.9.3 Check Serviceability

To find s_{max} :

Modular Ratio:

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

Tension Reinforcement Ratio:

$$\rho = \frac{A_s}{b \cdot d} \qquad \qquad \rho = 0.00146$$

$$k=\sqrt{(2\rho n)+(\rho n)^2}-(\rho n) \qquad \qquad k=0.134$$

$$d \cdot k = 10.91 \; \text{in} \; < \; d_{ledge} = 28 \; \text{in}$$

Therefore, the compression force acts over a rectangular

area.
$$j = 1 - \frac{k}{3}$$
 $j = 0.955$

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

$$f_{ss} = 12.2 \text{ ksi}$$

$$f_a = 0.6f_v$$
 $f_a = 36.00 \text{ ksi}$

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

$$d_c = cover + \frac{1}{2}d_{stirrup} + \frac{1}{2}d_{bar_A}$$
 $d_c = 3.58 in$

Exposure Condition Factor:

$$\gamma_e = 1.00$$

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

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

$$s_{Actual} = \frac{b_{stem} - 2d_c}{Bar A No - 1}$$
 $s_{Actual} = 5.81 \text{ in}$

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

For service loads, the stress on the cross-section is located as shown in Figure 4.72.

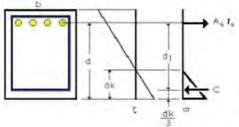


Figure 4.72 Stresses on the Cross Section for Service Loads of 45 Degrees Skewed ITBC

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

Service Load Bending Stress in outer layer of the reinforcing.

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

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)

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

4.4.9.4 Check Dead Load

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

$$M_a = A_s \cdot d \cdot j \cdot f_{dl} \cdot \frac{1ft}{12in}$$
 $M_a = 1556.7 \text{ kip} \cdot ft$ $M_{dl} = 563.1 \text{ kip} \cdot ft < M_a$ DeadLoadMom = "OK!"

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

Allowable Dead Load Moment

4.4.10 Flexural Reinforcement for Positive Bending (Bars B)

$$\begin{split} M_{dl} &= M_{posDL} & M_{dl} &= 379.0 \text{ kip} \cdot \text{ft} \\ M_{s} &= M_{posServ} & M_{s} &= 721.8 \text{ kip} \cdot \text{ft} \\ M_{u} &= M_{posUlt} & M_{u} &= 1080.5 \text{ kip} \cdot \text{ft} \end{split}$$

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	
$y_t = ybar$	$y_t = 34.5 \text{ in}$	Distance to the Center of Gravity of the Cap from the top of the Cap	
$f_{\rm r} = 0.24\sqrt{f_{\rm c}}$	$f_r = 0.537 \text{ ksi}$	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	
$M_{cr} = S \cdot f_r \cdot \frac{_{1ft}}{_{12in}}$	$M_{cr} = 3969.3 \text{ kip} \cdot \text{ft}$	Cracking Moment (AASHTO LRFD Eq. 5.6.3.3-1)	
$M_f = minimum of:$		Design the lesser of 1.2M _{cr} or	
$1.2M_{\rm cr} = 4763.2 \text{ kip} \cdot \text{ft}$		$1.33M_u$ when determining	
$1.33M_u = 1437.1 \text{ kip} \cdot \text{ft}$		mininum area of steel required.	

Thus, M_{r} must be greater than $M_{f}=1437.1\ kip\cdot ft$

4.4.10.2 Moment Capacity Design

Try,
$$11 \sim #11$$
's Bottom

BarBNo = 11

 $d_{\text{bar B}} = 1.41 \text{ in}$

 $A_{bar B} = 1.56 in^2$

 $A_s = BarBNo \cdot A_{bar_B}$

 $d = d_{s_pos}$

 $b = b_{stem}$

 $f_c = 5.0 \text{ ksi}$

 $f_v = 60 \text{ ksi}$

 $\beta_1 = 0.85 - 0.05(f_c - 4ksi)$

Bounded by: $0.65 \le \beta_1 \le 0.85$

 $c = \frac{A_s f_y}{0.85 f_c \beta_1 b}$

Number of bars in tension

Diameter of main reinforcing

bars

Area of main reinforcing bars

Area of steel in tension

Compressive Strength of Concrete

Yield Strength of Rebar

(AASHTO LRFD 5.6.2.2)

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.

$$a = c \cdot \beta_1$$

$$a = 5.77 in$$

 $A_s = 17.16 \text{ in}^2$

d = 81.36 in

b = 42 in

 $\beta_1 = 0.80$

c = 7.21 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.

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) \cdot \frac{1ft}{12in}$$

$$M_n = 6733.2 \, \text{kip} \cdot \text{ft}$$

$$\epsilon_s = 0.003 \cdot \frac{d-c}{c}$$

$$\varepsilon_{\rm s} = 0.031$$

Depth of Equivalent Stress Block (AASHTO LRFD 5.6.2.2)

Nominal Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.2-1)

Strain in Reinforcing at Ultimate

FlexureBehavior = "Tension Controlled"

$$\Phi_{\rm M} = 0.90$$

 $\varepsilon_{\rm s} > 0.005$

$$M_r = \Phi_M \cdot M_n$$

$$M_r = 6059.9 \text{ kip} \cdot \text{ft}$$

 $M_f = 1437.1 \text{ kip} \cdot \text{ft} < M_r$ MinReinfChk = "OK!"

 $M_u = 1080.5 \text{ kip} \cdot \text{ft} < M_r$ UltimateMom = "OK!"

4.4.10.3 Check Serviceability

To find s_{max} :

Modular Ratio:

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

Tension Reinforcement Ratio:

$$\rho = \frac{A_s}{b \cdot d} \qquad \qquad \rho = 0.005$$

$$k=\sqrt{(2\rho n)+(\rho n)^2}-(\rho n) \qquad \qquad k=0.234$$

$$d \cdot k = 19.04 \text{ in } < d_{stem} = 57.00 \text{ in}$$

Therefore, the compression force acts over a rectangular

$$f_{ss} = \frac{M_s}{A_s \cdot i \cdot d} \cdot \frac{12in}{1ft}$$
 $f_{ss} = 6.73 \text{ ksi}$

$$f_a = 0.6f_v$$
 $f_a = 36.00 \text{ ksi}$

$$f_{ss} < f_a$$
 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:

$$\gamma_e = 1.00$$

$$\beta_s = 1 + \frac{d_c}{0.7(h_{can} - d_c)}$$

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

Bars Inside Stirrup Bar S

Try: BarBInsideSNo = 5

$$s_{Actual} = \frac{b_{stem} - 2\left(cover \ \frac{1}{2}d_{bar_S} + \frac{1}{2}d_{bar_B}\right)}{BarBInsideSNo -}$$

$$s_{actual} < s_{max}$$

For service loads, the stress on the cross-section is located as shown in Figure 4.73.

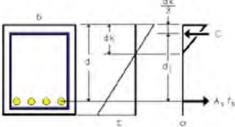


Figure 4.73 Stresses on the Cross Section for Bars B for Service Loads of 45 Degrees Skewed ITBC

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

Service Load Bending Stress in outer layer of the reinforcing.

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

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)

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

Number of Bars B that are inside Stirrup Bar S.

$$s_{Actual} = 8.71 in$$

ServiceabilityCheck = "OK!"

 $\beta_{\rm s} = 1.06$

Bars Outside Stirrup Bar S

BarBOutsideSNo = 11 - BarBInsideSNo

Stirrup Bar S.

BarBOutsideSNo = 6

$$s_{Actual} = \frac{2b_{ledge} + 2\left(cover \ \frac{1}{2}d_{bar_S} + \frac{1}{2}d_{bar_B} - cover \ \frac{1}{2}d_{bar_M} - \frac{1}{2}d_{bar_B}\right)}{BarBOutsideSNo}$$

 $s_{actual} = 8.31 in < s_{max}$

ServiceabilityCheck = "OK!"

4.4.10.4 Check Dead Load

Check allowable M_{dl}:

$$f_{dl} = 22 \text{ ksi}$$

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

Number of Bars B that are inside

$$M_a = A_s \cdot d \cdot j \cdot f_{dl} \cdot \frac{_{1ft}}{_{12in}}$$

$$M_a = 2360 \text{ kip} \cdot \text{ft}$$

 $M_{dl} = 379.0 \text{ kip} \cdot \text{ft} < M_a$

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.

<u>Distribution Width for Shear (AASHTO LRFD 5.8.4.3.2)</u>

Interior Girders

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

$$b_{s_Int} = 69.00 \ in$$

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

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

$$b_{m Int} = 93.50 in$$

Exterior Girders

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

$$b_{m_Ex}\ = 48.00\ in$$

4.4.11.2 Reinforcing Required for Shear Friction

AASHTO LRFD 5.7.4.1

$$\Phi = 0.90$$

$$\mu = 1.4$$

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

$$P_{\rm c} = 0$$

Recall:
$$d_e = 25.50$$
 in

(AASHTO LRFD 5.5.4)

"u" is 1.4 for monolithically placed concrete. (AASHTO LRFD

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)

5.7.4.4)

Minimum Reinforcing (AASHTO LRFD Eq. 5.7.4.2-1)

$$A_{vf_min} = \frac{^{0.05\,ksi\cdot A_{cv}}}{f_v}$$

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

$$a_{\text{vf_min}} = \frac{0.05 \text{ksi} \cdot d_e}{f_y}$$

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

 $a_{vf_min} = 0.26 \frac{in^2}{ft}$ Minimum Reinforcing required for

Interior Girders

$$A_{cv} = d_e \cdot b_{s,Int}$$

$$A_{cv} = 1759 \text{ in}^2$$

$$V_{u \text{ Int}} = 350.6 \text{ kip}$$

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

$$\Phi V_n \ge V_u$$

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

From "4.4.4.4 Load Summary".

(AASHTO LRFD Eq. 5.7.4.3-3)

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

$$A_{vf} = \frac{\frac{v_{u_Int}}{\Phi} - c_1 A_{cv}}{f_y} - P_c$$

$$A_{\rm vf} = 4.64 \, \rm in^2$$

$$a_{vf_Int} = \frac{A_{vf}}{b_{s_Int}}$$

$$a_{\text{vf_Int}} = 0.81 \frac{\text{in}^2}{\text{ft}}$$

 $a_{vf_Int} = 0.81 \frac{in^2}{ft}$ Required Reinforcing for Shear Friction per foot length of cap

Exterior Girders

$$\begin{split} A_{cv} &= d_e \cdot b_{s_Ext} & A_{cv} = 1224 \text{ in}^2 \\ V_{u_Ext} &= 350.6 \text{ kip} & \textit{From "4.4.4.4 Load Summary".} \\ V_n &= c_1 A_{cv} + \mu (A_{vf} f_y + P_c) & (\textit{AASHTO LRFD Eq. 5.7.4.3-3}) \\ \Phi V_n &\geq V_u & (\textit{AASHTO LRFD Eq. 5.7.4.3-1 & AASHTO LRFD Eq. 5.7.4.3-2}) \\ \Phi \cdot \left[c_1 A_{cv} + \mu (A_{vf} f_y + P_c) \right] \geq V_u & (\textit{AASHTO LRFD Eq. 5.7.4.3-2}) \\ A_{vf} &= \frac{\frac{V_{u_Ext}}{\Phi} - c_1 A_{cv}}{f_y} - P_c}{f_y} & A_{vf} &= 4.64 \text{ in}^2 & \textit{Required Reinforcing for Shear Friction} \\ a_{vf_Ext} &= \frac{A_{vf}}{b_{s_Ext}} & a_{vf_Ext} &= 1.16 \frac{\text{in}^2}{\text{ft}} & \textit{Required Reinforcing for Shear Friction per foot length of cap} \end{split}$$

4.4.11.3 Reinforcing Required for Flexure

AASHTO LRFD 5.8.4.2.1

per foot length of cap

Recall: $h=29.50 \ \text{in}$ $d_e=25.50 \ \text{in}$ $a_v=12 \ \text{in}$

From "4.4.5.2 Calculate Dimensions"

Interior Girders

$$\begin{array}{lll} V_{u_Int} = 350.6 \text{ kip} & \textit{From "4.4.4.4 Load Summary"}. \\ N_{uc_Int} = 0.2 \cdot V_{u_Int} & N_{uc_Int} = 70.1 \text{ kip} & (\textit{AASHTO LRFD 5.8.4.2.1}) \\ M_{u_Int} = V_{u_Int} \cdot a_v + N_{uc_Int} (h - d_e) & M_{u_Int} = 374 \text{ kip} \cdot \text{ft} & (\textit{AASHTO LRFD Eq. 5.8.4.2.1-1}) \\ \text{Use the following equations to solve for A_f:} \\ \Phi M_n \geq M_{u_Int} & (\textit{AASHTO LRFD Eq. 1.3.2.1-1}) \\ M_n = A_f f_y \left(d_e - \frac{a}{2} \right) & (\textit{AASHTO LRFD Eq. 5.6.3.2.2-1}) \\ c = \frac{A_f f_y}{\alpha_1 f_c \beta_1 b_{m_Int}} & (\textit{AASHTO LRFD Eq. 5.6.3.1.2-4}) \\ \alpha_1 = 0.85 & (\textit{AASHTO LRFD Eq. 5.6.3.1.2-4}) \\ a_1 = 0.80 & (\textit{AASHTO LRFD 5.6.2.2}) \\ a = c\beta_1 & (\textit{AASHTO LRFD 5.5.4.2}) \\ \text{Solve for A_f:} & A_f = 3.29 \text{ in}^2 & \textit{Required Reinforcing for Flexure} \\ a_{f_Int} = \frac{A_f}{b_{m_Int}} & a_{f_Int} = 0.42 \frac{\text{in}^2}{\text{ft}} & \textit{Required Reinforcing for Flexure} \\ \end{array}$$

Exterior Girders

$$V_{u Ext} = 350.6 \text{ kip}$$

From "4.4.4.4 Load Summary".

$$N_{uc Ext} = 0.2 \cdot V_{u Ext}$$

$$N_{uc Ext} = 70.1 \text{ kip}$$

(AASHTO LRFD 5.8.4.2.1)

$$M_{u_Ext} = V_{u_Ext} \cdot a_v + N_{uc_Ext}(h - d_e)$$
 $M_{u_Ext} = 374 \text{ kip} \cdot \text{ft}$

(AASHTO LRFD Eq. 5.8.4.2.1-1)

Use the following equations to solve for A_f:

$$\Phi M_n \ge M_{u Ext}$$

(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)

$$a = c\beta_1$$

$$0.75 \le \Phi = 0.65 + 0.15 \left(\frac{d_e}{c} - 1\right) \le 0.90$$

AASHTO LRFD 5.5.4.2

$$A_f = 3.32 \text{ in}^2$$

Required Reinforcing for Flexure

$$a_{f_Ext} = \frac{A_f}{b_{m Ext}}$$

$$a_{f_Ext} = 0.83 \frac{in^2}{ft}$$

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:

$$N_{uc_Int} = 0.2V_{u_Int}$$

 $N_{uc\ Int} = 70.1 \text{ kip}$

$$A_n = \frac{N_{uc_Int}}{\Phi f_v}$$

 $A_n = 1.30 \text{ in}^2$

Required Reinforcing for Axial Tension

$$a_{n_Int} = \frac{A_n}{b_{m_Int}}$$

 $a_{n_Int} = 0.17 \frac{in^2}{ft}$ Required Reinforcing for Axial Tension per foot length of cap

Exterior Girders:

$$N_{uc Ext} = 0.2V_{u Int}$$

 $N_{uc Ext} = 70.1 \text{ kip}$

$$A_n = \frac{N_{uc_Ext}}{\Phi f_y}$$

$$A_n=1.29\,\text{in}^2$$

Required Reinforcing for Axial Tension

$$a_{n_Ext} = \frac{A_n}{b_{m_Ext}}$$

$$a_{n_{\perp}Ext} = 0.32 \frac{in^2}{ft}$$

 $a_{n_Ext} = 0.32 \frac{in^2}{ft}$ Required Reinforcing for Axial Tension per foot length of cap

4.4.11.5 Minimum Reinforcing

$$a_{s_min} = 0.04 \frac{f_c}{f_v} d_e$$

$$a_{s_min} = 1.02 \frac{in^2}{ft}$$
 Minimum Required Reinforcing

4.4.11.6 Check Required Reinforcing

Actual Reinforcing:

$$a_{s} = \frac{A_{bar_M}}{s_{bar_M}} \qquad a_{s} = 1.16 \frac{in^{2}}{ft} \qquad \begin{array}{l} \textit{Primary Ledge Reinforcing} \\ \textit{Provided} \end{array}$$

$$a_{h} = \frac{A_{bar_N}}{s_{bar_N}} \qquad a_{h} = 1.16 \frac{in^{2}}{ft} \qquad \begin{array}{l} \textit{Auxiliary Ledge Reinforcing} \\ \textit{Provided} \end{array}$$

$$\underbrace{Checks:}_{S} A_{s} \geq A_{s_min} \qquad (\textit{AASHTO LRFD 5.8.4.2.1}) \qquad (\textit{AASHTO LRFD 5.8.4.2.2}) \qquad (\textit{AASHTO LRFD Eq. 5.8.4.2.2-5}) \\ A_{s} \geq \frac{2A_{vf}}{3} + A_{n} \qquad (\textit{AASHTO LRFD Eq. 5.8.4.2.2-5}) \qquad (\textit{AASHTO LRFD Eq. 5.8.4.2.2-6}) \end{array}$$

$$A_{h} \geq 0.5(A_{s} - A_{n}) \qquad (\textit{AASHTO LRFD Eq. 5.8.4.2.2-6})$$

Check Interior Girders:

Bar M:

$$\begin{array}{lll} \text{Check if:} & a_s \geq a_{s_min} & \textit{(AASHTO LRFD 5.8.4.2.1)} \\ & a_s \geq a_{f_Int} + a_{n_Int} & \textit{(AASHTO LRFD 5.8.4.2.2)} \\ & a_s \geq \frac{2a_{vf_Int}}{3} + a_{n_Int} & \textit{(AASHTO LRFD Eq. 5.8.4.2.2-5)} \\ & a_s = 1.16 \frac{in^2}{ft} \\ & a_{s_min} = 1.02 \, \frac{in^2}{ft} & < \, a_s \\ & a_{f_Int} + a_{n_Int} = 0.59 \frac{in^2}{ft} & < \, a_s \\ & & & & \\ \frac{2a_{vf_Int}}{3} + a_{n_Int} = 0.71 \frac{in^2}{ft} & < \, a_s \\ & & & \\ \end{array}$$

BarMCheck = "OK!"

Bar N:

Check if:
$$a_h \ge 0.5 \cdot (a_s - a_{n_Int})$$
 (AASHTO LRFD Eq. 5.8.4.2.2-6)
$$a_s = \text{The maximum of:} \qquad \text{"a_s" in this equation is the steel} \\ a_{f_Int} + a_{n_Int} \qquad \text{required for Bar M, based on the} \\ \frac{2a_{vf_Int}}{3} + a_{n_Int} \qquad \text{LRFD 5.8.4.2.2. This is derived from} \\ a_s = 0.71 \frac{\text{in}^2}{\text{ft}} \qquad \text{the suggestion that Ah should not be} \\ a_s = 0.71 \frac{\text{in}^2}{\text{ft}} \qquad \text{(Furlong \& Mirza pg. 73 \& 74)}$$

$$0.5 \cdot (a_s - a_{n_Int}) = 0.28 \frac{in^2}{ft} < a_h$$

BarNCheck = "OK!"

Check Exterior Girders:

Bar M:

Check if:
$$a_{s} \ge a_{s_min}$$
 (AASHTO LRFD 5.8.4.2.1)
$$a_{s} \ge a_{f_Ext} + a_{n_Ext}$$
 (AASHTO LRFD 5.8.4.2.2)
$$a_{s} \ge \frac{2a_{vf_Ext}}{3} + a_{n_Ext}$$
 (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:
$$a_h \geq 0.5 \cdot \left(a_s - a_{n_Ext}\right) \qquad (AASHTO\ LRFD\ Eq.\ 5.8.4.2.2-6)$$

$$a_s = \text{The maximum of:} \qquad "a_s" \text{ in this equation is the steel required}$$

$$a_{f_Ext} + a_{n_Ex} \qquad \text{for Bar M, based on the requirements for}$$

$$Bar\ M \text{ in AASHTO\ LRFD}\ 5.8.4.2.2. \text{ This}$$
 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)}
$$0.5 \cdot \left(a_s - a_{n_Ext}\right) = 0.42 \frac{\text{in}^2}{\text{ft}} < a_h$$

$$Bar\ NCheck = "OK!"$$

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.

$$s_{bar S} = 7.20 in$$

$$A_{hr} = 2stirrups \cdot A_{bar S}$$

$$A_v = 2legs \cdot A_{hr}$$

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

$A_{hr} = 0.88 \text{ in}^2$

$$A_{\rm v} = 1.76 \, \rm in^2$$

4.4.12.1 Check Minimum Transverse Reinforcement

$$b_v = b_{stem} \\$$

$$A_{v_min} = 0.0316\lambda \sqrt{f_c} \frac{b_v \cdot s_{bar_S}}{f_v}$$

 $\lambda = 1.0$ for normal weight concrete

$$b_v = 42 in$$

(AASHTO LRFD Eq. 5.7.2.5-1) (AASHTO LRFD 5.4.2.8)

$$A_{v_min} = 0.36 \ in^2$$

 $A_v > A_{v \text{ min}}$

MinimumSteelCheck = "OK!"

4.4.12.2 Check Service Limit State

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

Interior Girders

$$V_{all} = minimum of:$$

$$\frac{A_{hr}\cdot\left(\frac{2}{3}f_{y}\right)}{s_{bar_S}}\cdot\left(W+3a_{v}\right)=235\:\text{kip}$$

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)

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

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{har} s} \cdot S = 469 \text{ kip}$$

(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)

$$V_{all} = 235 \text{ kip}$$

$$V_{s Int} = 231.6 \text{ kip} < V_{all}$$

ServiceCheck = "OK!"

Exterior Girders

$$V_{all}$$
 = minimum of:

V_{all} for the Interior Girder

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{har\ S}} \cdot \left(\frac{W + 3a_v}{2} + c\right) = 235 \text{ kip}$$

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

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

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{bar,S}} \cdot \left(\frac{S}{2} + c\right) = 352 \text{ kip}$$

 $V_{\rm all} = 235 \, \rm kip$

$$V_{s Ext} = 231.6 \text{ kip} < V_{all}$$

(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.)

ServiceCheck = "OK!"

(AASHTO LRFD 5.8.4.3.5)

4.4.12.3 Check Strength Limit State

$$\Phi = 0.90$$

(AASHTO LRFD Eq. 5.5.4.2)

Interior Girders:

 $V_n = minimum of:$

$$\frac{A_{hr} \cdot f_y}{s_{har S}} \cdot S = 704 \text{ kip}$$

(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)

 $V_n = 704 \text{ kip}$

 $\Phi V_n = 634 \text{ kip}$

$$V_{u,Int} = 350.6 \text{ kip} < \Phi V_{n}$$

UltimateCheck = "OK!"

Exterior Girders:

 $V_n = minimum of:$

V_n for the Interior Girder

$$\frac{A_{\text{hr}} \cdot f_{y}}{s_{\text{bar S}}} \cdot \left(\frac{S}{2} + c\right) = 528 \text{ kip}$$

(AASHTO LRFD Eq. 5.8.4.3.5-2)

$$\left(0.063\sqrt{f_c} \cdot b_f \cdot d_f\right) + \frac{A_{hr} \cdot f_y}{s_{bar_S}} \left(\frac{W + 2d_f}{2} + c\right) = 747 \text{ kip } \frac{(AASHTO LRFD Eq. 5.8.4.3.5-3)}{(These equations are modified to$$

 $V_n = 528 \text{ kip}$

 $\Phi V_n = 475 \text{ kip}$

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

$$V_{\text{u Ext}} = 350.6 \text{ kip} < \Phi V_{\text{n}}$$

UltimateCheck = "OK!"

4.4.12.4 Check Combined Shear and Torsion

 $d_v = 80.59 \text{ in}$

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.

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 :

$$\begin{split} &\Phi_V = 0.90 \\ &v_u = \frac{|v_u - (\Phi_V \cdot V_p)|}{\Phi_V \cdot b_v \cdot d_v} \\ &v_u = 0.15 \text{ ksi} \\ &\frac{v_u}{f_c} = 0.03 \end{split}$$

Using Table B5.2-1 with
$$\frac{v_u}{f_c}=0.03$$
 and $\epsilon_x=0.001$ $\theta=36.4$ deg and $\beta=2.23$

$$\begin{split} \epsilon_x &= \frac{\frac{|M_u|}{d_v} + 0.5 N_u + 0.5 |V_u - V_p| \cot\theta - A_{ps} f_{po}}{2 (E_s A_s + E_p A_{ps})} \\ \text{where } &|M_u| = 504.8 \text{ kip} \cdot \text{ft must be} > \big| V_u - V_p \big| d_v = 3108 \text{ kip} \cdot \text{ft} \\ \epsilon_x &= 1.23 \times 10^{-3} \, > \, 1.00 \times 10^{-3} \\ \text{use } \epsilon_x &= 1.00 \times 10^{-3}. \end{split}$$

$$V_p = 0 \text{ kip}$$

$$\begin{aligned} A_c &= b_{stem} \cdot \frac{h_{cap}}{2} & A_c &= 1785 \text{ in}^2 \\ s &= s_{bar_S} & s &= 7.20 \text{ in} \end{aligned}$$

(AASHTO LRFD Eq. 5.5.4.2)

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

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.

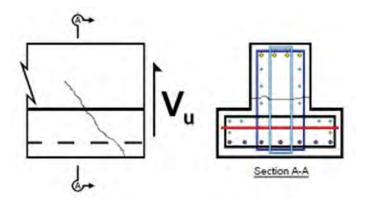
Strain halfway between the compressive and tensile resultants (AASHTO LRFD Eq. B5.2-3) If $\varepsilon_x < 0$, then use equation B5.2-5 and re-solve for ε_x .

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 " is zero as there is no prestressing.

(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

$$\begin{split} A_v &= 2 legs \cdot 2 stirrups \cdot A_{bar_S} & A_v &= 1.76 \ in^2 \\ A_t &= 1 leg \cdot A_{bar_S} & A_t &= 0.44 \ in^2 \\ A_{oh} &= (d_{stem}) \cdot (b_{stem} - 2 cover) + (d_{ledge} - 2 cover) \cdot (b_f - 2 cover) \\ & A_{oh} &= 4110 \ in^2 \\ A_0 &= 0.85 A_{oh} & A_0 &= 3493.5 in^2 \\ p_h &= (b_{stem} - 2 cover) + 2 \big(b_{ledge}\big) + (b_f - 2 cover) + 2 \big(h_{cap} - 2 cover\big) \\ p_h &= 334 \ in \end{split}$$

Equivalent Shear Force

$$V_{u_{-}Eq} = \sqrt{V_{u}^{2} + \left(\frac{0.9p_{h}T_{u}}{2A_{0}}\right)^{2}}$$
 $V_{u_{-}Eq} = 611.1 \text{ kip } (AASHTO LRFD Eq. B.5.2-1)$

Shear Steel Required

$$V_n$$
 = the lesser of:

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

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

Check maximum ΦV_n for section:

$$\begin{split} \Phi V_{n_max} &= \Phi \cdot \left(0.25 \cdot f_c \cdot b_v \cdot d_v + V_p \right) \\ \Phi V_{n_max} &= 3808 \text{ kip} \\ V_u &= 462.8 \text{ kip} < \Phi V_{n_max} \end{split}$$

Calculate required shear steel:

$$\begin{split} &V_{u} < \Phi V_{n} & (\textit{AASHTO LRFD Eq. 1.3.2.1-1}) \\ &V_{c} = 0.0316 \cdot \beta \cdot \sqrt{f_{c}} \cdot b_{v} \cdot d_{v} & V_{c} = 533 \text{ kip} & (\textit{AASHTO LRFD Eq. 5.7.3.3-3}) \\ &V_{u} < \Phi_{V} \cdot \left(V_{c} + V_{s} + V_{p}\right) & (\textit{AASHTO LRFD Eq. 5.7.3.3-4}) \\ &V_{s} = \frac{A_{v} \cdot f_{y} \cdot d_{v} \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha}{s_{req}} & (\textit{AASHTO LRFD Eq. 5.7.3.3-4}) \\ &a_{v_req} = \frac{\frac{V_{u}}{\Phi_{V}} - V_{c} - V_{p}}{f_{v} \cdot d_{v} \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha} & a_{v_req} = 0.00 \frac{in^{2}}{ft} \end{split}$$

Torsional Steel Required

$$\begin{split} \Phi_T &= 0.9 & (\textit{AASHTO LRFD 5.5.4.2}) \\ T_u &\leq \Phi_T T_n & (\textit{AASHTO LRFD Eq. 1.3.2.1-1}) \\ T_n &= \frac{2A_0 A_t f_y \cot \theta}{s_{bar_S}} & (\textit{AASHTO LRFD Eq. 5.7.3.6.2-1}) \\ a_{t_req} &= \frac{T_u}{\Phi_T 2A_0 f_y \cot \theta} & a_{t_req} &= 0.22 \frac{in^2}{ft} \end{split}$$

Total Required Transverse Steel

$$a_{req} = a_{v_req} + 2 sides \cdot a_{t_req} \qquad a_{req} = 0.44 \, \frac{in^2}{ft} \qquad \begin{array}{l} \textit{designed for the side of the section} \\ \textit{where the effects of shear and torsion} \\ a_{prov} = \frac{A_v}{s_{bar_S}} \qquad a_{prov} = 2.93 \, \frac{in^2}{ft} \qquad \begin{array}{l} \textit{are additive. (AASHTO LRFD} \\ \textit{C5.7.3.6.1)} \\ \end{array}$$

The transverse reinforcement is

Longitudinal Reinforcement

$$\begin{split} A_{ps}f_{ps} + A_{s}f_{y} &\geq \frac{|\mathsf{M}_{u}|}{\Phi d_{v}} + \frac{0.5N_{u}}{\Phi} + \cdots \\ &\qquad \qquad (\mathit{AASHTO LRFD Eq. 5.7.3.6.3-1}) \\ &\qquad \qquad \cot\theta \sqrt{\left(\left|\frac{V_{u}}{\Phi} - V_{p}\right| - 0.5V_{s}\right)^{2} + \left(\frac{0.45p_{h}T_{u}}{2A_{0}\Phi}\right)^{2}} \\ V_{s} &= a_{t_req} \cdot f_{y} \cdot d_{v} \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha \\ &\qquad \qquad (\mathit{AASHTO LRFD Eq. 5.7.3.3-4}) \\ &\qquad \qquad \qquad \qquad \qquad \qquad Bounded \ By: \ V_{s} &< \frac{V_{u}}{\Phi_{v}} \qquad \qquad V_{s} = 514.2 \ kip \qquad (\mathit{AASHTO LRFD Eq. 5.7.3.5-1}) \\ &\qquad \qquad \qquad \frac{|\mathsf{M}_{u}|}{\Phi_{f}d_{v}} + \frac{0.5N_{u}}{\Phi_{c}} + \cot\theta \sqrt{\left(\left|\frac{V_{u}}{\Phi_{v}} - V_{p}\right| - 0.5V_{s}\right)^{2} + \left(\frac{0.45p_{h}T_{u}}{2A_{0}\Phi_{T}}\right)^{2}} = 544 \ kip \\ &\qquad \qquad \qquad \text{Provided Force:} \end{split}$$

LongitudinalReinfChk = "OK!"

 $A_s f_v = 655.2 \text{ kip} > 544 \text{ kip}$

4.4.12.5 Maximum Spacing of Transverse Reinforcement

(AASHTO LRFD 5.7.2.6)

Shear Stress

$$v_u = \frac{|v_u - \Phi_V V_p|}{\Phi_V b_v d_v} \qquad \qquad v_u = 0.15 \text{ ksi} \qquad \textit{(AASHTO LRFD Eq. 5.7.2.8-1)}$$

$$0.125 \cdot f_c = 0.625 \text{ ksi}$$

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

$$s_{\text{max}} = \min(0.8d_{\text{v}}, 24\text{in})$$

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

$$s_{\text{max}} = \min(0.4d_{\text{v}}, 12\text{in})$$

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

TxDOT limits the maximum transverse reinforcement spacing to 12".

$$s_{max} = 12.00 \text{ in}$$

 $s_{bar S} = 7.20 \text{ in} < s_{max}$

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.

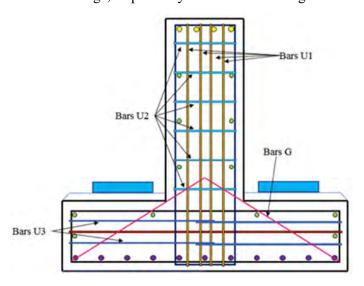


Figure 4.75 End Face Section View of 45 Degrees Skewed ITBC

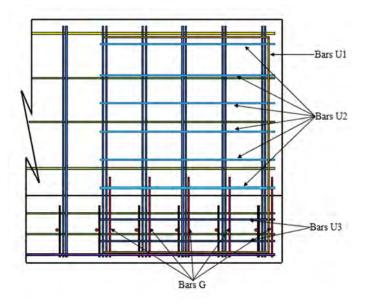


Figure 4.76 End Face Elevation View of 45 Degrees Skewed ITBC

4.4.14 Skin Reinforcement (Bars T)

Try $7 \sim \# 6$ bars in Stem and $3 \sim \# 6$ 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)

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

TxDOT typically uses: a = 6 in

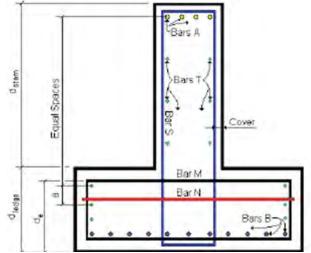


Figure 4.77 Section View for T Bars of 45 Degrees **Skewed ITBC**

(AASHTO LRFD 5.6.7)

4.4.14.1 Required Area of Skin Reinforcement

4.4.14.1 Required Area of Skin Reinforcement
$$A_{sk_Req} = 0.012 \cdot (d - 30)$$

$$A_{sk_Req} = 0.62 \frac{in^2}{ft}$$
(AASHTO LRFD Eq. 5.6.7-3)

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.

$$A_{sk_max} = max \left(\frac{\frac{A_{bar_A} \cdot BarANo}{4}}{\frac{d_{s_neg}}{2}}, \frac{\frac{A_{bar_B} \cdot BarBNo}{4}}{\frac{d_{s_pos}}{2}} \right)$$

$$A_{sk_max} = 1.27 \frac{in^2}{ft}$$

$$A_{skReq} = min(A_{sk_Req}, A_{sk_max})$$

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

4.4.14.2 Required Spacing of Skin Reinforcement

(AASHTO LRFD 5.6.7)

 $s_{req} = minimum of:$

$$\frac{A_{bar_T}}{A_{skReq}} = 8.52 \text{ in}$$

$$\frac{d_{s_neg}}{6} = 13.57 \text{ in}$$

$$\frac{d_{s_pos}}{6} = 13.56 \text{ in}$$

$$s_{req} = 8.52 in$$

4.4.14.3 Actual Spacing of Skin Reinforcement

Check T Bars spacing in Stem:

$$\begin{split} h_{top} &= d_{stem} - \left(cover + \frac{d_{bar_S}}{2} + \frac{d_{bar_A}}{2}\right) + \left(cover + \frac{d_{bar_M}}{2} + \frac{d_{bar_T}}{2}\right) \\ h_{top} &= 56.73 \text{ in} \end{split}$$

$$s_{skStem} = \frac{h_{top}}{NoTBarsStem + 1}$$

$$s_{skStem} = 7.09 in$$

$$s_{skStem} < s_{req}$$

SkinSpacing = "OK!"

Check T Bars spacing in Ledge:

$$\begin{split} h_{bot} = d_{ledge} - \left(cover + \frac{d_{bar_M}}{2} + \frac{d_{bar_T}}{2}\right) - \left(cover + \frac{d_{bar_S}}{2} + \frac{d_{bar_B}}{2}\right) \\ h_{bot} = 21.11 \ in \end{split}$$

$$s_{skLedge} = \frac{h_{bot} - a}{NoTBarsLedge}$$

$$s_{skLedge} = 7.56 in$$

$$s_{skLedge} < s_{req}$$

Check if "a" is less than s_{req}

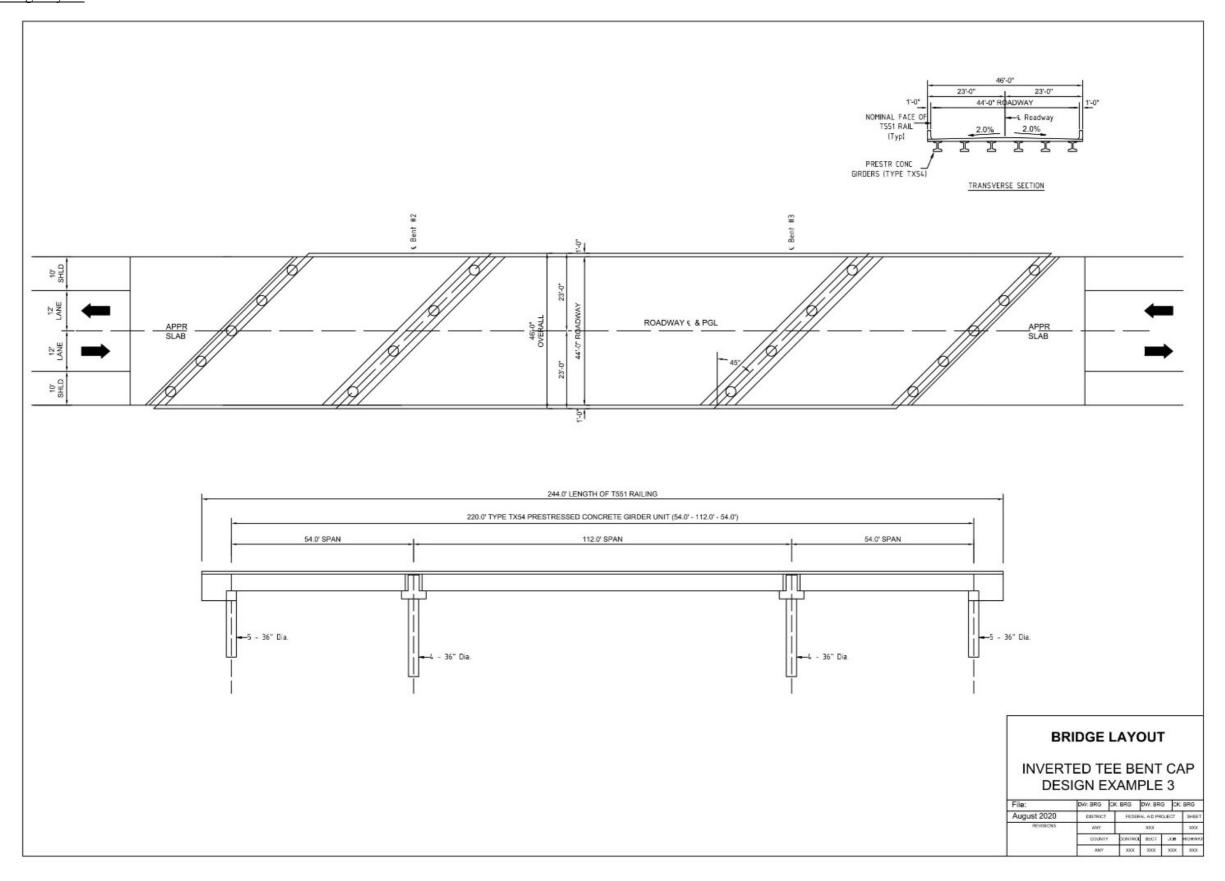
$$a = 6 in < s_{req}$$

Skin Reinforcement Summary:

Use $7 \sim \# 6$ bars in Stem and $3 \sim \# 6$ bars in Ledge on each side

4.4.15 Design Details and Drawings

4.4.15.1 Bridge layout



4.4.15.2 <u>CAP 18 Input File</u>

```
User Date (Today
CSJ Init if Blank) Comment
                     Proj
$Header Card 2 -----
CAP18 Version 6.00 ITBC Design Example 3, Skew = 45.00
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
                           XX X XX
             X X X
                                                  XXXXXXXXXX
Ś
Anly Opt (1=Working,
            |-Movable Load Data--| 2=Load Factor,3=Both)
Num Increment |Num Start Stop Step|Anly| Load Factors:
Ś
                        Ś
            Inc Length
$
             XX XXXXXXXXX
                 0.5
             92
Ś
  TABLE 2b
           Max # |-----Live Load Reduction Factors-----|
$
   Overlav
 Load Factor Lanes | 1 lane 2 lanes 3 lanes 4 lanes 5 lanes
Ś
   XXXXX X XXXX XXXX XXXX XXXX XXXX 1.50 3 1.2 1.0 0.85 0.65 0.65
STABLE 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
Ś
Ś
           XX XX XX XX XX
3 6 4 11 8
                                  VCP - Shear Control Points
 (Num Inputs)
  Left Lane Boundary Stations
 S
  Right Lane Boundary Stations
          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)
          (Stringers)
             6 22 38 54 70
                              86
  Station of Supports (two rows max)
 Ś
  Moment Control Point Stations (two rows max)
          (Mom CP)
            86
  Shear Control Point Stations (two rows max)
         $TABLE 4 - STIFFNESS AND LOAD DATA -----
            Bending Sidewalk, Cap &
Station 1 if Stiffness Slab Stringer Moving
From To Cont'd of Cap Loads Loads
           From To Cont'd of Cap Loads
                                                    Loads, DW
$XXXXXXXXXXXXX
            XXX
                8.66E+07
(CAP EI & DL)
                90
                                     -2.589
(DL Span1, Bm1)
                6
                                      -50.17
                                                    -5.04
                                      -50.17
(DL Span1, Bm2)
                                                    -5.04
(DL Span1, Bm3)
             38
                38
                                      -50.17
                                                    -5.04
(DL Span1, Bm4)
(DL Span1, Bm5)
             54
                 54
                                      -50.17
                                                    -5.04
             70
                70
                                      -50.17
                                                    -5.04
(DL Span1, Bm6)
(DL Span2, Bm1)
             86
                                      -50.17
                                                    -5.04
                                      -104.1
(DL Span2, Bm2)
             22
                                      -104.1
                                                    -10.5
(DL Span2, Bm3)
(DL Span2, Bm4)
             38
                38
                                      -104.1
                                                    -10.5
             54
                 54
                                      -104.1
                                                    -10.5
(DL Span2, Bm5)
             70
                                      -104.1
                70
                                                    -10.5
(DL Span2, Bm6)
                                      -104.1
                                                    -10.5
(Dist. Lane Ld)
                                             -4.92
            0
                20
(Conc. Lane Ld)
                 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) HIGHWAY PD- CONTROL- CODED PSF COUNTY NO IPE SECTION-JOB BY DATE NO 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) **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) 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): 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 LIST OF LOAD COEFFICIENTS CORRESPONDING TO NUMBER OF LANES LOADED

1.000

1.200

0.850

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CAP18 BENT CAP ANALYSIS Ver. 6.2 (Jul, 2011)

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 NUM OF NUM OF NUM MOM NUM SHEAR LANES STRINGERS SUPPORTS CONTR PTS 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

FIXED-OR-MOVABLE ------ FIXED-POSITION DATA ----- MOVABLESTA STA CONTD CAP BENDING SIDEWALK, STRINGER, OVERLAY POSITION
FROM TO IF=1 STIFFNESS SLAB LOADS CAP LOADS LOADS SLAB LOADS

(K-FT*FT) (K) (K) (K) (K)

2 90 0 86600000.000 0.000 -2.589 0.000 0.000 0.000 0.000 -50.170 -5.040 0.000 0.000 0.000 -50.170 -5.040 0.000 6 6 0 22 22 0 0.000 0.000 -50.170 -5.040 38 38 0 0.000 0.000 54 54 0 0.000 -50.170 -5.040 0.000 0.000 70 70 0 0.000 0.000 -50.170 -5.040 0.000 86 86 0 0.000 -50.170 -5.040 0.000 0.000 0.000 0.000 -104.100 -10.500 6 6 0 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 0.000 -104.100 -10.500 0.000 0.000 -104.100 -10.500 0.000 54 54 0 0.000 70 70 0 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 -21.300 0.000

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 -0.000087 -0.000076 -0.000065	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	0.0000FF	F 2	7 2	
5		-0.000044	-5.2 -11.7	-11.0	
6	4.24	-0.000033	-20.7	-99.0	
7			-152.4		
8			-286.7		
9			-423.7		
10	7.07		-563.		
11	7.78	0.000002	-470.	4 129.3	
12	8.49	0.000002	-380.	2 125.7	
13	9.19		-292.		
14	9.90	-0.000005	-207.	7 118.4	
15	10.61	-0.000011	-125	5.3 114.7	
16	11.31	-0.000017	-45.		
17	12.02	-0.000024	31.		
18	12.73	-0.000030	106		
19 20	13.44 14.14	-0.000036 -0.000041	247		
21	14.14	-0.000041	314		
22	15.56	-0.000044	379		
23	16.26	-0.000045	320		
24	16.20	-0.000043	259		
25	17.68	-0.000042	196		
26	18.38	-0.000033	129		
27	19.09	-0.000027	61.		
28	19.80				
29	20.51	-0.000021 -0.000015	-84.	1 -106.4	
30	21.21	-0.000009	-160		
31	21.92	-0.000004	-239		
32	22.63	-0.000001	-321		
33	23.33	0.000001	-405		
34	24.04	0.000000	-492	.5 44.5	
35	24.75	-0.000004	-342	7 210.1	
36	25.46	-0.000009	-195	5.4 206.4	
37	26.16	-0.000016	-50.	8 202.8	
38	26.87	-0.000023	91.		
39	27.58	-0.000029	110	.7 25.6	
40	28.28	-0.000035	127		
41	28.99	-0.000040	141		
42	29.70	-0.000045	153		
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) MOI	 MENT (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 71	49.50 50.20	-0.000046 -0.000044	379.0 314.7	-4.2 -92.7	
72	50.20	-0.000044	247.9	-92.7 -96.4	
73	51.62	-0.000036	178.4	-100.0	
74	52.33	-0.000030	106.4	-100.0	
75	53.03	-0.000030	31.8	-103.7	
76	53.74	-0.000024	-45.5	-111.0	
77	54.45	-0.000017	-125.3	-114.7	
78	55.15	-0.0000011	-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) (\star --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

					LAIN
	6	-20.7 0 1 2 3 0*	0.0 0.0 0.0 0.0	0 0.0 1 0.0 2 0.0 3 0.0	
	10	-563.1 0 1 2 3 0*	0.0 0.0 0.0 0.0	0 -249.3 1 2 1 -249.3 1 2 2 0.0 3 0.0	
1	22	379.0 0 1 2 3 0*	0.0	0 13 0 -47.2 2 36 1 12 1 -47.2 2 36 3 62 2 0.0 3 0.0	
3	34	-492.5 0 1 2 3 0*	0.0	3 62 0 -192.8 0 18 3 62 1 -164.8 1 12 2 -119.8 2 32 3 0.0 2*	
3	38	91.3 0 1 2 3 0*	118.2 118.2 4.5 0.0	2 32 0 -83.2 1 9 2 32 1 -83.2 1 9 3 62 2 0.0 3 0.0	
4	46	174.2 0 1 2 3 0*	98.1 98.1 0.0 0.0	2 36 0 -39.3 1 9 2 36 1 -39.3 1 9 2 -39.3 3 63 3 0.0 2*	
į	54	91.3 0 1 2 3 0*	118.2 118.2 4.5 0.0	2 40 0 -83.2 3 63 2 40 1 -83.2 3 63 1 10 2 0.0 3 0.0	
	58	-492.5 0 1 2 3 0*	26.4 26.4 0.0 0.0	1 9 0 -192.8 0 54 1 9 1 -164.8 3 60 2 -119.8 2 40 3 0.0 2*	

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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 1 2 3 0*	285.7 284.5 13.2 0.0	0 59 0 -47.2 2 36 3 60 1 -47.2 2 36 1 9 2 0.0 3 0.0 0*
82	-563.1 0 1 2 3 0*	0.0 0.0 0.0 0.0	0 -249.3 3 70 1 -249.3 3 70 2 0.0 3 0.0 0*
86	-20.7 0 1 2 3 0*	0.0 0.0 0.0 0.0	0 0.0 1 0.0 2 0.0 3 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 -88.1 1 2 0 0.0 0 1 2 -88.1 1 0.0 1 2 0.0 2 0.0 3 3 0.0 0.0 0* 125.7 12 44.8 1 6 0 -5.6 2 36 0 44.8 1 6 1 -5.6 1.6 3 62 2 0.0 1 2 36 2 3 3 0.0 0.0 0* 0* 32 -117.4 1.6 3 62 0 -54.6 0 15 1.6 3 62 1 -53.0 1 12 0 1 2 -11.2 2 32 3 0.0 2 0.0 3 0.0 0* 36 206.4 0 28 0 -7.8 3 63 87.6 0 84.1 2 32 1 -7.8 3 63 1 30.7 1 12 2 0.0 2 3 0.0 3 0.0 0* 2* 56 -206.4 0 7.8 1 9 0 -87.6 0 44 7.8 1 9 1 -84.1 2 40 1 2 0.0 -30.7 3 60 0.0 3 0.0 3 0* 2* 60 117.4
 54.6
 0
 57
 0
 -1.6
 1
 9

 53.0
 3
 60
 1
 -1.6
 1
 9

 11.2
 2
 40
 2
 0.0
 0 1 2 3 0.0 0.0 0* 80 -125.7 5.6 2 36 0 -44.8 3 66 0

2 36 1 -44.8 3 66

0.0

0.0

0.0

2 -1.6 1 9

3

0*

88.1 3 70 0 0.0 88.1 3 70 1 0.0

2

3

0*

1

2

3

1

3

0*

191.8

84

5.6

0.0

0.0

0.0

0.0

REACTION (K)

AT DEAD LD LANE POSITIVE LOAD AT LANE NEGATIVE LOAD AT STA EFFECT ORDER MAXIMUM LANE STA ORDER MAXIMUM LANE STA

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

-1 -0.71 0.0 0.0 0.0 0.0 0.0 0.0 0.0 1 0.71 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.	
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	
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	
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	
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	
5 3.54 -11.7 -11.7 -11.0 -11.0 6 4.24 -20.7 -20.7 -99.6 -152.4	
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	
6 4.24 -20.7 -20.7 -99.6 -152.4 7 4.95 -152.4 -227.2 -188.1 -293.9	
/ 4.95 -152.4 -227.2 -188.1 -293.9	
0 566 2067 4262 4010 2075	
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	
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	
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	
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	
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	
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	
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	
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	
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 6 103 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	

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA (DIST X	FT-K) (1	 ИОМ МА FT-К) (X - MOM (K) (F	 MAX + SHEAR ()	MAX - SHEAR
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	-1/6.9	-342./	-197.1	-321.2	
57	40.31	-317.6	-546.1	-200.8	-324.9	
58	41.01	-460.8	-///.2	-27.0	-88.3	
59	41.72	-3/5.3	-645.0	186.6	119.2	
60 61	42.43	-292.3	-515.5	170.2	115.5	
62	43.13	1242	-300.3	175.5	100.2	
63	43.04	-134.2	-200.5 -182.8	171.0	100.2	
64	45.35	73.6	-102.0	168.3	104.3	
65	45.25	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	

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TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

ST	A DIST	X MAX	+ MOM	MAX - MON	MAX +	SHEAR	MAX - SHEAR
	(FT)	(FT-K)	(FT-K)	(K)	(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		

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TABLE 7. MAXIMUM SUPPORT REACTIONS (WORKING STRESS)

STA DIST X		DIST X	MAX + REACT		MAX - REACT
	10	7.07	485.6	325	5.4
	34	24.04	517.3	32	7.3
	58	41.01	517.3	32	7.3
	82	57.98	485.6	32	5.4

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 -25.9 0 0.0 0.0 0.0 0.0 1 2 0.0 2 0.0 3 0.0 3 0.0 0* 0* 10 -714.9 0.0 0 -436.2 1 2 0 -436.2 1 2 0.0 1 1 2 0.0 2 0.0 3 0.0 3 0.0 0* 22 480.6
 499.9
 0
 13
 0
 -82.6
 2
 36

 497.9
 1
 12
 1
 -82.6
 2
 36

 23.1
 3
 62
 2
 0.0
 0 2 3 0.0 3 0.0 0* 0* 34 -623.9 46.2 3 62 0 -337.3 0 18 0 46.2 3 62 1 -288.5 1 12 1 2 -209.6 2 32 3 0.0 0.0 3 0.0 2* 0* 38 116.9 206.9 2 32 0 -145.6 1 9 206.9 2 32 1 -145.6 1 9 8.0 3 62 2 0.0 1 2 0.0 3 0.0 0* 0* 46 220.4 171.6 2 36 0 -68.9 1 9 171.6 2 36 1 -68.9 1 9 0.0 2 -68.9 3 63 0 1 2 0.0 0.0 2* 0* 54 116.9 206.9 2 40 0 -145.6 3 63 206.9 2 40 1 -145.6 3 63 8.0 1 10 2 0.0 0 2 3 0.0 3 0.0 0* 58 -623.9 46.2 1 9 0 -337.3 0 54 46.2 1 9 1 -288.5 3 60 0 2 -209.6 2 40 0.0 2 3 0.0 3 0.0

2*

MOMENT (FT-K)

0*

AT DEAD LD LANE POSITIVE LOAD AT LANE NEGATIVE LOAD AT STA EFFECT ORDER MAXIMUM LANE STA ORDER MAXIMUM LANE STA

70 480.6
 499.9
 0
 59
 0
 -82.6
 2
 36

 497.9
 3
 60
 1
 -82.6
 2
 36

 23.1
 1
 9
 2
 0.0
 0 1 2 3 0.0 3 0.0 0* 0* 82 -714.9 0.0 0 -436.2 3 70 0 1 0.0 -436.2 3 70 2 0.0 2 0.0 3 3 0.0 0.0 0* 86 -25.9 0 0.0 0 0.0 0.0 0.0 1 1 0.0 0.0 3 0.0 3 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

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

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

		MAX + I			 MAX + SHEAR K)	MAX - SHEAR
	-0.71	0.0	0.0		0.0	
o	0.00				0.0	
1	0.71	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	-6.5 -14.6	-14.6	-9.2 -13.7	-13.7	
6	4.24	-14.6 -25.9 -193.3 -363.9	-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 -248.2	-428.7 -433.2	
9	6.36	-537.8	-930.4	-248.2	-433.2	
10	7.07	-714.9 -565.0	-1238.4	-12.6	-93.0 153.1	
11	7.78	-565.0	-1061.9	257.9	132.1	
12	8.49 9.19	-406.9 -250.4	-888.7	253.3 248.8	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	-97.2 53.6 204.3 354.3	-228.2	235.0	129.2	
17	12.02	354.3	-/1.1	230.5	124.6	
18 19	12.73 13.44		67.8	225.9	120.1 115.5	
20	14.14	796.2		221.3 216.7		
21	14.85		308.0	216.7 212.1	106.3	
22	15.56		381.5	33.3	-17.6	
23	16.26		298 1	-104.0	-222.0	
24			211.2	-108.6		
25	17.68	769.1 609.7 447.6 284.0	120.3	-113.2		
26	18.38	447.6	25.7	-117.7		
27	19.09	284.0	-72.9	-122.3	-240.3	
28	19.80	133.3	-174.7	-126.9		
29	20.51	-17.3 -157.7	-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 -145.2	-258.6	
32	22.63	-356.6	-746.9	-145.2	-263.2	
33	23.33			-149.8	-267.8	
34	24.04	-568.4	-1122.0	133.4	26.0	
35	24.75	-389.9 -214.5	-789.9 -504.8	467.4 462.8	250.2	
36	25.46	-214.5	-504.8	462.8		
37	26.16	62.0 365.1	-273.8	458.2	241.1	
38	26.87	365.1	-57.8	247.0	128.4	
39	27.58	382.0 396.5	-22.0	48.3 43.7	15.8	
40		396.5	10.5			
41	28.99		39.8	39.2	6.6	
42	29.70		56.8		2.0	
43	30.41	423.9	68.2 76.3	30.0	-2.5	
44	31.11	426.8	/6.3	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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TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA	DIST X	MAX + M	IOM MA	X - MOM	MAX + SHEAR	MAX - SHEAR
		(FT-K) (F				
49	34.65	423.9	68.2	2.5	-30.0	
50	35.36	417.7 408.3	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	382.0 365.1	-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 -389.9 -568.4	-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	-460.9 -356.6 -255.5	-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	-17.3 133.3	-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	447.6 609.7	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	926.2 1080.5 939.3	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	796.2 650.7 503.5	151.1	-115.5	-221.3	
74	52.33	503.5	67.8	-120.1	-225.9	
75	53.03	354.3	-/1.1	-124.6	-230.5	
76	53.74	354.3 204.3 53.6	-228.2	-129.2	-235.0	
78	54.45	07.0	-300.3	120.0	-239.0	
79	55.15	-97.2 -250.4 -406.9	-332.U -710.7	1/2 0	-244.2	
80	56.57	-230.4	9007	-142.5	-240.0	
21	57.28	-565.0	-1061.7	-152.1	-257.9	
87	57.20	-565.0 -714.9 -537.8	-1238 4	93.0	12.6	
83	58.60	-714.9	-020.4	422.0	248.2	
84	50.03	-363.9	-625.6	128.7	243.6	
85	60.10	-193.3	-324.2	424.1	239.0	
86	60.81	-193.3 -25.9	-25.9	218.9	126.4	
	61.52	-14.6	-14.6	13.7	13.7	
88	62.23	-6.5	-6.5	9.2	9.2	
89	62.93	-6.5 -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

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TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

STA	DIST X	XAM)	+ MÓM	MAX - M	OM MAX -	+ SHEAR	MAX - SH	HEAR
(FT)	(FT-K)	(FT-K)	(K)	(K)			
93	65.76	0.0	0.0	0.0	0.0			

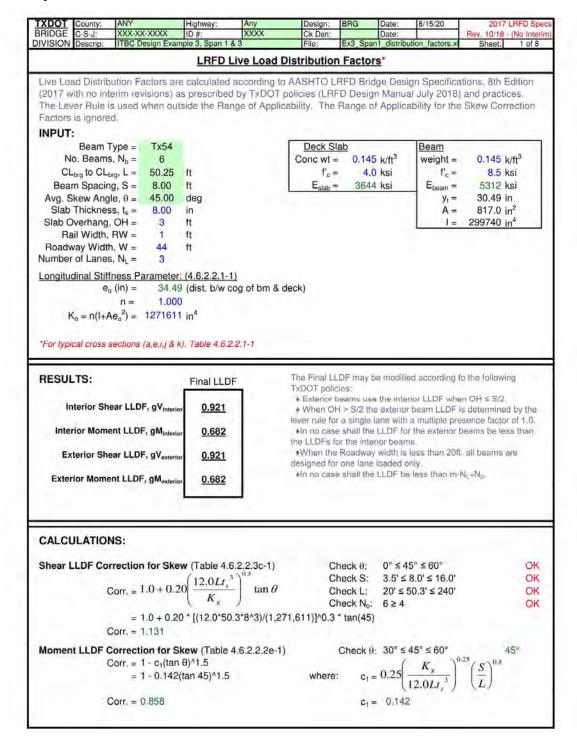
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TABLE 7. MAXIMUM SUPPORT REACTIONS (LOAD FACTOR)

STA	DIST X	MAX +	REACT	MAX - REACT
(FT)	(K)	(K)	
				-
10	7.07	689.7	409.	.4
34	24.04	741.8	409	0.2
58	41.01	741.8	409	0.2
82	57.98	689.7	409	0.4

4.4.15.4 <u>Live Load Distribution Factor Spreadsheet</u>

4.4.15.4.1 Spans 1 & 3

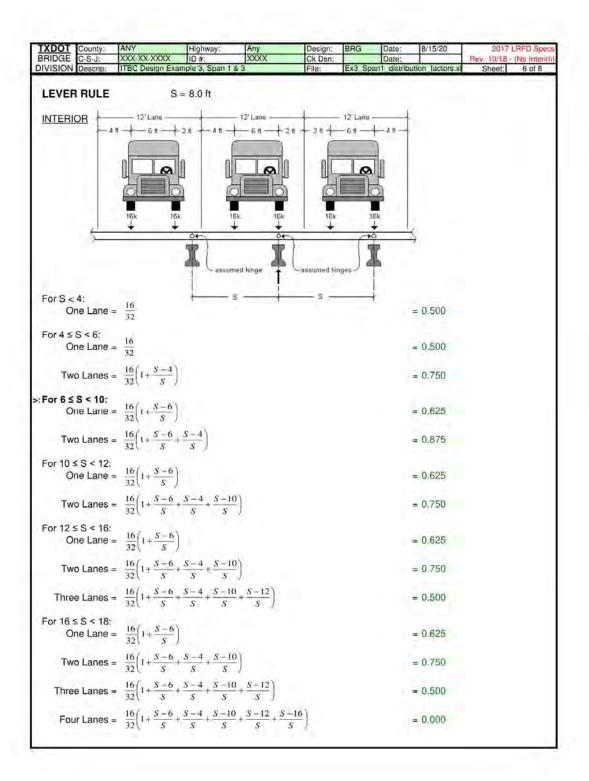


TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		RFD Spec
BRIDGE DIVISION	C-S-J: Descrip:	ITBC Design Exa	ID #: mple 3, Span 1 &	3	Ck Dsn: File:	Ex3 Span	Date:	tion factors.xl	Rev. 10/18 - Sheet:	(No Interin 2 of 8
INTER	IOR BE	AM:								
Shear I	LL Distrib	ution Per Lane	Table 4.6.2.2.	3a-1):						
	One La	ine Loaded								
		Lever Rule	(Table 3.6.1	.1.2)						
		mg = 0.6	625 * 1.2 =	0.750						
		Modify for	or Skew:							
			skew correc		1.131					
		A4 1505	mg = 0.750	* 1.131 =	0.848					
		$\frac{\text{Equation}}{g = 0.36}$	$6 + \left(\frac{S}{25}\right)$							
				0.000						
			+ (8 / 25) = or Skew:	0.680						
		woully it	skew correc	tion -	1.131					
			g = 0.680 *		0.769					
		Range of App	licability (ROA							
			3.5' ≤ 8.0' ≤		ОК					
			4.5" ≤ 8.0"		OK					
		Check L	20' ≤ 50.3'	≤ 240'	OK					
		Check N	l _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 Lo	paded							
		Lever Rule	(Table 3.6.1	.1.2)						
		200	ax(0.875 * 1.0, or Skew:	0.875 * 0.8	35, 0.875 * 0	.65) =	0.875			
			skew correc	ction =	1.131					
			mg = 0.875	* 1.131 =	0.990					
		Equation $g = 0.2$	$+\left(\frac{S}{12}\right) - \left(\frac{S}{2}\right)$	2.0						
			(12) (3:	25/42.0	0.014					
			or Skew:	35)~2.0 =	0,814					
		woully it	skew correc	ction =	1,131					
			g = 0.814 *		0.921					
		Range of App	licability (ROA		The second second	or one lar	ne loade	d)		
			from Table 4.					-/		
		gV _{int2+} =	0.921							
	TXDOT	Policy states g\								
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6	=	0.425					
		20ft ? Yes		-v/		-117	249-11	V 86		
		Policy states the								
>>		Policy states the	at ii W ≥ 20ft, ;	g V Interior 15 t	ne Maximun	of: gV _{in11}	gV _{int2+1}	m·N _L ÷N _b .		
	gV _{inte}	erior = 0.921								

BRIDGE	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	I have the law.	LRFD Spe
IVISION	C-S-J: Descrip:	ITBC Design Exam	ID #: ple 3. Span 1 & 3	XXXX	Ck Dsn:	Ex3 Span	Date:	ion factors.xl	Rev. 10/18 - Sheet:	3 of 8
NTER	IOR BE		pic of open i o a		11 1101	Life Option		TOTAL TRANSPORT	Oncot.	00.0
		ibution Per Lane	(Table 4 6 2 3	2h-1*						
Women		ne Loaded	(Table 4.0.2.2							
	One La		(Table 2.6.1	1 2)						
		Lever Rule mg = 0.62	(Table 3.6.1.	0.750						
				0.750						
		Modify for		Tana	0.050					
			skew correct mg = 0.750 *		0.858					
		en 1815	mg = 0.750	0.000 =	0.644					
		$\frac{\text{Equation}}{\text{g} = 0.06}$	$+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.4}$	$\int_{0.3}^{0.3} \left(\frac{K_g}{12Lt} \right)^{0.3}$	3					
			(8/14)^0.4 * (A		11/(12*50	.3*8^3))	0.1 =	0.591	
		Modify for		S. 3.2650 T. 2	O Salare	40.5555				
		1,455,6	skew correct	ion =	0.858					
			g = 0.591 * 0	.858 =	0.507					
		Range of Appli			-					
			3.5' ≤ 8.0' ≤			ОК				
			4.5" ≤ 8.0" ≤			OK				
			20' ≤ 50.3' ≤			ОК				
		Check N _b		2.17)		OK				
		10 10 10 10	10,000 ≤ 1,2	71.611≤7	000.000	OK				
		Use Equation f				4-6-	Tik-			
		gM _{int1} =	0.507	E-E-EN-1 NO	cause an c	illelia is	Q/No.			
	ATT 1.50		4 4							
	Two or	More Lanes Loa		2.41						
		Lever Rule	(Table 3.6.1.	107-1-17						
			(0.875 * 1.0, 0	0.875 * 0.85	, 0.875 * 0.	65) =	0.875			
		Modify for			2,340					
			skew correct		0.858					
			mg = 0.875 *	0.858 =	0.751					
		$\frac{\text{Equation}}{\text{g = 0.07}}$	$5 + \left(\frac{S}{9.5}\right)^{0.6} \left($	$\left(\frac{S}{L}\right)^{0.2} \left(\frac{K}{12L}\right)^{0.2}$	$\left(\frac{\zeta_g}{t^{3}}\right)^{0.1}$					
		a = 0.075	+ (8/9.5)^0.6	(8/50.3)^0	2 * (1.271.	611/(12*5	50.3*8^3))^0.1 =	0.795	
		Modify for		(4.55.5)	- 10-10			,,	.,	
			skew correct	ion =	0.858					
			g = 0.795 * 0		0.682					
		Range of Appli				or one lar	ne loade	d)		
		Use Equation f	rom Table 4.6	2.2.2b-1 be	cause all c	riteria is (OK.			
		gM _{int2+} =	0.682							
	TYDOT			M. M. m						
	INDO	Policy states gM m·N _L ÷N _b =			0.425					
	1-14/		0.85 * 3 / 6 =		0.425					
		20ft ? Yes	(r.W = 200 ~	M. in the	a Maximum	Maria	and m	N NI		
		Policy states that								
>>	-	Policy states that	n vv ≥ 20n, g	Winterior IS (Ne	= waximum	or giving	giviniz-	111-14/T-JAPA		
	gM _{inte}	erior = 0.682								

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	A CONTRACTOR OF THE PARTY OF TH	.RFD Specs
BRIDGE	C-S-J: Descrip:	ITBC Design Exa	ID#:	XXXX	Ck Dsn: File:	Ex3 So	Date:	ution factors.x	Rev. 10/18 - Sheet:	(No Interim) 4 of 8
	RIOR BE		anpic of open i		Ji ild.	Lau op	diti distrib	DUDIT_IGUIDIDIA	Sidet.	7010
		ution Per Lane	(Table 4.6.2.)	2.3b-1):						
<u> </u>		ne Loaded	Tradio Trainin	17.						
		Lever Rule	(Table 3.6	.1.1.2)						
			625 * 1.0 =		TxDOT us	es a mu	Itiple pre	sence factor	r of 1,0 for or	ne:
			or Skew:		lane loade				5. 114 151 51	
			skew corre	ection =	1,131					
			mg = 0.62	5 * 1.131 =	0.707					
		Use Lever Ru	ile, as per AA	SHTO LRF	D Table 4.6.2	2.2.3b-1	1			
		gV _{ext1} =	0.707							
	Two or	More Lanes L	oaded							
		Lever Rule	(Table 3.6	.1.1.2)						
		mg = M	ax(0.625 * 1.0	0, 0.625 * 0.	85, 0.625 * 0	.65) =	0.625			
			or Skew:							
			skew corre	ection =	1.131					
			mg = 0.62	5 * 1.131 =	0.707					
		Equation								
		$d_e = dis$	t. b/w CL web	to curb						
			- Rail Width							
			3ft - 1ft =	2.0	tt					
		e = 0.6	$+\left(\frac{d_e}{10}\right)$							
		e = 0.6	+ (2.0/10) =	0.800						
		g = e*g\	V _{int2+Eq}							
			00 * 0.921 =	0.737						
		Skew C	orrection is in	cluded in g	V(interior).					
		Range of App	olicability (RO	A) Checks	Interior	ROA is	implicitly	applied to t	he exterior b	eam.
		Check I	nterior Beam	ROA:	OK					
		Check of	d _e : -1.0' ≤ 2.0	' ≤ 5.5'	OK					
		Check N	N _b : 6 ≠ 3		OK					
		Use Equation	from Table 4	1.6.2.2.3b-1	because all	criteria i	s OK.			
		$gV_{ext2+} =$	0.737							
	TXDOT	Policy states g	V _{Exterior} must b	e ≥ gV _{interior}						
		gV _{interior} =	0.921							
	TXDOT	Policy states g	V _{Exterior} must b	e ≥ m·N _L ÷N	b					
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6	S =	0.425					
		S/2 ? Yes								
		20ft ? Yes	-111011-010	i di i	-14					
>>		Policy states th				a tha AA	a til matter	de all al	/ nad	
	IXDOI	Policy states th m·N _L ÷N _b .	at II OH > 5/a	and w<2	UIT, GV Exterior	s trie ivi	aximum (or, gv _{ext1} , gv	interior and	
	TYDOT	Policy states th	at if OH > QI	2 ans W > 2	Off aV-	s the M	avimum c	it aV aV	a av	
	TADOT	and m·N _L ÷N _b		alle VV = Z	Ore St. Exterior	o tile Mi	unitiditi C	A extl. A	ext2+> 9 v interior	6.1
	gV _{exte}									
	∠ y v exte	nor - 0.321								

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		RFD Spec
BRIDGE	C-S-J: Descrip:	ITRC Design Ex	ID #: ample 3, Span 1 &	XXXX	Ck Dsn: File:	Fx3 Sn	Date:	ution factors.x	Rev. 10/18 - Sheet:	5 of 8
	RIOR BE		ample of open 1 a		Triid.	LNU OF	uiti usula	DUCTI INDICTOR	Sildet.	5 01 0
		ribution Per Lar	ne (Table 4.6.2	2 24-11						
Momen		ne Loaded	ic (Table 4.0.2	.E.Ed Tj.						
	One Eu	Lever Rule								
			625 * 1.0 =	0.625	TypoTus	es a mi	Itinle pre	sence factor	r of 1,0 for or	10
		3.450.0000	for Skew:		lane loade				0 110 101 01	10
			skew correc	ction =	0.858					
			mg = 0.625		0.536					
		Use Lever Ri	ule as per AAS		Table 4.6.2	2.2d-1.				
		gM _{ext1} =	0.536							
	Tue or	More Lanes L	-							
	I WO OF	Lever Rule	(Table 3.6.1	(1.0)						
			ax(0.625 * 1.0,		DE 0 605 * 0	CE)	0.625			
		7071.0	for Skew:	0.025 0.0	55, 0.025 0	.05) =	0.023			
		widelity	skew correc	ction -	0.858					
			mg = 0.625		0.536					
		Equation	111g = 0.020	0.000 =	0.000					
			(d)							
		e = 0.7	$7 + \left(\frac{d_e}{9.1}\right)$							
			7 + (2.0/9.1) =		0.990					
					4144					
		g = e*gl	9 * 0.682 =	0.675						
			orrection inclu		ntorior)					
			olicability (ROA			DOA:-	tomorphism (16)	manifed to t	ka alikadas k	
			nterior Beam F		OK	HUAIS	ппристу	applied to t	he exterior b	eam.
			d _e : -1.0' ≤ 2.0'		OK					
			V _b : 6≠3	≥ 5.5	OK					
			from Table 4.	622241		aritaria l	COK			
		gMext2+ =	0.675	0.2.2.20-1	Decause all	interia i	SUN			
		O UNIK	-	7.5 mg						
	TXDOT	Policy states gl		e ≥ gM _{interior}						
	T 00T	gM _{interior} =	0.682							
	TXDOT	Policy states g	, million in the control of the cont							
			0.85 * 3 / 6	=	0.425					
		S/2 ? Yes								
		20ft ? Yes Policy states th	at if OH < S/2	aM- is	aM					
>>		Policy states th				ie the M	avimum	of: aM . al	M and	
	INDOI	m·N _L ÷N _h .	Idt II OI I > O/E	did W < E	ort, Blackderler	io tilo ivi	axiiiuiii	or. giviexiti gi	Winterior: Circ	
	TYDOT	Policy states th	at if OH > S/2	ans W > 20	Off aM-	is the M	avimum i	of: aM al	M. aM	
	TAWST	and m·N _L +N _b		W. 11 - 21	Au Au Ellelloi	eric ivi	SANTIGITI I	an Binesti A	inexisti Aurinien	DET
	gM _{ext}									
	giviext	enor - 0.002								



TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Spe
BRIDGE		XXX-XX-XXXX	ID #: ample 3, Span 1 &	XXXX	Ck Dsn: File:	Eu2 Con	Date:	ion factors.xl	Rev. 10/18 - Sheet:	7 of 8
IVISIOIV	Descrip.	I TOO Dealgh Ex	ample o, opan i o	J.	Trile,	LAU OPA	III QISUIDDI	ion idulois.a	Sileet.	7 01 0
LEVER	RULE	5	S = 8.0 ft							
INTERI	OR (con't)								
For 18 5	S < 22: ne Lane =	$= \frac{16}{32} \left(1 + \frac{S-6}{S} \right)$)				r i	0.625		
Tw	Lanes =	$= \frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$)			03	0.750		
Three	e Lanes =	$= \frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S-12}{S}$	- <u>18</u>)			-0.125		
Fou	r Lanes =	$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S-12}{S}$	$\left(\frac{-18}{S} + \frac{S-16}{S}\right)$			0.625		
For 22 s	S ≤ 24; ne Lane =	$= \frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$)				179	0.625		
Twe	Lanes =	$= \frac{16}{32} \left(1 + \frac{S-6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$)				0.750		
Three	e Lanes =	$= \frac{16}{32} \left(1 + \frac{S-6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S}{S}$	$\left(\frac{-18}{S}\right)$		10	-0.125		
Fou	r Lanes =	$= \frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S}{S}$	$\frac{-18}{S} + \frac{S - 16}{S}$	$+\frac{S-22}{S}$	l lu	-1.500		
		16k	4 ft - 2 ft	J.Sk.	41-				\$ = OH =	8.0 f
		он 🕴 х	s	- Sewimen	1 hinge			Rail Widti X = S+OH-	n = RW =	1.0 ft 8.0 ft
For X <	6: ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} \right)$					1.4	0.500		
For 6 ≤ Or	X < 12; ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	6					0.625		
For 12 s	X < 18; ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	-6)					0.625		
Twi	lanes =	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X}{S}\right)$	6 + X -12					0.375		

RIDGE	County:	ANY XXX-XX-XXXX	Highway:	Any XXXX	Design: Ck Dsn:		Date: 8.	15/20	2017 LRFD Spec
IVISION			ample 3, Span 1		File:	Ex3 Span1		factors.xl	Sheet: 8 of 8
	200								
LEVER	RULE								
EXTER	IOR (con'	t) S	= 8.0 f	ř.	OH =	3.0 ft			
LAILI	ion (con	RW			DH-RW-2ft =				
27770	100								
For 18 5	X < 24:	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	-6)				- 0	.625	
0	ie Lane =	32 5 5	1				= 0	.023	
Two	Lanes =	$=\frac{16}{32}\left(\frac{X}{S}+\frac{X}{S}\right)$	$\frac{-6}{4} + \frac{X - 12}{6} + \frac{X}{2}$	(-18)			= -	0.250	
		321 8 8	2	S)					
For 24 s	X < 30:	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	-6)					COL	
Of	ne Lane =	32 S S)				= 0	.625	
Twe	o Lanes -	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X}{S}\right)$	$\frac{6}{4} + \frac{X - 12}{4} + \frac{3}{2}$	(-18)			= -	250	
3		32\ S S	S	S				J.E.F.	
Three	e Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	$\frac{-6}{e} + \frac{X-12}{e} + \frac{X}{2}$	$\frac{X-18}{9} + \frac{X-2}{9}$	4)		4.	1.250	
		32(3 3	3	9 3	5.				
For 30 s	X < 36:	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X}{S}\right)$	6)				- 0	.625	
		32 (3	1				- 0	.02.0	
Two	Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{6}{100} + \frac{X - 12}{100} + \frac{X}{100}$	(-18)			-	0.250	
		State of the		COVE A C	V 45 -003				
Three	e Lanes =	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X-X-Y}{S}\right)$	$\frac{-6}{s} + \frac{x-12}{s} + \frac{2}{s}$	$\frac{C-18}{S} + \frac{X-2}{S}$	$\frac{4}{s} + \frac{x - 30}{s}$		= ~	2.625	
Ear 26 4	V = 10	35.43			-				
Or	ne Lane =	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X-X}{S}\right)$	-6				= 0	.625	
Two	Lanes =	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X-X}{S}\right)$	$\frac{6}{5} + \frac{X - 12}{5} + \frac{2}{5}$	$\left(\frac{C-18}{S}\right)$			= 4	0.250	
		16/ Y Y	6 V-12 1	C-18 X-2	4 Y - 30 \				
Three	e Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	$+\frac{x-12}{s}+\frac{x}{s}$	S + S	+ 2 30		= 1	2.625	
5.5		16 (X X -	6 X-12 2	C-18 X-2	4 X - 30	X -36)		Cole.	
Fou	r Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	5	S + S	S +	5	= *	4.375	
For 42 s	≤ X ≤ 48:	Le d'air in	61						
Or	ne Lane =	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X-X}{S}\right)$	-6				= 0	.625	
		16/X X-	6 X - 12 X	(-18)					
Two	Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	+ 3 + 2 + 3	5			-	0.250	
The	Llaces	16 (X . X -	6 X-12 D	C-18 X-2	4 (X - 30)			cor	
Inre	e Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	S	s s	S		= -	2.625	
Fou	r Lanes -	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	6 X-12 2	x - 18 + x - 2	4 + X = 30 +	X - 36 + X	-42	8 500	
, 00	Luiles -	32\S S	S	S S	S	S	s) -		
INTERIO	OR				EXTER	IOR			
	ne Loade	d	= 0.625			ne Loaded		-	0.625
	nes Load		= 0.875			nes Loade		-	0.625
	anes Loa		= 0.875			anes Load			0.625
	nes Load		= 0.875			nes Loade			0.625
Our La	nes Ludu	Cu	0.073		i oui La	iles LUdue	u	=	0.020

4.4.15.4.2 *Span 2*

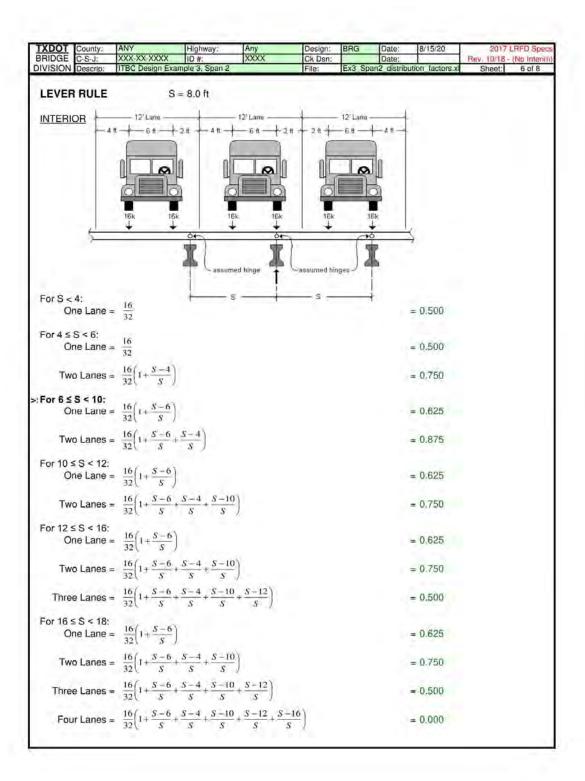
TXDOT County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Sp
BRIDGE C-S-J: DIVISION Descrip:	ITBC Design Exar	ID #: noie 3. Span 2	XXXX	Ck Dsn: File:	Ex3 Spar	Date:	ution factors.xl	Rev. 10/18 Sheet:	1 of 8
Tribinal Landson Principle			ive Load Di					Onderi	
Live Load Distribu	ution Factors ar						an Specifica	ations 8th F	dition
(2017 with no inte			100			545			
The Lever Rule is	s used when ou	tside the Ran	ge of Applica	bility. The	Range o	f Applica	ability for the	Skew Con	rection
Factors is ignored	d.								
INPUT:									
Beam T	ype = Tx54		- 1	Deck SI	ab		Beam		
No. Beams				Conc wt =	0.145	k/ft ³	weight =	0.145	c/ft ³
CL _{brg} to CL _b	$_{rg}$, L = 106.50	ft		f'c =		ksi	f'c =	8.5	ksi
Beam Spacing		ft		E _{slab} =	3644	ksi	E _{beam} =	5312	
Avg. Skew Angl		deg					$y_t =$		
Slab Thicknes		in					A =		
Slab Overhang,		ft					l=	299740 j	n*
Rail Width,		ft							
Roadway Width		ft							
Number of Lanes									
Longitudinal Stiffe									
e _o		9 (dist. b/w co	og of bm & de	eck)					
	n = 1.00								
$K_0 = n(I+A)$	e_a^2) = 127161	1 in ⁴							
are the second s	mark on branch & all	Other Transfer and							
For typical cross s	ections (a,e,l.j & l	k). Table 4.6.2.	2.1-1						
*For typical cross s	ections (a,e,i,j & i	k). Table 4.6,2.	2.1-1						
SCOTT AS	ections (a,e.i.j & i	30.27		The First (DE SOUR	a nemiddi		te the fathers	40
RESULTS:	ections (a,e,i,j & i	Final LLDF				e modifie	ed according	to the followi	ng
RESULTS:		Final LLDF		TxDOT polic	cies;		ed according		ng
RESULTS:	ear LLDF, gV _{interio}	Final LLDF	1	* Exterior b	cies; ceams use d > S/2 the	the inter	rior LLDF who	en OH ≤ S/2. is determine	d by the
RESULTS:	ear LLDF, gV _{interio}	Final LLDF	1	* Exterior b * When Ob-	cies; peams use d > S/2 the r a single l	the inter e exterior ane with	nior LLDF who beam LLDF a multiple pro	en OH ≤ S/2. is determine esence facto	d by the
RESULTS:		Final LLDF		TxDOT polic Exterior t When Of lever rule for In no case	cies; beams use d > S/2 the r a single l e shall the	the interest exterior and with	rior LLDF who beam LLDF a multiple pro r the exterior	en OH ≤ S/2. is determine esence facto	d by the
RESULTS: Interior She	ear LLDF, gV _{interio}	Final LLDF 0.947 0.596		TxDOT police * Exterior to * When Officer rule for *In no case the LLDFs for	cies; beams use of > S/2 the r a single l a shall the or the inte	the interest exterior and with	rior LLDF who beam LLDF a multiple pro r the exterior	en OH ≤ S/2. is determine esence facto beams be les	d by the r ol 1.0. ss than
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RESULTS: Interior She Interior Mome Exterior She Exterior Mome CALCULATION Shear LLDF Cor	ear LLDF, gV _{interio} ear LLDF, gM _{interio} ear LLDF, gM _{exterio} ear LLDF, gV _{interio} ear LLDF, gM _{exterio}	Final LLDF 0.947 0.596 0.947 0.596 ew (Table 4.6. $20 \left(\frac{12.0 Lt_s^3}{K_g} \right)$	2.2.2.3c-1) $\int_{0.3}^{0.3} \tan \theta$ 3.5*8^3)/(1,27)	TxDOT polic Exterior t When OI In no cass the LLDFs fr When Ihe designed for In no cass CC CC CC	peams uses a single a shall the part the interpretation of the in	e the interest of the interes	tior LLDF white beam LLDF a multiple profit the exterior is: less than 20f nly. less than more less than less	en OH < S/2: is determine: esence facto beams be les t, all beams . N _L +N _t ,	OK OK
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RESULTS: Interior She Interior Mome Exterior She Exterior Mome CALCULATION Shear LLDF Cor	ear LLDF, gV _{interio} ear LLDF, gM _{interio} ear LLDF, gV _{exterio} ear LLDF, gM _{exterio} ear LLDF, gV _{interio} ear LLDF, gM _{exterio}	Final LLDF 0.947 0.596 0.947 0.596 0.947 0.596 0.947 0.596 0	2.2.2.3c-1) $\int_{0.3}^{0.3} \tan \theta$ 3.5*8^3)/(1,27)	TxDOT polic Exterior to the When Oil When Oil In no case the LLDFs from the Mesigned for the Inno case C C C C C C C C C C C C C C C C C C C	cles; beams use; 1 > S/2 thir r a single lie shall the brithe inte Roadway r one lane a shall the heck 0: heck S: heck L: heck N _b : 3 * tan(45	e the interest of the interes	tior LLDF white beam LLDF a multiple program the exterior is less than 20f nly. 10	en OH < S/2: is determine: esence facto beams be les t, all beams . N _L +N _t ,	OK OK
RESULTS: Interior She Interior Mome Exterior She Exterior Mome CALCULATION Shear LLDF Cor	ear LLDF, gV _{interio} ear LLDF, gM _{interio} ear LLDF, gV _{exterio} ear LLDF, gM _{exterio} ear LLDF, gV _{interio} ear LLDF, gM _{exterio}	Final LLDF 0.947 0.596 0.947 0.596 0.947 0.596 ew (Table 4.6. $20\left(\frac{12.0Lt_s^3}{K_g}\right)$ 0 * [(12.0*106) skew (Table 4.6.	2.2.2.3c-1) $\int_{0.3}^{0.3} \tan \theta$ 3.5*8^3)/(1,27)	TxDOT polic Exterior t When OI I lever rule for In no case the LLDFs fr When the designed for In no case C C C	cles; beams use; 1 > S/2 thir r a single lie shall the brithe inte Roadway r one lane a shall the heck 0: heck S: heck L: heck N _b : 3 * tan(45	e the interest of the interes	tior LLDF white beam LLDF a multiple profit the exterior is: less than 20f nly. less than more less than less	en OH < S/2: is determine: esence facto beams be les t, all beams . N _L +N _t ,	OK OK
RESULTS: Interior She Interior Mome Exterior Mome Exterior Mome CALCULATION Shear LLDF Cor	ear LLDF, gV _{interio} ear LLDF, gM _{interio} ear LLDF, gV _{exterio} ear LLDF, gM _{exterio} ear LLDF, gV _{interio} ear LLDF, gM _{exterio}	Final LLDF 0.947 0.596 0.947 0.596 0.947 0.596 0.947 0.596 0	2.2.2.3c-1) $\int_{0.3}^{0.3} \tan \theta$ 3.5*8^3)/(1,27)	TxDOT polic Exterior to the When Oil When Oil In no case the LLDFs from the Mesigned for the Inno case C C C C C C C C C C C C C C C C C C C	cies; ceams use; > S/2 thir r a single a shall the or the inte Roadway r one lane a shall the heck θ: heck S: heck L: heck N _b : 3 * tan(45 Check θ : C ₁ =	e the interest of the interes	for LLDF which beam LLDF a multiple prior the exterior is less than 20 fully. It less than multiple prior the exterior is less than multiple prior the exterior is less than multiple prior that the exterior than the exterior tha	en OH < S/2: is determine: esence facto beams be les t, all beams . N _L +N _t ,	OK OK

DIDOL	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		.RFD Spe
RIDGE	C-S-J: Descrip:	ITBC Design Exam	ID #: nple 3, Span 2	XXXX	Ck Dsn: File:	Ex3 Spar	Date: n2 distribu	tion factors.xl	Rev. 10/18 - Sheet:	2 of 8
	OR BE									
Shear L	L Distrib	ution Per Lane (Table 4.6.2.2	.3a-1):						
		ne Loaded								
		Lever Rule	(Table 3.6.	1.1.2)						
			25 * 1.2 =	0.750						
		Modify fo								
		120	skew corre	ction =	1.164					
			mg = 0.750	* 1.164 =	0.873					
		Equation	100							
		g = 0.36	$+\left(\frac{S}{25}\right)$							
		g = 0.36	+ (8 / 25) =	0.680						
		Modify to	r Skew:							
			skew corre	ction =	1-164					
			g = 0.680 *	1.164 =	0.792					
		Range of Appl	icability (RO	A) Checks						
			3.5' ≤ 8.0'		OK					
			4.5" ≤ 8.0"		OK					
			20' ≤ 106.5	5' ≤ 240'	OK					
		Check N	, 6≥4		OK					
		Use Equation		.6.2.2.3a-1 l	because all o	criteria is	OK.			
		gV _{int1} =	0.792							
	Two or	More Lanes Lo	aded							
		Lever Rule	(Table 3.6.	1.1.2)						
		mg = Ma: Modify fo	k(0.875 * 1.0 r Skew:	, 0.875 * 0.8	5, 0.875 * 0	.65) =	0.875			
			skew corre	ction =	1.164					
			mg = 0.875	* 1.164 =	1.019					
		Equation	1011	2.0						
		g = 0.2 -	$-\left(\frac{3}{12}\right) - \left(\frac{3}{2}\right)$	5						
		0-02:	(8 / 12) - (8 /	35\\\2 0 -	0.814					
		Modify fo		00) 2.0 -	0,014					
		widaiiy to	skew corre	ction =	1.164					
			g = 0.814 *		0.947					
		Range of Appl			(same as f	or one la	ne loade	d)		
		Use Equation						٠,		
			0.947	ore resident	zedude un c	1110110110	O			
	TXDOT	Policy states gV	nterior must be	$\geq m \cdot N_L + N_b$						
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6	=	0.425					
		20ft ? Yes	August and							
		Policy states tha								
>>		Policy states tha	t if W ≥ 20ft,	gV _{Interior} is the	ne Maximum	of: gV _{int1}	gV _{int2+}	$m \cdot N_L \div N_b$.		
	gV _{inte}	rior = 0.947	1							

SERIOGE Section Series SXXXXXXXXXX 10 st. SXXXXXXXXX SXXXXXXXXX SXXXXXXXXXX	TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017	LRFD Spec
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 0.889 Equation g = 0.06 + $\left(\frac{S_1}{I_1}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_x}{I_1^2 L I_x}\right)^{0.1}$ 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 * 516.0 OK Check L; 20* \$ \$ 106.5 \$ \$ 240 OK Check L; 20* \$ \$ 106.5 \$ \$ 240 OK Check K; 10.00 \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$	BRIDGE				XXXX		Ev2 So		tion factors vi		
Moment LL Distribution Per Lane (Table 4.6.2.2.2b-1): One Lane Loaded Lever Rule (Table 3.6.1.1.2) $mg = 0.625^{\circ} 1.2 = 0.750$ Modify for Skew: $skew$ correction = 0.919 $mg = 0.750^{\circ} 0.919 = 0.689$ Equation $g = 0.06 + (S \cdot \frac{1}{14})^{0.4} (\frac{S}{L})^{0.3} (\frac{K_e}{12LI_s})^{0.1}$ $g = 0.06 + (8/4)/0.4^{\circ} (8/106.5)/0.3^{\circ} (1.271.611/(12^{\circ}106.5^{\circ}8^{\circ}3))/0.1 = 0.453$ Modify for Skew: $skew$ correction = 0.919 $g = 0.453^{\circ} 0.919 = 0.416$ Range of Applicability (ROA) Checks Check S: 3.5' s 8.0' s 16.0' Check L: 20' s 108.5' s 240' Check N ₃ : 6 2 4 OK Check N ₃ : 7 1.0.0875 \cdot 0.85.0.875 \cdot 0.65) = 0.875 Modify for Skew: $skew$ correction = 0.919 $mg = 0.875^{\circ} 0.919 = 0.804$ Equation $g = 0.075 + (g.9.5)^{\circ} 0.6^{\circ} (g.106.5)^{\circ} 0.2^{\circ} (1.271.611/(12^{\circ}106.5^{\circ}8^{\circ}3))^{\circ} 0.1 = 0.649$ Modify for Skew: $skew$ correction = 0.919 $g = 0.075 + (g.9.5)^{\circ} 0.6^{\circ} (g.106.5)^{\circ} 0.2^{\circ} (1.271.611/(12^{\circ}106.5^{\circ}8^{\circ}3))^{\circ} 0.1 = 0.649$ Modify for Skew: $skew$ correction = 0.919 $g = 0.416$ Equation $g = 0.075 + (g.9.5)^{\circ} 0.6^{\circ} (g.106.5)^{\circ} 0.2^{\circ} (1.271.611/(12^{\circ}106.5^{\circ}8^{\circ}3))^{\circ} 0.1 = 0.649$ Modify for Skew: $skew$ correction = 0.919 $g = 0.643^{\circ} 0.919 = 0.596$ Range of Applicability (ROA) Checks (same as for one lane loaded) Use Equation from Table 4.6.2.2.2.b-1 because all criteria is OK. $gM_{m2} = 0.0596$ TxDOT Policy states that if W < 20ft, gM _{matter} is the Maximum of: gM _{m2} , and $m \cdot N_{a} + N_{b}$. $m \cdot N_{a} \cdot N_{b} = 0.85^{\circ} 3 / 6 = 0.425$ Is W ≥ 20ft? Yes TxDOT Policy states that if W < 20ft, gM _{matter} is the Maximum of: gM _{m2} , and $m \cdot N_{a} + N_{b}$.		100000		ipie o, opan z		Trile,	JENO OP	anz distribu	non igulore.si	. Stieet.	3010
One Lane Loaded Lever Rule (Table 3,5,1.1.2) $mg = 0.625 \cdot 1.2 = 0.750$ Modify for Skew: $skew correction = 0.919$ $mg = 0.750 \cdot 0.919 = 0.689$ $\frac{Equation}{g} = 0.06 + \left(\frac{11}{14}\right)^{04} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12L_L^3}\right)^{0.1}$ $g = 0.06 + (8/14)^{4}0.4 \cdot (8/106.5)^{4}0.3 \cdot (1,271,611/(12^{4}106.5^{8}A^{3}))^{4}0.1 = 0.453$ Modify for Skew: $skew correction = 0.919$ $g = 0.453 \cdot 0.919 = 0.416$ Range of Applicability (ROA) Checks Check \$1.5 \cdot 8.0 \cdot 9.16.0 \cdot 0K Check \$1.5 \cdot 9.0 \cdot 10.6.5 \cdot 9.2 \cdot 20 \cdot 0K Check \$1.5 \cdot 9.0 \cdot 10.6.5 \cdot 9.2 \cdot 20 \cdot 0K Check \$1.5 \cdot 9.0 \cdot 10.6.5 \cdot 9.2 \cdot 10.6.5 \cdot 9.2 \cdot 10.6.5 \cdot 9.2 \cdot 9.0 \cdot 0K Check \$1.5 \cdot 9.5 \cdot 9.0 \cdot 9.2 \cdot 10.6.5 \cdot 9.2 \cdot 9.0 \cdot 9.				(Table 4.6.)	2.2.2b-1):						
	Monto			Tradio Hon	17.						
Modify for Skew:			Lever Rule	(Table 3.6	.1.1.2)						
$ \begin{aligned} & \text{skew correction} = & 0.919 \\ & \text{mg} = 0.750 \cdot 0.919 = & 0.689 \end{aligned} $ $ \begin{aligned} & \text{Equation} \\ & \text{g} = 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_S}{12L_I}\right)^{3} \cdot 1 \\ & \text{g} = 0.06 + (8/14)^{0.4} \cdot (8/106.5)^{0.3} \cdot (1.271,611/(12^{*}106.5^{*}8^{*}3))^{*}0.1 = 0.453 \end{aligned} $ $ \begin{aligned} & \text{Modify for Skew:} \\ & \text{skew correction} = & 0.919 \\ & \text{g} = 0.453 \cdot 0.919 = & 0.416 \end{aligned} $ $ \begin{aligned} & \text{Range of Applicability (ROA) Checks} \\ & \text{Check } \mathbf{k}: & 4.5^{\circ} \leq 8.0^{\circ} \leq 15.0^{\circ} & \text{OK} \\ & \text{Check } \mathbf{k}: & 4.5^{\circ} \leq 8.0^{\circ} \leq 12.0^{\circ} & \text{OK} \\ & \text{Check } \mathbf{k}: & 4.5^{\circ} \leq 8.0^{\circ} \leq 12.0^{\circ} & \text{OK} \\ & \text{Check } \mathbf{k}: & 20^{\circ} \leq 106.5^{\circ} \leq 240^{\circ} & \text{OK} \\ & \text{Check } \mathbf{k}: & 20^{\circ} \leq 106.5^{\circ} \leq 240^{\circ} & \text{OK} \\ & \text{Check } \mathbf{k}: & 20^{\circ} \leq 106.5^{\circ} \leq 240^{\circ} & \text{OK} \\ & \text{Check } \mathbf{k}: & 20^{\circ} \leq 106.5^{\circ} \leq 240^{\circ} & \text{OK} \end{aligned} $ $ \begin{aligned} & \text{Check } \mathbf{k}: & 20^{\circ} \leq 106.5^{\circ} \leq 240^{\circ} & \text{OK} \\ & \text{Check } \mathbf{k}: & 20^{\circ} \leq 106.5^{\circ} \leq 240^{\circ} & \text{OK} \end{aligned} $ $ \begin{aligned} & \text{Check } \mathbf{k}: & 20^{\circ} \leq 106.5^{\circ} \leq 240^{\circ} & \text{OK} \\ & \text{Check } \mathbf{k}: & 20^{\circ} \leq 10.0000000000000000000000000000000000$			mg = 0.63	25 * 1.2 =	0.750						
$ \begin{aligned} &\text{mg} = 0.750 ^{\circ} 0.919 = & 0.689 \\ &\frac{\text{Equation}}{g} = 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_x}{12Lt_x^3}\right)^{0.1} \\ &g = 0.06 + (8/14)^{\circ} 0.4 ^{\circ} (8/106.5)^{\circ} 0.3 ^{\circ} (1.271.611/(12^{\circ}106.5^{\circ}8^{\circ}3))^{\circ} 0.1 = 0.453 \\ &\text{Modify for Skew:} \\ &\text{skew correction} = & 0.919 \\ &g = 0.453 ^{\circ} 0.919 = & 0.416 \\ &\frac{\text{Range of Applicability (ROA) Checks}}{\text{Check S:} } = 0.55 ^{\circ} 0.9 ^{\circ} 16.0^{\circ} \text{ OK} \\ &\text{Check L:} 2.0^{\circ} s + 3.0^{\circ} s + 12.0^{\circ} \text{ OK} \\ &\text{Check L:} 2.0^{\circ} s + 106.5^{\circ} s + 240^{\circ} \text{ OK} \\ &\text{Check K}_{g^{\circ}} \cdot 10,000 \leq 1,271.611 \leq 7,000,000 \text{ OK} \\ &\text{Use Equation from Table 4.6.2.2.2.6b-1 because all criteria is GNL.} \\ &\frac{gM_{\text{re1}}}{g} = 0.418 \\ &\frac{Q.418}{M_{\text{re1}}} = 0.418 \\ &\frac{Q.418}{M_{\text{re1}}} = 0.919 \\ &\text{mg} = 0.875 ^{\circ} 0.919 = 0.804 \\ &\frac{Equation}{g} = 0.075 + (8/9.5)^{\circ} 0.6 ^{\circ} (8/106.5)^{\circ} 0.2 ^{\circ} (1,271.611/(12^{\circ}106.5^{\circ}8^{\circ}3))^{\circ} 0.1 = 0.649 \\ &\text{Modify for Skew:} \\ &\text{skew correction} = 0.919 \\ &\text{g} = 0.649 ^{\circ} 0.919 = 0.596 \\ &\frac{Q}{M_{\text{re2},0}} = 0.696 \\ &\frac{Q}{M_{\text{re2},0}} = 0.85 ^{\circ} 3 / 6 = 0.425 \\ &\text{Is W} \geq 201t ? \text{Ves} \\ &\text{Is W} \geq 201t \text{Policy states that if W} \geq 201t, gM_{\text{Miniture}} \text{ is the Maximum of } gM_{\text{max},0}, gM_{\text{matz},+}, m.N_{\text{L}} \wedge N_{\text{R}}, \\ &\text{TxDOT Policy states that if W} \geq 201t, gM_{\text{Miniture}} \text{ is the Maximum of } gM_{\text{max},0}, gM_{\text{matz},+}, m.N_{\text{L}} \wedge N_{\text{R}}, \\ &\text{TxDOT Policy states that if W} \geq 201t, gM_{\text{Miniture}} \text{ is the Maximum of } gM_{\text{max},0}, gM_{\text{matz},+}, m.N_{\text{L}} \wedge N_{\text{R}}, \\ &\text{TxDOT Policy states that if W} \geq 201t, gM_{\text{Miniture}} \text{ is the Maximum of } gM_{\text{max},+}, gM_{\text{matz},+}, m.N_{\text{L}} \wedge N_{\text{R}}, \\ &\text{TxDOT Policy states that if W} \geq 201t, gM_{\text{Miniture}} \text{ is the Maximum of } gM_{\text{max},+}, gM_{\text{max},+}, m.N_{\text{L}} \wedge N_{\text{R}}, \\ &\text{TxDOT Policy states that if W} \geq 201t, gM_{\text{max},+}, gM_{\text{max},+}, gM_{\text{max},+}, gM_{\text{max},+}, gM_{\text{max},+}, gM_{max$			Modify fo	r Skew:							
Equation $g = 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_s}{12Lt^3}\right)^{0.1}$ $g = 0.06 + (8/14)^{\circ}0.4 \cdot (8/106.5)^{\circ}0.3 \cdot (1,271,611/(12^{\circ}106.5^{\circ}8^{\circ}3))^{\circ}0.1 = 0.453$ Modify for Skew: $skew correction = 0.919$ $g = 0.453 \cdot 0.919 = 0.416$ Range of Applicability (ROA) Checks $Check S: 3.5' \le 8.0' \le 18.0' OK$ $Check L: 20' \le 106.5' \le 240' OK$ $Check L: 20' \le 106.5' \le 240' OK$ $Check K_0: 6 \ge 4 OK$ $Check K_0: 10,000 \le 1,271,611 \le 7,000,000 OK$ Use Equation from Table 4,6.2.2.2b-1 because all criteria is OR. $gM_{\text{Not}1} = 0.418$ Two or More Lanes Loaded $\frac{L_{\text{ever Rule}}}{M_{\text{Not}1}} \left(\text{Table 3.6.1.1.2}\right)$ $mg = \text{Max}(0.875 \cdot 1.0, 0.875 \cdot 0.85, 0.875 \cdot 0.65) = 0.875$ Modify for Skew: $skew correction = 0.919$ $mg = 0.875 \cdot 0.919 = 0.804$ $\frac{Equation}{M_{\text{off}1}} g = 0.075 + (8/9.5)^{\circ}0.6 \cdot (8/106.5)^{\circ}0.2 \cdot (1,271,611/(12^{\circ}106.5^{\circ}8^{\circ}3))^{\circ}0.1 = 0.849$ Modify for Skew: $skew correction = 0.919$ $g = 0.649 \cdot 0.919 = 0.596$ 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. $gM_{\text{ret2}} = 0.596$ $TxDOT Policy states that if W < 201t, gM_{\text{bellion}}$ is the Maximum of: gM _{pert} and m·N ₁ =N ₁ ,				skew corre	ection =	0.919					
$g = 0.06 + (\frac{S}{14})^{11} (\frac{S}{2LL_1})^{12} (\frac{K}{2LL_1})^{13}$ $g = 0.06 + (8/14)^{10} (A^* (8/106.5)^{10} 0.3^* (1.271,611/(12^*106.5^*8^*3))^* 0.1 = 0.453$ Modify for Skew: $skew correction = 0.919$ $g = 0.453^* 0.919 = 0.416$ Range of Applicability (ROA) Checks $Check S: 3.5^* 8.0^* \le 16.0^* OK$ $Check L: 20^* \le 106.5^* \le 240^* OK$ $Check N_0: 6 \ge 4$ $Check N_0: 6 \ge 4$ $Check N_0: 10.000 \le 1.271,611 \le 7.000,000 OK$ $Use Equation from Table 4.6.2.2.2b-1 because all criteria is OR. gM_{\text{red}} = 0.416 Two or More Lanes Loaded Lever Rule (Table 3.6.1.1.2) mg = Max(0.875^* 1.0, 0.875^* 0.85, 0.875^* 0.65) = 0.875 Modify for Skew: skew correction = 0.919 mg = 0.875^* 0.919 = 0.804 Equation g = 0.075 + (\frac{S}{9.5})^{0.6} (\frac{S}{2})^{0.2} (\frac{K_s}{12LL_s})^{0.1} g = 0.075 + (8/9.5)^{0.0} (s^* (8/106.5)^{0.0} 2^* (1.271,611/(12^*106.5^*8^*3))^{0.1} = 0.649 Modify for Skew: skew correction = 0.919 g = 0.649^* 0.919 = 0.596 Range of Applicability (ROA) Checks (same as for one lane loaded) Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK. gM_{\text{red},s} = 0.85^* 3 / 6 = 0.825 Ix DOT Policy states gM_{\text{petanor}} must be \ge m \cdot N_L \in N_0 m \cdot N_L \in N_0 m \cdot N_L \in N_0 = 0.85^* 3 / 6 = 0.425 Ix W \ge 20ft; 2^* Yes Tx DOT Policy states that if W < 20ft, gM_{\text{measure}} is the Maximum of: gM_{\text{pet},s}} and m \cdot N_L \in N_0 = 10.450 = 0.450 = 0.450 = 0.450 = 0.450 = 0.450 = 0.425 = 0.425 = 0.425 = 0.425 = 0.425 = 0.450 = 0.4$				mg = 0.756	0 * 0.919 =	0.689					
Modify for Skew:			$\frac{\text{Equation}}{\text{g} = 0.06}$	$+\left(\frac{S}{14}\right)^{0.4}$	$\left(\frac{S}{L}\right)^{0.3} \left(\frac{K}{12L}\right)^{0.3}$	$\left(\frac{8}{t}\right)^{0.1}$					
$skew correction = 0.919 \\ g = 0.453 * 0.919 = 0.416 \\ \hline Range of Applicability (ROA) Checks \\ Check S: 3.5' \le 8.0' \le 16.0' OK \\ Check L: 20' \le 106.5' \le 240' OK \\ Check L: 20' \le 106.5' \le 240' OK \\ Check N_b: 6 \ge 4 OK \\ Check K_g: 10,000 \le 1,271,611 \le 7,000,000 OK \\ Use Equation from Table 4.6.2.2.2b-1 because all criteria is OR. gM_{mat} = 0.416 \\ gM_{mat} = 0.416 \\ \hline Two or More Lanes Loaded \\ \underline{lever Rule} (Table 3.6.1.1.2) \\ mg = Max(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875 \\ Modify for Skew: skew correction = 0.919 \\ mg = 0.875 * 0.919 = 0.804 \\ \hline Equation \\ g = 0.075 + (8/9.5)^{0.6} (\frac{S}{L})^{0.2} (\frac{K_g}{12L_g}^3)^{0.1} \\ g = 0.075 + (8/9.5)^{0.6} (8/106.5)^{0.2} * (1,271,611/(12*106.5'8^3))^{0.1} = 0.649 \\ Modify for Skew: skew correction = 0.919 \\ g = 0.649 * 0.919 = 0.596 \\ \hline Range of Applicability (ROA) Checks (same as for one lane loaded) \\ Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK gM_{mat2} = 0.8596 \\ \hline TxDOT Policy states gM_{patenor} must be \ge m \cdot N_L \ne N_b \\ m \cdot N_L \Rightarrow N_c \Rightarrow N_L \Rightarrow N_c = 0.85 * 3 / 6 = 0.425 \\ \hline Is W \ge 201? 7 * Yes \\ \hline TxDOT Policy states that if W < 20ft, gM_{matps} is the Maximum of: gM_{mat}, gM_{mat2}, m \cdot N_L \Rightarrow N_b \Rightarrow TxDOT Policy states that if W < 20ft, gM_{matps} is the Maximum of: gM_{mat}, gM_{mat2}, m \cdot N_L \Rightarrow N_b \Rightarrow TxDOT Policy states that if W < 20ft, gM_{matps} is the Maximum of: gM_{mat}, gM_{mat2}, m \cdot N_L \Rightarrow N_b$			g = 0.06	+ (8/14)^0.4	* (8/106.5)^0	0.3 * (1,271,	611/(12	106.5*8^	3))^0.1 =	0.453	
$\begin{array}{c} g = 0.453 * 0.919 = & \underline{0.416} \\ \hline \textbf{Range of Applicability (ROA) Checks} \\ \hline \textbf{Check S:} & 3.5' \leq 8.0' \leq 16.0' & OK \\ \hline \textbf{Check L:} & 4.5'' \leq 8.0'' \leq 12.0'' & OK \\ \hline \textbf{Check L:} & 20' \leq 106.5' \leq 240' & OK \\ \hline \textbf{Check N_0:} & 6 \geq 4 & OK \\ \hline \textbf{Check N_0:} & 6 \geq 4 & OK \\ \hline \textbf{Check N_0:} & 10,000 \leq 1,271,611 \leq 7,000,000 & OK \\ \hline \textbf{Use Equation from Table 4,6.2.2.2b-1 because all criteria is OK.} \\ \hline \textbf{gM}_{\text{ret}} & 0.416 \\ \hline \textbf{Two or More Lanes Loaded} \\ \hline \textbf{Lever Rule} & (Table 3.6.1.1.2) \\ \hline \textbf{mg} & \textbf{max}(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875 \\ \hline \textbf{Modify for Skew:} \\ \hline \textbf{skew correction} & 0.919 \\ \hline \textbf{mg} & 0.875 * 0.919 = 0.804 \\ \hline \textbf{Equation} \\ \hline \textbf{g} & 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_x}{12L_x}\right)^{0.1} \\ \hline \textbf{g} & 0.075 + (8/9.5)^{0.0}.0.8 * (8/106.5)^{0.0}.2 * (1,271,611/(12*106.5*8^3))^{0.0}.1 = 0.649 \\ \hline \textbf{Modify for Skew:} \\ \hline \textbf{skew correction} & 0.919 \\ \hline \textbf{g} & 0.649 * 0.919 = 0.596 \\ \hline \textbf{Range of Applicability (ROA) Checks} & (same as for one lane loaded) \\ \hline \textbf{Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.} \\ \hline \textbf{gM}_{\text{max},2} & 0.596 \\ \hline \textbf{TxDOT Policy states gM}_{\text{interior}} \text{ must be \geq m·N_L & N_g} \\ \hline \textbf{m·N}_L & N_b & 0.85 * 3 / 6 = 0.425 \\ \hline \textbf{Is W \geq 20ft ? Yes} \\ \hline \textbf{TxDOT Policy states that if W $<$ 20ft, gM}_{\text{interior}} \text{ is the Maximum of: gM}_{\text{interior}} \text{ and m·N}_L & N_b.} \\ \hline \textbf{TxDOT Policy states that if W $<$ 20ft, gM}_{\text{interior}} \text{ is the Maximum of: gM}_{\text{interior}} \text{ mon N}_L & N_b.} \\ \hline \textbf{TxDOT Policy states that if W $<$ 20ft, gM}_{\text{interior}} \text{ is the Maximum of: gM}_{\text{interior}} \text{ mon N}_L & N_b.} \\ \hline \textbf{TxDOT Policy states that if W $<$ 20ft, gM}_{\text{interior}} \text{ is the Maximum of: gM}_{\text{interior}} \text{ mon N}_L & N_b.} \\ \hline \textbf{TxDOT Policy states that if W $<$ 20ft, gM}_{\text{interior}} \text{ is the Maximum of: gM}_{\text{interior}} \text{ mon N}_L & N_b.} \\ \hline \textbf{TxDOT Policy states that if W $<$ 20ft, gM}_{\text{interior}} \text{ is the Maximum of: gM}_{\text{interior}} \text{ mon N}_L & N_b.} \\ \hline TxDOT Policy states that if W $<$$			Modify fo	r Skew:							
Range of Applicability (ROA) Checks Check S: $3.5' \le 8.0' \le 16.0'$ Check L; $4.5'' \le 8.0'' \le 12.0''$ OK Check L, $20' \le 106.5' \le 240'$ OK Check N ₀ : $6 \ge 4$ Check N ₃ : $10.000 \le 1,271.611 \le 7.000.000$ Use Equation from Table $4.6.2.2.2b-1$ because all criteria is OR. gM _{mat} = 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 + $\left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_{\times}}{12LL_{\star}}\right)^{0.1}$ g = 0.075 + (8/9.5)\(^{\text{0.6}}\) 6.8 * (8/106.5)\(^{\text{0.2}}\) 0.2 * (1,271.611/(12*106.5*8^3))\(^{\text{0.1}}\) 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 _{mat2+} = 0.596 TxDOT Policy states gM _{intenor} must be $\ge m \cdot N_L \div N_b$ m* $N_L \div N_b = 0.85 * 3 / 6 = 0.425$ Is W ≥ 2017 ? Yes TxDOT Policy states that if W < 2016 , gM _{intenor} is the Maximum of: gM _{int1+} and m* $N_L \div N_b$.				skew corre	ection =	0.919					
Check S; $3.5' \le 8.0' \le 16.0'$ OK Check L; $20' \le 106.5' \le 240'$ OK Check L, $20' \le 106.5' \le 240'$ OK Check N _b : $6 \ge 4$ OK Check N _g : $10.000 \le 1,271.611 \le 7,000.000$ OK Check N _g : $10.000 \le 1,271.611 \le 7,000.000$ OK Use Equation From Table 4,6.2.2.2b-1 because all criteria is OK. gM _{mat} = 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 $g = 0.075 + \left(\frac{S}{9.5}\right)^{0.0} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_x}{12LL_x^3}\right)^{0.1}$ $g = 0.075 + (8/9.5)^{0.0} (6.8' (8/106.5)^{0.0}.2 * (1,271.611/(12*106.5*8^3))^{0.0}.1 = 0.649$ 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 _{max} = 0.596 TxDOT Policy states gM _{interior} must be $\ge m \cdot N_L \div N_b$ $m \cdot N_L \div N_b = 0.85 * 3 / 6 = 0.425$ Is W ≥ 2017 ? Yes TxDOT Policy states that if W ≤ 2011 , gM _{interior} is the Maximum of: gM _{int1} , and m · N _L ÷ N _b .				g = 0.453	* 0.919 =	0.416					
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 + $\left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_x}{12LI_x^3}\right)^{0.1}$ 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 _{intenor} must be ≥ m·N _L = N _b m·N _L = N _b = 0.85 * 3 / 6 = 0.425 Is W ≥ 20ft; Yes TxDOT Policy states that if W < 20ft, gM _{intenior} is the Maximum of: gM _{int1} and m·N _L = N _b . TxDOT Policy states that if W < 20ft, gM _{intenior} is the Maximum of: gM _{int1} and m·N _L = N _b .											
Check L: $20^{\circ} \le 106.5^{\circ} \le 240^{\circ}$ OK Check N _b : $6 \ge 4$ OK Check N _b : $6 \ge 4$ OK Check N _b : $10,000 \le 1,271,611 \le 7,000,000$ OK Use Equation from Table 4.6.2.2.2b-1 because all criteria is OR. $gM_{\text{mat}} = 0.416$ Two or More Lanes Loaded $\frac{\text{Lever Rule}}{\text{Lever Rule}} = (\text{Table 3.6.1.1.2})$ $mg = \text{Max}(0.875^{\circ} 1.0, 0.875^{\circ} 0.85, 0.875^{\circ} 0.65) = 0.875$ Modify for Skew: $\text{skew correction} = 0.919$ $mg = 0.875^{\circ} 0.919 = 0.804$ $\frac{\text{Equation}}{\text{g} = 0.075} + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_x}{12Lt_x^3}\right)^{0.1}$ $g = 0.075 + (8/9.5)^{0.6} \cdot (8/106.5)^{0.2} \cdot (1,271,611/(12^*106.5^*8^*3))^{0.1} = 0.649$ Modify for Skew: $\text{skew correction} = 0.919$ $g = 0.649^{\circ} 0.919 = 0.596$ $\frac{\text{Range of Applicability (ROA) Checks}}{\text{Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.}$ $gM_{\text{mt2*}} = 0.596$ $\text{TxDOT Policy states gM}_{\text{premor}}$ must be $\ge \text{m·N}_L \in \text{N}_b$ $\text{m·N}_L \in \text{N}_b = 0.85^{\circ} 3/6 = 0.425$ Is W $\ge 20\text{ft ? Yes}$ $\text{TxDOT Policy states that if W \le 20\text{ft}, gM_{\text{interior}} is the Maximum of: gM_{\text{inter}} and \text{m·N}_L \in \text{N}_b.$			(C) 'SE' (S)		-1.6.3						
Check N_b : $6 \ge 4$ OK Check N_g : $10,000 \le 1,271,611 \le 7,000,000$ OK Use Equation from Table 4.6.2.2.2b-1 because all criteria is OR. $gM_{\text{init}} = 0.416$ Two or More Lanes Loaded $\frac{\text{Lever Rule}}{\text{Impulsion of Nore of Nore Lanes Loaded}}$ $\frac{\text{Lever Rule}}{\text{Impulsion of Nore Lanes Loaded}}$ $\frac{\text{Lever Rule}}{\text{Impulsion of Nore lanes Loaded}}$ $\frac{\text{Lever Rule}}{\text{Impulsion of Nore of Nore lanes}}$ $\frac{\text{Impulsion of Nore lanes}}{\text{Impulsion of Nore lanes}}$ $\text{Impulsion$			P. C. C. C. C.				100				
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 OR $gM_{\text{int1}} = 0.416$ Two or More Lanes Loaded Lever Rule (Table 3.6.1.1.2) $mg = \text{Max}(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875$ Modify for Skew: $skew$ correction = 0.919 $mg = 0.875 * 0.919 = 0.804$ Equation $g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_x}{12Lt_x^3}\right)^{0.1}$ $g = 0.075 + (8/9.5)^{0.6} * (8/106.5)^{0.2} * (1,271.611/(12*106.5*8^3))^{0.1} = 0.649$ 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. $gM_{\text{int2}*} = 0.596$ TxDOT Policy states gM_{justrior} must be ≥ m·N _L ÷N _b m ·N _L ÷N _b = 0.85 * 3 / 6 = 0.425 Is W ≥ 20ft ? Yes TxDOT Policy states that if W < 20ft, gM_{intarior} is the Maximum of: $gM_{\text{int1}*}$ and m ·N _L ÷N _b .					5' ≤ 240'						
Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK $gM_{\text{int1}} = 0.416$ Two or More Lanes Loaded Lever Rule (Table 3.6.1.1.2) $mg = \text{Max}(0.875 * 1.0, 0.875 * 0.85, 0.875 * 0.65) = 0.875$ Modify for Skew: $skew correction = 0.919$ $mg = 0.875 * 0.919 = 0.804$ Equation $g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12LL_s^3}\right)^{0.1}$ $g = 0.075 + (8/9.5)^{0.0} \cdot (8/106.5)^{0.0} \cdot (1.271.611/(12*106.5*8^3))^{0.1} = 0.649$ 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. $gM_{\text{int2}, v} = 0.596$ TxDOT Policy states gM_{justrior} must be $\geq m \cdot N_L \pm N_b$ $m \cdot N_L \pm N_b = 0.85 * 3 / 6 = 0.425$ Is $W \geq 20ft$? Yes TxDOT Policy states that if $W < 20ft$, gM_{intarior} is the Maximum of: $gM_{\text{int1}, v}$ and $m \cdot N_L \pm N_b$. TxDOT Policy states that if $W \geq 20ft$, gM_{intarior} is the Maximum of: $gM_{\text{int1}, v}$ and $m \cdot N_L \pm N_b$.					1 074 644 2	7 000 000					
$\begin{array}{lll} & \textbf{Two or More Lanes Loaded} \\ & \underline{\textbf{Lever Rule}} & (\textbf{Table 3.6.1.1.2}) \\ & mg = \texttt{Max}(0.875 ^* 1.0, 0.875 ^* 0.85, 0.875 ^* 0.65) = & 0.875 \\ & \texttt{Modify for Skew:} \\ & \texttt{skew correction} = & 0.919 \\ & mg = 0.875 ^* 0.919 = & 0.804 \\ & \underline{\textbf{Equation}} \\ & g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_x}{12Lt_x^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 \\ & \texttt{Modify for Skew:} \\ & \texttt{skew correction} = & 0.919 \\ & g = 0.649 ^* 0.919 = & 0.596 \\ & \underline{\textbf{Range of Applicability (ROA) Checks}} & (\texttt{same as for one lane loaded}) \\ & \texttt{Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK} \\ & gM_{\text{int2}*} = & 0.596 \\ & \texttt{TxDOT Policy states gM}_{\text{Internor}} \text{ must be $\geq m \cdot N_L \stackrel{.}{=} N_b$} \\ & m \cdot N_L \stackrel{.}{=} N_b = & 0.85 ^* 3 / 6 = & 0.425 \\ & \texttt{Is W} \geq 20ft ? \text{Yes} \\ & \texttt{TxDOT Policy states that if W} < 20ft, gM_{\text{Internor}} \text{ is the Maximum of: } gM_{\text{int1}} \text{ and } m \cdot N_L \stackrel{.}{=} N_b. \\ & \Rightarrow \texttt{TxDOT Policy states that if W} \geq 20ft, gM_{\text{Internor}} \text{ is the Maximum of: } gM_{\text{int1}} \text{ and } m \cdot N_L \stackrel{.}{=} N_b. \\ & \Rightarrow \texttt{TxDOT Policy states that if W} \geq 20ft, gM_{\text{Internor}} \text{ is the Maximum of: } gM_{\text{int2}*} + m \cdot N_L \stackrel{.}{=} N_b. \\ & \Rightarrow \texttt{TxDOT Policy states that if W} \geq 20ft, gM_{\text{Internor}} \text{ is the Maximum of: } gM_{\text{int2}*} + m \cdot N_L \stackrel{.}{=} N_b. \\ & \Rightarrow \texttt{TxDOT Policy states that if W} \geq 20ft, gM_{\text{Internor}} \text{ is the Maximum of: } gM_{\text{int2}*} + m \cdot N_L \stackrel{.}{=} N_b. \\ & \Rightarrow \texttt{TxDOT Policy states that if W} \geq 20ft, gM_{\text{Internor}} \text{ is the Maximum of: } gM_{\text{int2}*} + m \cdot N_L \stackrel{.}{=} N_b. \\ & \Rightarrow \texttt{TxDOT Policy states that if W} \geq 20ft, gM_{\text{Internor}} \text{ is the Maximum of: } gM_{\text{int2}*} + m \cdot N_L \stackrel{.}{=} N_b. \\ & \Rightarrow \texttt{TxDOT Policy states that if W} \geq 20ft, gM_{\text{Internor}} \text{ is the Maximum of: } gM_{\text{int2}*} + m \cdot N_L \stackrel{.}{=} N_b. \\ & \Rightarrow \texttt{TxDOT Policy states that if W} \geq 20ft, gM_{\text{int2}*} + m \cdot N_L \stackrel{.}{=} N_b. \\ & \Rightarrow \texttt{TxDOT Policy States that if W} \geq 20ft, gM_{\text{int2}*$								City			
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_x}{12Lt_x^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·N _L ÷N _b $ m·N_L÷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·N_L÷N_b. $			Contract of the Contract of th		1,6.2,2,20-1 0	ecause all	oriteria is	OK.			
$\begin{array}{lll} mg = \text{Max}(0.875 \ ^{\circ} 1.0, 0.875 \ ^{\circ} 0.85, 0.875 \ ^{\circ} 0.65) = & 0.875 \\ \text{Modify for Skew:} & \text{skew correction} = & 0.919 \\ mg = 0.875 \ ^{\circ} 0.919 = & 0.804 \\ \hline \\ \frac{\text{Equation}}{g} = & 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_{\chi}}{12Lt_{\chi}^{3}}\right)^{0.1} \\ g = & 0.075 + (8/9.5)^{\circ} 0.6 \ ^{\circ} (8/106.5)^{\circ} 0.2 \ ^{\circ} (1,271,611/(12^{\circ} 106.5^{\circ} 8^{\circ} 3))^{\circ} 0.1 = 0.649 \\ \text{Modify for Skew:} & \text{skew correction} = & 0.919 \\ g = & 0.649 \ ^{\circ} 0.919 = & 0.596 \\ \hline \\ \text{Range of Applicability (ROA) Checks} & \text{(same as for one lane loaded)} \\ \text{Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.} \\ gM_{\text{int2}*} = & 0.596 \\ \hline \\ \text{TxDOT Policy states gM}_{\text{Interior}} \text{ must be } \geq \text{m·N}_{L} \dot{=} \text{N}_{b} \\ m \cdot \text{N}_{L} \dot{=} \text{N}_{b} = & 0.85 \ ^{\circ} 3 \ / 6 = & 0.425 \\ \hline \text{Is W} \geq \text{20ft ? Yes} \\ \hline \text{TxDOT Policy states that if W} < \text{20ft, } gM_{\text{Interior}} \text{ is the Maximum of: } gM_{\text{int1}} \text{ and } \text{m·N}_{L} \dot{=} \text{N}_{b}. \\ \hline \Rightarrow \text{TxDOT Policy states that if W} \geq \text{20ft, } gM_{\text{Interior}} \text{ is the Maximum of: } gM_{\text{int1}*} \text{ and } \text{m·N}_{L} \dot{=} \text{N}_{b}. \\ \hline \Rightarrow \text{TxDOT Policy states that if W} \geq \text{20ft, } gM_{\text{interior}} \text{ is the Maximum of: } gM_{\text{int2}*} \text{ grid m·N}_{L} \dot{=} \text{N}_{b}. \\ \hline \end{cases}$		Two or	More Lanes Lo	aded							
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_{\pi}}{12Lt_{\pi}^{3}}\right)^{0.1} $ $ g = 0.075 + (8/9.5)^{\circ}0.6 * (8/106.5)^{\circ}0.2 * (1,271,611/(12*106.5*8^{\circ}3))^{\circ}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_{\text{int2}*} = 0.596 $ TxDOT Policy states gM_{interior} must be $\geq m \cdot N_L \pm N_b$ $ m \cdot N_L \pm N_b = 0.85 * 3 / 6 = 0.425 $ Is $W \geq 20ft$? Yes $ TxDOT Policy states that if W < 20ft, gM_{\text{interior}} is the Maximum of: gM_{\text{int1}} and m \cdot N_L \pm N_b. \Rightarrow TxDOT Policy states that if W \geq 20ft, gM_{\text{interior}} is the Maximum of: gM_{\text{int1}} and m \cdot N_L \pm N_b.$			Lever Rule	(Table 3.6	.1.1.2)						
$ \begin{aligned} &\text{skew correction} = & 0.919 \\ &\text{mg} = 0.875 * 0.919 = & 0.804 \end{aligned} $ $ \begin{aligned} &\frac{\text{Equation}}{g} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_s}{12LL_s}^3\right)^{0.1} \\ &g = 0.075 + (8/9.5)^{\circ}0.6 * (8/106.5)^{\circ}0.2 * (1,271,611/(12*106.5*8^{\circ}3))^{\circ}0.1 = 0.649 \end{aligned} $ $ \begin{aligned} &\text{Modify for Skew:} \\ &\text{skew correction} = & 0.919 \\ &g = 0.649 * 0.919 = & 0.596 \end{aligned} $ $ \begin{aligned} &\text{Range of Applicability (ROA) Checks} \\ &\text{Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.} \\ &gM_{\text{int2+}} = & 0.596 \end{aligned} $ $ \begin{aligned} &\text{TxDOT Policy states gM}_{\text{Interior}} \text{ must be } \geq \text{m·N}_L \dot{+} \text{N}_b \\ &\text{m·N}_L \dot{+} \text{N}_b = & 0.85 * 3 / 6 = & 0.425 \end{aligned} $ $ \begin{aligned} &\text{Is } W \geq 20\text{ft ? Yes} \\ &\text{TxDOT Policy states that if } W < 20\text{ft, gM}_{\text{Interior}} \text{ is the Maximum of: gM}_{\text{int1}} \text{ and m·N}_L \dot{+} \text{N}_b.} \end{aligned} $			mg = Max	k(0.875 * 1.0	0, 0.875 * 0.8	5, 0.875 * 0	.65) =	0.875			
$\begin{aligned} &\text{mg} = 0.875 * 0.919 = & & & & & & & \\ & & & & & \\ & & & & &$			Modify fo	r Skew:							
$\frac{\text{Equation}}{g} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_x}{12Lt_x^3}\right)^{0.1}$ $g = 0.075 + (8/9.5)^{\circ}0.6 \cdot (8/106.5)^{\circ}0.2 \cdot (1,271,611/(12^*106.5^*8^*3))^{\circ}0.1 = 0.649$ Modify for Skew: $\text{skew correction} = 0.919$ $g = 0.649 \cdot 0.919 = 0.596$ $\text{Range of Applicability (ROA) Checks} \text{(same as for one lane loaded)}$ $\text{Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK.}$ $gM_{\text{int2+}} = 0.596$ $\text{TxDOT Policy states } gM_{\text{interior}} \text{ must be } \geq \text{m·N}_L \dot{+} \text{N}_b$ $\text{m·N}_L \dot{+} \text{N}_b = 0.85 \cdot 3 / 6 = 0.425$ $\text{Is W } \geq 20\text{ft ? Yes}$ $\text{TxDOT Policy states that if W } \leq 20\text{ft, } gM_{\text{interior}} \text{ is the Maximum of: } gM_{\text{int1}} \text{ and m·N}_L \dot{+} \text{N}_b$ $\Rightarrow \text{TxDOT Policy states that if W } \geq 20\text{ft, } gM_{\text{interior}} \text{ is the Maximum of: } gM_{\text{int1}} \text{ and m·N}_L \dot{+} \text{N}_b.$											
$g = 0.075 + \left(\frac{S}{9.5}\right) \left(\frac{S}{L}\right) \left(\frac{K_x}{12Lt_x^3}\right)$ $g = 0.075 + (8/9.5)^{\circ}0.6 * (8/106.5)^{\circ}0.2 * (1,271,611/(12*106.5*8^{\circ}3))^{\circ}0.1 = 0.649$ 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) $Use \ Equation \ from \ Table \ 4.6.2.2.2b-1 \ because \ all \ criteria \ is \ OK.$ $gM_{int2+} = 0.596$ $TxDOT \ Policy \ states \ gM_{interior} \ must \ be \ \ge m \cdot N_L \div N_b$ $m \cdot N_L \div N_b = 0.85 * 3 / 6 = 0.425$ Is $W \ge 20ft$? Yes $TxDOT \ Policy \ states \ that \ if \ W < 20ft, \ gM_{interior} \ is \ the \ Maximum \ of: \ gM_{int1} \ and \ m \cdot N_L \div N_b.$ $>> TxDOT \ Policy \ states \ that \ if \ W \ge 20ft, \ gM_{interior} \ is \ the \ Maximum \ of: \ gM_{int1} \ and \ m \cdot N_L \div N_b.$				mg = 0.87	5 * 0.919 =	0.804					
$g = 0.075 + (8/9.5)^{\circ}0.6 * (8/106.5)^{\circ}0.2 * (1,271,611/(12*106.5*8^{\circ}3))^{\circ}0.1 = 0.649$ 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) $Use \ Equation \ from \ Table \ 4.6.2.2.2b-1 \ because \ all \ criteria \ is \ OK.$ $gM_{int2+} = 0.596$ $TxDOT \ Policy \ states \ gM_{interior} \ must \ be \ \ge m\cdot N_L \doteq N_b$ $m\cdot N_L \doteq N_b = 0.85 * 3 / 6 = 0.425$ Is $W \ge 20ft$? Yes $TxDOT \ Policy \ states \ that \ if \ W < 20ft, \ gM_{interior} \ is \ the \ Maximum \ of: \ gM_{int1} \ and \ m\cdot N_L \doteq N_b.$ $>> TxDOT \ Policy \ states \ that \ if \ W \ge 20ft, \ gM_{interior} \ is \ the \ Maximum \ of: \ gM_{int1} \ and \ m\cdot N_L \doteq N_b.$			$\frac{\text{Equation}}{\text{g} = 0.07}$	$5 + \left(\frac{S}{9.5}\right)^0$	$^{6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{1}{12}\right)^{0.2}$	$\left(\frac{K_g}{2Lt_s^3}\right)^{0.1}$					
$g = 0.649 * 0.919 = \underbrace{0.596}_{0.596}$ 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. $gM_{int2+} = \underbrace{0.596}_{0.596}$ TxDOT Policy states $gM_{interior}$ must be $\geq m \cdot N_L \pm N_b$ $m \cdot N_L \pm N_b = 0.85 * 3 / 6 = \underbrace{0.425}_{0.425}$ Is W \geq 20ft? Yes TxDOT Policy states that if W $<$ 20ft, $gM_{interior}$ is the Maximum of: gM_{int1} and $m \cdot N_L \pm N_b$. >> TxDOT Policy states that if W \geq 20ft, $gM_{interior}$ is the Maximum of: gM_{int1} and gM_{int2+} gM_{int2+} $m \cdot N_L \pm N_b$.			g = 0.075	+ (8/9.5)^0	.6 * (8/106.5)	^0.2 * (1,27	1,611/(1	2*106.5*8	3^3))^0.1 =	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. $gM_{\text{int2+}} = 0.596$ $\text{TxDOT Policy states } gM_{\text{Interior}} \text{ must be } \geq \text{m·N}_L \doteq \text{N}_b$ $\text{m·N}_L \doteq \text{N}_b = 0.85 * 3 / 6 = 0.425$ Is W \geq 20ft? Yes $\text{TxDOT Policy states that if W } < 20\text{ft, } gM_{\text{Interior}} \text{ is the Maximum of: } gM_{\text{int1}} \text{ and } \text{m·N}_L \doteq \text{N}_b$ $\text{TxDOT Policy states that if W } \geq 20\text{ft, } gM_{\text{Interior}} \text{ is the Maximum of: } gM_{\text{int1}} = gM_{\text{int2+}} + \text{m·N}_L \doteq \text{N}_b.$				skew corre	ection =	0.919					
Use Equation from Table 4.6.2.2.2b-1 because all criteria is OK. $gM_{int2+} = 0.596$ $TxDOT Policy states gM_{interior} must be \ge m \cdot N_L \div N_b m \cdot N_L \div N_b = 0.85 * 3 / 6 = 0.425 is W \ge 20ft? Yes TxDOT Policy states that if W < 20ft, gM_{interior} is the Maximum of: gM_{int1} and m \cdot N_L \div N_b. \Rightarrow TxDOT Policy states that if W \ge 20ft, gM_{interior} is the Maximum of: gM_{int2+} m \cdot N_L \div N_b.$				g = 0.649	* 0.919 =	0.596					
$\begin{split} gM_{\text{int2+}} &= & 0.596 \\ \text{TxDOT Policy states } gM_{\text{Interior}} \text{ must be } \geq m \cdot N_L \dot{=} N_b \\ & m \cdot N_L \dot{+} N_b = & 0.85 \text{ * } 3 \text{ / } 6 = & 0.425 \\ \text{Is } W \geq 20\text{ft ?} & \text{Yes} \\ \text{TxDOT Policy states that if } W < 20\text{ft, } gM_{\text{Interior}} \text{ is the Maximum of: } gM_{\text{Int1}} \text{ and } m \cdot N_L \dot{=} N_b. \\ \text{>> } \text{TxDOT Policy states that if } W \geq 20\text{ft, } gM_{\text{Interior}} \text{ is the Maximum of: } gM_{\text{Int1}} \text{ and } m \cdot N_L \dot{=} N_b. \end{split}$			Range of Appl	cability (RO	A) Checks	(same as f	or one la	ane loade	d)		
TxDOT Policy states $gM_{Interior}$ must be $\geq m \cdot N_L \div N_b$ $m \cdot N_L \div N_b = 0.85 * 3 / 6 = 0.425$ Is W \geq 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 \geq 20ft, $gM_{Interior}$ is the Maximum of: $gM_{Int1} = gM_{Int1} = gM_{Int1} = gM_{Int2} + m \cdot N_L \div N_b$.			Use Equation	rom Table 4	1.6.2.2.2b-1 b	ecause all	oriteria is	OK.			
$\begin{array}{ll} m\cdot N_L \div N_b = & 0.85 \text{ " } 3 \text{ / } 6 = & \underline{0.425} \\ \text{Is W} \geq \text{20ft ?} & \text{Yes} \\ \text{TxDOT Policy states that if W} < 20\text{ft, } gM_{\text{Interior}} \text{ is the Maximum of: } gM_{\text{Int1}} \text{ and } m\cdot N_L \div N_b. \\ \text{>> TxDOT Policy states that if W} \geq 20\text{ft, } gM_{\text{Interior}} \text{ is the Maximum of: } gM_{\text{Int1}} - gM_{\text{Int2}+} m\cdot N_L \div N_b. \\ \end{array}$			gM _{int2+} =	0.596							
Is W \geq 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 \geq 20ft, $gM_{interior}$ is the Maximum of: gM_{int1} , gM_{int2+} , $m \cdot N_L \div N_b$.		TXDOT	Policy states gM	Interior must b	e ≥ m·N _L ÷N _b						
TxDOT Policy states that if W < 20ft, gM_{Interior} is the Maximum of: gM_{Int1} and $m\cdot N_L \div N_D$. >> TxDOT Policy states that if W \ge 20ft, gM_{Interior} is the Maximum of: gM_{Int1} , $gM_{\text{IntE}+}$, $m\cdot N_L \div N_D$.											
>> TxDOT Policy states that if W ≥ 20fl, gM _{Interior} is the Maximum of gM _{Int2+} m·N _L ÷N _b .		Is W ≥	20ft ? Yes								
		TXDOT	Policy states tha	t if W < 20ft	, gM _{I/(e/io)} is t	he Maximun	n of: gM	and m	N _L ÷N _b -		
gM _{interior} = 0.596	>>	TXDOT	Policy states tha	t if W ≥ 20ft	gM _{interior} is t	he Maximun	n of gM	781 - gM _{Int2+}	+ m·NL÷Nb		
		gM _{inte}	erior = 0.596								

TXDOT	County:	ANY	Highway:	Апу	Design:	BRG	Date:	8/15/20	and the same of th	RFD Specs
BRIDGE	C-S-J: Descrip:	ITBC Design Ex	ID#:	XXXX	Ck Dsn: File:	Ex3 Sn	Date:	ution factors.x	Rev. 10/18 - (No Interim) 4 of 8
	RIOR BE		ample of open a		It iid.	Line Op	ant distrib	bioit_idutore.s	J. Dileot.	7010
		ution Per Lane	(Table 4.6.2.)	2.3b-1):						
311041		ne Loaded	Tradio Hollan	17.						
1		Lever Rule	(Table 3.6	.1.1.2)						
			625 * 1.0 =		TxDOT us	es a mu	Itiple pre	sence factor	r of 1,0 for an	e
			for Skew:		lane loade				9, 114 (9) (3)	
			skew corre	ection =	1,164					
			mg = 0.62	5 * 1.164 =	0.728					
		Use Lever Ru	ile. as per AA	SHTO LRF	D Table 4.6.2	2.2.3b-1				
		gV _{ext1} =	0.728							
	Two or	More Lanes L	oaded							
		Lever Rule	(Table 3.6	.1.1.2)						
		mg = M	ax(0.625 * 1.0	0, 0.625 * 0.8	85, 0.625 * 0	.65) =	0.625			
		Modify I	or Skew:							
			skew corre	ection =	1.164					
			mg = 0.62	5 * 1.164 =	0.728					
		Equation								
			t. b/w CL web							
			- Rail Width							
			3ft - 1ft =	2.0	tt.					
		e = 0.6	$+\left(\frac{d_e}{10}\right)$							
		e = 0.6	+ (2.0/10) =	0.800						
		$g = e^*g^*$	Vint2+Eq.							
			00 * 0.947 =	0.758						
		Skew C	orrection is in	cluded in g	/(interior).					
		Range of App	olicability (RC	A) Checks	Interior	ROA is	implicitly	applied to t	he exterior be	eam.
		Check I	nterior Beam	ROA:	OK					
		Check of	d _e : -1.0' ≤ 2.0	0' ≤ 5.5'	OK					
		Check I	N _b : 6 ≠ 3		OK					
		Use Equation	from Table 4	4.6.2.2.3b-1	because all	criteria i	s OK.			
		$gV_{ext2+} =$	0.758							
	TXDOT	Policy states g	V _{Exterior} must b	oe ≥ gV _{interior}						
		gV _{interior} =	0.947							
	TXDOT	Policy states g	V _{Exterior} must b	oe ≥ m·N _L ÷N	ь					
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6	ô =	0.425					
		S/2 ? Yes								
		20ft ? Yes	-111 011 - 011	. cu 10	-14					
>>		Policy states th				a tha AA	atilization m	de all al	1	
	IXDOI	Policy states the m·N _L ÷N _b .	ial II OH > 5/6	z and vv < z	UIT, GV Exterior	is the ivi	aximum c	or, gv _{ext1} , gv	interior, and	
	TYDOT	Policy states th	at if OH > 9/	2 ans W > 2	Off aV-	s the M	avimum o	of aV aV	/ a aV	
	TADOT	and m·N _L ÷N _b		alia VV E Z	9 Y Exterior	o the M	annum C	y extly 9 v	ext2+> 9 v interior	
1 7	gV _{exte}									
	y v exte	nor - 0.947								

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Specs
RIDGE	C-S-J: Descrip:	ITBC Design Exa	ID #: ample 3. Span 2	XXXX	Ck Dsn:	Ex3 Spa	Date:	ition factors.xl	Rev. 10/18 - Sheet:	5 of 8
	RIOR BE		.,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,		1, 1101				Cindetif	
		ribution Per Lan	e (Table 4.6.)	2.2.2d-1):						
ionici.		ne Loaded	o (Tuoic 4.o.	17.						
		Lever Rule								
			625 * 1.0 =	0.625	TxDOT us	es a mult	inle pres	ence factor	of 1.0 for a	пe
		3.450.0000	or Skew:	3,010	lane loade				9/ 1/4/9/ 9	(IE)
		and the	skew corre	ection =	0.919					
			mg = 0.62	5 * 0.919 =	0.574					
		Use Lever Ru	le as per AA	SHTO LRF	Table 4.6.2	2.2d-1.				
		gM _{ext1} =	0.574							
	Two or	More Lanes L	habaa							
	I WO OI	Lever Rule	(Table 3.6	112						
			ax(0.625 * 1.0	and the second second	85 0 625 * 0	65) -	0.625			
		70071.0	or Skew:	, 0.025 0.	00, 0.020	.00) =	0.025			
		wideling i	skew corre	ection =	0.919					
				5 * 0.919 =	0.574					
		Equation		9,010	-					
			_ (d.)							
		e = 0.7	$7 + \left(\frac{d_e}{9.1}\right)$							
		e = 0.77	+ (2.0/9.1) =		0.990					
		g = e*gf								
			0.596 =	0.590						
		12.	orrection incli		interior)					
		Range of App				BOA is in	molicitly	applied to th	ne exterior h	neam
		and the second second	nterior Beam		OK	HOAR	присту	applied to ti	(C CALCITOT E	Juani.
			d _e : -1.0' ≤ 2.0		OK					
			N _b : 6 ≠ 3	. = 0.0	ОК					
		Use Equation		1.6.2.2.2d-1		riteria is	OK			
		gM _{ext2+} =	0.590	(O)EIEIEO	occuped an	JIMOTIA IO	010			
	THROT	O GALLY		Makad						
	IXDOI	Policy states gl	0.596	be z givi _{interio}						
	TYDOT	gM _{interior} = Policy states gl	Assess	he > m.N	i.					
	12001	$m \cdot N_1 \div N_h =$	0.85 * 3 / 6		0.425					
	Is OH <	S/2 ? Yes	0.03 370		0.425					
	000000	20ft ? Yes								
>>		Policy states th	at if OH & S/2	2, gM _{Exterior} is	s qM _{interior}					
		Policy states th				is the Ma	ximum c	f: gM _{ext1} , gN	Amterior, and	
		$m \cdot N_L \div N_b$								
	TXDOT	Policy states th	at if OH > S/2	2 ans W ≥ 2	Off, gMExterior	s the Ma	ximum o	f: gM _{ext1} , gN	A _{ext2+1} gM _{mter}	norr
		and m. N _{L+} N _b			-			4.000		
	gM _{ext}									



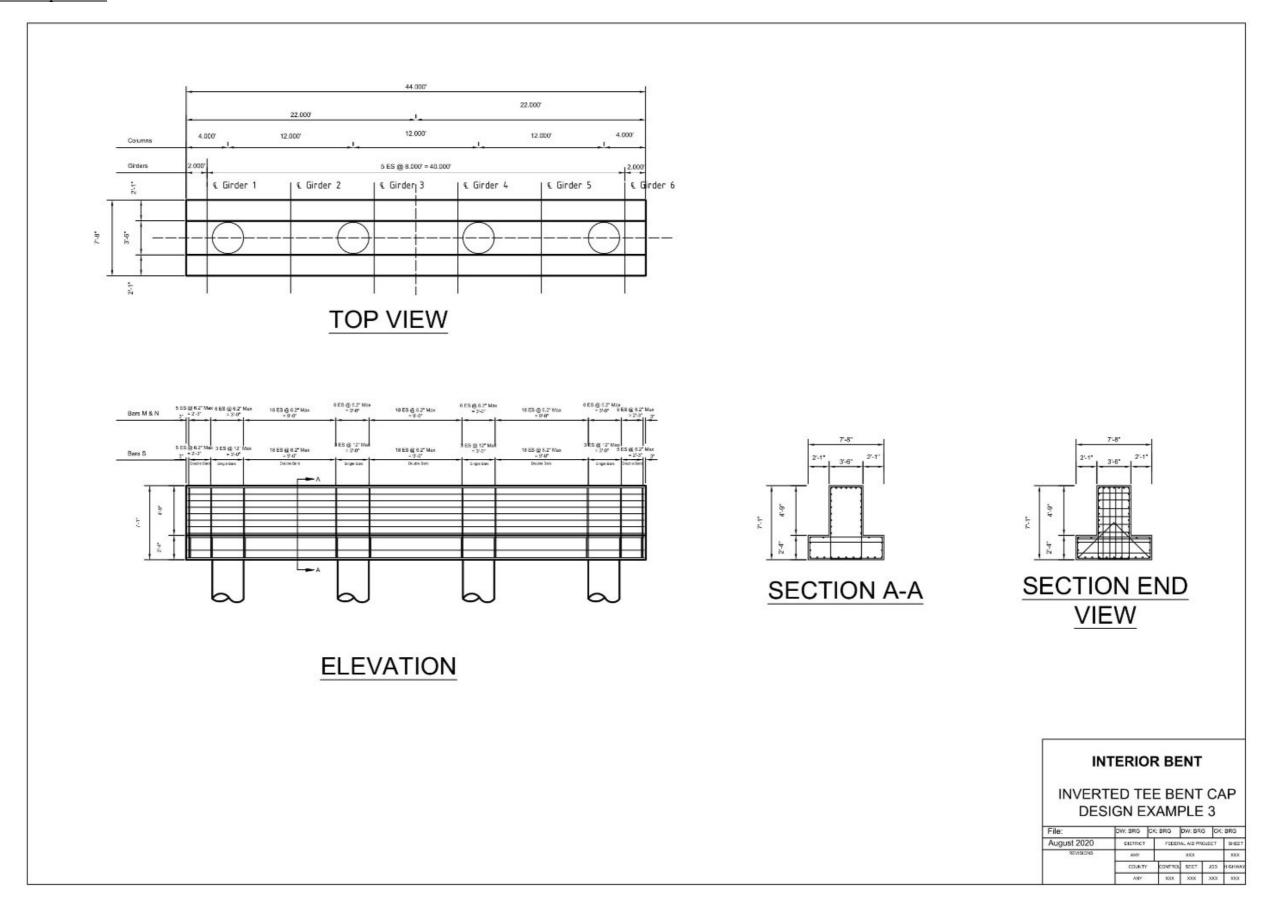
XDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Spe
RIDGE		ITBC Design Exa	ID#:	XXXX	Ck Dsn: File:	Eu2 Con	Date:	ion factors.xl	Rev. 10/18 - Sheet:	7 of 8
VISION	Descrip.	I TOC Design Exa	mple 5, Sparr 2		Trile,	LAU OPA	iz_uistribut	UII_IQUIUIS.AI	Sheet	7 01 0
	RULE		= 8.0 ft							
	OR (con't)									
For 18 s	s S < 22: ne Lane =	$\frac{16}{32}\bigg(1+\frac{S-6}{S}\bigg)$					100	0.625		
Tw	o Lanes =	$\frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$	$\frac{S-4}{S} + \frac{S-10}{S}$)			0,5	0.750		
		$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$					-	-0.125		
		$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$\frac{S-4}{S} + \frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S-12}{S}$	$\left(\frac{-18}{S} + \frac{S-16}{S}\right)$		19	0.625		
For 22 s	≤ S ≤ 24; ne Lane =	$\frac{16}{32}\bigg(1+\frac{S-6}{S}\bigg)$					119	0.625		
Tw	o Lanes =	$\frac{16}{32}\bigg(1+\frac{S-6}{S}\bigg)$	$\frac{S-4}{S} + \frac{S-10}{S}$)				0.750		
Three	e Lanes =	$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$\frac{S-4}{S} + \frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S}{S}$	$\left(\frac{-18}{S}\right)$		110	-0.125		
Fou	r Lanes =	$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S-12}{S}$	$\frac{-18}{S} + \frac{S - 16}{S}$	$+\frac{S-22}{S}$		-1.500		
		16k	16k	D fi	4 11				S =	8.0 f
	_	он — — х-	_s	assumed	1 hinge		,	Rail Width X = S+OH-		3.0 f 1.0 f 8.0 f
For X <	6: ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} \right)$					1.5	0.500		
For 6 ≤ Or	X < 12; ne Lane =	$\frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					0.625		
For 12 s	≤ X < 18; ne Lane =	$\frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					0.625		
O										

BRIDGE	County:	ANY XXX-XX-XXXX	Highway: ID #:	Any	Design: Ck Dsn:	_	Date: 8	3/15/20	2017 LRFD Spec
IVISION			ample 3, Span 2	JAAAA	File:	Ex3 Span2		factors.xl	Sheet: 8 of 8
6	200								
LEVER	RULE								
EXTER	IOR (con'	t) S	= 8.0 f	t.	OH =	3.0 ft			
	(,	RW			H-RW-2ft =				
For 18	X < 24:								
O	ne Lane =	$=\frac{16}{32}\left(\frac{X}{S}+\frac{X}{S}\right)$	-6				= 1	0.625	
		22/10/10	7.3K. 10. (88) - 32 - 13	c_10\					
Tw	b Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	$\frac{3}{5}$	5			= 1	0.250	
For 24 s	X < 30:	107.0	-1						
O	ne Lane =	$=\frac{16}{32}\left(\frac{X}{S}+\frac{X-X}{S}\right)$	(0)				= (0.625	
	4	16 (X X -	6 X-12 X	(- 18)				Gun	
Tw	o Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	S	S			=	0.250	
Three	e Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	$-6 + \frac{X - 12}{4} +$	x - 18 + x - 2	4)		= -	1.250	
		32 (5 5	S	2 2	7.			1000	
For 30 s	X < 36:	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X}{S}\right)$	-6)				- 1	0.625	
		32 3	1				-	0.020	
Tw	Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	$\frac{-6}{5} + \frac{X - 12}{5} + \frac{3}{5}$	(-18)			-	0.250	
		Allegen III			x - 307				
Thre	e Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	+ S + S	s + s	+ 3 3 3		=	2.625	
For 36 5	X < 42:	16/ Y Y	.63						
O	ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	7				= 1	0.625	
Ton	o Longo	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X}{S}\right)$	6 + X - 12 + 3	K = 18				0.050	
I W	b Lanes =	32 S S	S	S)			_	0.250	
Thre	e Lanes =	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X}{S}\right)$	$\frac{6}{6} + \frac{X-12}{6} + \frac{3}{2}$	$\frac{x-18}{c} + \frac{x-2}{c}$	$\frac{4}{4} + \frac{X - 30}{C}$		11.00	2.625	
			4 2 2			9 167			
Fou	r Lanes =	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X-X}{S}\right)$	$\frac{-6}{S} + \frac{A-12}{S} + \frac{2}{S}$	$\frac{1-18}{S} + \frac{A-2}{S}$	$\frac{4}{S} + \frac{X - 30}{S} + \frac{1}{S}$	5	= 9	4.375	
For 42 s	≤ X ≤ 48:								
O	ne Lane =	$=\frac{16}{32}\left(\frac{X}{5} + \frac{X}{5}\right)$	-6				= (0.625	
		16 (X X -	6 X-12	(-18)					
Tw	o Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	+ 32+	S			=	0.250	
Three	e lanes -	$=\frac{16}{32}\left(\frac{X}{5} + \frac{X}{5}\right)$	-6 + X - 12 + 3	x - 18 + x - 2	4 + X - 30			2.625	
		Neva N	9	0 0					
Fou	r Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	$\frac{-6}{S} + \frac{X - 12}{S} + \frac{3}{S}$	$\frac{x-18}{s} + \frac{x-2}{s}$	$\frac{4}{S} + \frac{X - 30}{S} + \frac{1}{S}$	$\frac{X-36}{S} + \frac{X}{S}$	$\left(\frac{-42}{S}\right) = 0$	6.500	
INTERIO	OR				EXTER	IOR			
	ne Loade	d	= 0.625			ne Loaded		-	0.625
	nes Load		= 0.875			nes Loade		=	0.625
Three L	anes Loa	ded	= 0.875		Three L	anes Load	led	-	0.625
Equal a	nes Load	ed	= 0.875		Four La	nes Loade	d		0.625

4.4.15.5 Concrete Section Shear Capacity Spreadsheet

	0.100	ANY			Descrip:	ITBC Design E	xample 3 - Ben	12	
	Highway: C-S-J:	XXXXXXX			Design:	BRG C	k Dsn:	BRG	
lexas	3000	A CONTRACTOR OF THE PARTY OF TH		20.00.00	Design:	4444		2.02	
of Transportation	Bridge I	277.34.77		ev: 09/26/08			Date:	Aug-20	
CONCRETE SECTION SHEA	AR CAPA	CITY BY A	ASHTO L	RFD BRID	GE DESIG	N SPECIFIC	ATIONS, FO	URTH EDIT	ON, 2007
Resistance Factors:			Units:	US					
φ _V =	0.9								
фм =	0.9								
ψ _N =	0.75								
Concrete:			Mild Steel:			Prestressed			
f'c =		ksi	fy =	60		fpu =	270 k		
Ec =	4070	ksi	Es =	29000	ksi	Ep =	28500 k	si	
					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	(
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	7,7
Prestressed steel area, Aps	in^2	0	0	0	0	0	0	0	(
Prestress shear, Vp	kip	0	0	.0	0	0	0	0	(
Average prestress, fps	ksi	0	0	0	0	0	0	0	(
Torsional moment, Tu	kip-ft	773	387	387	773	773	387	387	77.
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	0.4
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 ε _x	kip	2069 6.83E-04	2099 7.07E-04	2411 4.33E-04	1816 1.00E-03	2060 6.88E-04	2075 7.31E-04	2404 4.39E-04	1816 1.00E-03
ex A	den	1000000	3.51.21	29.40	36.40	32.90	10.00	29.50	A CALL OF
9	deg	32.80 2.440	33.10 2.420	29.40	2.230	2.430	33.42 2.400	2,680	36.40 2.230
Reg'd Shear reinf. Av/S	in^2/in	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Reg'd Torsion reinf. At/S	in^2/in	0.000	0.008	0.000	0.000	0.000	0.008	0.000	0.000
Maximum stirrup spacing, Smax	in	24.0	24.0	24.0	24.0	24.0	24.0	24.0	24.0
Conclusion									
	inforcing	OK	OK	OK	OK	OK	OK	OK	OK
Shear Re	moreing				OK	OK	OK	OK	OK

4.4.15.6 Bent Cap Details



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

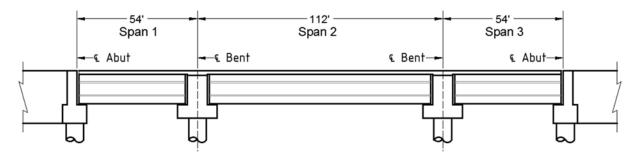


Figure 4.78 Spans of the Bridge with 60 Degrees Skewed ITBC

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

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

 $f_c' = 5 \text{ ksi}$

 $w_c = 150 \text{ pcf (for weight)}$

 $w_c = 145 \text{ pcf (for Modulus of Elasticity calculation)}$

"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

 $f_y = 60 \text{ ksi}$

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

Bents

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

Define Variables

Back Span	Forward Span	!	
Span1 = 54ft	Span2 = 112ft	į.	Span Length
GdrSpa1 = 8ft	GdrSpa2 = 8ft	-	Girder Spacing (Normalized values)
GdrNo1 = 6	GdrNo2 = 6		Number of Girders in Span
GdrWt1 = 0.851klf	GdrWt2 = 0.85	51klf	Weight of Girder
Haunch1 = 2in	Haunch2 = 3.7	75in	Size of Haunch
<u>Bridge</u>			
Skew = 60deg			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 = 0.372 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			
$f_c = 5ksi$			Concrete Strength
$w_{cE} = 0.145 kcf$			Unit Weight of Concrete for E_c
$E_{c} = 33000 \cdot w_{cE}^{1.5} \cdot \sqrt{s}$	$\overline{\mathrm{f_c}}$	$E_c = 4074 \text{ ksi}$	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
$D_{column} = 36in$			Diameter of Columns

IM = 33%

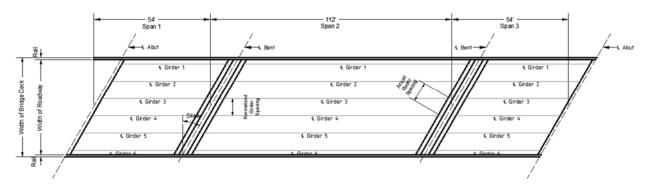


Figure 4.79 Top View of the 60 Degrees Skewed ITBC with Spans and Girders

4.5.2 Determine Cap Dimensions

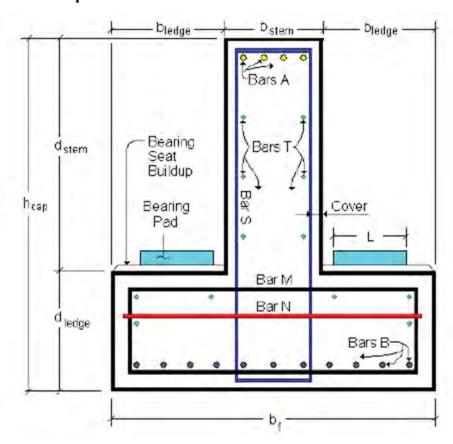


Figure 4.80 Section View of 60 Degrees Skewed ITBC

4.5.2.1 Stem Width

 $b_{stem} = at least D_{column} + 3in$

Use: $b_{stem} = 42 \text{ in}$

4.5.2.2 Stem Height

Distance from Top of Slab to Top of Ledge:

 $D_{Slab_to_Ledge} = SlabThk + Haunch2 + GirderD + BrgPa$

 $D_{Slab to Ledge} = 70.00 in$

StemHaunch = 3.75 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)

Haunch2 is the larger of the two haunches.

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.

$$d_{stem} = D_{Slab_to_Ledge} - SlabThk - StemHaunch - 0.5in$$

$$d_{stem} = 57.75 in$$

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

4.5.2.3 Ledge Width

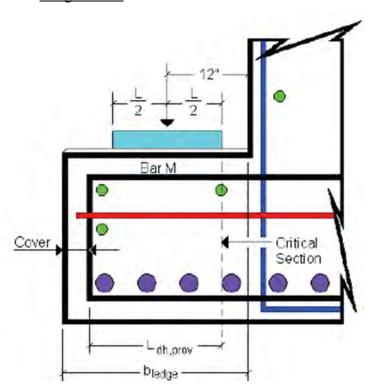


Figure 4.81 Ledge Section of 60 Degrees ITBC

cover = 2.5 in

L = 15 in

Determine the Required Development Length of Bar M:

Try # 7 Bar for Bar M.

$$d_{bar\ M} = 0.875 \text{ in}$$

$$A_{bar_M}=0.60\,in^2$$

Basic Development Length

$$L_{\rm dh} = \frac{38.0 \cdot d_{\rm bar_M}}{60} \cdot \left(\frac{f_{\rm y}}{\sqrt{f_{\rm c}}}\right)$$

 $L_{dh} = 14.87 \text{ in}$

(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"?

The stem must accommodate ½" of bituminous fiber.

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

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

" $L_{dh,prov}$ " 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 circular 15" Dia. for 60° skewed beents, as shown in the IGEB standard.

SideCover = cover
$$-\frac{d_{bar_M}}{2}$$
 = 2.06 in

$$TopCover = cover - \frac{d_{bar_{M}}}{2} = 2.06 \text{ in}$$

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)

M.

The dimension "cover" is

(AASHTO LRFD 5.10.8.2.4a)

"Side Cover" and "Top Cover" are the clear cover on the side

and top of the hook respectively.

measured from the center of Bar

The Required Development Length:

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

Therefore,

$$L_{dh req} = 14.87 in$$

$$b_{ledge_min} = L_{dh_req} + cover + 12in - \frac{L}{2}$$
 $b_{ledge_min} = 21.87 in$

Use:

$$b_{ledge} = 25 in$$

$$b_f = 2 \cdot b_{ledge} + b_{stem}$$

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

4.5.2.4 Ledge Depth

Width of Bottom Flange:

Use a Ledge Depth of 28".

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

Total Depth of Cap:

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

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.

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

 $b_f = 92 in$

4.5.2.5 Summary of Cross Sectional Dimensions

$$b_{stem} = 42 in$$

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

$$b_{ledge} = 25 in$$

$$d_{ledge} = 28 in$$

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

4.5.2.6 Length of Cap

First define Girder Spacing and End Distance:

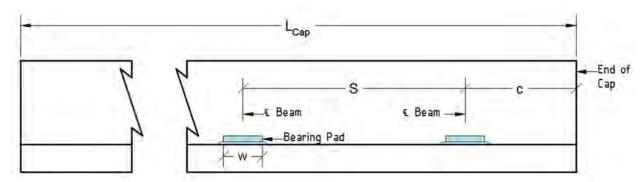


Figure 4.82 Elevation View of 60 Degrees Skewed ITBC

$$S=8 \text{ ft} \\ c=2 \text{ ft} \\ \text{"c" is the distance from the Center} \\ \text{Line of the Exterior Girder to the} \\ \text{Edge of the Cap measured along} \\ \text{the Cap.} \\ L_{Cap}=S\cdot (\text{GdrNo1}-1)+2c \\ L_{Cap}=44 \text{ ft} \\ \text{Length of Cap} \\ \text{Length of C$$

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 in W = 15 in Width of Bearing Pad Width of Bearing Pad

4.5.3 Cross Sectional Properties of Cap

$$\begin{split} A_g &= d_{ledge} \cdot b_f + d_{stem} \cdot b_{stem} & A_g = 4970 \text{ in}^2 \\ ybar &= \frac{d_{ledge} \cdot b_f \cdot \left(\frac{1}{2} d_{ledge}\right) + d_{stem} \cdot b_{stem} \cdot \left(d_{ledge} + \frac{1}{2} d_{stem}\right)}{A_g} & ybar = 34.5 \text{ in} & \textit{Distance from bottom of the cap to the center of gravity of the cap} \\ I_g &= \frac{b_f \cdot d_{ledge}^3}{12} + b_f \cdot d_{ledge} \cdot \left(ybar - \frac{1}{2} d_{ledge}\right)^2 + \frac{b_{stem} \cdot d_{stem}^3}{12} + \cdots \\ b_{stem} \cdot d_{stem} \cdot \left[ybar - \left(d_{ledge} + \frac{1}{2} d_{stem}\right)\right]^2 & I_g = 3.06 \times 10^6 \text{ in}^4 \end{split}$$

4.5.4 Cap Analysis

4.5.4.1 <u>Cap Model</u>

Assume:

4 Columns Spaced @ 12'-0"

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

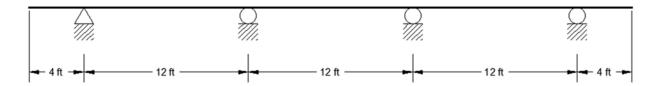


Figure 4.83 Continuous Beam Model for 60 Degrees Skewed ITBC

TxDOT does not consider frame action for typical multi-column bents (BDM-LRFD, Ch. 4, Sect. 5, Structural Analysis).

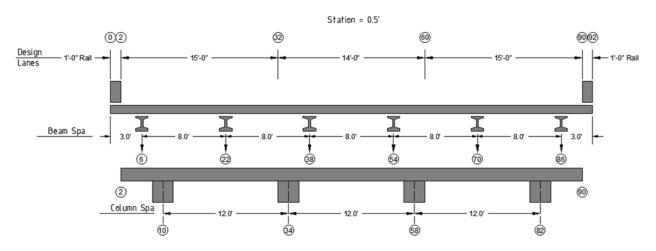


Figure 4.84 Cap 18 Model of 60 Degrees Skewed ITBC

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.

$$station = 0.5 ft$$

Station increment for CAP 18

Recall:

$$E_c = 4074 \text{ ksi}$$
 $I_g = 3.06 \times 10^6 \text{ in}^4$

$$E_c I_g = 1.25 \times 10^{10} \; \text{kip} \cdot \text{in}^2 / \; \left(12 \frac{\text{in}}{\text{ft}}\right)^2 \; E_c I_g = 8.66 \times 10^7 \text{kip} \cdot \text{ft}^2$$

4.5.4.1.1 Dead Load

SPAN 1

Values used in the following equations can be found on "4.5.1 Design Parameters"

$$Rail1 = \frac{2 \cdot RailWt \cdot \frac{Span_1}{2}}{\min(GdrNo1,6)} \qquad \qquad Rail1 = 3.44 \frac{kip}{girder}$$

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

$$Slab1 = w_c \cdot GdrSpa1 \cdot SlabThk \cdot \frac{Span1}{2} \cdot 1.10$$

Slab1 = $23.76 \frac{\text{kip}}{\text{girder}}$ Increase slab DL by 10% to account for haunch and thickened slab ends.

$$Girder1 = GdrWt1 \cdot \frac{Span1}{2}$$

$$Girder1 = 22.98 \frac{kip}{girder}$$

$$DLRxn1 = (Rail1 + Slab1 + Girder1)$$

 $DLRxn1 = 50.17 \frac{kip}{girder}$

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

$$Overlay1 = w_{Olay} \cdot GdrSpa1 \cdot OverlayThk \cdot \frac{Span1}{2}$$

Overlay
$$1 = 5.04 \frac{\text{kip}}{\text{girder}}$$

Design for future overlay.

SPAN 2

$$Rail2 = \frac{2 \cdot RailWt \cdot \frac{Span2}{2}}{min(GdrNo2,6)}$$

Rail2 =
$$7.13 \frac{\text{kip}}{\text{girder}}$$

$$Slab2 = w_c \cdot GdrSpa2 \cdot SlabThk \cdot \frac{Span2}{2} \cdot 1.10$$

$$Slab2 = 49.28 \frac{kip}{girder}$$

$$Girder2 = GdrWt1 \cdot \frac{Span2}{2}$$

$$Girder2 = 47.66 \frac{kip}{girder}$$

$$DLRxn2 = (Rail2 + Slab2 + Girder2)$$

$$DLRxn2 = 104.07 \frac{kip}{girder}$$

$$Overlay2 = w_{Olay} \cdot GdrSpa2 \cdot OverlayThk \cdot \frac{Span2}{2}$$

Overlay2 =
$$10.45 \frac{\text{kip}}{\text{girder}}$$

CAP

Cap =
$$w_c \cdot A_g = 5.177 \frac{kip}{ft} \cdot \frac{0.5ft}{station}$$

$$Cap = 2.589 \frac{kip}{station}$$

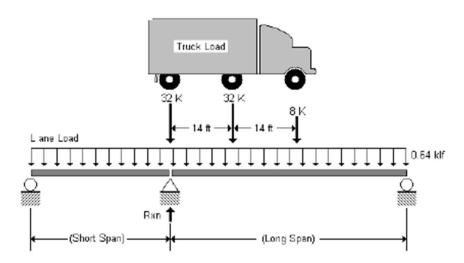


Figure 4.85 Live Load Model of 60 Degrees Skewed ITBC

LongSpan = max(Span1, Span2)

LongSpan = 112 ft

ShortSpan = min(Span1, Span2)

ShortSpan = 54 ft

IM = 0.33

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

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

$$Truck = 32kip + 32kip \cdot \left(\frac{LongSpan - 14ft}{LongSpan}\right) + 8kip \cdot \left(\frac{LongSpan - 28ft}{LongSpan}\right)$$

$$Truck = 66.00 \frac{kip}{lane}$$

LLRxn = Lane + Truck
$$\cdot$$
 (1 + IM)
LLRxn = 140.90 $\frac{\text{kip}}{\text{lane}}$

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.

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)

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

$$P = 21.28 \text{ kip}$$

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

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

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

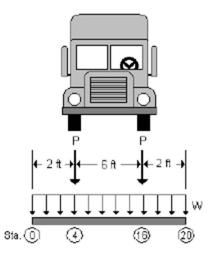


Figure 4.86 Live Load Model of 60 Degrees Skewed ITBC for CAP18

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)

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

Factors".

4.5.4.1.3 Cap 18 Data Input

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

<u>Limit States</u> (AASHTO LRFD 3.4.1)

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

Service I

Live Load and Dynamic Load Allowance LL+IM = 1.00

Dead Load and Wearing Surface DC & DW = 1.00

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

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.

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

	$\underline{\mathbf{Max} + \mathbf{M}}$	<u> Max - M</u>
Dead Load:	$M_{posDL} = 582.2 \text{ kip} \cdot \text{ft}$	$M_{\text{negDL}} = -844.9 \text{ kip} \cdot \text{ft}$
Service Load:	$M_{posServ} = 1067.0 \text{ kip} \cdot \text{ft}$	$M_{negServ} = -1267.9 \text{ kip} \cdot \text{ft}$
Factored Load:	$M_{posUlt} = 1585.8 \text{kip} \cdot \text{ft}$	$M_{negIJlt} = -1812.0 \text{ kip} \cdot \text{ft}$

4.5.4.2 Girder Reactions on Ledge

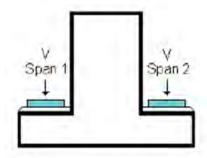


Figure 4.87 Girder Reactions on the Ledge of 60 Degrees Skewed ITBC

Dead Load

$$DLSpan1 = Rail1 + Slab1 + Girder1$$

$$DLSpan1 = 50.17 \frac{kip}{girder}$$

Overlay1 =
$$5.04 \frac{\text{kip}}{\text{girder}}$$

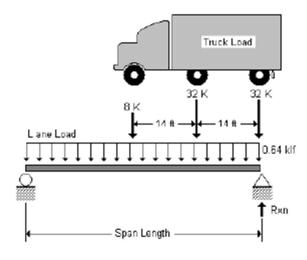
$$DLSpan2 = Rail2 + Slab2 + Girder2$$

$$DLSpan2 = 104.07 \frac{kip}{girder}$$

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

$$LaneSpan1 = 0.64klf \cdot \left(\frac{Span1}{2}\right)$$

$$LaneSpan1 = 17.28 \frac{kip}{lane}$$

$$LaneSpan2 = 0.64klf \cdot \left(\frac{Span2}{2}\right)$$

$$LaneSpan2 = 35.84 \frac{kip}{lane}$$

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

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

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

$$\begin{split} TruckSpan2 &= 32kip + 32kip \cdot \left(\frac{Span2 - 14ft}{Span2}\right) + 8kip \cdot \left(\frac{Span2 - 28ft}{Span2}\right) \\ TruckSpan2 &= 66.00 \frac{kip}{lane} \end{split}$$

IM = 0.33

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

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

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

LLRxnSpan2 = $123.62 \frac{\text{kip}}{\text{girder}}$

 $gV_{Span1_Int} = 0.999$

 $gV_{Span1_Ext} = 0.999$

 $gV_{Snan2 Int} = 1.045$

 $gV_{Span2 Ext} = 1.045$

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

$$LLSpan1Int = gV_{Span1_Int} \cdot LLRxnSpan1 \qquad LLSpan1Int = 96.40 \frac{kip}{girder}$$

$$LLSpan1Ext = gV_{Span1_Ext} \cdot LLRxnSpan1 \qquad LLSpan1Ext = 96.40 \frac{kip}{girder}$$

$$LLSpan2Int = gV_{Span2_Int} \cdot LLRxnSpan2 \qquad LLSpan2Int = 129.18 \frac{kip}{girder}$$

LLSpan2Ext =
$$gV_{Span2_Ext} \cdot LLRxnSpan2$$
 LLSpan2Ext = $129.18 \frac{kip}{girder}$

Span 1

Interior Girder

$$V_{s Span1Int} = DLSpan1 + Overlay1 + LLSpan1Int$$

$$V_{s \text{ Span1Int}} = 152 \text{ kip}$$

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)

$$V_{u \text{ Span1Ext}} = 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 + 1.75 \cdot LLSpan1Ext$$

$$V_{u \text{ Span}_{1}\text{Ext}} = 239 \text{ kip}$$

Span 2

Interior Girder

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

$$V_{s \text{ Span2Int}} = DLSpan2 + Overlay2 + LLSpan2Int$$

$$V_{s,Snan2Int} = 244 \text{ kip}$$

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

$$V_{u \; Span2Int} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot LLSpan2Int$$

$$V_{u,Span,2Int} = 372 \text{ kip}$$

Exterior Girder

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

$$V_{s_Span2Ext} = DLSpan2 + Overlay2 + LLSpan2Ext$$

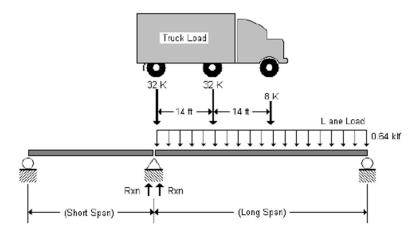
$$V_{s \text{ Span}2\text{Ext}} = 244 \text{ kip}$$

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

$$V_{u \text{ Span}2\text{Ext}} = 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot LLSpan2\text{Ext}$$

$$V_{u \text{ Span}2\text{Ext}} = 372 \text{ kip}$$

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

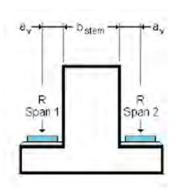


Figure 4.90 Loads on the Ledge of 60 Degrees Skewed ITBC for Torsion

$$a_v = 12 \text{ in}$$

" 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 =
$$a_v + \frac{1}{2}b_{stem}$$

$$LeverArm = 33 in$$

Interior Girders

Girder Reactions

$$\begin{split} R_{u_Span1} &= 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 \\ R_{u_Span1} &= 70 \text{ kip} \\ R_{u_Span2} &= 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2_Int} \\ & \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)] \\ R_{u_Span2} &= 372 \text{ kip} \end{split}$$

Torsional Load

$$T_{u_Int} = \left| R_{u_Span1} - R_{u_Span2} \right| \cdot LeverArm$$

$$T_{u_Int} = 830 \; kip \cdot ft$$

Exterior Girders

Girder Reactions

$$\begin{split} R_{u_Span1} &= 1.25 \cdot DLSpan1 + 1.5 \cdot Overlay1 \\ R_{u_Span1} &= 70 \text{ kip} \\ R_{u_Span2} &= 1.25 \cdot DLSpan2 + 1.5 \cdot Overlay2 + 1.75 \cdot gV_{Span2_Ext} \\ & \cdot [LaneSpan2 + TruckSapn2 \cdot (1 + IM)] \\ R_{u_Span2} &= 372 \text{ kip} \end{split}$$

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

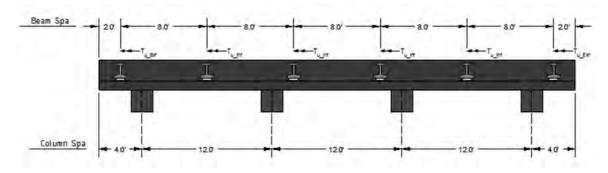


Figure 4.91 Elevation View of 60 Degrees Skewed ITBC with Torsion Loads

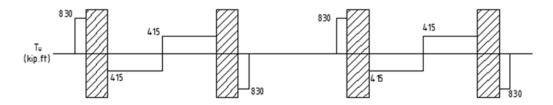


Figure 4.92 Torsion Diagram of 60 Degrees Skewed ITBC

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

$$T_u = 830 \text{ kip} \cdot \text{ft}$$
 Maximum Torsion on Cap

4.5.4.4 Load Summary

Ledge Loads

Interior Girder

Service Load

$$V_{s Int} = max(V_{s Span1Int}, V_{s Span2Int})$$
 $V_{s Int} = 243.7 \text{ kip}$

Factored Load

$$V_{u \text{ Int}} = \max(V_{u \text{ Span1Int}}, V_{u \text{ Span2Int}})$$
 $V_{u \text{ Int}} = 371.8 \text{ kip}$

Exterior Girder

Service Load

$$V_{s_Ext} = max(V_{s_Span1Ext}, V_{s_Span2Ext})$$
 $V_{s_Ext} = 243.7 \text{ kip}$

Factored Load

$$V_{u_Ext} = max(V_{u_Span1Ext} \text{ , } V_{u_Span2Ext}) \qquad \qquad V_{u_Ext} = 371.8 \text{ kip}$$

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: $M_{\text{negServ}} = -1267.9 \text{ kip} \cdot \text{ft}$

Factored Load: $M_{negUlt} = -1812.0 \text{ kip} \cdot \text{ft}$

Maximum Torsion and Concurrent Shear and Moment (Strength I)

 $T_u = 830 \text{ kip} \cdot \text{ft}$ Located two stations away from

centerline of column.

 $V_{
m u}=481.8~{
m kip}$ $V_{
m u}$ and $M_{
m u}$ values are from

 $M_u = 769.1 \text{ kip} \cdot \text{ft}$ CAP18

4.5.5 Locate and Describe Reinforcing

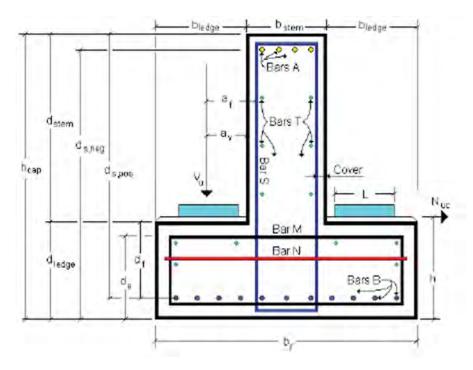


Figure 4.93 Section View of 60 Degrees Skewed ITBC

Recall:

 $b_{stem} = 42 in$

 $d_{stem} = 57 in$

 $b_{ledge} = 25 in$

 $d_{ledge} = 28 \ in$

 $b_f = 92 \ in$

 $h_{cap}=85\ in$

cover = 2.5 in

4.5.5.1 Describe Reinforcing Bars

Use # 11 bars for Bar A

$$A_{bar_A} = 1.56 \ in^2 \qquad \qquad d_{bar_A} = 1.410 \ in$$

Use # 11 bars for Bar B

$$A_{bar_B} = 1.56 \ in^2 \qquad \qquad d_{bar_B} = 1.410 \ in$$

Use # 7 bars for Bar M

$$A_{bar_M} = 0.60 \, in^2$$
 $d_{bar_M} = 0.875 in$ Bar M was considered. Bar M must be # 7 or smaller to allow it

Use # 7 bars for Bar N

 $A_{bar\ N}=0.60\ in^2$ $d_{bar \ N} = 0.875 \text{ in}$

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

M.

fully develop.

In the calculation of b_{ledge} , # 7

$$A_{bar_S} = 0.44 \text{ in}^2$$
 $d_{bar_S} = 0.75 \text{ in}$

Use # 6 bars for Bar T

Use # 6 bars for Bar S

$$A_{bar_T} = 0.44 \ in^2 \qquad \qquad d_{bar_T} = 0.75 \ in$$

4.5.5.2 Calculate Dimensions

$$d_{s_neg} = h_{cap} - cover - \frac{1}{2} d_{bar_S} - \frac{1}{2} d_{bar_A} \qquad \qquad d_{s_neg} = 81.42 \ in$$

$$d_{s_pos} = h_{cap} - cover - \frac{1}{2} max(d_{ba_S}, d_{bar_M}) \ - \frac{1}{2} d_{bar_B} \qquad \quad d_{s_pos} = 81.36 \ in$$

$$a_v = 12 in$$

$$a_f = a_v + cover$$
 $a_f = 14.50 in$

$$d_e = d_{ledge} - cover$$
 $d_e = 25.50 in$

$$d_f = d_{ledge} - cover - \frac{1}{2} d_{bar_M} - \frac{1}{2} d_{bar_B} \qquad \qquad d_f = 24.36 \text{ in}$$

$$h = d_{ledge} + BrgSeat$$
 $h = 29.50 in$

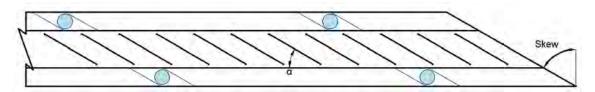


Figure 4.94 Plan View of 60 Degrees Skewed ITBC

$$\alpha = 30 \deg$$

Recall:

L = 15 in

W = 15 in

Angle of Bars S (Angle from the horizontal)

Dimension of Bearing Pad (15" Dia. Circular Bearing Pad)

4.5.6 **Check Bearing**

The load on the bearing pad propagates along a truncated pyramid whose top has the area A₁ and whose base has the area A2. A1 is the loaded area (the bearing pad area: LxW). A2 is the area of the lowest rectangle contained wholly within the support (the Inverted Tee Cap). A2 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.

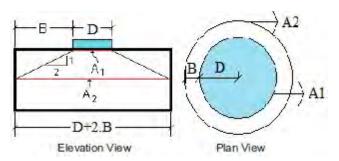


Figure 4.95 Bearing Check for 60 Degrees Skew Angle

Resistance Factor (ϕ) = 0.7

$$A_1 = \frac{\pi}{4} d_1^2$$

$$d_1 = 15in$$
, $A_1 = 176.71 in^2$

(AASHTO LRFD 5.5.4.2)

Area under Bearing Pad

Interior Girders

$$\begin{split} B &= \min \left[\left(b_{\mathrm{ledge}} - a_{\mathrm{v}} \right) - \frac{1}{2} L, \left(a_{\mathrm{v}} + \frac{1}{2} b_{\mathrm{stem}} \right) \right. \\ &\left. - \frac{1}{2} L, 2 d_{\mathrm{ledge}}, \frac{1}{2} S - \frac{1}{2} W \right] \end{split}$$

B = 5.5 in.

Diameter of truncated area, $d_2 = = d_1 + 2 \cdot B$

Base of the truncated pyramid, $A_2 = \frac{\pi}{4} \ d_2^{\ 2}$

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

$$d_2 = 26 \text{ in}$$

$$A_2 = 530.93 \text{ in}^2$$

Modification factor

$$m = \min\left(\sqrt{\frac{A_2}{A_1}}, 2\right) = 1.73 \text{ and } 2 \quad m = 1.73$$

AASHTO LRFD Eq. 5.6.5-3

 $\phi V_n = \phi \ 0.85 \ f_c \ A_1 \ m$

 $\phi V_n = 909.48 \text{ kips}$

AASHTO LRFD Eqs. 5.6.5-1 and 5.6.5-2.

 $V_{u \text{ Int}} = 371.8 < \phi V_{n}$

BearingChk = "OK!"

Vu int from "4.5.4.4 Load Summary".

Exterior Girders

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

B=5.5 in.

"B" is the distance from perimeter of A_1 to the perimeter of A2 as seen in the above figure

Diameter of truncated area, $d_2 = = d_1 + 2 \cdot B$

Base of the truncated pyramid, $A_2 = \frac{\pi}{4} d_2^2$

 $d_2 = 26 \text{ in}$

 $A_2 = 530.93 in^2$

Modification factor

$$m = min(\sqrt{\frac{A_2}{A_1}}, 2) = 1.73 \text{ and } 2 \quad m = 1.73$$

AASHTO LRFD Eq. 5.6.5-3

 $\phi V_n = \phi \quad 0.85 \quad f_c \quad A_1 \quad m$

 $\phi V_n = 909.48 \text{ kips}$

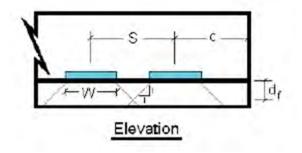
AASHTO LRFD Eqs. 5.6.5-1 and 5.6.5-2:

 $V_{u \text{ ext}} = 371.8 \text{ kips} < \Phi V_n$

BearingChk= "OK!"

Vu ext from "4.5.4.4 Load Summary".

4.5.7 Check Punching Shear



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

Figure 4.96 Punching Shear Check for 60 Degrees Skew Angle

Resistance Factor (ϕ) = 0.90

AASHTO LRFD 5.5.4.2.

Determine if the Shear Cones Intersect

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

$$\frac{1}{2}S - \frac{1}{2}W = 40.5$$
 in

$$d_f = 24.36 \text{ in}$$

$$\operatorname{Is} \frac{1}{2} b_{\text{stem}} + a_{\text{v}} - \frac{1}{2} L \ge d_{\text{f}}?$$

$$\frac{1}{2}b_{\text{stem}} + a_{\text{v}} - \frac{1}{2}L = 25.5 \text{ in}$$

 $d_{\text{f}} = 24.36 \text{ in}$

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

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

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

Interior Girders

$$V_n = 0.125 \ \text{?} \ \lambda \sqrt{f_c'} \ b_o \ d_f$$

$$V_n = 581.39 \,\mathrm{kips}$$

$$b_o = \frac{\pi}{2} * \left(D + d_f \right) + D$$

$$b_o = 76.82 in$$

 $\phi V_n = 523.25 \text{ kips}$

$$V_{u \text{ Int}} = 371.25 \text{ kips} < \phi V_{n}$$

$$V_{u_int}$$
 from "4.5.4.4 Load Summary".

Exterior Girders

$$\begin{array}{lll} V_n = & \min[\, 0.125 \cdot \sqrt{f_c} & V_n \, = \, 424.96 \, \text{kips} & \textit{AASHTO LRFD} \\ & \cdot \left(\frac{\pi}{4} \cdot (D + d_f) + \frac{D}{2} + c \, \right) & 5.8.4.3.4-3 \, \textit{and} \\ & \cdot d_f, 0.125 \cdot \sqrt{f_c} \cdot \frac{\pi}{2} \cdot (D + d_f) & \\ & + D \,] \end{array}$$

$$\phi V_n = 382.46 \text{ kips}$$

$$V_{u_ext} = 371.8 \text{ kips} < \phi V_n$$
 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

Interior Girders

 $= \min (63 \text{ in}, 96 \text{ in})$

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

$$b_{s Int} = 63 in$$

$$A_{cv} = b_{s \text{ Int}} \cdot d_e \qquad \qquad A_{cv} = 1606.5 in^2$$

Exterior Girders

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

$$= \min [69, 96, 48]$$

$$=48 \text{ in}$$

$$A_{cv} = b_{s_ext} \cdot d_e \qquad \qquad A_{cv} = 1224 \text{ in 2}$$

Interior Girders

$$V_n = min(0.2 \cdot f_c \cdot A_{cv}, 0.8 \cdot A_{cv})$$
 $V_n = 1285.2 \text{ kips}$ AASHTO LRFD 5.8.4.2.2-1 and 5.8.4.2.2-2

$$\phi V_n = 1156.68 \text{ kips}$$

$$V_{u_Int} = 371.68 \text{ kips} < \phi V_n$$
 ShearFrictionChk= "OK!" V_{u_int} from "4.5.4.4 Load Summary"

٠

Exterior Girders

$$V_{n} = min(0.2 \cdot f_{c} \cdot A_{cv}, 0.8 \cdot A_{cv})$$
 $V_{n} = 979.2 \text{ kips}$ AASHTO LRFD 5.8.4.2.2-1 and 5.8.4.2.2-2

$$\varphi V_n \, = 881 \, \text{kips}$$

$$V_{u_{\text{ext}}} = 371.81 \text{ kips} < \phi V_{n}$$
 ShearFrictionChk= "OK!" $V_{u_{\text{ext}}}$ from "4.5.4.4 Load Summary".

4.5.9 Flexural Reinforcement for Negative Bending (Bars A)

$$\begin{split} M_{dl} &= \left| M_{negDL} \right| & M_{dl} = 844.9 \text{ kip} \cdot \text{ft} \\ M_{s} &= \left| M_{negServ} \right| & M_{s} = 1267.9 \text{ kip} \cdot \text{ft} \\ M_{u} &= \left| M_{negUlt} \right| & M_{u} = 1812.0 \text{ kip} \cdot \text{ft} \end{split}$$

4.5.9.1 Minimum Flexural Reinforcement

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

	Gross Moment of Inertia
	Depth of Cap
	Distance to the Center of Gravity of the Cap from the bottom of the Cap
$f_r = 0.537 \text{ ksi}$ $y_t = 50.50 \text{ in}$	Modulus of Rupture (BDM- LRFD, Ch. 4, Sect. 5, Design Criteria)
	Distance from Center of Gravity to extreme tension fiber
$S = 6.06 \times 10^4 \text{ in}^3$	Section Modulus for the extreme tension fiber
$M_{cr} = 2711.8 \text{kip} \cdot \text{ft}$	Cracking Moment (AASHTO LRFD Eq. 5.6.3.3-1)
	Design the lesser of $1.2M_{cr}$ or
	$1.33M_u$ when determining
mininum area of steel required	mininum area of steel required.
	$y_t = 50.50 \text{ in}$ $S = 6.06 \times 10^4 \text{ in}^3$

4.5.9.2 Moment Capacity Design

Try,
$$7 \sim #11$$
's Top

BarANo = 7

 $d_{bar\ A} = 1.410 \text{ in}$

 $A_{\text{bar A}} = 1.56 \text{ in}^2$

 $A_s = BarANo \cdot A_{bar_A}$

 $d_{stirrup} = d_{bar_S}$

 $d = d_{s_neg} \\$

 $b = b_f$

 $f_c = 5.0 \text{ ksi}$

 $f_v = 60 \text{ ksi}$

 $\beta_1 = 0.85 - 0.05(f_c - 4ksi)$

Bounded by: $0.65 \le \beta_1 \le 0.85$

 $c = \frac{A_s f_y}{0.85 \ _c \beta_1 b}$

Number of bars in tension

Diameter of main reinforcing

bars

Area of main reinforcing bars

Area of steel in tension

Diameter of shear reinforcing

bars

Compressive Strength of Concrete

Yield Strength of Rebar

(AASHTO LRFD 5.6.2.2)

Depth of Cross Section under

Compression under Ultimate Load

(AASHTO LRFD Eq. 5.6.3.1.2-4)

Depth of Equivalent Stress Block

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.

$$a = c \cdot \beta_1$$

$$a = 1.67 in$$

 $A_s = 10.92 \text{ in}^2$

d = 81.42 in

b = 92 in

 $\beta_1 = 0.80$

c = 2.09 in

 $d_{\text{stirrup}} = 0.75 \text{ in}$

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.

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) \cdot \frac{1 ft}{12 in}$$

$$M_n = 4400 \text{ kip} \cdot \text{ft}$$

$$\epsilon_s = 0.003 \cdot \frac{d-c}{c}$$

$$\varepsilon_{\rm s}=0.114$$

Nominal Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.2-1)

(AASHTO LRFD 5.6.2.2)

Strain in Reinforcing at Ultimate

 $\epsilon_{\rm s} > 0.005$

FlexureBehavior = "Tension Controlled"

$$\Phi_{\text{M}} = 0.90$$

$$M_r = \Phi_M M_n$$

$$M_f = 2410 \text{ kip} \cdot \text{ft} < M_r$$

$$M_{ij} = 1812 \text{ kip} \cdot \text{ft} < M_{r}$$

$M_r = 3960 \text{ kip} \cdot \text{ft}$

(AASHTO LRFD 5.6.2.1)

(AASHTO LRFD 5.5.4.2)

Factored Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.1-1)

4.5.9.3 Check Serviceability

To find s_{max} :

Modular Ratio:

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

Tension Reinforcement Ratio:

$$\rho = \frac{A_s}{b \cdot d} \qquad \qquad \rho = 0.00146$$

$$k=\sqrt{(2\rho n)+(\rho n)^2}-(\rho n) \qquad \qquad k=0.134$$

$$d \cdot k = 10.91 \text{ in } < d_{ledge} = 28 \text{ in}$$

Therefore, the compression force acts over a rectangular

area.
$$j = 1 - \frac{k}{3}$$
 $j = 0.955$

$$f_{ss} = \frac{M_s}{A_s \cdot j \cdot d} \cdot \frac{12in}{1ft}$$
 $f_{ss} = 17.92 \text{ ksi}$

$$f_a = 0.6f_v$$
 $f_a = 36.00 \text{ ksi}$

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

 $d_c = cover + \frac{1}{2}d_{stirrup} + \frac{1}{2}d_{bar_A}$ $d_c = 3.58 in$

Exposure Condition Factor:

$$\gamma_e = 1.00$$

$$\beta_{s} = 1 + \frac{d_{c}}{0.7(h_{cap} - d_{c})}$$
 $\beta_{s} = 1.06$

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

$$s_{Actual} = \frac{b_{stem} - 2d_c}{Bar A No-}$$
 $s_{Actual} = 5.81 \text{ in}$

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

For service loads, the stress on the cross-section is located as shown in Figure 4.97.

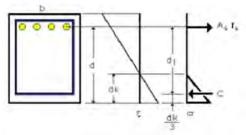


Figure 4.97 Stresses on the Cross Section for Service Loads of 60 Degrees Skewed ITBC

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

Service Load Bending Stress in outer layer of the reinforcing.

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

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)

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

4.5.9.4 Check Dead Load

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

$$M_a = A_s \cdot d \cdot j \cdot f_{dl} \cdot \frac{1ft}{12in}$$
 $M_a = 1556.7 \text{ kip} \cdot ft$ $M_{dl} = 844.9 \text{ kip} \cdot ft < M_a$ DeadLoadMom = "OK!"

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

Allowable Dead Load Moment

4.5.10 Flexural Reinforcement for Positive Bending (Bars B)

$$\begin{split} M_{dl} &= M_{posDL} & M_{dl} &= 582.2 \text{ kip} \cdot \text{ft} \\ M_{s} &= M_{posServ} & M_{s} &= 1067.0 \text{ kip} \cdot \text{ft} \\ M_{u} &= M_{posUlt} & M_{u} &= 1585.8 \text{ kip} \cdot \text{ft} \end{split}$$

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
$y_t = ybar$	$y_t = 34.5 \text{ in}$	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
$M_{cr} = S \cdot f_r \cdot \frac{_{1ft}}{_{12in}}$	$M_{cr} = 3969.3 \text{ kip} \cdot \text{ft}$	Cracking Moment (AASHTO LRFD Eq. 5.6.3.3-1)
$M_f = minimum of:$		Design the lesser of $1.2M_{cr}$ or
$1.2M_{cr} = 4763.2 \text{ kip} \cdot \text{ft}$		$1.33M_u$ when determining
$1.33 M_u = 2109.1 \mathrm{kip} \cdot \mathrm{ft}$		mininum area of steel required.

4.5.10.2 Moment Capacity Design

Try,
$$11 \sim #11$$
's Bottom

BarBNo = 11

 $d_{bar B} = 1.41 in$

 $A_{bar B} = 1.56 in^2$

 $A_s = BarBNo \cdot A_{bar B}$

 $d = d_{s \text{ nos}}$

 $b = b_{stem}$

 $f_c = 5.0 \text{ ksi}$

 $f_v = 60 \text{ ksi}$

 $\beta_1 = 0.85 - 0.05(f_c - 4ksi)$

Bounded by: $0.65 \le \beta_1 \le 0.85$

 $c = \frac{A_s f_y}{0.85 \ c \beta_1 b}$

Number of bars in tension

Diameter of main reinforcing

bars

Area of main reinforcing bars

Area of steel in tension

Compressive Strength of Concrete

Yield Strength of Rebar

(AASHTO LRFD 5.6.2.2)

Depth of Cross Section under

Compression under Ultimate Load

(AASHTO LRFD Eq. 5.6.3.1.2-4)

c = 7.21 inThis "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.

$$a = c \cdot \beta_1$$

$$a = 5.77 in$$

 $A_s = 17.16 \text{ in}^2$

d = 81.36 in

b = 42 in

 $\beta_1 = 0.80$

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.

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) \cdot \frac{1 f t}{12 i n}$$

$$\varepsilon_{\rm s} = 0.003 \cdot \frac{\rm d-c}{\rm c}$$

$$\varepsilon_{\rm s} > 0.005$$

Depth of Equivalent Stress Block (AASHTO LRFD 5.6.2.2)

 $M_n = 6733.2 \text{ kip} \cdot \text{ft}$

 $\varepsilon_s = 0.031$

Nominal Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.2-1)

Strain in Reinforcing at Ultimate

FlexureBehavior = "Tension Controlled"

$$\Phi_{\rm M} = 0.90$$

$$M_r = \Phi_M \cdot M_n$$

$$M_r = 6059.9 \text{ kip} \cdot \text{ft}$$

Factored Flexural Resistance (AASHTO LRFD Eq. 5.6.3.2.1-1)

$$M_f = 2109.1 \text{ kip} \cdot \text{ft} < M_r$$
 MinReinfChk = "OK!"

$$M_u = 1585.8 \, \text{kip} \cdot \text{ft} < M_r$$

4.5.10.3 Check Serviceability

To find s_{max} :

Modular Ratio:

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

Tension Reinforcement Ratio:

$$\rho = \frac{A_s}{b \cdot d} \qquad \qquad \rho = 0.005$$

$$k = \sqrt{(2\rho n) + (\rho n)^2} - (\rho n)$$
 $k = 0.234$

$$d \cdot k = 19.04 \text{ in } < d_{stem} = 57.00 \text{ in}$$

Therefore, the compression force acts over a rectangular area.

$$j = 1 - \frac{k}{3}$$
 $j = 0.922$

$$f_{ss} = \frac{M_s}{A_s \cdot i \cdot d} \cdot \frac{12in}{1ft} \qquad \qquad f_{ss} = 9.95 \text{ ksi}$$

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

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

 $d_c = cover + \frac{1}{2}d_{stirrup} + \frac{1}{2}d_{bar_B}$ $d_c = 3.64 in$

Exposure Condition Factor:

$$\gamma_e = 1.00$$

$$\beta_s = 1 + \frac{d_c}{0.7(h_{can} - d_c)}$$

$$\beta_s = 1.06$$

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

Bars Inside Stirrup Bar S

Try: BarBInsideSNo = 5

$$s_{Actual} = \frac{b_{stem} - 2\left(cover + \frac{1}{2}d_{bar_S} + \frac{1}{2}d_{bar_B}\right)}{BarBInsideSNo -}$$

$$s_{actual} < s_{max}$$

For service loads, the stress on the cross-section is located as shown in Figure 4.98.

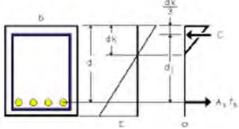


Figure 4.98 Stresses on the Cross Section for Bars B for Service Loads of 60 Degrees Skewed ITBC

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

Service Load Bending Stress in outer layer of the reinforcing.

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

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)

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

Number of Bars B that are inside Stirrup Bar S.

$$s_{Actual} = 8.71 \text{ in}$$

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} : $f_{dl} = 22 \text{ ksi}$

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

$$M_a = A_s \cdot d \cdot j \cdot f_{dl} \cdot \frac{_{1ft}}{_{12in}}$$

$$M_a = 2360 \text{ kip} \cdot \text{ft}$$

Allowable Dead Load Moment

$$M_{dl} = 582.2 \text{ kip} \cdot \text{ft} < M_a$$

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.

$$s_{bar_M} = 5.80 \text{ in}$$

$$s_{bar \ N} = 5.80 \text{ in}$$

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

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

4.5.11.1 Determine Distribution Widths

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

<u>Distribution Width for Shear (AASHTO LRFD 5.8.4.3.2)</u>

Interior Girders

$$b_{s_Int} = min(W + 4a_v, S)$$

 $b_{s_Int} = 63.00 in$

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

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

$$b_{m \text{ Int}} = 87.50 \text{ in}$$

Exterior Girders

$$b_{m_{\text{Ext}}} = \min(W + 5a_f, 2c, S)$$
$$b_{m_{\text{Ext}}} = 48.00 \text{ in}$$

4.5.11.2 Reinforcing Required for Shear Friction

AASHTO LRFD 5.7.4.1

$$\Phi = 0.90$$

$$u = 1.4$$

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

Minimum Reinforcing (AASHTO LRFD Eq. 5.7.4.2-1)

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

Recall:
$$d_e = 25.50$$
 in

(AASHTO LRFD 5.5.4)

"u" 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.

$$a_{\text{vf_min}} = \frac{0.05 \text{ksi} \cdot d_e}{f_v}$$

 $A_{vf_min} = \frac{0.05 \text{ ksi} \cdot A_{cv}}{f_{v}}$

$$a_{\text{vf_min}} = 0.26 \frac{\text{in}^2}{\text{ft}}$$

 $a_{vf_min} = 0.26 \frac{in^2}{ft}$ Minimum Reinforcing required for

Interior Girders

$$A_{cv} = d_e \cdot b_{s,Int}$$

 $a_{vf_Int} = \frac{A_{vf}}{b_{s_Int}}$

$$A_{cv} = 1606.5 \text{ in}^2$$

$$V_{u \text{ Int}} = 371.8 \text{ kip}$$

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

$$\Phi V_n \geq V_u$$

$$\Phi \cdot \left[c_1 A_{cv} + \mu \left(A_{vf} f_v + P_c \right) \right] \ge V_{u}$$

From "4.5.4.4 Load Summary".

(AASHTO LRFD Eq. 5.7.4.3-3)

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

$$A_{vf} = \frac{\frac{V_{u.Int}}{\Phi} - c_1 A_{cv}}{f_y} - P_c$$

$$A_{vf} = 4.92 \text{ in}^2$$

$$f_{\text{Int}} = 0.94 \frac{\text{in}^2}{\text{ft}}$$

Required Reinforcing for Shear Friction

 $a_{vf_Int} = 0.94 \frac{in^2}{ft}$ Required Reinforcing for Shear Friction per foot length of cap

Exterior Girders

$$\begin{split} A_{cv} &= d_e \cdot b_{s_Ext} & A_{cv} = 1224 \text{ in}^2 \\ V_{u_Ext} &= 371.8 \text{ kip} & \textit{From "4.5.4.4 Load Summary".} \\ V_n &= c_1 A_{cv} + \mu (A_{vf} f_y + P_c) & (\textit{AASHTO LRFD Eq. 5.7.4.3-3}) \\ \Phi V_n &\geq V_u & (\textit{AASHTO LRFD Eq. 5.7.4.3-1 & AASHTO LRFD Eq. 5.7.4.3-2}) \\ \Phi \cdot \left[c_1 A_{cv} + \mu (A_{vf} f_y + P_c) \right] \geq V_u & (\textit{AASHTO LRFD Eq. 5.7.4.3-2}) \\ A_{vf} &= \frac{\frac{V_{u_Ext}}{\Phi} - c_1 A_{cv}}{f_y} - P_c}{f_y} & A_{vf} &= 4.92 \text{ in}^2 & \textit{Required Reinforcing for Shear Friction} \\ a_{vf_Ext} &= \frac{A_{vf}}{b_{s_Ext}} & a_{vf_Ext} &= 1.23 \frac{\text{in}^2}{\text{ft}} & \textit{Required Reinforcing for Shear Friction per foot length of cap} \end{split}$$

4.5.11.3 Reinforcing Required for Flexure

 $V_{u \text{ Int}} = 371.8 \text{ kip}$

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 "4.5.5.2 Calculate Dimensions"

From "4.5.4.4 Load Summary".

Interior Girders

$$\begin{split} N_{uc_Int} &= 0.2 \cdot V_{u_Int} & N_{uc_Int} = 74.4 \text{ kip} & (\textit{AASHTO LRFD 5.8.4.2.1}) \\ M_{u_Int} &= V_{u_Int} \cdot a_v + N_{uc_Int} (h - d_e) & M_{u_Int} = 397 \text{ kip} \cdot \text{ft} & (\textit{AASHTO LRFD Eq. 5.8.4.2.1-1}) \\ Use the following equations to solve for A_f: \\ & \Phi M_n &\geq M_{u_Int} & (\textit{AASHTO LRFD Eq. 1.3.2.1-1}) \\ M_n &= A_f f_y \left(d_e - \frac{a}{2} \right) & (\textit{AASHTO LRFD Eq. 5.6.3.2.2-1}) \\ c &= \frac{A_f f_y}{\alpha_1 f_c \beta_1 b_{m_Int}} & (\textit{AASHTO LRFD Eq. 5.6.3.1.2-4}) \\ \alpha_1 &= 0.85 & (\textit{AASHTO LRFD Eq. 5.6.3.1.2-4}) \\ a &= c \beta_1 & (\textit{AASHTO LRFD 5.6.2.2}) \\ a &= c \beta_1 & (\textit{AASHTO LRFD 5.6.2.2}) \\ a &= c \beta_1 & (\textit{AASHTO LRFD 5.5.4.2}) \\ Solve for A_f: & A_f &= 3.50 \text{ in}^2 & \textit{Required Reinforcing for Flexure} \\ a_{f_Int} &= \frac{A_f}{b_{m_Int}} & a_{f_Int} &= 0.48 \frac{\text{in}^2}{\text{ft}} & \textit{Required Reinforcing for Flexure} \\ &= per foot length of cap \\ \end{split}$$

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)

$$M_{u Ext} = V_{u Ext} \cdot a_v + N_{uc Ext}(h - d_e) M_{u Ext} = 397 \text{ kip} \cdot \text{ft}$$

Use the following equations to solve for A_f:

$$\Phi M_n \ge M_{u Ext}$$

(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)

$$a = c\beta_1$$

$$0.75 \le \Phi = 0.65 + 0.15 \left(\frac{d_e}{c} - 1\right) \le 0.90$$

AASHTO LRFD 5.5.4.2

Solve for
$$A_f$$
:

$$A_f = 3.53 \text{ in}^2$$

Required Reinforcing for Flexure

$$a_{f_Ext} = \frac{A_f}{b_{m_Ext}}$$

$$a_{f_Ext} = 0.88 \frac{in^2}{ft}$$

Required Reinforcing for Flexure

per foot length of cap

4.5.11.4 Reinforcing Required for Axial Tension

 $\Phi = 0.90$

AASHTO LRFD 5.5.4.2

(AASHTO LRFD 5.8.4.2.2)

Interior Girders:

$$N_{uc_Int} = 0.2 V_{u_Int}$$

 $N_{uc\ Int} = 74.4 \text{ kip}$

$$A_n = \frac{N_{uc_Int}}{\Phi f_v}$$

 $A_n = 1.38 \text{ in}^2$

Required Reinforcing for Axial Tension

$$a_{n_Int} = \frac{A_n}{b_{m_Int}}$$

 $a_{n_Int} = 0.19 \frac{in^2}{ft}$ Required Reinforcing for Axial Tension per foot length of cap

Exterior Girders:

$$N_{uc Ext} = 0.2V_{u Int}$$

 $N_{uc Ext} = 74.4 \text{ kip}$

$$A_n = \frac{N_{uc_Ext}}{\Phi f_y}$$

 $A_n = 1.38 \text{ in}^2$

Required Reinforcing for Axial Tension

$$a_{n_Ext} = \frac{A_n}{b_{m_Ext}}$$

$$a_{n_{\perp}Ext} = 0.35 \frac{in^2}{ft}$$

 $a_{n_Ext} = 0.35 \frac{in^2}{ft}$ Required Reinforcing for Axial Tension per foot length of cap

4.5.11.5 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

4.5.11.6 Check Required Reinforcing

Actual Reinforcing:

$$a_{s} = \frac{A_{bar_M}}{s_{bar_M}} \qquad a_{s} = 1.24 \frac{in^{2}}{ft} \qquad \begin{array}{l} \textit{Primary Ledge Reinforcing} \\ \textit{Provided} \end{array}$$

$$a_{h} = \frac{A_{bar_N}}{s_{bar_N}} \qquad a_{h} = 1.24 \frac{in^{2}}{ft} \qquad \begin{array}{l} \textit{Auxiliary Ledge Reinforcing} \\ \textit{Provided} \end{array}$$

$$\underbrace{Checks:}_{S} A_{s} \geq A_{s_min} \qquad (AASHTO LRFD 5.8.4.2.1)$$

$$A_{s} \geq A_{f} + A_{n} \qquad (AASHTO LRFD 5.8.4.2.2)$$

$$A_{s} \geq \frac{2A_{vf}}{3} + A_{n} \qquad (AASHTO LRFD Eq. 5.8.4.2.2-5)$$

$$A_{h} \geq 0.5(A_{s} - A_{n}) \qquad (AASHTO LRFD Eq. 5.8.4.2.2-6)$$

Check Interior Girders:

Bar M:

$$\begin{array}{lll} \text{Check if:} & a_{s} \geq a_{s_min} & \textit{(AASHTO LRFD 5.8.4.2.1)} \\ & a_{s} \geq a_{f_Int} + a_{n_Int} & \textit{(AASHTO LRFD 5.8.4.2.2)} \\ & a_{s} \geq \frac{2a_{vf_Int}}{3} + a_{n_Int} & \textit{(AASHTO LRFD Eq. 5.8.4.2.2-5)} \\ & a_{s} = 1.24 \frac{in^{2}}{ft} \\ & a_{s_min} = 1.02 \, \frac{in^{2}}{ft} & < \, a_{s} \\ & a_{f_Int} + a_{n_Int} = 0.67 \frac{in^{2}}{ft} & < \, a_{s} \\ & \frac{2a_{vf_Int}}{3} + a_{n_Int} = 0.82 \frac{in^{2}}{ft} & < \, a_{s} \end{array}$$

BarMCheck = "OK!"

Bar N:

Check if:
$$a_h \geq 0.5 \cdot \left(a_s - a_{n_Int}\right)$$
 (AASHTO LRFD Eq. 5.8.4.2.2-6)
$$a_s = \text{The maximum of:} \qquad \text{"a_s" in this equation is the steel} \\ a_{f_Int} + a_{n_Int} \qquad \text{required for Bar M, based on the} \\ \frac{2a_{vf_Int}}{3} + a_{n_Int} \qquad \text{LRFD 5.8.4.2.2. This is derived from} \\ a_s = 0.82 \frac{\text{in}^2}{\text{ft}} \qquad \text{the suggestion that Ah should not be} \\ \text{less than $A_{p}/2$ nor less than $A_{vp}/3$} \\ \text{(Furlong \& Mirza pg. 73 \& 74)}$$

$$0.5 \cdot (a_s - a_{n_Int}) = 0.32 \frac{in^2}{ft} < a_h$$

BarNCheck = "OK!"

Check Exterior Girders:

Bar M:

Check if:
$$a_{s} \ge a_{s_min}$$
 (AASHTO LRFD 5.8.4.2.1)
$$a_{s} \ge a_{f_Ext} + a_{n_Ext}$$
 (AASHTO LRFD 5.8.4.2.2)
$$a_{s} \ge \frac{2a_{vf_Ext}}{3} + a_{n_Ext}$$
 (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:
$$a_h \geq 0.5 \cdot \left(a_s - a_{n_Ext}\right) \qquad (AASHTO \ LRFD \ Eq. \ 5.8.4.2.2-6)$$

$$a_s = \text{The maximum of:} \qquad "a_s" \ in \ this \ equation \ is \ the \ steel \ required$$

$$for \ Bar \ M, \ based \ on \ the \ requirements \ for \ Bar \ M \ in \ AASHTO \ LRFD \ 5.8.4.2.2. \ This \ is \ derived \ from \ the \ suggestion \ that \ Ah$$

$$a_s = 1.15 \frac{\text{in}^2}{\text{ft}} \qquad should \ not \ be \ less \ than \ A_f/2 \ nor \ less \ than}$$

$$a_s = 1.15 \frac{\text{in}^2}{\text{ft}} \qquad should \ not \ be \ less \ than \ A_f/2 \ nor \ less \ than}$$

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$$a_s = 1.15 \frac{\text{in}^2}{\text{ft}} \qquad should \ not \ be \ less \ than}$$

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.

$$s_{bar S} = 6.80 in$$

$$A_{hr} = 2stirrups \cdot A_{bar S}$$

$$A_v = 2legs \cdot A_{hr}$$

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

$$A_{hr} = 0.88 \text{ in}^2$$

$$A_v = 1.76 \text{ in}^2$$

4.5.12.1 Check Minimum Transverse Reinforcement

$$b_v = b_{stem} \\$$

$$A_{v_min} = 0.0316\lambda \sqrt{f_c} \frac{b_v \cdot s_{bar_S}}{f_v}$$

 $\lambda = 1.0$ for normal weight concrete

$$b_v = 42 \text{ in}$$

(AASHTO LRFD Eq. 5.7.2.5-1) (AASHTO LRFD 5.4.2.8)

$$A_{v_min} = 0.34 \ in^2$$

$$A_v > A_{v \text{ min}}$$

MinimumSteelCheck = "OK!"

4.5.12.2 Check Service Limit State

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

Interior Girders

$$V_{all} = minimum of:$$

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{S_{har} s} \cdot (W + 3a_v) = 249 \text{ kip}$$

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)

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

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{har} s} \cdot S = 497 \text{ kip}$$

 $V_{all} = 249 \, \mathrm{kip}$

 $V_{s Int} = 243.7 \text{ kip} < V_{all}$

(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)

ServiceCheck = "OK!"

Exterior Girders

 V_{all} = minimum of:

Vall for the Interior Girder

$$\frac{A_{\text{hr}} \cdot \left(\frac{2}{3} f_y\right)}{s_{\text{har S}}} \cdot \left(\frac{W + 3a_v}{2} + c\right) = 249 \text{ kip}$$

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

(BDM-LRFD Ch.4, Sect. 5, Design Criteria

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

$$\frac{A_{hr} \cdot \left(\frac{2}{3} f_y\right)}{s_{bar,S}} \cdot \left(\frac{S}{2} + c\right) = 373 \text{ kip}$$

 $V_{all} = 249 \text{ kip}$

$$V_{s_Ext} = 243.7 \text{ kip} < V_{all}$$

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.)

ServiceCheck = "OK!"

(AASHTO LRFD 5.8.4.3.5)

4.5.12.3 Check Strength Limit State

$$\Phi = 0.90$$

(AASHTO LRFD Eq. 5.5.4.2)

Interior Girders:

 $V_n = minimum of:$

$$\frac{A_{hr} \cdot f_y}{s_{har S}} \cdot S = 745 \text{ kip}$$

(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

$$\frac{A_{\text{hr}} \cdot f_y}{s_{\text{bar S}}} \cdot \left(\frac{S}{2} + c\right) = 560 \text{ kip}$$

(AASHTO LRFD Eq. 5.8.4.3.5-2)

$$\left(0.063\sqrt{f_c} \cdot b_f \cdot d_f\right) + \frac{A_{hr} \cdot f_y}{s_{bar, S}} \left(\frac{W + 2d_f}{2} + c\right) = 808 \text{ kip } \frac{(AASHTO \ LRFD \ Eq. \ 5.8.4.3.5-3)}{m_{eq}}$$

 $V_n = 560 \text{ kip}$

 $\Phi V_n = 504 \text{ kip}$

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

$$V_{u Ext} = 371.8 \text{ kip} < \Phi V_{n}$$

UltimateCheck = "OK!"

4.5.12.4 Check Combined Shear and Torsion

 $d_v = 80.59 \text{ in}$

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.

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 :

$$\begin{split} &\Phi_V = 0.90 \\ &v_u = \frac{|v_u - (\Phi_V \cdot V_p)|}{\Phi_V \cdot b_V \cdot d_V} \\ &\frac{v_u}{f_c} = 0.03 \end{split} \qquad v_u = 0.16 \text{ ksi} \label{eq:vu}$$

Using Table B5.2-1 with
$$\frac{v_u}{f_c}=0.03$$
 and $\epsilon_x=0.001$ $\theta=36.4$ deg and $\beta=2.23$

$$\begin{split} \epsilon_x &= \frac{\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p| cot\theta - A_{ps} f_{po}}{2(E_s A_s + E_p A_{ps})} \\ \text{where } &|M_u| = 769.1 \text{ kip} \cdot \text{ft must be} > \big|V_u - V_p\big| d_v = 3236 \text{ kip} \cdot \text{ft} \\ &\epsilon_x = 1.23 \times 10^{-3} > 1.00 \times 10^{-3} \\ &\text{use } \epsilon_x = 1.00 \times 10^{-3}. \end{split}$$

$$V_p = 0 \text{ kip}$$

$$\begin{aligned} A_c &= b_{stem} \cdot \frac{h_{cap}}{2} & A_c &= 1785 \text{ in}^2 \\ s &= s_{bar_S} & s &= 6.80 \text{ in} \end{aligned}$$

(AASHTO LRFD Eq. 5.5.4.2)

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

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.

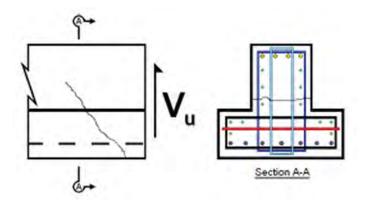
Strain halfway between the compressive and tensile resultants (AASHTO LRFD Eq. B5.2-3) If $\varepsilon_x < 0$, then use equation B5.2-5 and re-solve for ε_x .

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

" V_p " is zero as there is no prestressing.

(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

$$\begin{split} A_v &= 2 legs \cdot 2 stirrups \cdot A_{bar_S} & A_v &= 1.76 \ in^2 \\ A_t &= 1 leg \cdot A_{bar_S} & A_t &= 0.44 \ in^2 \\ A_{oh} &= (d_{stem}) \cdot (b_{stem} - 2 cover) + (d_{ledge} - 2 cover) \cdot (b_f - 2 cover) \\ & A_{oh} &= 4110 \ in^2 \\ A_0 &= 0.85 A_{oh} & A_0 &= 3493.5 in^2 \\ p_h &= (b_{stem} - 2 cover) + 2 \big(b_{ledge}\big) + (b_f - 2 cover) + 2 \big(h_{cap} - 2 cover\big) \\ p_h &= 334 \ in \end{split}$$

Equivalent Shear Force

$$V_{u_{-}Eq} = \sqrt{V_{u}^{2} + \left(\frac{0.9p_{h}T_{u}}{2A_{0}}\right)^{2}}$$
 $V_{u_{-}Eq} = 624.3 \text{ kip } (AASHTO LRFD Eq. B.5.2-1)$

Shear Steel Required

$$V_n$$
 = the lesser of:

$$V_c + V_s + V_p$$
 (AASHTO LRFD Eq. 5.7.3.3-1)
0.25 · f_c · b_v · d_v + V_p (AASHTO LRFD Eq. 5.7.3.3-2)

Check maximum ΦV_n for section:

$$\begin{split} \Phi V_{n_max} &= \Phi \cdot \left(0.25 \cdot f_c \cdot b_v \cdot d_v + V_p \right) \\ \Phi V_{n_max} &= 3808 \text{ kip} \\ V_u &= 481.8 \text{ kip} < \Phi V_{n_max} &\qquad & \\ &\qquad &\qquad & \\ &\qquad &\qquad & \\ &\qquad &\qquad & \\ &\qquad &\qquad & \\ &\qquad &\qquad & \\ &\qquad & \\ &\qquad & \\ &\qquad &\qquad &$$

Calculate required shear steel:

$$\begin{split} V_{u} &< \Phi V_{n} \\ V_{c} &= 0.0316 \cdot \beta \cdot \sqrt{f_{c}} \cdot b_{v} \cdot d_{v} \\ V_{c} &= 533 \text{ kip} \end{split} \qquad \textit{(AASHTO LRFD Eq. 1.3.2.1-1)} \\ V_{u} &< \Phi_{V} \cdot \left(V_{c} + V_{s} + V_{p}\right) \\ V_{s} &= \frac{A_{v} \cdot f_{y} \cdot d_{v} \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha}{s_{req}} \\ a_{v_req} &= \frac{\frac{V_{u}}{\Phi_{V}} - V_{c} - V_{p}}{f_{t} \cdot d_{v} \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha} \\ a_{v_req} &= 0.004 \frac{\sin^{2}}{f_{t}} \end{split}$$

Torsional Steel Required

$$\begin{split} \Phi_T &= 0.9 & (\textit{AASHTO LRFD 5.5.4.2}) \\ T_u &\leq \Phi_T T_n & (\textit{AASHTO LRFD Eq. 1.3.2.1-1}) \\ T_n &= \frac{2A_0 A_t f_y \cot \theta}{s_{bar_S}} & (\textit{AASHTO LRFD Eq. 5.7.3.6.2-1}) \\ a_{t_req} &= \frac{T_u}{\Phi_T 2A_0 f_y \cot \theta} & a_{t_req} &= 0.23 \frac{in^2}{ft} \end{split}$$

Total Required Transverse Steel

$$a_{req} = a_{v_req} + 2 sides \cdot a_{t_req} \qquad a_{req} = 0.46 \frac{in^2}{ft} \qquad \begin{array}{l} \textit{designed for the side of the section} \\ \textit{where the effects of shear and torsion} \\ a_{prov} = \frac{A_v}{s_{bar_S}} \qquad a_{prov} = 3.10 \frac{in^2}{ft} \qquad \begin{array}{l} \textit{are additive. (AASHTO LRFD} \\ \textit{C5.7.3.6.1)} \\ \end{array}$$

The transverse reinforcement is

Longitudinal Reinforcement

$$\begin{split} A_{ps}f_{ps} + A_sf_y &\geq \frac{|M_u|}{\Phi d_v} + \frac{0.5N_u}{\Phi} + \cdots \\ & cot\theta \sqrt{\left(\left|\frac{V_u}{\Phi} - V_p\right| - 0.5V_s\right)^2 + \left(\frac{0.45p_hT_u}{2A_0\Phi}\right)^2} \\ V_s &= a_{t_req} \cdot f_y \cdot d_v \cdot (cot\theta + cot\alpha) \cdot sin\alpha \\ & Bounded \ By: \ V_s < \frac{V_u}{\Phi_V} \\ & V_s = 535.3 \ kip \\ & \frac{|M_u|}{\Phi_f d_v} + \frac{0.5N_u}{\Phi_c} + cot\theta \sqrt{\left(\left|\frac{V_u}{\Phi_V} - V_p\right| - 0.5V_s\right)^2 + \left(\frac{0.45 \ hT_u}{2A_0\Phi_T}\right)^2} = 614 \ kip \end{split}$$
 Provided Force:

$$A_s f_v = 655.2 \text{ kip} > 614 \text{ kip}$$

LongitudinalReinfChk = "OK!"

4.5.12.5 Maximum Spacing of Transverse Reinforcement

(AASHTO LRFD 5.7.2.6)

Shear Stress

$$v_u = \frac{|v_u - \Phi_V v_p|}{\Phi_V b_v d_v}$$

$$v_u = 0.16 \text{ ksi} \qquad (AASHTO LRFD Eq. 5.7.2.8-1)$$

$$0.125 \cdot f_c = 0.625 \text{ ksi}$$

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

$$s_{\text{max}} = \min(0.8d_{\text{v}}, 24\text{in})$$

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

$$s_{\text{max}} = \min(0.4d_{\text{v}}, 12\text{in})$$

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

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

$$s_{max} = 12.00 \text{ in}$$
 Detailing)

$$s_{\text{bar_S}} = 6.80 \text{ in } < s_{\text{max}}$$
 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.

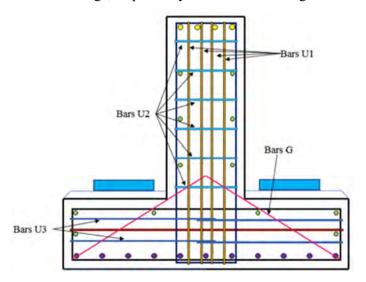


Figure 4.100 End Face Section View of 60 Degrees ITBC

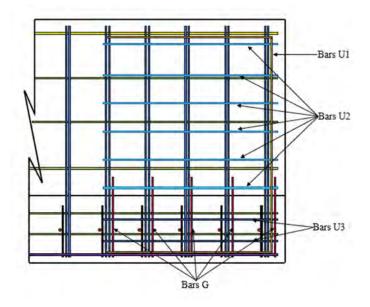


Figure 4.101 End Face Elevation View of 60 Degrees ITBC

4.5.14 Skin Reinforcement (Bars T)

Try $7 \sim \# 6$ bars in Stem and $3 \sim \# 6$ 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

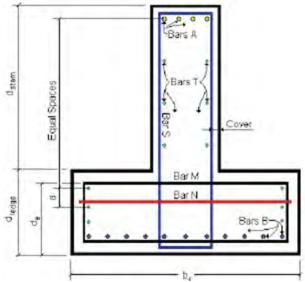


Figure 4.102 Section View for T Bars of 60 Degrees Skewed ITBC

(AASHTO LRFD 5.6.7)

4.5.14.1 Required Area of Skin Reinforcement

$$A_{\rm sk_Req} = 0.012 \cdot (d - 30)$$

$$A_{sk_Req} = 0.62 \frac{in^2}{ft}$$
 (AASHTO LRFD Eq. 5.6.7-3)

 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)

 $s_{req} = minimum of:$

$$\frac{A_{bar_T}}{A_{skReq}} = 8.52 \text{ in}$$

$$\frac{d_{s_neg}}{6} = 13.57 \ in$$

$$\frac{d_{s_{pos}}}{6} = 13.56 \text{ in}$$
 & 12 in

$$s_{reg} = 8.52 in$$

4.5.14.3 Actual Spacing of Skin Reinforcement

Check T Bars spacing in Stem:

$$\begin{split} h_{top} &= d_{stem} - \left(cover + \frac{d_{bar_S}}{2} + \frac{d_{bar_A}}{2}\right) + \left(cover + \frac{d_{bar_M}}{2} + \frac{d_{bar_T}}{2}\right) \\ h_{top} &= 56.73 \text{ in} \end{split}$$

$$s_{skStem} = \frac{h_{top}}{NoTBarsStem+}$$

$$s_{skStem} = 7.09 in$$

$$s_{skStem} < s_{req}$$

Check T Bars spacing in Ledge:

$$\begin{split} h_{bot} = d_{ledge} - \left(cover + \frac{d_{bar_M}}{2} + \frac{d_{bar_T}}{2}\right) - \left(cover + \frac{d_{bar_S}}{2} + \frac{d_{bar_B}}{2}\right) \\ h_{bot} = 21.11 \ in \end{split}$$

$$s_{skLedge} = \frac{h_{bot} - a}{NoTBarsLedge}$$

$$s_{skLedge} = 7.56 in$$

$$s_{skLedge} < s_{req}$$

SkinSpacing = "OK!"

Check if "a" is less than s_{req}

$$a = 6 in < s_{req}$$

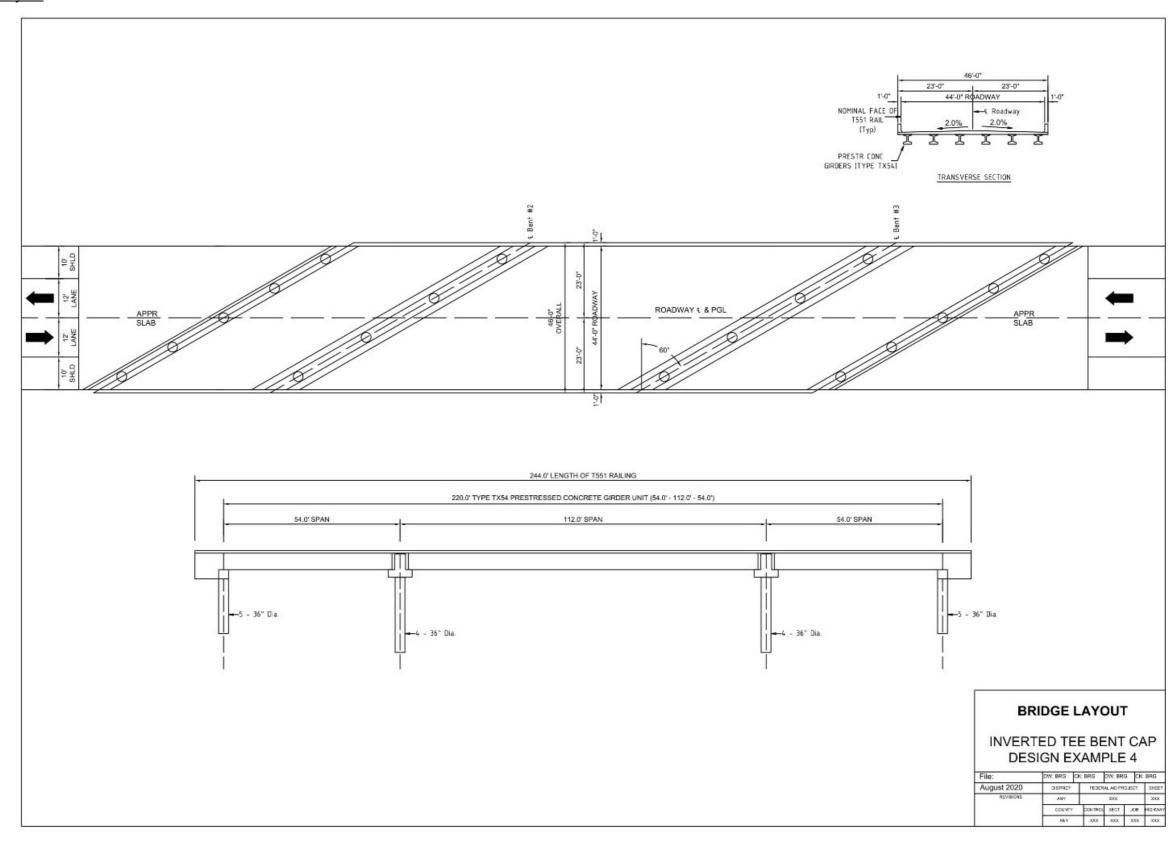
SkinSpacing = "OK!"

Skin Reinforcement Summary:

Use $7 \sim \# 6$ bars in Stem and $3 \sim \# 6$ bars in Ledge on each side

4.5.15 Design Details and Drawings

4.5.15.1 Bridge Layout



4.5.15.2 <u>AP 18 Input File</u>

```
CAP18 Version 6.00 ITBC Design Example 4, Skew = 60.00
$Problem Card ----
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
                                             S
                      X X X X
                                                                                  XXXXXXXXXX
Anly Opt (1=Working,
                    |-Movable Load Data--| 2=Load Factor,3=Both)
Num Increment |Num Start Stop Step|Anly| Load Factors:
Ŝ
                                       Inc Length
S
S
                     XX XXXXXXXX
                          0.5
                     92
Ś
    TABLE 2b
                  Max # |-----Live Load Reduction Factors-----|
    Overlay
$

        Load Factor
        Lanes | 1 lane
        2 lanes
        3 lanes
        4 lanes
        5 lanes

        XXXXX
        X
        XXXX
        XXXXX
        XXXX
        XXXX
        XXXX

$
STABLE 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 XX VCP - Shear Control Points
Ś
Ś
  XX XX XX XX XX XX (Num Inputs) 3 6 4 11 8
                                                       VCP - Shear Control Points
    Left Lane Boundary Stations
  Ś
$
  Station of Stringers (two rows max, may be at tenths of stations, XX.X)
                6 22 38 54 70
                                                  86
   Station of Supports (two rows max)
  Ś
S
   Moment Control Point Stations (two rows max)
               (Mom CP)
                    86
    Shear Control Point Stations (two rows max)
              $TABLE 4 - STIFFNESS AND LOAD DATA -----
                   Bending Sidewalk, Cap &
Station 1 if Stiffness Slab Stringer Moving
From To Cont'd of Cap Loads Loads Loads
                  From To Cont'd of Cap Loads
$XXXXXXXXXXXXX XXX
                         (CAP EI & DL)
                          90
                                   8.66E+07
                                                            -2.589
                                                                                     -5.04
(DL Span1, Bm1)
                      6
                           6
                                                             -50.17
(DL Span1, Bm2)
                     22
                           22
                                                             -50.17
                                                                                     -5.04
(DL Span1, Bm3)
                                                             -50.17
                                                                                     -5.04
(DL Span1, Bm4)
(DL Span1, Bm5)
                           54
                                                             -50.17
                                                                                     -5.04
                     54
                     70
                           70
                                                             -50.17
                                                                                     -5.04
(DL Span1, Bm6)
(DL Span2, Bm1)
                     86
                           86
                                                             -50.17
                                                                                     -5.04
                                                             -104.1
                                                                                     -10.5
(DL Span2, Bm2)
                                                             -104.1
                                                                                     -10.5
(DL Span2, Bm3)
(DL Span2, Bm4)
                     38
                                                             -104.1
                           38
                                                                                     -10.5
                     54
                           54
                                                             -104.1
                                                                                     -10.5
(DL Span2, Bm5)
(DL Span2, Bm6)
                     70
                           70
                                                             -104.1
                                                                                     -10.5
                                                             -104.1
                                                                                     -10.5
                                                                         -4.92
(Dist. Lane Ld)
                          20
(Conc. Lane Ld)
                     4
                           4
                                                                         -21.3
(Conc. Lane Ld)
                    16
                          16
                                                                         -21.3
```

4.5.15.3 CAP 18 Output File

TEXAS DEPARTMENT OF TRANSPORTATION (TxDOT) AUG 12, 2020 PAGE 1 CAP18 BENT CAP ANALYSIS Ver. 6.2 (Jul, 2011) HIGHWAY PD- CONTROL- CODED COUNTY NO IPE SECTION-JOB BY DATE NO 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) **ENVELOPES TABLE NUMBER** OF MAXIMUMS 2 3 4 KEEP FROM PRECEDING PROBLEM (1=YES) 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): 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

LIST OF LOAD COEFFICIENTS CORRESPONDING TO NUMBER OF LANES LOADED

2 3 1.000

1.200

AUG 12, 2020 TEXAS DEPARTMENT OF TRANSPORTATION (TxDOT) PAGE 2
CAP18 BENT CAP ANALYSIS Ver. 6.2 (Jul, 2011)

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 NUM OF NUM OF NUM MOM NUM SHEAR LANES STRINGERS SUPPORTS CONTR PTS 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

FIXED-OR-MOVABLE ------ FIXED-POSITION DATA ----- MOVABLESTA STA CONTD CAP BENDING SIDEWALK, STRINGER, OVERLAY POSITION
FROM TO IF=1 STIFFNESS SLAB LOADS CAP LOADS LOADS SLAB LOADS
(K-FT*FT) (K) (K) (K) (K)

(1811-11) (18) (18) (18)

2 90 0 86600000.000 0.000 -2.589 0.000 0.000 0.000 0.000 -50.170 -5.040 0.000 0.000 0.000 -50.170 -5.040 0.000 6 6 0 22 22 0 0.000 0.000 -50.170 -5.040 0.000 38 38 0 54 54 0 0.000 0.000 -50.170 -5.040 0.000 70 70 0 0.000 -50.170 -5.040 0.000 0.000 86 86 0 0.000 0.000 -50.170 -5.040 0.000 0.000 0.000 -104.100 -10.500 6 6 0 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 0.000 0.000 0.000 0.000 -4,920 0.000 0.000 0.000 0.000 -21.300 0.000 0.000 0.000 0.000 -21.300 0 20 0 4 4 0 0.000 16 16 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) DEFLECTIO	N (FT) MC	MENT (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 23	22.00	-0.000152	582.2 497.6	3.0	
24	23.00	-0.000150	497.6	-87.1	
25	24.00 25.00	-0.000141 -0.000128	313.0	-92.3 -97.5	
26	26.00	-0.000128	213.0	-97.5 -102.7	
27	27.00	-0.000112	107.7	-102.7	
28	28.00	-0.000093	-2.7	-113.0	
29	29.00	-0.000072	-118.3	-118.2	
30	30.00	-0.000032	-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	

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TABLE 4A. DEAD LOAD RESULTS (WORKING STRESS)

STA	DIST X (FT)	DEFLECTION	(FT) MO	MENT (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	

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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
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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

-41.4 6 0.0 0 0.0 0 1 0.0 1 0.0 2 0.0 0.0 3 0.0 3 0.0 0* 0*

10 -844.9 0.0 0 -352.5 1 2 -352.5 0.0 1 2 0.0 2 0.0 3 0.0 3 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*

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*

278.0 46 2 36 0 -55.6 1 9 2 36 1 -55.6 1 9 138.7 0 138.7 1 2 -55.6 3 63 2 0.0 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*

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 DEAD LD LANE POSITIVE LOAD AT LANE NEGATIVE LOAD AT STA EFFECT ORDER MAXIMUM LANE STA ORDER MAXIMUM LANE STA

70	582.2 0 1 2 3 0*	404.0 402.4 18.7 0.0	0 3 1	59 60 9 3 0*	0 -66. 1 -66. 2 0.0 0.0		2 36 2 36
82	-844.9 0 1 2 3 0*	0.0 0.0 0.0 0.0		0 1 2 3 0*	-352.5 -352.5 0.0 0.0	3	70 70
86	-41.4 0 1 2 3	0.0 0.0 0.0 0.0		0 1 2 3	0.0 0.0 0.0 0.0		

SHEAR (K)

AT DEAD LD LANE POSITIVE LOAD AT LANE NEGATIVE LOAD AT STA EFFECT ORDER MAXIMUM LANE STA ORDER MAXIMUM LANE STA

8	-200.9 0 1 2 3 0*	0.0 0.0 0.0 0.0	0 -88.1 1 2 1 -88.1 1 2 2 0.0 3 0.0 0*
12	139.6 0 1 2 3 0*	44.8 44.8 1.6 0.0	1 6 0 -5.6 2 36 1 6 1 -5.6 2 36 3 62 2 0.0 3 0.0
32	-133.7 0 1 2 3 0*	1.6 1.6 0.0 0.0	3 62 0 -54.6 0 15 3 62 1 -53.0 1 12 2 -11.2 2 32 3 0.0 0*
36	221.6 0 1 2 3 2*	87.6 84.1 30.7 0.0	0 28 0 -7.8 3 63 2 32 1 -7.8 3 63 1 12 2 0.0 3 0.0 0*
56	-221.6 0 1 2 3 0*	7.8 7.8 0.0 0.0	1 9 0 -87.6 0 44 1 9 1 -84.1 2 40 2 -30.7 3 60 3 0.0 2*
60	133.7 0 1 2 3 0*	54.6 53.0 11.2 0.0	0 57 0 -1.6 1 9 3 60 1 -1.6 1 9 2 40 2 0.0 3 0.0 0*
80	-139.6 0 1 2 3 0*	5.6 5.6 0.0 0.0	2 36 0 -44.8 3 66 2 36 1 -44.8 3 66 2 -1.6 1 9 3 0.0 0*
84	200.9 0 1 2 3 0*	88.1 88.1 0.0 0.0	3 70 0 0.0 3 70 1 0.0 2 0.0 3 0.0 0*

REACTION (K)

AT DEAD LD LANE POSITIVE LOAD AT LANE NEGATIVE LOAD AT STA EFFECT ORDER MAXIMUM LANE STA ORDER MAXIMUM LANE STA

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

((FT-K) (FT-K)		MAX + SHEAR K)	MAX - SHEAR
-1	-1 00	0.0	0.0	0.0	0.0	
0	0.00	0.0 0.0 0.0 -2.6 -10.4	0.0	0.0	0.0	
1	1.00	0.0	0.0	0.0 -1.3 -5.2	0.0	
2	2.00 3.00	-2.6	2.6	-1.3 -5.2	-1.3 -5.2	
	4.00	-10.4	-10.4	-10.4	-10.4	
5	5.00	-23.3	-23.3	-15.5	-15.5	
6	6.00	-23.3 -41.4	-41.4	-105.6	-158.5	
7	7.00	-234.5 -432.8	-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 -59.6	
10	10.00	-844.9	-1267.9	-13.7	-59.6	
11	11.00		-1072.8	198.6	138.1	
12	12.00	-493.5	-882.9	193.4 188.3	133.0	
13 14	14.00	-319.7 -151.1			400 6	
15	15.00		-518.6 -344.2	177 0	117.4	
16	16.00	174.5	-175.0	172.7	112.3	
17	17.00	174.5 332.8	-10.9	167.5	107.1	
18	18.00	488.0 638.9	135.9	162.4	101.9	
19	19.00	638.9	235.3	157.2	96.7	
20	20.00	786.0 928.5	329.4	152.0 146.8	91.5	
21	21.00	928.5	418.3	146.8	86.4	
22	22.00	1067.0 917.7 763.7 605.2	502.1	18.9	-10.2	
23	23.00	917.7	410.2	-85.3	-152.7	
24 25	24.00	/63./ 605.2	312.8	-90.4	-157.9	
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	275.0 115.8	-133.1	-106.0 -111.2	-178.6	
29	29.00	-45.8	-257.9	-116.3	-183.8	
30	30.00	-45.8 -201.8	-388.8	-121.5	-183.8 -188.9	
31	31.00	-325.9 -455.1	-575.3	-126.7	-194.1 -199.3	
32	32.00	-455.1	-770.5	-131.9	-199.3	
33	33.00	-589.6 -729.2	-970.9 -1176.6	-137.0		
34	34.00	-/29.2	-11/6.6	87.7	26.4	
35 36	35.00	-509.1	-832.4	341.6	217.5	
37	37.00	-294.2	-320.0	330.4	207.1	
38	38.00	312.9	-28.9	184.8	117.0	
39	39.00	-725.2 -509.1 -294.2 -0.1 312.9 345.8	19.3	45.5	26.9	
40	40.00	374.1	62.2	40.4		
41	41.00	397.8	62.2 100.0	35.2	21.8 16.6	
42	42.00	416.9	125.3	30.0	11.4	

TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA (DIST X	MAX + N FT-K) (иом мах FT-К) (X - MOM K) (F	MAX + SHEAR () 6.2 1.1 -4.1 -9.3 -14.5 -19.7 -24.8 -30.0 -35.2 -40.4 -45.5 -184.8 -331.2 -336.4 -341.6 -87.7 137.0 131.9 126.7 121.5 116.3 111.2 106.0 100.8 95.6 90.4 85.3 -18.9 -146.8 -152.0 -157.2 -162.4 -167.5 -177.9 -183.1 -188.3 -193.4 -198.6 13.7 206.1	MAX - SHEAR
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	/63./	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	/86.0	329.4	-91.5	-152.0	
73	73.00	638.9	235.3	-96./	-15/.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	1/4.5	-1/5.0	-112.3	-1/2./	
77	77.00	13.0	-344.2	-117.4	-1/7.9	
78	78.00	-151.1	-518.6	-122.6	-183.1	
79	79.00	-319./	-698.2	-127.8	-188.3	
80	80.00	-493.5	-882.9	-133.0	-193.4	
81	81.00	-6/1.3	-10/2.8	-138.1	-198.6	
82	82.00	-844.9	-1267.9	59.6	13./	
83	83.00	-636.3	-953.5	311.8	206.1	
0-1	84.00	-844.9 -636.3 -432.8 -234.5	-644.3	306.6	200.9	
85	85.00	-234.5 -41.4	-340.3	301.4	195./	
86	86.00	-41.4	-41.4	158.5	105.6	

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TABLE 6. ENVELOPES OF MAXIMUM VALUES (WORKING STRESS)

STA	DIST X	MAX+	мом м	1AX - MON	MAX + SHE	AR MAX - SHEAR
((FT)	(FT-K) (FT-K)	(K)	(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	

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TABLE 7. MAXIMUM SUPPORT REACTIONS (WORKING STRESS)

STA (DIST X FT)	MAX + (K)	REACT (K)	MAX - REACT
10	10.00	514.7	354	
34	34.00	554.9	364	
58	58.00	554.9	364	
82	82.00	514.7	354	.6

TABLE 5. MULTI-LANE LOADING SUMMARY (LOAD FACTOR) (*--CRITICAL NUMBER OF LANE LOADS)

MOMENT (FT-K)

0*

AT DEAD LD LANE POSITIVE LOAD AT LANE NEGATIVE LOAD AT STA EFFECT ORDER MAXIMUM LANE STA ORDER MAXIMUM LANE STA

MOMENT (FT-K)

AT DEAD LD LANE POSITIVE LOAD AT LANE NEGATIVE LOAD AT STA EFFECT ORDER MAXIMUM LANE STA ORDER MAXIMUM LANE STA

70 737.4 707.0 0 59 0 -116.8 2 36 704.1 3 60 1 -116.8 2 36 32.7 1 9 2 0.0 0.0 3 0.0 1 2 3 0* 0* 82 -1071.7 0 0.0 0 -616.9 3 70 1 0.0 -616.9 3 70 1 2 0.0 2 0.0 3 0.0 3 0.0 86 -51.8 0.0 0 0.0 0 0.0 0.0 1 1 2 0.0 0.0 3 0.0 3 0.0 0* 0*

BENT CAP ANALYSIS Ver. 6.2 (Jul, 2011)

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 -255.0 0 -154.2 1 2 1 -154.2 1 2 0 0.0 0.0 1 2 0.0 2 0.0 3 0.0 3 0.0 0* 0* 176.7 12 78.4 1 6 0 -9.7 2 36 0 78.4 1 6 1 -9.7 2.7 3 62 2 0.0 1 2 36 2 3 3 0.0 0.0 0* 0* 32 -168.9 0 1 0.0 3 0* 0* 36 280.9 153.2 0 28 0 -13.6 3 63 0 3 0.0 3 0.0 0* 2* 56 -280.9 13.6 1 9 0 -153.2 0 44 0 13.6 1 9 1 -147.2 2 40 1 2 -53.7 3 60 0.0 3 0.0 3 0.0 0* 2* 60 168.9 95.6 0 57 0 -2.7 1 9 92.7 3 60 1 -2.7 1 9 19.5 2 40 2 0.0 0 1 2 3 0.0 0.0 0* 80 -176.7 9.7 2 36 0 -78.4 3 66 9.7 2 36 1 -78.4 3 66 1 2 -2.7 1 9 2 0.0 3 0.0 3 0.0 0* 0* 255.0 84 154.2 3 70 0 0.0 154.2 3 70 1 0.0 0 1 2 0.0 0.0 3 3 0.0 0.0 0* 0*

REACTION (K)

AT DEAD LD LANE POSITIVE LOAD AT LANE NEGATIVE LOAD AT STA EFFECT ORDER MAXIMUM LANE STA ORDER MAXIMUM LANE STA

TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

	FT)	(FT-K) (FT-K) ((K) (I	 MAX + SHEAR <)	MAX - SHEAR
-1	4.00	0.0	0.0	0.0	0.0	
0	0.00	0.0 0.0 0.0 -3.2 -12.9 -29.1 -51.8	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	0.00	-548.8 -807.0	-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	-1071.7 -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	9/3.9	267.4	225.5	119.7	
20	20.00	-839.4 -597.4 -359.6 -128.3 97.6 321.6 542.6 760.7 973.9 1183.0 1386.6	383.9	219.0	113.2	
21	21.00	1386.6	493.8	212.5	106.7	
22	22.00	1585.8 1365.1	597.3	31.8	-19.1	
23 24	24.00	1120.1	340.5	112.0	-225.4 -231.9	
25	25.00	1138.6 907.0	2149.5	120.4	-231.9	
26	26.00	660.5	72.0	126.4	-230.4	
27	27.00	428.2	76.6	-120.0	-244.0	
28	28.00	203.0	-70.0	-133.3	-257.8	
29	29.00	669.5 428.2 203.0 -23.8	-394.9	-146.3	-264.2	
30	30.00	-/38 U	-5054	-13//	-//11/	
31	31.00	-394.0	-830.5 -1108.4	-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	-725.4 -900.8	-1683.6	132.6	-283.7 -290.1 25.2 271.1 264.6 258.1 143.6	
35	35.00	-626.4 -358.5	-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	533.5 566.2	20.4	55.1	22.6	
41	41.00	593.4 615.2	72.3	48.6	16.1	
42	42.00	615.2	104.9	42.2	9.6	
	43.00	630.6 639.4	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

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TABLE 6. ENVELOPES OF MAXIMUM VALUES (LOAD FACTOR)

		MAX + N (FT-K) (f			 MAX + SHEAR ()	MAX - SHEAR
49	49.00	630.6	127.5	-3.1	-35./	
50	50.00	615.2 593.4 566.2	72.2	-9.6	-42.2	
51	57.00	595.4	20.4	-10.1	-40.0 -55.1	
52	52.00	500.2	20.4	-22.0	-55.1	
22	55.00	333.3 40E 4	102.0	142.6	767.0	
55	55.00	533.5 495.4 50.5	102.0	259.1	-202.1	
56	56.00	250.5	760 1	-256.1	-4/3.3 -/101.0	
57	57.00	-358.5 -626.4 -900.8	-1102.1	-204.0	-401.0	
58	58.00	-900.4	-1683.6	-25.2	-400.2	
50	50.00	725.4	1202.0	200.1	172.0	
60	60.00	-725.4 -556.5 -394.0	-1392.6	290.1	165.7	
61	61.00	-330.3	-830.5	203.7	159.7	
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	-238.0 -23.8 203.0	-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	428.2 669.5 907.0	214.8	238.4	120.4	
68	68.00	1138.6	349.5	231.9	113.9	
69	69.00	1138.6 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	1386.6 1183.0	383.9	-113.2	-219.0	
73	73.00	973.9	267.4	-119.7	-225.5	
74	74.00	760.7 542.6	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	321.6 97.6 -128.3	-771.5	-152.0	-257.8	
79	79.00	-359.6	-1021.9	-158.5	-264.3	
80	80.00	-597.4 -839.4	-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	-1071.7 -807.0 -548.8	-918.9	440.0	255.0	
85	85.00	-297.1	-482.1	433.6	248.5	
86	86.00	-51.8 -29.1	-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

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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)

.....

ST		X MAX (FT-K)			IOM MAX (K)	(+ SHEAR	MAX -	SHEAR
93	93.00	0.0	0.0	0.0	0.0			

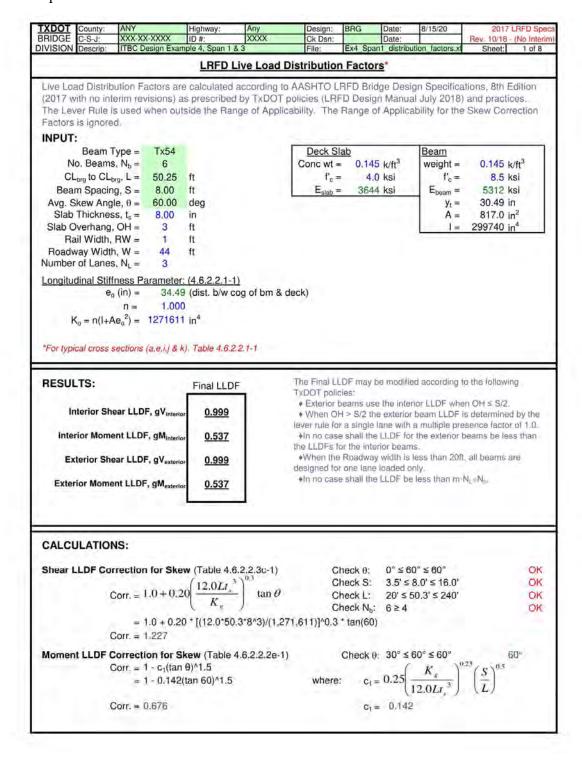
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TABLE 7. MAXIMUM SUPPORT REACTIONS (LOAD FACTOR)

			-
STA DIS		+ REACT (K)	MAX - REACT
10 10.	00 726	.1 445	5.8
34 34.	00 788.	.8 456	5.2
58 58.	00 788.	.8 456	5.2
82 82.	00 726.	.1 445	5.8

4.5.15.4 <u>Live Load Distribution Factor Spreadsheet</u>

4.5.15.4.1 Spans 1 & 3

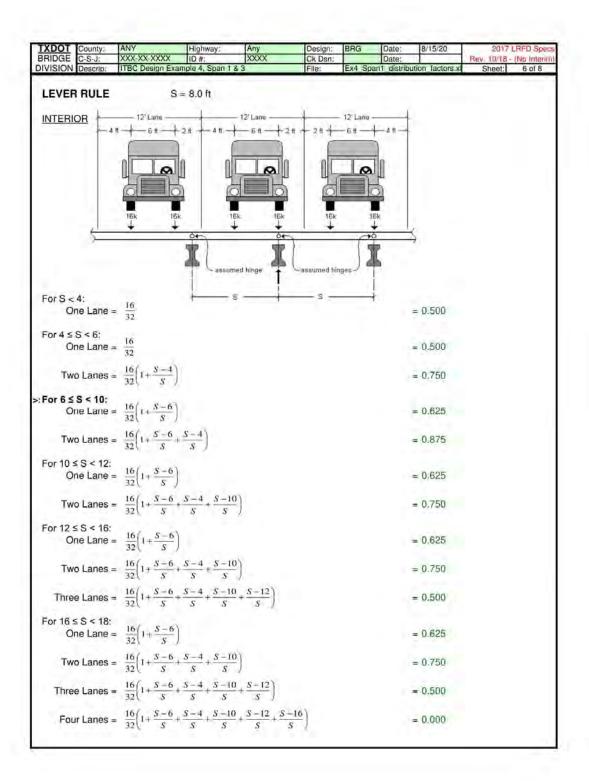


TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Spece
BRIDGE DIVISION	C-S-J: Descrip:	ITBC Design Exa	ID #:	XXXX & 3	Ck Dsn: File:	Ex4 So	Date:	ution factors.xl	Rev. 10/18 - Sheet:	2 of 8
	IOR BE		inple if epair !		p 1107	12		ment manerens	- Oncot.	20.0
		ution Per Lane	Table 4.6.2.2	2.3a-1):						
_		ne Loaded								
		Lever Rule	(Table 3.6.	1.1.2)						
		mg = 0.6	525 * 1.2 =	0.750						
		Modify for	or Skew:							
			skew corre	ection =	1.227					
			mg = 0.750	1.227 =	0.920					
		Equation $g = 0.36$	$6+\left(\frac{S}{25}\right)$							
			+ (8 / 25) =	0.680						
			or Skew:	46.44						
		100	skew corre	ction =	1.227					
			g = 0.680 *	1.227 =	0.834					
		Range of App	licability (RO	A) 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	l _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 Lo	paded							
		Lever Rule	(Table 3.6.	1.1.2)						
		200337.7.20	ax(0.875 * 1.0 or Skew:	, 0.875 * 0.8	35, 0.875 * 0	.65) =	0.875			
			skew corre	ction =	1.227					
			mg = 0.875	5 * 1.227 =	1.074					
		Equation g = 0.2	$+\left(\frac{S}{12}\right)-\left(\frac{S}{3}\right)$	$\left(\frac{S}{35}\right)^{2.0}$						
			+ (8 / 12) - (8 or Skew:	/ 35)^2.0 =	0,814					
			skew corre	ection =	1.227					
			g = 0.814 *	1.227 =	0.999					
		Range of App	licability (RO	A) Checks	(same as f	or one la	ane loade	ed)		
		Use Equation $gV_{int2+} =$	from Table 4 0.999	.6.2.2.3a-1	because all o	criteria is	OK.			
	TXDOT	Policy states g\	/literior must be	e ≥ m·N _L ÷N _e						
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6	=	0.425					
	Is W≥2	20ft ? Yes								
	TXDOT	Policy states the	at if W < 20tt.	gV _{interior} is t	he Maximun	of: gV	and m	NL+Nb		
>>	TXDOT	Policy states the	at if W ≥ 20ft,	gV _{Inletior} is t	he Maximum	of: gVin	11. gV _{int2+}	$m-N_L+N_b$.		
	gV _{inte}	erior = 0.999	11							

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	I have the same	LRFD Sper
BRIDGE	C-S-J: Descrip:	ITBC Design Exar	nple 4, Span 1 a	XXXX & 3	Ck Dsn:	Ex4 Spa	Date:	tion factors.x	Rev. 10/18 - Sheet:	3 of 8
INTER	IOR BE				1					
		ribution Per Lane	(Table 4.6.2	2.2b-1):						
Monto		ne Loaded	112010 1101	17.						
	One Eu	Lever Rule	(Table 3.6.	112)						
			25 * 1.2 =	0.750						
		Modify fo		0.700						
		Widding to	skew corre	ection =	0.676					
			mq = 0.750		0.507					
		Equation			- W.F					
		g = 0.06	$s + \left(\frac{S}{14}\right)^{0.4}$	$\left(\frac{S}{L}\right)^{0.3} \left(\frac{K}{12L}\right)^{0.3}$	r 3					
				* (8/50.3)^0.3	* *	11/(12*5	0.3*8^3))	^0.1 =	0.591	
		Modify fo		12/2/2007	y venue		21000.00			
		172.76	skew corre	ection =	0.676					
			g = 0.591 *	0.676 =	0.400					
		Range of Appl			3					
			3.5' ≤ 8.0'	.,		OK				
		Check ts	4.5" ≤ 8.0'	'≤ 12.0"		OK				
		Check L:	20' ≤ 50.3'	≤ 240'		ок				
		Check N	: 6≥4			OK				
		Check K	: 10,000 ≤ 1	1,271,611 ≤ 7	,000,000	OK				
		Use Equation				criteria is	OK.			
		gM _{int1} =	0.400	ATTEMPT TO	20-10-27, 10-13	7,110,110				
	Two or	More Lanes Lo	- 14 - 3							
	I WO OI	Lever Rule	(Table 3.6.	1 1 2)						
				, 0.875 * 0.8	5 0.875 * 0	651 -	0.875			
		Modify fo		, 0.073 0.0	J, 0.073 U	.00) -	0.075			
		lviddily ic	skew corre	ection -	0.676					
				5 * 0.676 =	0.592					
		Equation			- 10.1					
		g = 0.07	$75 + \left(\frac{S}{9.5}\right)^{0.5}$	$\frac{6}{L}$ $\left(\frac{S}{L}\right)^{0.2} \left(\frac{1}{12}\right)^{0.2}$	$\left(\frac{K_g}{Lt^{3}}\right)^{0.1}$					
				6 * (8/50.3)^4	4 /	611/(12	50.3*8^3	3))^0.1 =	0.795	
		Modify fo				38 - 19		*E. 12.10		
			skew corre	ection =	0.676					
			g = 0.795 *	0.676 =	0.537					
		Range of Appl	icability (RO	A) Checks		or one la	ne loade	d)		
		Use Equation						ō(=		
		gM _{int2+} =	0.537			,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,				
	TYDOT			No Mark						
	TXUUT.	Policy states glV								
	In Tay or	$m \cdot N_L \div N_b =$	0.85 * 3 / 6	=	0.425					
		20ft ? Yes Policy states tha	111W - 200	nM. is i	na Mavimus	Ala da	and m	N. = N.		
65										
>>		Policy states that	7	Alvintation (2 ()	ie waxiiiuli	, or givin	HI AMMS+	4 mundEsight		
	gM _{inte}	erior = 0.537								

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	2017 LRFD Spec
BRIDGE	C-S-J: Descrip:	ITBC Design Ex	ID #: ample 4. Span 1 &	XXXX 3	Ck Dsn:	Ex4 So	Date:	ution factors.x	Rev. 10/18 - (No Interim Sheet: 4 of 8
	RIOR BE			-	1, 1101				
		ution Per Lane	(Table 4 6 2 2	3b-11:					
Oneu I		ne Loaded	114010 1.0.2.2.	00 17.					
	One Eu	Lever Rule	(Table 3,6,1	1.21					
			625 * 1.0 =		TypoTus	es a mi	Itinla pro	conce facto	r of 1,0 for one
		30.000	for Skew:	0.000	lane loade				di Na idi dila
		.,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	skew correc	tion =	1.227				
			mg = 0.625		0.767				
		Use Lever Ri	ule, as per AAS		-	2.2.3b-1			
		gV _{ext1} =	0.767						
	Tue or	More Lanes L							
	I WO OI	Lever Rule	(Table 3.6.1	1.01					
			ax(0.625 * 1.0,		25 0 625 * D	65) -	0.606		
			for Skew:	0.025 0.0	35, 0.025	.00) –	0.023		
		wideling	skew correct	tion =	1,227				
			mg = 0.625						
		Equation	mg = olone		011.01				
			t. b/w CL web t	o curb					
			- Rail Width						
			3ft - 1ft =	2.0	ii.				
			A STATE OF THE PARTY OF THE PAR	770					
		e = 0.6	$6 + \left(\frac{d_e}{10}\right)$						
				0.800					
		g = e*g'	10,000	2144					
			v int2+Eq 00 * 0.999 =	0.799					
			orrection is inc		//interior\				
			plicability (ROA			BOA is	implicitly	applied to t	he exterior beam.
		-	nterior Beam F		OK	non is	mphothy	applied to t	ne extend beam.
		17072	d _e : -1.0' ≤ 2.0'		OK				
			N _b : 6 ≠ 3	- 0.0	OK				
			from Table 4.	6 2 2 3b-1		criteria i	s OK		
		gV _{ext2+} =	0.799	0.2.2.00	Decador an	arrior i	0,11		
	TYDOT		-	Vac					
	IXDOI	Policy states g	0.999	≥ y v interior					
	TYDOT	Policy states g		> m.NN					
	TADOT	$m \cdot N_L \div N_b =$	0.85 * 3 / 6		0.425				
	Is OH <	S/2 ? Yes	0.05 370		0.420				
		20ft ? Yes							
>>		Policy states th	at if OH ≤ S/2,	gV _{Exterior} is	gV _{interior} .				
		Policy states th				is the M	aximum o	of: gV _{ext1} , gV	Interior, and
		$m \cdot N_L \div N_b$.							
	TXDOT	Policy states th	nat if OH > S/2	ans W ≥ 20	oft, gV _{Exterior}	s the M	aximum c	ft gV _{ext1} , gV	exiz+- gV _{interior}
		and m·N _L +N _t							
	gV _{ext}								
	ext	2.003	_						

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Spece
BRIDGE	C-S-J: Descrip:	ITBC Design Exa	ID #: mple 4. Span 1 8	XXXX & 3	Ck Dsn: File:	Ex4 So	Date: an1 distrib	ution factors.x	Rev. 10/18 - Sheet:	5 of 8
	RIOR BE				1, 1101				- Oncorn	
		ribution Per Lan	e (Table 4.6.2	2.2d-1):						
o.		ne Loaded	o (Table 170)							
		Lever Rule								
			625 * 1.0 =	0.625	TxDOT us	es a mu	Itiple pres	sence factor	r of 1.0 for a	ne
		Modify fo	or Skew:		lane loade					
			skew corre	ction =	0.676					
			mg = 0.625	5 * 0.676 =	0.423					
		Use Lever Ru	le as per AAS	SHTO LRFE	Table 4.6.2	.2.2d-1.				
		gM _{ext1} =	0.423							
	Two or	More Lanes Lo	aded							
		Lever Rule	(Table 3.6.	1.1.2)						
			ax(0.625 * 1.0		85. 0.625 * 0	.65) =	0.625			
			or Skew:	,		,				
			skew corre	ction =	0.676					
			mg = 0.625	6 * 0.676 =	0.423					
		Equation			-					
			(d.)							
		e = 0.7	$7 + \left(\frac{d_e}{9.1}\right)$							
		e = 0.77	+ (2.0/9.1) =		0.990					
		g = e*gN	Ames co							
			* 0.537 =	0.532						
			orrection inclu		nterior).					
		Range of App				ROA is	implicitly	applied to t	he exterior b	oeam.
			nterior Beam		OK	1141710	,	Applica is .	,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	
		Check d	e: -1.0' ≤ 2.0'	' ≤ 5.5'	OK					
		Check N	l _b : 6≠3		OK					
		Use Equation	from Table 4	.6.2.2.2d-1	because all	criteria is	s OK			
		gM _{ext2+} =	0.532							
	TYDOT	Policy states gl	/- must h	Mn < a						
	1,001	gM _{interior} =	0.537	- Amilitado						
	TXDOT	Policy states gN	- annual	e ≥ m·N. ≟N	I.					
	1,001		0.85 * 3 / 6		0.425					
	Is OH <	S/2 ? Yes	0.00		0.720					
		20ft ? Yes								
>>	TXDOT	Policy states the	at if OH ≤ S/2	, gM _{Exterior} is	gM _{interior}					
		Policy states the				is the M	aximum (of: gM _{exit} , gl	Minterior, and	
		$m \cdot N_L + N_b$								
	TXDOT	Policy states the	at if OH > S/2	ans W ≥ 2	Oft, gM _{Exterior}	is the M	aximum o	of: gM _{ext1} , gl	M _{ext2+} , gM _{mie}	norr
		and m·N _L +N _b								
	gM _{exte}	erior = 0.537								



TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Spe
BRIDGE		XXX-XX-XXXX	ID #: ample 4, Span 1 &	XXXX	Ck Dsn: File:	End Con	Date:	ion factors.xl	Rev. 10/18 - Sheet:	7 of 8
IVISIOIV	Descrip.	IT DC Dealgh Ex	ample 4, Span 1 &	J.	Trile.	LAT OPA	II UISUIDUI	ion_iaciors.si	Sileet.	7 01 0
LEVER	RULE	\$	S = 8.0 ft							
INTERI	OR (con't)								
For 18 5	S < 22: ne Lane =	$= \frac{16}{32} \left(1 + \frac{S-6}{S} \right)$)				13	0.625		
Tw	Lanes =	$= \frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$)			03	0.750		
Three	e Lanes =	$= \frac{16}{32} \left(1 + \frac{S-6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S-12}{S}$	18			-0.125		
Fou	r Lanes =	$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S-12}{S}$	$\left(\frac{-18}{S} + \frac{S-16}{S}\right)$		10	0.625		
For 22 s	S ≤ 24; ne Lane =	$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$)				174	0.625		
Twe	Lanes =	$= \frac{16}{32} \left(1 + \frac{S-6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$)				0.750		
Three	e Lanes =	$= \frac{16}{32} \left(1 + \frac{S-6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S}{S}$	$\left(\frac{-18}{S}\right)$		10	-0.125		
Fou	r Lanes =	$= \frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S-12}{S}$	$\frac{-18}{S} + \frac{S - 16}{S}$	$+\frac{S-22}{S}$	1	-1.500		
		16k	1 h	15k	t hinge				S = OH =	8.0 ft 3.0 ft
		OH X	s	ł				Rail Widti X = S+OH-		1.0 ft 8.0 ft
For X <	6: ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} \right)$					1.9	0.500		
For 6 ≤ Or	X < 12; ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	-6)					0.625		
For 12 s	X < 18; ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	-6)					0.625		
Twi	Lanes =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	$\frac{6}{4} + \frac{X - 12}{1}$					0.375		

BRIDGE	County:	ANY XXX-XX-XXXX	Highway:	Any XXXX	Design: Ck Dsn:	_	Date: Date:	8/15/20	2017 LRFD Sper Rev. 10/18 - (No Interir
IVISION			kample 4, Span 1		File:	Ex4 Span1		n factors.xl	Sheet: 8 of 8
	2								
LEVER	RULE								
EXTER	IOR (con'	t) S	s = 8.0 f	t	OH =	3.0 ft			
-		RW	= 1.0 f	t X = S+0	OH-RW-2ft =	8.0 ft			
For 18	X < 24:								
O	ne Lane =	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X}{S}\right)$	-6				=	0.625	
		224.0	0. 188 - 02 - 1	V = 18 \					
Tw	o Lanes =	$\frac{10}{32} \left(\frac{x}{s} + \frac{x}{s} \right)$	$\frac{-6}{S} + \frac{X - 12}{S} + \frac{3}{S}$	S			=	-0.250	
For 24 s	≤ X < 30:	16/V V	61						
O	ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	(2)				=	0.625	
	Acies	16 (X X	-6 X-12 X	(81-7				0.000	
IW	o Lanes =	32 5 5	$\frac{-6}{S} + \frac{X - 12}{S} + \frac{3}{S}$	S			=	-0.250	
Three	e Lanes =	$\frac{16}{37}\left(\frac{X}{x} + \frac{X}{x}\right)$	$\frac{-6}{3} + \frac{X - 12}{5} + \frac{3}{3}$	$\frac{x-18}{9} + \frac{x-2}{9}$	4)		-	-1.250	
		32(3 3		2 2	5.				
For 30 S	≤ X < 36: ne Lane =	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X}{S}\right)$	-6)					0.625	
		32 (3	1	3 068				0,020	
Tw	o Lanes =	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X}{S}\right)$	$\frac{-6}{s} + \frac{x-12}{s} + \frac{x}{s}$	$\left(\frac{s-18}{s}\right)$			=	-0.250	
		Allerana allerana			$4 \cdot X = 30$				
Thre	e Lanes =	32 8 5	$\frac{-6}{s} + \frac{X - 12}{s} + \frac{3}{s}$	s s	S		=	-2.625	
For 36 5	≤ X < 42:	16/X X	-6)					St. of Char	
O	ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$					=	0.625	
Tw	o Lanes =	16 X + X	$\frac{-6}{5} + \frac{x-12}{5} + \frac{x}{2}$	$\kappa = 18$			1.2	-0.250	
		2200		A . X	a Araty			Cinado	
Thre	e Lanes =	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X}{S}\right)$	$\frac{-6}{S} + \frac{x-12}{S} + \frac{x}{S}$	$\frac{x-18}{s} + \frac{x-2}{s}$	$\frac{4}{4} + \frac{X - 30}{S}$		11.5	-2.625	
			4 2 2 2			X -36			
Fou	r Lanes =	32 5 + 5	$\frac{-6}{S} + \frac{X - 12}{S} + \frac{2}{S}$	S + S	- + S +	5	=	-4.375	
For 42	≤ X ≤ 48:	16/ V V	6)						
O	ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	5				=	0.625	
Two	o Lance	16 (X , X -	$\frac{-6}{5} + \frac{X-12}{5} + \frac{3}{5}$	(-18)				-0.250	
1.00	o Lanes =	32 8 5	S	S)				-0.230	
Three	e Lanes =	$=\frac{16}{32}\left(\frac{X}{6} + \frac{X}{6}\right)$	$\frac{-6}{2} + \frac{X - 12}{2} + \frac{A}{2}$	$\frac{x-18}{5} + \frac{x-2}{5}$	$\frac{4}{4} + \frac{X - 30}{8}$		=	-2.625	
		200		0 0		V 36 V	-425		
Fou	r Lanes =	$=\frac{10}{32}\left(\frac{x}{s} + \frac{x}{s}\right)$	$\frac{-6}{S} + \frac{X - 12}{S} + \frac{3}{2}$	$\frac{1}{S} + \frac{\lambda - 2}{S}$	+ x - 30 +	S + X	$\frac{-42}{S}$ =	-6.500	
Autoria	25.				KER-SP	. 01			
One La	OR ne Loade	d	- 0.625		One La	<u>IOH</u> ne Loaded			0.625
	nes Loade		= 0.625 = 0.875			nes Loaded		=	0.625
	anes Loa		= 0.875			anes Loade		-	0.625
	nes Load		= 0.875			nes Loade		_	0.625
. 551 60		77	5.075		, our ca	LOUGE			-1040

4.5.15.4.2 Span 2

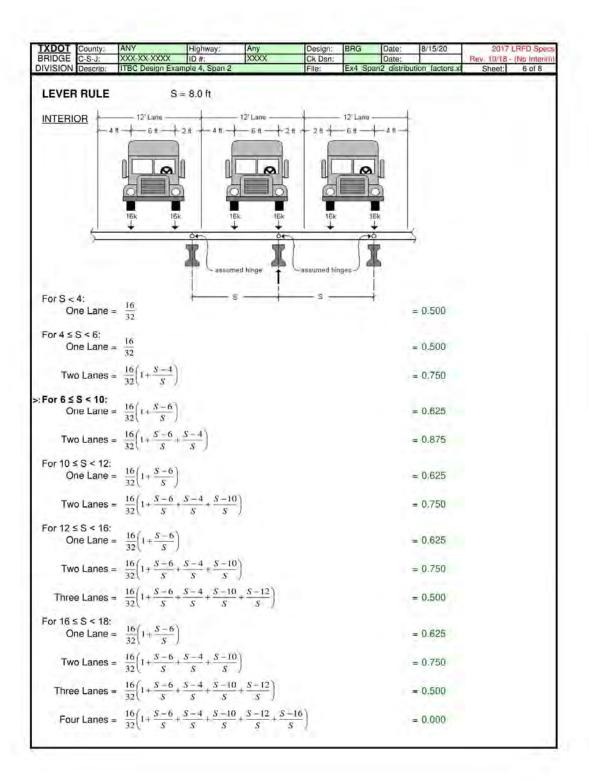
BRIDGE	C-S-J:	ANY XXX-XX	-XXXX	Highway: ID #:	Any	Design: Ck Dsn:	BRG	Date:	8/15/20	2017 LRF Rev. 10/18 - (No	
	Descrip:			ple 4, Span 2	10000	File:	Ex4 Span		tion_factors.xl		of 8
				LRFD L	ive Load Di	stribution	Factor	<u>s</u> *			
(2017) The Le	with no interver Rule is	erim rev s used v	isions) a	s prescribed	by TxDOT po	olicies (LR	D Design	Manua	July 2018)	ations, 8th Editi and practices Skew Correct	
INPUT	s is ignore r.	a,									
INFO	Beam T	vne =	Tx54		1	Deck S	ah		Beam		-
1	No. Beams	2.1	6			Conc wt =		k/ft ³	weight =	0.145 k/ft ³	
	L _{brg} to CL _b		and the later	ft		f'c =		ksi	f'c =	8.5 ksi	
	am Spacin	7	8.00	ft		E _{slab} =			E _{beam} =	5312 ksi	
	Skew Ang	~	60.00	dea	11/4	-siau	30-14	,,,,,	V ₁ =	30.49 in	
	Thicknes		8.00	in					A =	817.0 in ²	
	Overhang,		3	ft					1=	299740 in4	
	ail Width,		1	ft							
	way Width		44	ft							
Vumbe	er of Lanes	s, N _L =	3								
Longitu	udinal Stiff	ness Pa	rameter	(4.6.2.2.1-1	1)						
		(in) =			og of bm & de	eck)					
		n=	1.000								
	$K_0 = n(1+A)$	$(e_0^2) =$	1271611	in ⁴							
	a circle										
*For tvo	oical cross s	sections	a.e.i.i & k). Table 4.6.2	2.1-1						
- 97				/2 / doing State							
RESU	LTS:			Final LLD		TxDOT police	ies.			to the following	
- 1	Interior She	ear LLDF	, gV _{interio}	1.045		When O	1 > S/2 the	exterior		en OH ≤ 3/2, is determined by esence factor of	
Inte	erior Mome	ent LLDF	, gM _{interio}	0.529		the LLDFs I	or the inter	ior beam	5.	beams be less th	lan
								middle in I	ace than 2011	Company of the Compan	
	xterior She		, gV _{exterio}	1.045						, all beams are	
E	xterior She	ear LLDF				designed for	one lane	loaded or			
E		ear LLDF				designed for	one lane	loaded or	nly_		
E	xterior She	ear LLDF				designed for	one lane	loaded or	nly_		
Exte	xterior She	ear LLDF				designed for	one lane	loaded or	nly_		
Exte	xterior She	ear LLDF				designed for	one lane	loaded or	nly_		
Exte	xterior She	ear LLDF	, gM _{exterio}	0.529		designed for *In no case	one lane shall the	loaded or	nly. less than m		O
Exte	erior Mome	nt LLDF	, gM _{exterio}	0.529 w (Table 4.6	5.2.2.3c-1)	designed for #In no case	one lane shall the heck θ:	loaded or LLDF be	nly. less than m· 0° ≤ 60°		
Exte	erior Mome	nt LLDF	, gM _{exterio}	0.529 w (Table 4.6	5.2.2.3c-1)	designed for *In no case C	heck 6:	0° ≤ 60 3.5' ≤ 8	nly. less than m· 0° ≤ 60° 3.0' ≤ 16.0'	$N_L = N_{L^p}$	OF
Exte	erior Mome	nt LLDF	, gM _{exterio}	0.529	5.2.2.3c-1)	designed for left no case C C C	heck 0: heck S: heck L:	0° ≤ 60 3.5' ≤ 120' ≤ 1	nly. less than m· 0° ≤ 60°	$N_L = N_{L^p}$	OF
Exte	erior Mome	nt LLDF NS: rrection Corr. = 1	for Ske	w (Table 4.6) $0 \left(\frac{12.0Lt_s}{K_g} \right)$	$3.2.2.3c-1$) $\frac{3}{3}$ $\tan \theta$	designed for all no case	heck θ: heck S: heck L: heck N _b :	0° ≤ 60 3.5' ≤ 1 20' ≤ 1 6 ≥ 4	nly. less than m· 0° ≤ 60° 3.0' ≤ 16.0'	$N_L = N_{L^p}$	OF
Exte	EULATION	NS: rrection corr. = 1	for Ske .0 + 0.20	w (Table 4.6) $0 \left(\frac{12.0Lt_s}{K_g} \right)$	5.2.2.3c-1)	designed for all no case	heck θ: heck S: heck L: heck N _b :	0° ≤ 60 3.5' ≤ 1 20' ≤ 1 6 ≥ 4	nly. less than m· 0° ≤ 60° 3.0' ≤ 16.0'	$N_L = N_{L^p}$	OF
Exte	CULATION	NS: rrection Corr. = 1 Corr. = 1	for Ske .0+0.2	w (Table 4.6) $0 \left(\frac{12.0Lt_s}{K_s} \right) \cdot [(12.0^{+}10^{-})]$	3.2.2.3c-1) $ \begin{array}{c} 3.2.2.3c-1) \\ 5 \\ - \end{array} $ $ \tan \theta $ 6.5*8^3)/(1,27)	designed for all no case	heck θ: heck S: heck L: heck N _b : 3 * tan(60	0° ≤ 60 3.5' ≤ 1 6 ≥ 4	nly. less than m· 0° ≤ 60° 3.0' ≤ 16.0' 06.5' ≤ 240'	N _L +N _L ,	OF
Exte	CULATION LLDF Cor	NS: rrection = 1 Corr. = 1 Correction	for Ske .0+0.2 .0+0.26	w (Table 4.6 $0 \left(\frac{12.0 L t_s}{K_g} \right)$ (12.0*10 kew (Table 4.6)	$3.2.2.3c-1$) $\frac{3}{3}$ $\tan \theta$	designed for all no case	heck θ: heck S: heck L: heck N _b :	0° ≤ 60 3.5' ≤ 8 20' ≤ 1 6 ≥ 4	nly. less than m· 0° ≤ 60° 3.0' ≤ 16.0' 06.5' ≤ 240'	N _L +N _L ,	OH
Exte	CULATION LLDF Cor	NS: rrection = 1 Corr. = 1 Corr. = 1 Corr. = 1	for Ske .0 + 0.2 .0 + 0.26 .284 on for Si	w (Table 4.6) $0 \left(\frac{12.0Lt_s}{K_g} \right) * [(12.0^{+}10) \text{kew} (Table + \theta)^{-}1.5]$	$\begin{array}{c} 3.2.2.3c-1) \\ 5.2.2.3c-1) \\ -1 & \tan \theta \\ 6.5^*8^3)/(1,27) \\ 4.6.2.2.2e-1) \end{array}$	C C C C C 71,611)]^0.3	heck 0: heck S: heck L: heck N _b : 3 * tan(60	0° ≤ 60 3.5' ≤ 8 20' ≤ 1 6 ≥ 4	nly. less than m· 0° ≤ 60° 3.0' ≤ 16.0' 06.5' ≤ 240'	N _L +N _L .	OH
Exte	CULATION LLDF Cor	NS: rrection = 1 Corr. = 1 Corr. = 1 Corr. = 1	for Ske .0 + 0.2 .0 + 0.26 .284 on for Si	w (Table 4.6 $0 \left(\frac{12.0 L t_s}{K_g} \right)$ (12.0*10 kew (Table 4.6)	$\begin{array}{c} 3.2.2.3c-1) \\ 5.2.2.3c-1) \\ -1 & \tan \theta \\ 6.5^*8^3)/(1,27) \\ 4.6.2.2.2e-1) \end{array}$	designed for all no case	heck 0: heck S: heck L: heck N _b : 3 * tan(60	0° ≤ 60 3.5' ≤ 8 20' ≤ 1 6 ≥ 4	nly. less than m· 0° ≤ 60° 3.0' ≤ 16.0' 06.5' ≤ 240'	N _L +N _L .	OH
Exte Exte CALC Shear	CULATION LLDF Cor	NS: rrection = 1 Corr. = 1 Corr. = 1 Corr. = 1	for Ske .0+0.2 .0+0.26 .284 on for Si - c ₁ (tan - 0.081)	w (Table 4.6) $0 \left(\frac{12.0Lt_s}{K_g} \right) * [(12.0^{+}10) \text{kew} (Table + \theta)^{-}1.5]$	$\begin{array}{c} 3.2.2.3c-1) \\ 5.2.2.3c-1) \\ -1 & \tan \theta \\ 6.5^*8^3)/(1,27) \\ 4.6.2.2.2e-1) \end{array}$	C C C C C 71,611)]^0.3	heck θ: heck S: heck L: heck N _b : 3 * tan(60	0° ≤ 60 3.5' ≤ 8 20' ≤ 1 6 ≥ 4	nly. less than m· 0° ≤ 60° 3.0' ≤ 16.0' 06.5' ≤ 240'	N _L +N _L .	OK OK OK

INTERI	C-S-J: Descrip: OR BE	ITBC Design Exam	ID #:	XXXX	Ck Dsn:		Date:	J 0- 10-4	Rev. 10/18 -	(No Interi
INTERI					File:	Ex4 Spa	n2 distribu	tion factors.xl	Sheet:	2 of 8
		AM:								
	L Distrib	ution Per Lane (Table 4.6.2.2	(3a-1):						
		ne Loaded	Tuoio monera	17.						
		Lever Rule	(Table 3.6.)	1.1.2)						
			25 * 1.2 =	0.750						
		Modify fo		277.50						
			skew corre	ction =	1.284					
			mg = 0.750		0.963					
		Equation		1756-						
		g = 0.36	$+\left(\frac{S}{s}\right)$							
		7	+ (8 / 25) =	0.680						
		Modify fo	r Skew:							
			skew corre		1.284					
			g = 0.680 *	1.284 =	0.873					
		Range of Appl	icability (RO)	A) Checks						
			3.5' ≤ 8.0'		OK					
		Check t _s :	4.5" ≤ 8.0"	'≤ 12.0"	OK					
		Check L:	20' ≤ 106.5	5' ≤ 240'	OK					
		Check N	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 Los	aded							
		Lever Rule	(Table 3.6.	1.1.2)						
		mg = Max	x(0.875 * 1.0	The second second	35, 0.875 * 0	.65) =	0.875			
		Modify fo								
		100000	skew corre	ction =	1,284					
			mg = 0.875	* 1.284 =	1.124					
		Equation		10						
		g = 0.2	$\left(\frac{S}{S}\right) - \left(\frac{S}{S}\right)$	5)						
			(12) (3	5)						
		7.1	(8 / 12) - (8 /	35)^2.0 =	0.814					
		Modify fo								
			skew corre		1.284					
			g = 0.814 *	1.284 =	1.045					
		Range of Appl	icability (ROA	A) Checks	(same as f	for one la	ne loade	d)		
		Use Equation	from Table 4	.6.2.2.3a-1	because all	criteria is	OK.			
		$gV_{int2+} =$	1.045							
	TXDOT	Policy states gV	laterior must be	≥ m·NL÷Ne	y .					
		$m \cdot N_L \div N_b =$	0.85 * 3 / 6		0.425					
	Is W ≥ 2	20ft ? Yes								
		Policy states tha	t if W < 20tt.	gV _{interior} is t	he Maximum	of: gV _{mi}	and mil	VL+Nb		
>>		Policy states tha								
	gV _{inte}		7			- 111	2,000			

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	A CONTRACTOR OF THE PARTY OF TH	LRFD Specs
BRIDGE	C-S-J: Descrip:	ITBC Design Exam	ID #: ple 4. Span 2	XXXX	Ck Dsn: File:	Ex4 Span	Date:	tion factors.xl	Rev. 10/18 - Sheet:	(No Interim)
	IOR BE		pic 1, opair 2		Tr.iid.	LAT OPEN	L_000100	non industrial	- Onder.	0010
		ribution Per Lane	(Table 4.6.2	2.2b-1):						
Montal		ne Loaded	Tradio Hom	17.						
		Lever Rule	(Table 3.6.)	1.1.2)						
		mg = 0.62	a contract	0.750						
		Modify for		330,000						
			skew corre	ction =	0.815					
			mg = 0.750	* 0.815 =	0.611					
		Equation	2 04 .		₹0.1					
		g = 0.06	$+\left(\frac{S}{14}\right)^{3/4}$	$\left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12Lt}\right)^{0.3}$	3					
		g = 0.06 +	- (8/14)^0.4	(8/106.5)^0.	3 * (1,271,6	611/(12*1	06.5*8*3	3))^0.1 =	0.453	
		Modify for	Skew:							
			skew corre	ction =	0.815					
			g = 0.453*	0.815 =	0.369					
		Range of Appli	cability (RO)	A) Checks						
		Check S:	3.5' ≤ 8.0'	≤ 16.0'		OK				
		Check ts:	4.5" ≤ 8.0"	≤ 12.0"		OK				
		Check L:	20' ≤ 106.5	5' ≤ 240'		OK				
		Check N _b	6≥4			OK				
		Check K _g	10,000 ≤ 1	,271,611 ≤ 7,	,000,000	OK				
		Use Equation f gM _{int1} =	rom Table 4 0.369	6.2.2.2b-1 be	ecause all o	criteria is	OK.			
	Two or	More Lanes Loa	ded							
	222.20	Lever Rule	(Table 3.6.	1.1.2)						
			The second second	0.875 * 0.85	0.875 * 0.	.65) =	0.875			
		Modify for		1000			2,000			
		10000	skew corre	ction =	0.815					
			mg = 0.875	* 0.815 =	0.713					
		Equation	c = > 0.0	V=>02/	70.1					
		g = 0.07	$5+\left(\frac{S}{9.5}\right)^{-1}$	$\left(\frac{S}{L}\right)^{0.2} \left(\frac{K}{12L}\right)^{0.2}$	$\left(\frac{L_{g}}{Lt_{s}^{3}}\right)$					
		g = 0.075 Modify for	+ (8/9.5)^0.0	6 * (8/106.5)^	0.2 * (1,27	1,611/(12	*106.5*8	3^3))^0.1 =	0.649	
			skew corre	ction =	0.815					
			g = 0.649 *	0.815 =	0.529					
		Range of Appli	cability (RO	A) Checks	(same as f	or one lar	ne loade	d)		
		Use Equation f	rom Table 4	6.2.2.2b-1 be	ecause all o	oriteria is	OK.			
		gM _{int2+} =	0.529							
	TXDOT	Policy states gM	must be	$\geq m/N_{\perp} \pm N_{\perp}$						
	14001	$m \cdot N_L \div N_b =$	0.85 * 3 / 6		0.425					
	Is W > 3	20ft ? Yes	0.00		MATERIA					
		Policy states that	If W < 2011.	gMiller is th	e Maximun	n of: aM	and m-	Ni ÷Ni		
>>		Policy states that								
	gM _{inte}		1	S		31110	- 2	101		
	- Similar	7.025	4							

EXTERIOR BEAM: Sheet 4 of 8	TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20	A CONTRACTOR OF THE PARTY OF TH	RFD Specs
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 Modify for Skew:	and the second second second		ITBC Design Exa		XXXX	Ck Dsn:	Ex4 Spa	Date:	ution factors vi		
Shear LL Distribution Per Lane (Table 4.6.2.2.3b-1): One Lane Loaded				mple 4, Opair 2		Triid.	LAT OPA	L USUID	DITOTI_IGUIDIGI.SI	Silber	4010
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. lane loaded lane load	many to district			(Table 4 6 2 2	3h-11.						
Lever Rule	Oneas L			Table 4.0.2.2	.00-17.						
mg = 0.625 * 1.0 = 0.625		One La		(Table 3.6.)	1 1 2)						
Modify for Skew: Iane loaded on the exterior beam. skew correction = 1.284						TypoTus	es a mult	inla aro	eance factor	of 1 D for or	30
Skew correction = 1.284 mg = 0.625 * 1.284 = 0.803			34.5.000		0.023					di No idi di	ie.
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 - Rall Width d _n = 3t · 1t = 2.0 tt e = 0.6 + (Moony I		ction -						
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 = 31 · 11t = 2.0 tt e = 0.6 + (2.010) = 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 Check Interior Beam ROA: OK Check d _e : -1.0* ≤ 2.0* ≤ 5.5* OK Check N _e : 6 ≠ 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 _{Extensor} must be ≥ gV _{extensor} gV _{interior} = 1.045 TXDOT Policy states gV _{Extensor} must be ≥ m·N ₁ ÷N _b m·N ₁ ÷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 and W < 20ft, gV _{Extensor} is the Maximum of: gV _{ext1} , gV _{entsor} , and m·N ₁ ÷N _b . TXDOT Policy states that if OH > S/2 and W < 20ft, gV _{Extensor} is the Maximum of: gV _{ext1} , gV _{entsor} , and m·N ₁ ÷N _b . TXDOT Policy states that if OH > S/2 and W < 20ft, gV _{Extensor} is the Maximum of: gV _{ext1} , gV _{entsor} , and m·N ₁ ÷N _b . TXDOT Policy states that if OH > S/2 and W < 20ft, gV _{Extensor} is the Maximum of: gV _{ext1} , gV _{entsor} , and m·N ₁ ÷N _b . TXDOT Policy states that if OH > S/2 and W < 20ft, gV _{Extensor} is the Maximum of: gV _{ext1} , gV _{entsor} , and m·N ₁ ÷N _b . TXDOT Policy states that if OH > S/2 and W < 20ft, gV _{Extensor} is the Maximum of: gV _{ext1} , gV _{entsor} , and m·N ₁ ÷N _b . TXDOT Policy states that if OH > S/2 and W < 20ft, gV _{Extensor} is the Maximum of: gV _{ext1} , gV _{entsor} , and m·N ₁ ÷N _b .				411510 42115							
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Range of Applicability (ROA) Checks Check Interior Beam ROA: Check de: -1.0' ≤ 2.0' ≤ 5.5' Check Nb: 6 ≠ 3 Use Equation from Table 4.6.2.2.3b-1 because all criteria is OK. gVext2+ = 0.836 TXDOT Policy states gVexterior must be ≥ gVinterior gVinterior = 1.045 TXDOT Policy states gVexterior must be ≥ m·NL÷Nb m·NL÷Nb = 0.85 * 3 / 6 = 0.425 Is OH ≤ S/2 ? Yes Is W ≥ 20ft ? Yes >> TXDOT Policy states that if OH > S/2 and W < 20ft, gVexterior is the Maximum of: gVext1, gVext2+, gVinterior. TXDOT Policy states that if OH > S/2 ans W ≥ 20ft, gVexterior is the Maximum of: gVext1, gVext2+, gVinterior.						Very Control					
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. $gV_{ext2*} = 0.836$ $TxDOT Policy states gV_{Exterior} must be \ge gV_{Interior} gV_{Interior} = 1.045 TxDOT Policy states gV_{Exterior} must be \ge m \cdot N_L \div N_b m \cdot N_L \div N_b = 0.85 * 3 / 6 = 0.425 Is OM \le S/2? Yes Is W \ge 20ft? Yes STXDOT Policy states that if OM \le S/2, gV_{Exterior} is gV_{Interior}. TxDOT Policy states that if OM \le S/2 and OM \le S/2 and OM \le S/2 is the Maximum of: gV_{ext1}, gV_{Interior}, and gV_{Interior}. TxDOT Policy states that if OM > S/2 and OM \le S/2, gV_{Exterior} is the Maximum of: gV_{ext1}, gV_{Interior}, and gV_{Interior}.$							-6.1				
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. $gV_{ext2*} = 0.836$ $TxDOT Policy states gV_{Exterior} must be \ge gV_{Interior} gV_{Interior} = 1.045 TxDOT Policy states gV_{Exterior} must be \ge m \cdot N_L \div N_D m \cdot N_L \div N_D = 0.85 * 3 / 6 = 0.425 Is OH \le S/2? Yes Is W \ge 20ft? Yes TxDOT Policy states that if OH \le S/2, gV_{Exterior} is gV_{Interior}. TxDOT Policy states that if OH \ge S/2 and OH \le S/2 and OH \le S/2 is the Maximum of: gV_{ext1*}, gV_{Interior*}, and gV_{Interior*}. TxDOT Policy states that if OH \ge S/2 and OH \le S/2$			A STATE OF THE STA				ROA is it	nplicitly	applied to the	ne exterior b	eam.
Check N _b : 6 ≠ 3 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 ≥ gV _{Interior} gV _{Interior} = 1.045 TxDOT Policy states gV _{Exterior} must be ≥ m·N _L ÷N _b m·N _L ÷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·N _L ÷N _b . TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, gV _{Exterior} is the Maximum of: gV _{ext1+} gV _{Interior} , and			170777117	Walter and Taleston of							
Use Equation from Table 4.6.2.2.3b-1 because all criteria is OK. $gV_{\text{ext2+}} = 0.836$ $TxDOT Policy states gV_{\text{Exterior}} \text{ must be } \ge gV_{\text{interior}}$ $gV_{\text{interior}} = 1.045$ $TxDOT Policy states gV_{\text{Exterior}} \text{ must be } \ge m \cdot N_L \div N_b$ $m \cdot N_L \div N_b = 0.85 \div 3 / 6 = 0.425$ Is OH $\le S/2$? Yes Is W ≥ 20 ft? Yes $STXDOT Policy states that if OH \le S/2, gV_{\text{Exterior}} is gV_{\text{interior}}. TxDOT Policy states that if OH \le S/2 and W \le 20ft, gV_{\text{Exterior}} is the Maximum of: gV_{\text{ext1+}} gV_{\text{interior}} and m \cdot N_L \div N_b. TxDOT Policy states that if OH \ge S/2 ans W \ge 20ft, gV_{\text{Exterior}} is the Maximum of: gV_{\text{ext1+}}, gV_{\text{interior}}.$					≤ 5.5'						
$gV_{\text{ext2+}} = \underbrace{0.836}$ $TxDOT\ Policy\ states\ gV_{\text{Exterior}\ must\ be} \geq gV_{\text{interior}\ }$ $gV_{\text{interior}} = \underbrace{1.045}$ $TxDOT\ Policy\ states\ gV_{\text{Exterior}\ must\ be} \geq m \cdot N_L \div N_b$ $m \cdot N_L \div N_b = \underbrace{0.85 \ ^*\ 3 \ / 6} = \underbrace{0.425}$ $Is\ OH \leq S/2\ ? \ Ves$ $Is\ W \geq 20ft\ ? \ Ves$ $Is\ W \geq 20ft\ ? \ Ves$ $TxDOT\ Policy\ states\ that\ if\ OH \leq S/2\ , \ gV_{Exterior\ } is\ gV_{interior\ }$ $TxDOT\ Policy\ states\ that\ if\ OH > S/2\ and\ W < 20ft\ , \ gV_{Exterior\ } is\ the\ Maximum\ of:\ gV_{ext1+\ } gV_{interior\ } and\ m \cdot N_L \div N_b.$ $TxDOT\ Policy\ states\ that\ if\ OH > S/2\ ans\ W \geq 20ft\ , \ gV_{Exterior\ } is\ the\ Maximum\ of:\ gV_{ext1+\ } gV_{interior\ } is\ ft$					2.3.3.57			503			
$TxDOT \ Policy \ states \ gV_{\text{Exterior}} \ must \ be \geq gV_{\text{Interior}} \ gV_{\text{Interior}} = \frac{1.045}{1.045}$ $TxDOT \ Policy \ states \ gV_{\text{Exterior}} \ must \ be \geq m \cdot N_L \div N_b \ m \cdot N_L \div N_b = 0.85 \cdot 3 / 6 = 0.425$ $Is \ OH \leq S/2 \ Yes \ Is \ W \geq 20ft \ ? \ Yes \ Is \ W \geq 20ft \ ? \ Yes \ SV_{\text{Exterior}} \ Is \ gV_{\text{Interior}} \ SV_{\text{exterior}} \ is \ the \ Maximum \ of: \ gV_{\text{ext1}} \ gV_{\text{Interior}}, \ and \ m \cdot N_L \div N_b.$ $TxDOT \ Policy \ states \ that \ if \ OH > S/2 \ ans \ W \geq 20ft, \ gV_{\text{Exterior}} \ is \ the \ Maximum \ of: \ gV_{\text{ext1}}, \ gV_{\text{Interior}}, \ gV_{Int$			F-17		.6.2.2.3b-1	because all	criteria is	OK.			
gV _{interior} = 1.045 TxDOT Policy states gV _{Exterior} must be ≥ m·N _L ÷N _b m·N _L ÷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·N _L ÷N _b . TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, gV _{Exterior} is the Maximum of: gV _{ext1} , gV _{interior} , gV _{interior} .			No. of the last of	STATE OF THE PARTY							
TxDOT Policy states gV _{Exterior} must be ≥ m·N _L ÷N _D m·N _L ÷N _D = 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·N _L ÷N _D . TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, gV _{Exterior} is the Maximum of: gV _{ext1} , gV _{Interior} , gV _{Interior} .		TXDOT	Policy states g\	/Exterior must be	e ≥ gV _{interior}						
m·N _L ÷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·N _L ÷N _b . TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, gV _{Exterior} is the Maximum of: gV _{ext1} , gV _{interior} , gV _{interior} .			gV _{interior} =	1.045							
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·N _L ÷N _D . TxDOT Policy states that if OH > S/2 ans W ≥ 20ft, gV _{Exterior} is the Maximum of: gV _{ext1} , gV _{ext2} , gV _{interior} ,		TXDOT	Policy states g\	Exterior must be	$0.01 \pm 10^{-1} \text{ M} \le 0.01 \pm 10^{-1}$	b					
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·N _L ÷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} .				0.85 * 3 / 6	=	0.425					
>> 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·N _L ÷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} .											
TxDOT Policy states that if OH > S/2 and W < 20ft, gV _{Exterior} is the Maximum of; gV _{ext1} , gV _{interior} , and m·N _L ÷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} ,				LEUTH DA		45.7					
$m \cdot N_L \div N_b$. TxDOT Policy states that if OH > S/2 ans W \ge 20ft, gV_{Exterior} is the Maximum of: $gV_{\text{ext}1}$, $gV_{\text{gxl}2+}$, gV_{interior} .	>>						70.4				
TxDOT Policy states that if OH > S/2 ans W \geq 20ft, gV_{Exterior} is the Maximum of: gV_{ext} , gV_{ext} , gV_{interior} ,		TXDOT		at if OH > S/2	and W < 2	Off, gV _{Exterior}	s the Ma	ximum c	or, gV _{ext1} , gV	interior, and	
			The state of the s	111 611 611	JC 3 (2) - 5 - 5	si iu		GOLDON OF	r dress the		
and m·N ₁ ÷N _n .		TXDOT			ans W ≥ 2	off, gV _{Exterior} i	s the Ma	kimum c	it: gV _{ext1} , gV	exiz+- gV _{interior}	4
	1 4		and m·N _L ÷N _b								
gV _{exterior} = 1.045		gV _{exte}	erior = 1.045								

TXDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Spece
BRIDGE	C-S-J: Descrip:	ITBC Design Exa	ID#:	XXXX	Ck Dsn:	Ev4 Sn	Date:	ution factors.x	Rev. 10/18 - Sheet:	- (No Interim
	RIOR BE		ilipie 4, opan 2		Trile;	EX4 OP	anz_usuioi	JUOIT_IBCIOIS.X	Sheet	5010
		ibution Per Lane	/Toble 4.6.5	2 24 11						
Momen		ne Loaded	1 (Table 4.0.2							
	One Lai	Lever Rule								
			25 * 1.0 =	0.625	TypoTue	20 2 001	ltiple pros	ennen Facto	r of 1,0 for o	nd n
		Modify fo		0.023	lane loade				W 1,0 101 0	(IE
		(Modify 10	skew corre	ction =	0.815					
			211211-2211-	* 0.815 =						
		Use Lever Ru				2.2d-1				
		gM _{ext1} =	0.509							
	Tue or	More Lanes Lo	-							
	I WO OF	Lever Rule	(Table 3.6.	11.0						
			x(0.625 * 1.0	the state of the same	85 0 625 * 0	65) -	0.625			
		Modify fo		, 0.025 0.0	35, 0.025 0.	05) -	0.025			
		wiodily it	skew corre	ction =	0.815					
			mg = 0.625		0.509					
		Equation								
			(d.)							
		e = 0.77	$7 + \left(\frac{d_e}{9.1}\right)$							
		e = 0.77	+ (2.0/9.1) =		0.990					
		g = e*gN	lour es							
			* 0.529 =	0.524						
		12.	rrection inclu		nterior).					
		Range of App				ROA is	implicitly	applied to t	he exterior b	neam.
		and the second second	terior Beam		OK	1101110	p.	applied to t	(c chiche)	
			: -1.0' ≤ 2.0'		OK					
		Check N	The second purchase		ОК					
		Use Equation	from Table 4	.6.2.2.2d-1	because all o	riteria is	OK.			
		gM _{ext2+} =	0.524							
	TYDOT	Policy states gM	f must h	e > aM.						
	1,001	gM _{interior} =	0.529	- givintenor						
	TXDOT	Policy states gN	- Contraction	e≥m·N,÷N	L					
	111001		0.85 * 3 / 6		0.425					
	Is OH ≤	S/2 ? Yes	0.00		01.100					
		Oft? Yes								
>>	TXDOT	Policy states that	at if OH ≤ S/2	, gM _{Exterior} is	gMinterior:					
	TXDOT	Policy states that	at if OH > S/2	and W < 20	Oft, gM _{Exterior} i	s the M	aximum o	of: gM _{ext1} , gl	Minterior, and	
		$m \cdot N_L + N_b$								
	TXDOT	Policy states that	at if OH > S/2	ans W ≥ 20	Off, gM _{Exterior} i	s the M	aximum d	of: gM _{ext1} , gl	M _{ext2+} , gM _{m/e}	norr
		and m·N _L +N _b .								
	gM _{exte}	erior = 0.529								
			_							



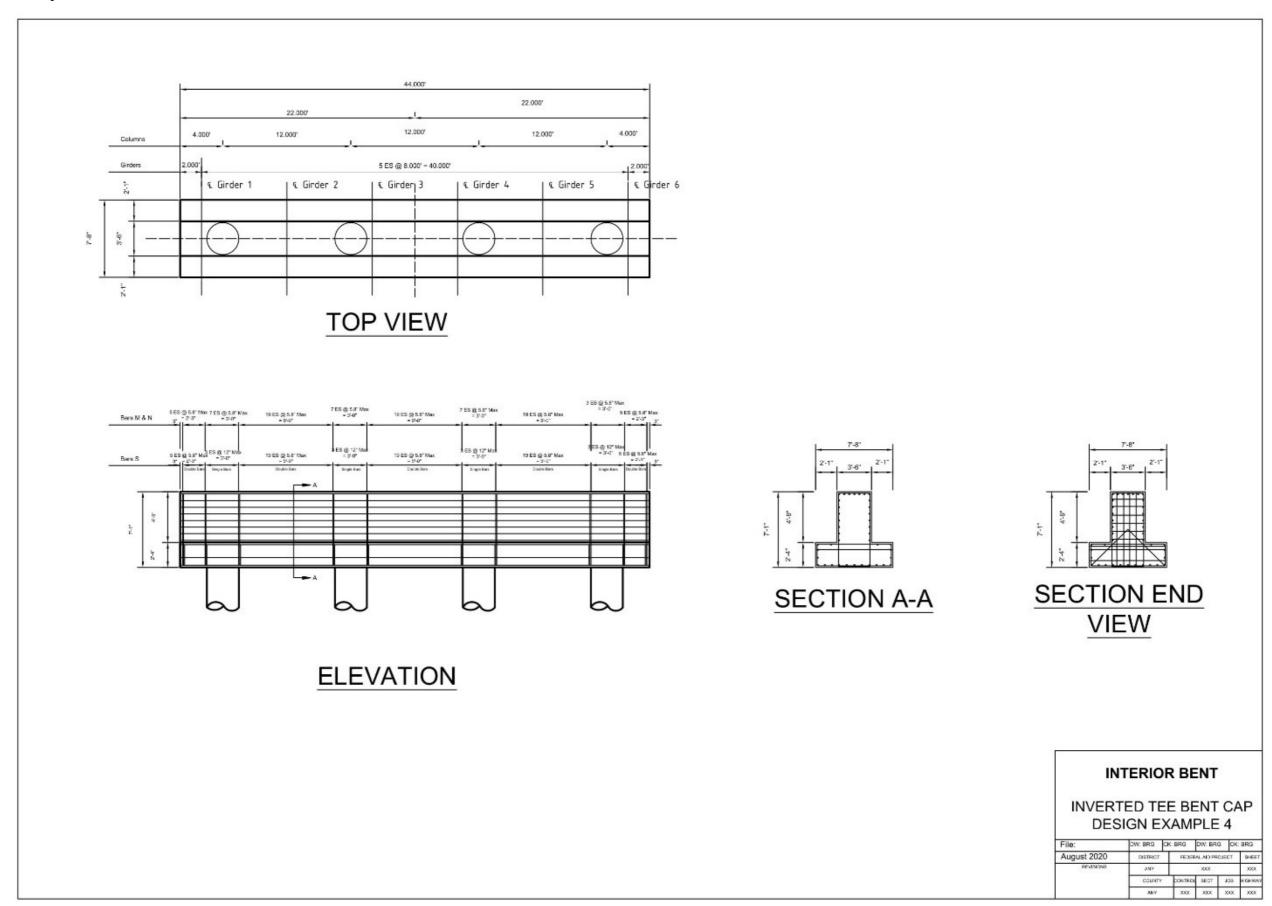
XDOT	County:	ANY	Highway:	Any	Design:	BRG	Date:	8/15/20		LRFD Spe
RIDGE		ITBC Design Ex	ID#:	XXXX	Ck Dsn: File:	Eud Con	Date:	ion factors.xl	Rev. 10/18 - Sheet:	7 of 8
VISION	Descrip.	I TOO Dealgh Ext	imple 4, Opan 2		Trile,	LAT OPA	ILE_UISUIDUI	UII_IGUIUIS:AI	Sheet	7 01 0
	RULE		= 8.0 ft							
	OR (con't)									
For 18 s	S < 22: ne Lane =	$\frac{16}{32}\bigg(1+\frac{S-6}{S}\bigg)$					r a	0.625		
Tw	b Lanes =	$\frac{16}{32}\bigg(1+\frac{S-6}{S}$	$+\frac{S-4}{S}+\frac{S-10}{S}$)			0,5	0.750		
		5-7 6	$+\frac{S-4}{S}+\frac{S-10}{S}$				-	-0.125		
		$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S-12}{S}$	$\left(\frac{-18}{S} + \frac{S-16}{S}\right)$		1	0.625		
For 22 s	S S ≤ 24; ne Lane =	$\frac{16}{32} \left(1 + \frac{S - 6}{S} \right)$					119	0.625		
Tw	o Lanes =	$\frac{16}{32}\bigg(1+\frac{S-6}{S}$	$+\frac{S-4}{S}+\frac{S-10}{S}$)				0.750		
Three	e Lanes =	$-\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S}{S}$	$\left(\frac{-18}{S}\right)$		110	-0.125		
Fou	r Lanes =	$\frac{16}{32}\left(1+\frac{S-6}{S}\right)$	$+\frac{S-4}{S}+\frac{S-10}{S}$	$+\frac{S-12}{S}+\frac{S-12}{S}$	$\frac{-18}{S} + \frac{S - 16}{S}$	$+\frac{S-22}{S}$		-1.500		
		16k	4 ft - 2 ft		54 6 7				S=	8.0 f
	-	он — х-	s	Sesumen	f hinge			Rail Width X = S+OH-	OH = 1 = RW =	3.0 f 1.0 f 8.0 f
For X <	6: ne Lane =	$\frac{16}{32} \left(\frac{X}{S} \right)$						0.500		
For 6 ≤ Or	X < 12; ne Lane =	$\frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					0.625		
For 12 s	X < 18; ne Lane =	$\frac{16}{32} \left(\frac{X}{S} + \frac{X - X}{S} \right)$	6)					0.625		
O		22.0								

BRIDGE	County:	ANY XXX-XX-XXX	Highway:	Any XXXX	Design: Ck Dsn:	_	Date: 8	/15/20	2017 LRFD Spec Rev. 10/18 - (No Interin
IVISION			xample 4, Span		File:		distribution	factors.xl	Sheet: 8 of 8
LEVER	RULE								
EXTER	IOR (con'	t)	S = 8.0	ft	OH =	3.0 ft			
		and the same of th	V = 1.0	ft X = S+0	OH-RW-2ft =	8.0 ft			
For 18	X < 24:								
O	ne Lane =	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X}{S}\right)$	-6 c				= 0	.625	
		224.0	M 32	V = 18 \					
Tw	o Lanes =	$=\frac{10}{32}\left(\frac{x}{s}\right)^{\frac{1}{2}}$	$\frac{-6}{S} + \frac{X-12}{S} + \frac{X}{S}$	<u>x</u> (0)			= -	0.250	
For 24 s	≤ X < 30:	16/V V	-61						
O	ne Lane =	$=\frac{16}{32}\left(\frac{X}{S}+\frac{X}{S}\right)$	$\frac{-6}{5}$				= 0	.625	
	Acies	16 (X X	-6 X-12	X - 18				0.000	
IW	o Lanes =	32 5	$\frac{-6}{S} + \frac{X-12}{S} +$	S			= -	0.250	
Three	e Lanes =	$\frac{16}{3\pi}\left(\frac{X}{x} + \frac{X}{x}\right)$	$-6 + \frac{X-12}{2} $	$\frac{X-18}{S} + \frac{X-1}{S}$	24		= -	1.250	
		32 (3	2 2	2 2	Ž.				
For 30 S	≤ X < 36: ne Lane =	$=\frac{16}{32}\left(\frac{X}{S}+\frac{X}{S}\right)$	-6				= 0	.625	
		02.0	. /	O sie				.000	
Tw	o Lanes =	$=\frac{16}{32}\left(\frac{X}{S} + \frac{X}{S}\right)$	$\frac{-6}{S} + \frac{X-12}{S} +$	$\frac{X-18}{S}$			= 1	0.250	
		Alleren J		3. V. A. I	$(24 \ X - 30)$			0.000	
Thre	e Lanes =	32 8	s s	$\frac{X-18}{S} + \frac{X-2}{S}$	S		= -	2.625	
For 36 5	≤ X < 42:	16/X X	-61						
O	ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	S				= 0	.625	
Tw	o Lanes =	$16\left(\frac{X}{x} + \frac{X}{x}\right)$	$\frac{-6}{5} + \frac{x-12}{5} +$	X-18			120	0.250	
		3201		A . X	S. A. S.			O.III.OO	
Thre	e Lanes =	$=\frac{16}{32}\left(\frac{X}{S}+\frac{X}{S}\right)$	$\frac{-6}{S} + \frac{X-12}{S} +$	$\frac{X-18}{S} + \frac{X-2}{S}$	$\frac{14}{s} + \frac{X - 30}{S}$		-	2.625	
						X -36)			
Fou	r Lanes =	$=\frac{1}{32}\left(\frac{1}{S}\right)^{+}$	S + S	$\frac{X-18}{S} + \frac{X-2}{S}$	-+-S	S	= *	4.375	
For 42	≤ X ≤ 48:	16/ V V	-61						
O	ne Lane =	$= \frac{16}{32} \left(\frac{X}{S} + \frac{X}{S} \right)$	5				= 0	.625	
Two	o Lance	16 (X , X	$\frac{-6}{8} + \frac{X-12}{5} +$	X-18				0.250	
1.00	o Lanes =	32 8	S S	S)			-	0.200	
Three	e Lanes =	$=\frac{16}{32}\left(\frac{X}{2}+\frac{X}{2}\right)$	$\frac{-6}{6} + \frac{X-12}{6} +$	$\frac{X-18}{5} + \frac{X-2}{5}$	$\frac{24}{4} + \frac{\dot{X} - 30}{2}$		= -	2.625	
		N= 1.0	и о	0 0		V 36 V	-421		
Fou	r Lanes =	$=\frac{10}{32}\left(\frac{x}{5} + \frac{x}{5}\right)$	$\frac{-6}{S} + \frac{8-12}{S} +$	$\frac{X-18}{S} + \frac{X-2}{S}$	+ 30 +	S + X	s = -	6.500	
Autoria	25.				K-Br-32				
One La	on ne Loade	d	= 0.625		One La	ne Loaded		-	0.625
	nes Loade		= 0.875			nes Loaded		-	0.625
	anes Loa		= 0.875			anes Load		-	0.625
	nes Load		= 0.875			nes Loade		=	0.625
		77	5.575		. 55, 65				-7

4.5.15.5 Concrete Section Shear Capacity Spreadsheet

=	Highway:								
Texas	C-S-J:	XXXXXXX			Design:	BRG C	k Dsn:	BRG	
Department of Transportation	Bridge	Division	R	ev: 09/26/08			Date:	Aug-20	
CONCRETE SECTION SHEA	AR CAP	ACITY BY A	ASHTO L	RFD BRID	GE DESIG	N SPECIFIC	ATIONS, FO	URTH EDIT	ION, 200
Resistance Factors:			Units:	US					
h _v =	0.9								
b _M =	0.9								
h _N =	0.75								
Concrete:			Mild Steel:			Prestressed	Steel:		
fc=	5	ksi	fy =	60	ksi	fpu =	270 k	si	
Ec =	4070		Es =	29000		Ep =	28500 k		
					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	91
Shear force, Vu	kip	255	270.8	165.7	481.8	264.6	283.7	165	44
Axial force, Nu (+ if tensile)	kip	0	0	0	0	0	0	0	1000
Web width, bv	in	42.00	42.00	42.00	42.00	42.00	42.00	42.00	42.0
Shear depth, dv Mild steel reinf, area, As	in	80.59 10.92	80.59 10.92	80.59 10.92	80.59 10.92	80.59 10.92	80.59 10.92	80.59	80.5
Conc area on tension side, Ac	in^2	1785	1785	1785	1785	1785	1785	10.92 1785	178
Area of stirrups, Av	in^2	1.76	1,76	1.76	1.76	1.76	1,76	1.76	1.7
Stirrup spacing, s	in	6.8	6.8	6.8	6.8	6.8	6.8	6.8	6.
Prestressed steel area, Aps	in^2	0	0	0	0	0	0	Ol	-
Prestress shear, Vp	kip	o	0	0	0	0	0	0	
Average prestress, fps	ksi	0	0	0	0	0	0	0	
Torsional moment, Tu	kip-ft	830	415	415	830	830	415	415	83
Shear flow area, Ao	in^2	3493.5	3493.5	3493.5	3493.5	3493.5	3493.5	3493.5	3493.
Area of one leg of stirrup, At	in^2	0.44	0.44	0.44	0.44	0.44	0.44	0.44	0.4
Perimeter of stirrup, Ph	in	334	334	334	334	334	334	334	33
Calculated Values									
Уc	kip	576.4	569.2	624.2	533.3	571.6	564.4	614.7	533.
Vs	kip	1784.9	1812.7	2077.9	1569,9	1763.2	1791.6	2039,4	1569,
φVn	kip	2125	2144	2432	1893	2101	2120	2389	189
εx	555	7.10E-04	7.48E-04	4.86E-04	1.00E-03	7.33E-04	7.80E-04	5.20E-04	1.00E-0
θ	deg	33.20 2.410	33.70 2.380	30.30 2.610	36.40 2.230	33.50 2.390	34.00 2.360	30.75 2.570	36.4 2.23
Reg'd Shear reinf. Av/S	in^2/in	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
Reg'd Torsion reinf. At/S	in^2/in	0.000	0.000	0.008	0.019	0.000	0.009	0.008	0.00
Maximum stirrup spacing, Smax	in	24.0	24.0	24.0	22.5	24.0	24.0	24.0	22.
Conclusion									
	einforcing	OK	OK						
Shear Re				OK	OK	OK	OK	OK	OK

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.
- 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 $(0^{\circ}, 30^{\circ}, 45^{\circ}, \text{ and } 60^{\circ})$.

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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	A	AASHTO LRFD 2017		
Section Number	Title or Content	Section Number	Title or Content		
Eq. 1.3.2.1-1	$\sum \eta_i \gamma_i Q_i \le \Phi R_n = R_r$	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	$c = \frac{A_{ps}f_{ps} + A_{s}f_{s} - A_{s}f_{s}}{0.85f_{c}\beta_{1}b}$	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	$M_{n} = A_{ps} f_{ps} \left(d_{p} - \frac{a}{2} \right) + A_{s} f_{s} \left(d_{s} - \frac{a}{2} \right) - A'_{s} f'_{s} \left(d'_{s} - \frac{a}{2} \right) + 0.85 f_{c} (b - b_{w}) h_{f} \left(\frac{a}{2} - \frac{h_{f}}{2} \right)$	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-	NR		
5.7.3.4	Control of Cracking by Distribution of Reinforcement	5.6.7	NR		
Eq. 5.7.3.4-1	$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$	Eq. 5.6.7-1	NR		
Eq. 5.7.3.4-2	$A_{sk} \ge 0.012(d_l - 30) \le \frac{A_s + A_{ps}}{4}$	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	$P_n = 0.85 f_c A_1 m$	Eq. 5.6.5-2	NR		
Eq. 5.7.5-3	$m = \sqrt{\frac{A_2}{A_1}} \le 2.0$	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	A	ASHTO LRFD 2017
Section Number	Title or Content	Section Number	Title or Content
Eq. 5.8.2.5-1	$A_{v} \ge 0.0316\sqrt{f_{c}} \frac{b_{v}s}{f_{y}}$	Eq. 5.7.2.5-	$A_v \ge 0.0316\lambda\sqrt{f_c}\frac{b_v s}{f_y}$
5.8.2.7	Shear and Torsion – Minimum Spacing of Transverse Reinforcement	5.7.2.6	NR
Eq. 5.8.2.7-1	$s_{max} = 0.8d_v \le 24.0in$	Eq. 5.7.2.6-	NR
Eq. 5.8.2.7-2	$s_{max} = 0.4d_v \le 12.0in$	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	$d_e = \frac{A_{ps}f_{ps}d_p + A_sf_yd_s}{A_{ps}f_{ps} + A_sf_y}$	Eq. 5.7.2.8-	NR
Eq. 5.8.3.3-1	$V_n = V_c + V_s + V_p$	Eq. 5.7.3.3-	NR
Eq. 5.8.3.3-2	$V_n = 0.25 f_c b_\nu d_\nu + V_p$	Eq. 5.7.3.3-	NR
Eq. 5.8.3.3-3	$V_c = 0.0316\beta\sqrt{f_c}b_vd_v$	Eq. 5.7.3.3-	$V_c = 0.0316\beta\lambda\sqrt{f_c}b_vd_v$
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-	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	$\varepsilon_{S} = \frac{\frac{ M_{u} }{d_{v}} + 0.5N_{u} + V_{u} - V_{p} - A_{ps} f_{po}}{E_{S} A_{S} + E_{p} A_{ps}}$	Eq. 5.7.3.4.2-4	NR
Eq. 5.8.3.6.2-1	$T_n = \frac{2A_o A_t f_y \cot \theta}{s}$	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-	NR
Eq. 5.8.4.1-2	$V_{ri} \ge \Phi V_{ul}$	Eq. 5.7.4.3-	NR
Eq. 5.8.4.1-3	$V_{ni} = cA_v + \mu (A_{vf}f_y + P_c)$	Eq. 5.7.4.3-	NR
5.8.4.3	Cohesion and Friction Factors	5.7.4.4	NR

	AASHTO LRFD 2010	A	ASHTO LRFD 2017
Section Number	Title or Content	Section Number	Title or Content
Eq. 5.8.4.4-1	$A_{vf} \ge \frac{0.05A_{cv}}{f_y}$	Eq. 5.7.4.2-	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	$V_n = 0.2 f_c b_w d_e$	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	$A_s \ge \frac{2A_{vf}}{3} + A_n$	Eq. 5.8.4.2.2-5	NR
Eq. 5.13.2.4.2-6	$A_h \ge 0.5(A_s - A_n)$	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	5.8.4.3.4			
	$\Phi V_n = \Phi 0.125 \sqrt{f_c} (W + 2L + 2d_f) * d_f$				
	$\Phi V_n = \Phi \min(0.125\sqrt{f_c}\left(\frac{1}{2}W + L + d_f + c\right)d_f, 0.125\sqrt{f_c}\left(W + 2L + 2d_f\right)\cdot d_f)$		$\Phi V_n = \Phi \cdot \lambda \cdot 0.125 \cdot \sqrt{f_c} \cdot (W + 2L + 2d_f) \cdot d_f$		
	$ \left \begin{array}{c} \mathcal{L}_{j} u_{f}, 0.123 \sqrt{J_{c}} \left(W + 2L + 2u_{f} \right) \cdot u_{f} \right) \end{array} \right $		$\Phi V_n = \Phi \cdot \lambda \cdot \min(0.125 \cdot \sqrt{f_c})$		
			$\cdot \left(\frac{1}{2}W + L\right)$		
			$+d_f+c$		
			$ d_f, 0.125 \cdot \sqrt{f_c} $ $ (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	$V_n = \left(0.063\sqrt{f_c}b_f d_f\right) + \frac{A_{hr}f_y}{s}\left(W + 2d_f\right)$	Eq. 5.8.4.3.5-3	$V_n = \left(0.063\lambda\sqrt{f_c}b_fd_f\right) + \frac{A_{hr}f_y}{s}\left(W + 2d_f\right)$		
Appendix B5	General Procedure for Shear Design with Tables	NR	NR		
Eq. B5.2-1	$\varepsilon_{\chi} = \frac{\frac{ M_u }{d_v} + 0.5N_u + 0.5 V_u - V_p \cot\theta - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})}$	Eq. B5.2-3	NR		
Eq. B5.2-3	$\varepsilon_{x} = \frac{\frac{ M_{u} }{d_{v}} + 0.5N_{u} + 0.5 V_{u} - V_{p} \cot\theta - A_{ps}f_{po}}{2(E_{c}A_{c} + E_{s}A_{s} + E_{p}A_{ps})}$	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	$E_c = 33000 K_1 w_c^{1.5} \sqrt{f_c}$		

	AASHTO LRFD 2010	AASHTO LRFD 2017		
Section Number	Lifle or Content		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	$E_c = 1820\sqrt{f_c}$	