Influence of the LRFD Bridge Design Specifications on Seismic Design in Alabama

by

Paul Jeffrey Coulston

A thesis submitted to the Graduate Faculty of
Auburn University
in partial fulfillment of the
requirements for the Degree of
Master of Science

Auburn, Alabama
May 9, 2011

Approved by

Justin Marshall, Chair, Assistant Professor of Civil Engineering
Anton K. Schindler, Associate Professor Civil Engineering
Hassan Abbas, Assistant Professor of Civil Engineering

Abstract

The Alabama Department of Transportation (ALDOT) is in the process of transitioning from the AASHTO Standard Specification for Highway Bridges to the LRFD Bridge Design Specifications. One significant difference between the two specifications is the seismic design provisions. From a practical point of view, the desire is that typical details can be developed for the worst case scenarios which can be implemented for bridges throughout the state without a significant cost premium. To determine the effects of the updated seismic provisions on current practice, an initial study of existing bridges was completed. Three typical, multi-span, prestressed concrete I-girder bridges were selected for the study. In order to bracket the demands for typical bridges, the primary bridge geometry variables were span length, pier height and pier configuration. The bridges' Earthquake Resisting Systems were re-designed for the worst conditions for the state of Alabama. This paper discusses the changes made to the three bridges in order to meet the requirements.

After the re-design of the three bridges, a few conclusions were drawn. There was an increase in the connection between the substructure and substructure. Also, the amount of hoop reinforcing in the columns, drilled shafts and struts was increased. It was concluded that typical details could not be created for the worst seismic scenario in Alabama.

Acknowledgments

First of all, the author would like to thank all those whose help and encouragement led to the completion of this project. Thank you to Dr. Justin Marshall for his instruction and aid throughout this project. Also, thank you to the Alabama Department of Transportation for allowing me to be a part of this project and their guidance along the way. The author would like to thank the Highway Research Center at Auburn University for funding this study. Thank you to all my fellow students and professors who helped me to complete this project and graduate school classes.

I thank God for giving me the ability to complete this work, and knowing through him all things are possible. The Lord has truly blessed me immensely. Thank you to the Coulston and Piskorz families. The love and support of both families has helped me to accomplish my goals.

A final and special thank you goes to my loving and beautiful wife, Bethany Coulston. Her support and motivation helped me to complete this project. Thank you for supporting me through the good and bad times. I am not deserving of you, and I thank God everyday for you. Since you are my one and only true love, I would like to dedicate this work to you.

Table of Contents

List of Figures

List of Tables

List of Symbols

A_{sp} = area of hoop reinforcement

b_v = column diameter

B_o = column diameter

Cover = distance of concrete cover

D’ = diameter of hoop

d_{bl} = diameter of longitudinal bar

D_{sp} = diameter of hoop reinforcement

E = modulus of elasticity

f'_c = compressive stress of concrete

f_y = yield stress of steel

g = gravity

h = column height

H_o = clear height of column

I = moment of Inertia

K = stiffness

L = length of bridge

l_p = length of plastic hinge

P = point load

s = spacing of hoop reinforcement

T = period of structure

W = weight of substructure and superstructure

δ = deflection

Λ = factor for column end restraint condition

Chapter 1 : Introduction

1.1 Problem Statement

Alabama Department of Transportation (ALDOT) has been designing bridges using the AASHTO Standard Specifications for Highway Bridges (Standard Specification) (AASHTO 2002); however, due to Federal Highways Administration (FHWA) requirements, ALDOT will begin designing bridges in accordance with AASHTO LRFD Bridge Design Specifications (LRFD Specification) (AASHTO 2007). ALDOT has chosen to design the bridges with the AASHTO Guide Specifications for LRFD Seismic Bridge Design (Guide Specification) (AASHTO 2008). One of the significant differences between the two specifications is the seismic design provisions. Under the Standard Specification most of the state was classified as Seismic Performance Category A which required minimal seismic detailing and no additional analysis. The new requirements will influence future bridge design. With changes in the ground acceleration maps, it is expected that the substructure elements, the superstructure-to-substructure connections, and the foundations will see the most change. From a practical point of view, ALDOT's Bridge Bureau wants to develop typical details for the worst case scenarios which can be implemented for bridges throughout the state without a significant cost premium.

1.2 Objectives and Scope

Since typical details were trying to be developed, it was important to define the most important geometric features of the bridge. It was determined that one important feature was the pier height and the span length. The span length has a significant effect on the period and seismic force of the bridge. For a bridge with long spans, the bridge will have a long period of vibration, and a bridge with short spans will have a short period of vibration. Equation 1.1 shows the equation to calculate the structure's period of vibration. The pier height affects the stiffness of the bridge which in turn changes the period of vibration. When calculating the stiffness for the bridge piers the height of the pier is cubed which makes it an influential variable. The deflection equation for a cantilever beam or column with a point load on the end is shown in Equation 1.2. All the other variables in design, such as bridge width, foundation type, etc., were to remain somewhat constant. ALDOT was asked to choose three bridges to be re-designed with these parameters in mind. The three concrete bridges they chose were a short span bridge with short pier heights, a long span bridge with short pier heights, and a long span bridge with tall pier heights.

$$\delta = \frac{P * h^3}{E * I} \qquad \textbf{Equation 1.1}$$

$$T = 2 * \Pi \sqrt{\frac{W}{K * g}} \qquad \textbf{Equation 1.2}$$

This study was done to evaluate the seismic bridge design according to the Standard Specification and update the seismic design according to the LRFD Specification. This study was done to evaluate Three existing standard, multi-span, prestressed concrete I-girder bridges were selected to be redesigned according to the AASHTO Guide Specifications for LRFD Seismic Bridge Design and AASHTO LRFD Bridge Design Specifications. The main objectives were the following:

1. To determine the effects of LRFD seismic provisions on design and detailing of critical elements in the bridge lateral load resisting system.
2. To determine if typical, economically feasible details can be utilized for all the selected bridges.

1.3 Document Organization

This document is organized into five chapters. Chapter 1 is an introduction to the project. Chapter 2 contains a literature review and the comparison of the three design specifications investigated during the project. Chapter 3 describes the design process for the Guide Specification and the LRFD Specification. Chapter 4 shows the results from the re-design of the bridges. Lastly, Chapter 5 contains the conclusions that were made based on the case study design results.

Chapter 2 : Literature Review

2.1 Introduction

The three specifications discussed in this section have some differences. The design philosophy behind the Guide Specification is different from both the LRFD Specification and the Standard Specification. As the knowledge of earthquakes and fault zones have increased, the specifications have been updated to accommodate for the new information found. The specifications are always changing as new knowledge is discovered.

2.2 History

The compilation of the Standard Specification began in 1921 with the organization of the Committee on Bridges and Structures of the American Association of State Highway Officials (AASHO). Starting in 1921, the specifications were gradually developed. As several actions of the specification were approved, they were made available in mimeographed form for use by the State Highway Departments and other organizations. A complete specification was available in 1926 and revised in 1928. The first edition of the Standard Specification was printed in 1931. The last edition before switching to the LRFD design philosophy was printed in 2002 (AASHTO 2002).

The body of knowledge related to the design of bridges has grown immensely since 1931. The pace of advancements in bridge design are growing so rapidly that to accommodate this growth the Subcommittee of Bridges and Structures has been granted authority under AASHTO's governing documents to approve and issue Bridge Interims each year. In 1986, the Subcommittee submitted a request to the AASHTO Standing Committee on Research to undertake an assessment of U.S. bridge design specifications, to review foreign design specifications and codes, to consider alternative design philosophies to those underlying the Standard Specifications, and to render recommendations based on these investigations (AASHTO 2007). The investigation was completed in 1987, and it was found that the Standard Specification included discernible gaps, inconsistencies, and even some conflicts (AASHTO 2007).

The Standard Specification did not include the most recently developed design philosophies, load-and-resistance factor design (LRFD), which was gaining popularity in other areas of structural engineering. Until 1970, the sole design philosophy in the Standard Specification was known as working stress design (WSD). "WSD establishes allowable stresses as a fraction or percentage of a given material's load-carrying capacity, and requires that calculated design stresses not exceed those allowable stresses" (AASHTO 2007). The next design philosophy added was the load factor design (LFD). LFD reflects the variable predictability of certain loads types, such as vehicular loads and wind forces, through adjust design factors. A further philosophical extension was LRFD which takes variability in the behavior of structural elements into account in an explicit manner. "LRFD relies on extensive use of statistical methods, but sets forth the results in a manner readily usable by bridge designers and analysts" (AASHTO 2007).

After the 1971 San Fernando earthquake, significant effort was expended to develop comprehensive design guidelines for the seismic design of bridges. "That effort led to updates of both the AASHTO and Caltrans design provisions and ultimately resulted in the development of ATC-6, *Seismic Design Guidelines for Highway Bridges*, which was published in 1981" (AASHTO 2008). It was adopted as a Guide Specification in 1983 by AASHTO. In 1991, the guidelines were formally adopted into the Standard Specification, and then revised and reformatted as Division I-A of the Standard Specification. After damaging earthquakes in the 1980s and 1990s, it became apparent that improvements to the seismic design practice were needed. Several efforts were made by different groups in the development of ATC-32, *Improved Seismic Design Criteria for California Bridges: Provisional Recommendations* in 1996, the development of Caltrans' *Seismic Design Criteria*, the development of publication of MCEER/ATC-49 (NCHRP 12-49), *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges* in 2003, and the development of the South Carolina *Seismic Design Specification* in 2001. In 2005, work began to identify and consolidate the best practices from these four documents. "The resulting document was founded on displacement-based design principles, recommended a 1000-yr return period earthquake ground motion, and comprised a new set of guidelines for seismic design of bridges" (AASHTO 2008). In 2007, a technical review team refined the document into the Guide Specifications that were adopted in 2007 by AASHTO. In the following year more revisions were made, and then in 2008, the 2007 document and the revisions were approved as the Guide Specification (AASHTO 2008). A 2010 Interim was published for the Guide Specification which affected steel bridge design.

2.3 Comparison of the Seismic Design Specifications

This section describes the major differences between the seismic provisions of the Guide Specification, LRFD Specification, and the Standard Specification. The differences can be broken down into the following categories:

- New Ground Acceleration Maps

  The Standard Specification uses ground acceleration maps created by the United States Geological Survey (USGS) in 1988. The map assumes the soil condition to be rock. The seismic loads represented by the acceleration coefficients have a 10% probability of exceedance in 50 years which corresponds to a return period of approximately 475 years. For the Guide Specification and LRFD Specification, the ground acceleration maps depict probabilistic ground acceleration and spectral response for 7% probability of exceedance in 75 years which corresponds to a return period of approximately 1000 years. Also, the Guide Specification and LRFD Specification have maps for peak ground acceleration (PGA), 0.2 second spectral response acceleration (S_S), and 1.0 second spectral response acceleration (S_1) instead of just one map. Table 2.1 shows some of the spectral response values for different locations in Alabama. The accelerations are higher in north Alabama. The change in the ground acceleration maps generates an increase in design earthquake load for Alabama bridges. These changes are due to larger return period for the design earthquake and the significant amount of research completed on the New Madrid and East Tennessee faults.

Table 2.1: Oseligee Creek Bridge Bent 3 Design Changes

Location	PGA (g)	S_S (g)	S_1 (g)
Huntsville, AL	0.082	0.186	0.067
Birmingham, AL	0.083	0.171	0.058
Muscle Shoals, AL	0.087	0.207	0.076
Montgomery, AL	0.041	0.094	0.043

- New Design Spectral Shape

In the AASHTO Standard Specification, the spectral acceleration maximum is 2.5 x Acceleration coefficient, A. Unless the ground acceleration is greater than or equal to 0.3, then the maximum spectral acceleration is 2.0 x A. The response coefficient is decaying a rate of $1/T^{2/3}$. The response spectrum decreases at a rate of 1/T, but because of the concerns associated with inelastic response of longer period bridges, it was decided that the ordinates of the design coefficients and spectra should not decrease as rapidly as 1/T but should be proportional to $1/T^{2/3}$ (AASHTO 2002). The region decreasing at a rate of 1/T is the acceleration sensitive region. The design spectrum in the Guide Specification and LRFD Specification decreases at a rate of 1/T. In Figure 2.1, the response spectrums for the Guide Specification and Standard Specification are compared. The Guide Specification design response spectrum includes the short-period transition from the acceleration coefficient, A_S, to the peak response region, S_{DS}, unlike the Standard Specification. This transition is effective for all modes, including the fundamental vibration modes. According to the Guide Specification, the use of the peak response down to zero period is felt to overly conservative, particularly for a displacement-based design. (AASHTO 2008)

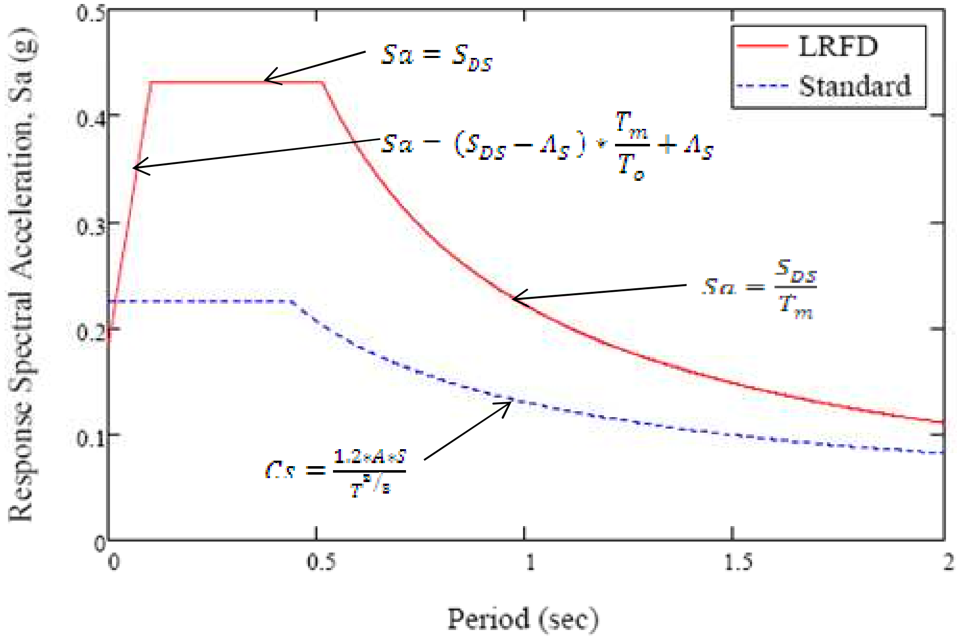


Figure 2.1: Comparison of the response spectrum for the Standard Specification and Guide Specification located at the Northeast corner of Alabama

- Importance/Operational Classification

When assigning a classification to a bridge, the basis of classification shall include social/survival and security/defense requirements and also consider possible future changes in conditions and requirements (AASHTO 2007). The Standard Specification has two different importance classifications which are Essential and Other. Essential bridges are defined by the Standard Specification as bridges that must continue to function after an earthquake (AASHTO 2002). The LRFD Specification has three operational classifications which are Critical, Essential, and Other. The LRFD Specification defines a Critical bridge as a bridge that must remain open to all traffic after a design earthquake and useable by emergency vehicles and security/defense purposes immediately after a large earthquake, which is an earthquake with a 2,500 year return period. Essential bridges are defined as bridges that at a minimum are open to emergency vehicles and for security/defense purposes immediately after the design earthquake, which is an earthquake with a 1,000 year return period (AASHTO 2007). If the bridge is critical or essential, then the design requirements will be more strenuous. The Guide Specification only specifically addresses Conventional bridges, which is defined as Other bridges by the LRFD Specification.

- Site Factors

The Standard Specification has four different soil profiles which correspond to four site factors. The seismic performance category is first chosen, and then according to the soil condition, the elastic seismic response coefficient is amplified. The Guide Specification and LRFD Specification have six soil profiles which correspond to six soil factors. The site class is chosen before the seismic design category is selected in contrast with the Standard Specification. Therefore, the spectral response coefficients can either be increased or decreased based on the site class. The soil factors affect the spectral response coefficients which in turn affects the seismic design category (SDC).

- Design Approaches

The design philosophy is different between the LRFD Specification and the Guide Specification. The Guide Specification is a displacement based design; while, the LRFD Specification and Standard Specification are force based designs. A displacement based design has to meet certain displacement limits set by the specification. In a force based design, response modification factors are used to modify the elastic forces. Since columns are assumed to deform inelastically where seismic forces exceed their design level, it is appropriate to divide the seismic elastic forces by a response modification factor (AASHTO 2007). Either approach is considered acceptable in the design of bridges.

2.4 Previous Research

In Virginia, a similar project was done by Widjana in 2003 to investigate the effects of the new LRFD deign procedures. He investigated a steel and concrete bridge. The location of the bridges was taken into account in his research. Widjana found that there was a significant increase in the time required to design a bridge according to the LRFD Guidelines than the Standard Specification. After re-designing the two bridges, the following changes in the detailing were discovered:

- Column shear reinforcement in potential plastic hinge zones,
- Transverse reinforcement for confinement at plastic hinges,
- Spiral spacing,
- Moment resisting connection between members (column/beam and column/footing joints),
- Minimum required horizontal joint shear reinforcement,
- Lap splices at bottom of the column, which are not permitted,
- Column joint spiral reinforcement to be carried into the pier cap beam, and
- Transverse reinforcement in cap beam-to-column joints. (Widjana 2003)

Also, the cost increase from the changes in design was evaluated. The impact of the changes affected the construction cost of the steel girder bridge by 1.0% and the prestressed concrete I-girder bridge by 0.2% (Widjana 2003).

2.5 Conclusion

The design specifications have undergone many changes over the years. The design philosophies of the specifications have changed as each new edition is published. The new knowledge gained from research and experiences is helping to develop the best design philosophy for the design of bridges.

Chapter 3 : Seismic Bridge Design

3.1 Introduction

The design processes for the three bridges chosen for this project are discussed in this chapter. The three bridges chosen as case studies are typical concrete bridges that were designed and built in Alabama. The bridges will be designed for the worst case for seismic hazard in Alabama. The design process of the Guide Specification and LRFD Specification are described throughout this chapter. Two typical design worksheets can be viewed in Appendix A and B for both of the design processes. These worksheets provide a clearer understanding of the design process.

Each bridge in this study had been previously designed according to the Standard Specification, which required a minimum amount of seismic design for the state of Alabama. The maximum Seismic Performance Category (SPC) for Alabama was category A. For SPC A, no detailed seismic analysis was required. The design requirements were a minimum support length and the connections for the substructure to superstructure were designed for 0.2 times the dead load reactions. As will be seen in this chapter, there was an increase in the design effort when using the new specifications.

A crucial part in any design is knowing the load path for the structure. The bridge deck distributes the dead, live and lateral loads to the girders. The girders then distribute the load into the bent cap beams and abutments. The connection between the girders and the substructure must be able to transfer the force between the connecting elements. The cap beams and abutments were assumed to remain elastic. The cap beam and abutment beams transfer the load into the columns, and then the columns transfer the load to the drilled shafts. Figure 3.1 shows a cross section that aids in visualizing the load path. The connections between the elements are crucial in a continuous load path because if the connections fail, undesirable performance results in possible structural failure.

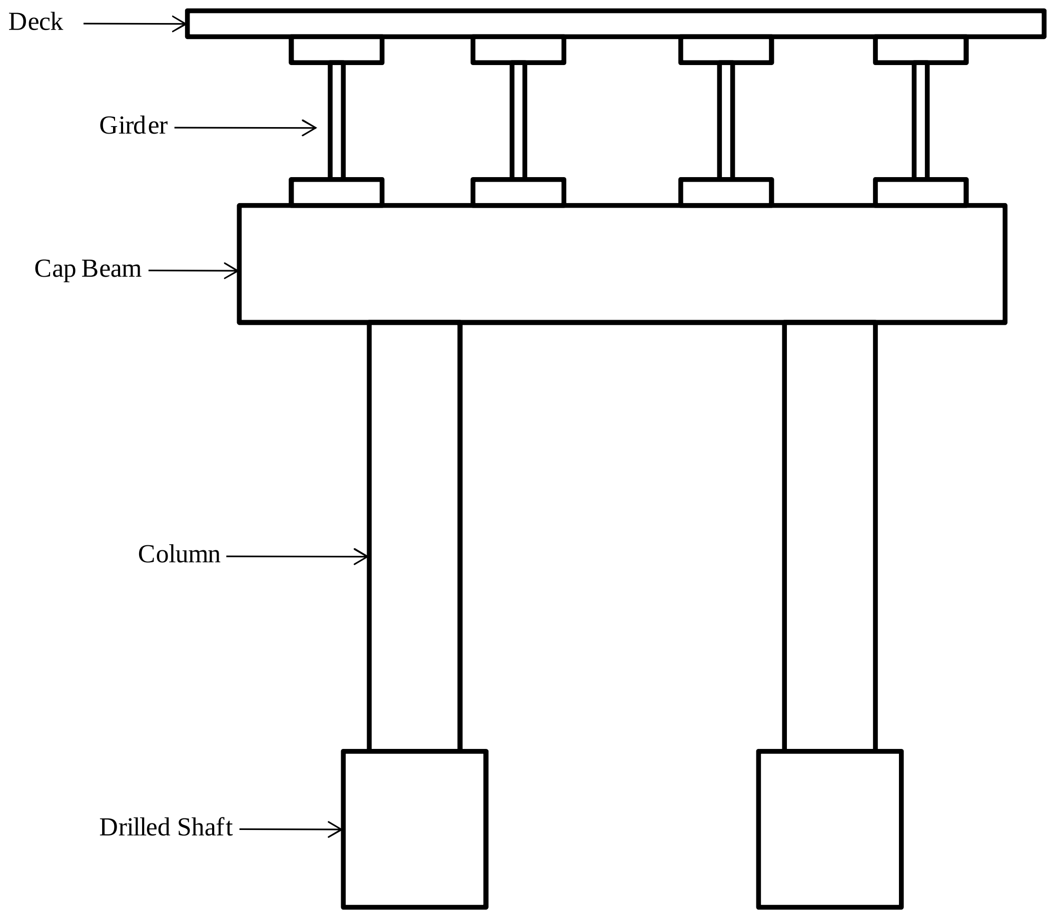


Figure 3.1: Load Path Diagram

3.2 Design Process

The initial seismic calculations and checks were the same for the three bridges. This is the case because the worst case hazard for seismic design in Alabama was chosen for this project. Therefore, the actual location of the bridges in Alabama for re-design is irrelevant. The steps, checks and calculations described in this section are the same for the three bridges designed.

3.3 Design Process for the Guide Specification

3.3.1 Initial Steps for the Design

A design worksheet was created in Mathcad (Parametric Technology Corporation 2007) for concrete bridges in order to facilitate the seismic design process. The input data will be assigned to a variable which will represent the input data. The units are hard coded into the worksheet. This was done because the units were causing problems later in the worksheet. The units used are pounds, inches and feet. Mathcad allows the data and the calculations to be seen easily. The worksheet was designed to have all the design checks required for seismic design of concrete bridges in SDC B. Most information that needs to be modified for bridge design is at the beginning of the worksheet. Other values will need to be assigned to variables later in the worksheet. The main purpose of the design worksheet is to aid in the seismic bridge design process.

The first step in the seismic design process was to input all the information about the bridge into the design worksheet. The length, width, span lengths, deck thickness, column diameters, drilled shaft diameters, column heights, girder areas, guard rail areas, and cap beam volumes were input into the sheet. Using that information, the column areas and drilled shaft areas were calculated. Some dimensions which are easier to input as feet were then converted to inches.

3.3.2 Applicability of Specification

The Guide Specification supplies flowcharts which can be followed to ensure all the requirements for seismic design are checked. First of all, the initial sizing of columns should be done for strength and service load combinations defined in the LRFD Specification. If the Guide Specification is going to be used for the design, the first task is to verify that the Guide Specification can be used. The Guide Specification applies to the design and construction of Conventional Bridges to resist the effects of earthquake motions. Critical and Essential bridges are not specifically addressed in the Guide Specification. The bridges analyzed and designed in this research are classified as Conventional Bridges.

3.3.3 Performance Criteria

The next item to be investigated was the performance criteria the bridges will be assigned. According to the Guide Specification, bridges shall be designed for life safety performance objective considering a seismic hazard corresponding to a 7% probability of exceedance in 75 years. Life Safety implies that the bridge has a low probability of collapse but may suffer significant damage and that significant disruption to service is possible. Also, partial or complete replacement may be required after the a design seismic event. If a higher performance criterion is desired by the owner, the bridge can be designed to that higher criterion. The bridges in this project were designed for life safety.

3.3.4 Foundation Investigation and Liquefaction

A foundation investigation needs to be done in the location that the bridge will be built. For bridges in SDC B and where loose to very loose saturated sands are in the subsurface profile, the potential of liquefaction should be considered since the liquefaction of these soils could affect the stability of the structure. The Guide Specification commentary discusses when liquefaction needs to be considered. It was assumed that the bridges considered in this thesis would not be affected by liquefaction.

3.3.5 Earthquake Resisting System

For SDC B, the identification of an earthquake resisting system (ERS) should be considered. For the selected bridges, a Type 1 earthquake resisting system was chosen. Type 1 structures have ductile substructures with essentially an elastic superstructure. This category includes conventional plastic hinging in columns and abutments that limits inertial forces by full mobilization of passive soil resistance. Also included are foundations that may limit inertial forces by in-ground hinging (AASHTO 2008).

3.3.6 General Design Response Spectrum

A response spectrum was created for the bridges. The response spectrum is created by using the seismic hazard maps and the site classification. The USGS seismic parameters CD-ROM, accompanying the Guide Specification, with the seismic hazard maps will determine the accelerations based on the latitude and longitude. The maps in the Guide Specification are for Site Class B. For this project, it was assumed that the soil condition would be Site Class D. In northeast and northwest Alabama, which are regions of high seismic activity, Site Class D is a good assumption as the worst case scenario for the soil conditions. After determining these values, a response spectrum was created by using a series of equations. The next step in the design process was to select the seismic design category. The SDC is used to permit different requirements for methods of analysis, minimum support lengths, column design details and foundation design. This is based on the 1-second period design spectral acceleration which is dependent on location and site classification. For all the bridges in this project, the Seismic Design Category is B. After this step in the process, the design will change depending on the bridge.

3.3.7 Displacement Demand Analysis

Since the Guide Specification is displacement based, a displacement demand analysis was done on the bridges. The first step in this process was to select an analysis procedure. The applicability of the procedure is determined by the regularity of a bridge, which is a function of the number of spans and the distribution of weight and stiffness. According to Section 4.2 of the Guide Specification, regular bridges shall be taken as those having fewer than seven spans; no abrupt or unusual changes in weight, stiffness, or geometry; and that satisfy requirements specified about the maximum subtended angle (curved bridge), span length ratio from span-to-span, and maximum bent/pier stiffness ratio from span-to-span. All the bridges that were designed were able to use analysis Procedures 1 or 2. Procedure 1, which is an equivalent static analysis, was chosen to determine the displacement demands for these bridges. Both the uniform load method and single-mode spectral analysis were used in the analysis of the bridges.

Both the uniform load method and single-mode spectral analysis are allowable analysis procedures to estimate the fundamental period. The uniform load method is suitable for regular bridges that respond principally in their fundamental mode of vibration. It is an equivalent static method of analysis that uses a uniform lateral load to approximate the effect of seismic loads. This method calculates the displacements with reasonable accuracy, but the method can overestimate the transverse shears at the abutments by up to 100 percent. The single-mode spectral analysis is slightly more complicated. The analysis procedure is based on the fundamental mode of vibration in either the longitudinal or transverse direction. The mode shape is found by applying a uniform load horizontal to the bridge and calculating the corresponding deformed shape. Both methods can be seen in Appendix A; however since the results from the two methods were similar, the uniform load method was chosen for in the design because it is simpler.

Before either method can be utilized, an analytical bridge model must be created. SAP 2000 Bridge Modeler (Computer & Structures 2007) was used to build a model that represented the bridge. The bridge modeler allows the designer the ability to create cross-sections that accurately represent the bridge. When creating the model, it was assumed that the drilled shafts would be considered fixed at the rock line and the contribution of the soil resistance would be neglected. When the Guide Specification was being created, the issue of the abutment contribution to the earthquake resisting system was heavily debated. However, according to the Guide Specification, the abutments should not be included in the earthquake resisting system. Therefore, the abutment's restraint was restricted to the vertical direction only because of the difficulty in modeling the soil pressure. If the abutment beam is supported by drilled shafts, which are considered structural elements, it was decided to use the abutments as part of the earthquake resisting system. If the abutments have drilled shafts, they are not relying on soil pressure to resist forces.

After building the model, a uniform load was applied. According to both methods, a uniform 1.0 kip/ft or kip/in load, p_o was converted into point loads that were applied to the joints along the bridge deck. Figure 3.2 shows the transverse and longitudinal loadings. After the load was applied and the model analyzed, the displacement of the structure was determined. For the uniform load method, the maximum deflections in the longitudinal and transverse directions were determined. The calculations for the stiffness, weight, period, spectral acceleration and the equivalent static earthquake loading can be seen in the sample calculation in Appendix A starting on page 116. Once the deflection was known, the stiffness of the bridge in both directions was calculated, and the stiffness equation can be viewed in Equation 3.1. The weight of the bridge was determined in order to be able to calculate the period of the bridge. A program was created in the Mathcad worksheet to calculate the spectral acceleration of the structure. Once all of these calculations were completed, then the equivalent static earthquake load, p_e, was determined. All the above elements were calculated for both the longitudinal and transverse directions. There is also a displacement magnifier that must be applied to structures with a short period. The magnifier is dependent on the bridges S_{DS}, S_{D1} and the structure period. The assumption that displacements of an elastic system will be the same as those of an elasto-plastic system is not valid for short-period structures that are expected to perform inelastically (AASHTO 2008). If the displacement magnifier is applicable, the displacement is multiplied by the magnifier. Instead of re-inputting the new loading into the SAP model, the Guide Specification allows the designer to scale the displacements by p_e/p_o.

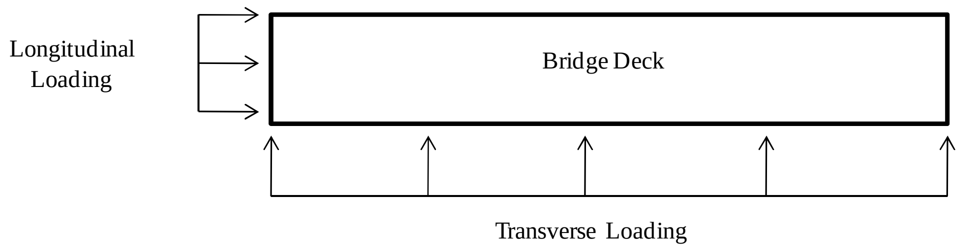


Figure 3.2: Loading Directions

$$K = \frac{p_o * L}{\delta_{\max}} \qquad \textbf{Equation 3.1}$$

The single-mode spectral analysis was also used to analyze the bridge. The process was more complex than the uniform load method. A bridge model was built, and a uniform load was converted into point loads and applied at the joints along the bridge deck. After analyses, the displacement along the deck was found. This was a time consuming task, because no way was determined to find the displacement of the joints only along the deck edge. In the end, this had to be done by looking at each joint individually. With the displacements put into a table and graphed, a best-fit line was fitted to the data. The equation of this line was used to calculate the shape functions α, β and γ. The equation for α, β and γ are shown in Equation 3.2, Equation 3.3 and Equation 3.4, respectively. The factor $v_s(x)$ is the displacement along the length of the bridge, and $w(x)$ is the unfactored dead load of the superstructure and substructure along the length of the bridge. These factors are later used to determine the period and equivalent static earthquake load. The response spectral acceleration was also calculated for this method. The equivalent static earthquake load is a line function that can be applied to the structure, and the force and the deflection along the length of the bridge can be seen in Appendix A. This method was more time consuming than the uniform load method but is more accurate for non-standard bridges.

$$\alpha = \int v_s(x)dx \qquad \textbf{Equation 3.2}$$

$$\beta = \int w(x)v_s(x)dx \qquad \textbf{Equation 3.3}$$

$$\gamma = \int w(x)v_s^2(x)dx \qquad \textbf{Equation 3.4}$$

3.3.8 Column Design

After the analysis has been completed, column design can begin. The first step was to verify that the columns of each bent meet the deflection criteria. For seismic loading, the a load factor of 1.0 is used in column design. The deflection at the top of the bent was found for both the transverse and longitudinal direction. The Guide Specification contains a simplified equation for bridges in SDC B or C which can be used instead of doing a more rigorous pushover analysis. The simplified equation for SDC B is displayed in Equation 3.5. Equation 3.6 shows the calculation for the x variable in Equation 3.5. The equations are primarily intended for determining the displacement capacities of bridges with single- and multi-column reinforced concrete piers for which there is no provision for fusing or isolation between the superstructure and substructure during design event accelerations. The equations are calibrated for columns that have clear heights that are greater than or equal to 15 ft. The formulas are not intended for use with configuration of bents with struts at mid-height (AASHTO 2008). The equations are a function of column clear height, column diameter and end restraint condition, such as fixed or pinned. If the equation for SDC B is not satisfied, then the allowable displacement capacity can be increased by meeting detailing requirements of a higher SDC, or a pushover analysis can be done. If the equation is not satisfied, it means the bridge is more prone to fail in shear.

$$\Delta_c = 0.12H_o \times (-1.27\ln(x) - 0.32) \geq 0.12H_o \qquad \textbf{Equation 3.5}$$

$$x = \frac{\Lambda * B_o}{H_o} \qquad \textbf{Equation 3.6}$$

A pushover analysis was done on all of the bridges that were investigated in this project. A pushover analysis is an incremental analysis that captures the overall nonlinear behavior of the elements by pushing them laterally to initiate plastic action. Each increment of loading pushes the frame laterally, through all possible stages, until the potential collapse mechanism is achieved. The Nonlinear Static Procedure is expected to provide a more realistic measure of behavior than may be obtained from elastic analysis procedures. SAP Bridge Modeler has the ability to do the seismic design of a bridge. By setting the SDC to D in the SAP seismic design program, a pushover analysis will be completed by SAP. The bridge's displacement demand and capacity are calculated during this analysis process.

After completing the displacement capacity check, the minimum support lengths for the girders were calculated for the bridge. The support length was checked at each abutment and bent. The minimum support lengths are a function of span length, column height and the skew angle. Since the bridge is in SDC B, the minimum support length must be increased by 150% as required by Article 4.12.2.

According to the Guide Specification, the shear demand for a column in SDC B shall be determined on the basis of the lesser of: the force obtained from a linear elastic seismic analysis or the force corresponding to plastic hinging of the column including overstrength. It is recommended that the plastic hinging forces be used whenever practical. Both methods are included in the design worksheet. In order to know which case is more practical, a moment capacity analysis was completed. When the bridge was analyzed by SAP 2000, a linear elastic analysis has been done; therefore, the linear elastic loads can be taken directly from the model. However, the loads coming out of SAP 2000 need to be amplified by p_e/p_o.

The moment capacity of the column was found by creating an interaction diagram using PCA Column (PCA 2004); although, any column design program could be used. A worksheet was set up using Microsoft Excel to help keep the information organized. To find the moment capacity of the column, the axial dead load was input into PCA Column along with the moment due to the dead load. The moment capacity must be amplified by an overstrength factor which depends on the yield stress of the reinforcement being used. The amplification factor for ASTM A706 reinforcement and ASTM A615 Grade 60 reinforcement is 1.2 and 1.4, respectively (AASHTO 2008). After the moment capacity was determined, the shear force in the column was calculated by equilibrium based on the moment capacity at the top and bottom and the column length. A model of the bent was created in order to apply the shear force at the center of mass of the superstructure and to determine the axial forces in the column due to overturning. The axial load from overturning is added to and subtracted from the dead load axial force. The reason for subtracting the seismic axial load is because the column could be in tension, or uplift, instead of compression. Next, the shear force of the column bent needs to be recalculated by re-entering the new axial forces into PCA column. The new shear force for the bent must be within 10% of the previous shear force. If not, the designer must iterate until the shear force is within 10% of the previous value. The interaction diagram was used to verify that the elastic loads on the column from the model did not exceed the failure envelope of the interaction diagram. Both the uplift and compressive cases must be checked for the axial load. As long as all the points fall within the failure envelope interaction diagram, the column design strength is sufficient.

After calculating the shear force, the plastic hinge length needs to be determined. The plastic hinge length is a function of the height of the column to the point of fixity, yield stress of the reinforcing and longitudinal bar diameter. The maximum of equations 3.7 and 3.8 is the plastic hinge length. The plastic hinge region is a function of the column diameter, plastic hinge length and the location where the moment exceeds 75% of the maximum plastic moment. A program was written in Mathcad to calculate both the plastic hinge length and plastic hinge region. These programs can be seen in Appendix A on page 128.

$$lp = 0.08 \times h + (0.15 \times f_y \times d_{bl})$$ **Equation 3.7**

$$lp = 0.03 \times f_y \times d_{bl}$$ **Equation 3.8**

The next step was to determine the column shear capacity in the plastic hinge region. The column diameter, spacing of lateral reinforcing, area of lateral reinforcing, diameter of lateral reinforcing, column cover and diameter of the hoop were input at the beginning of the design process. If values need to be altered later in the design, the designer can make the necessary changes. The concrete shear capacity is calculated first. According to Article 8.6.2 of the Guide Specification, a reinforcement ratio, ρ_s, for the column is calculated, and it was verified that ρ_s times the reinforcing yield stress is less than or equal to 0.35. Then a ratio for the ductility of the column was created in order to determine the compressive stress, v_c, of the column. This compressive stress was multiplied by the affected area, which is 0.8 times the gross area, and the shear capacity of the concrete was determined by this. A program was written in Mathcad to calculate the compressive stress on the column and can be viewed in Appendix A. If the axial load on the column is not compressive, then the concrete shear strength is equal to zero.

After calculating the concrete shear capacity, it was time to calculate the shear reinforcement capacity. The maximum shear capacity is dependent on the number of shear planes, area of the spiral, yield strength of the transverse reinforcement, diameter of the reinforcement, spacing of the transverse reinforcing, compressive stress of the concrete and the affected area of the column. A program was written in Mathcad to calculate the shear strength of the transverse reinforcement. After the shear reinforcement capacity and concrete shear capacity were calculated, they were summed together and multiplied by a phi factor, which is 0.9 for shear. The combined shear capacity was checked against the applied shear force to verify that the combined shear capacity was greater than or equal to the shear force.

There are several checks that need to be made in order verify the longitudinal and shear reinforcement are sufficient. For transverse reinforcing, the minimum ratio is required to be greater than or equal to 0.003 according to Article 8.6.5. If the transverse reinforcing does not meet this criteria, the spacing or the bar size of the seismic hoop can be modified. The maximum longitudinal reinforcement ratio must be less than 0.04 times the area of the column, and the minimum longitudinal reinforcement ratio is 0.007 times the area of the column. If the minimum longitudinal reinforcing does not meet the standard, then the column size can be decreased, or the longitudinal reinforcing can be increased. If the longitudinal reinforcing exceeds the maximum, the section size can be increased, or the longitudinal reinforcing can be decreased.

3.3.9 Seismic Design Category B Detailing

The spacing requirements within the plastic hinge region are stricter than outside the hinge region. The Guide Specification has specific requirements of how the hooks for the transverse reinforcing must be bent. The hoop requirements specify that the bar shall be a closed or continuously wound tie. A closed tie may be made of several reinforcing elements with 135 degree hooks having a six-diameter but not less than 3 in. extension at each end. A continuously wound tie shall have at each end a 135 degree hook with a six-diameter but not less than 3 in. extension that engages the longitudinal reinforcing. In Figure 3.1, two options are shown for the hook detail. The Illinois Department of Transportation allows for the seismic hoops to be mechanically spliced or welded, and the details of this connection are shown in Figure 3.2 (Tobias et. al. 2008). The maximum spacing of the transverse reinforcing is governed by the column diameter and diameter of the longitudinal reinforcing. The Guide Specification has a maximum spacing set at six inches. According to Article 8.8.9, the smallest of the following shall be used as the maximum spacing in the plastic hinge zone: one fifth of the column diameter, six times the diameter of the longitudinal reinforcing or six inches. Then, the maximum spacing needs to be checked to ensure that it will provide sufficient strength.

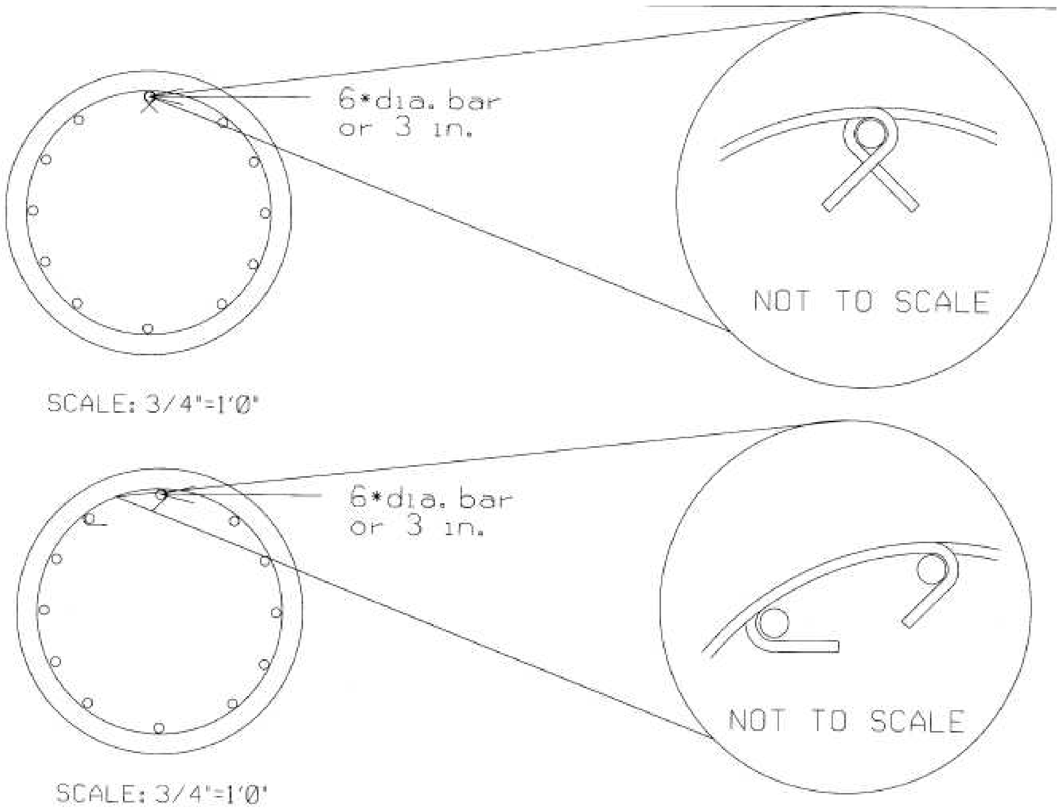


Figure 3.3: Seismic Hoop Detail

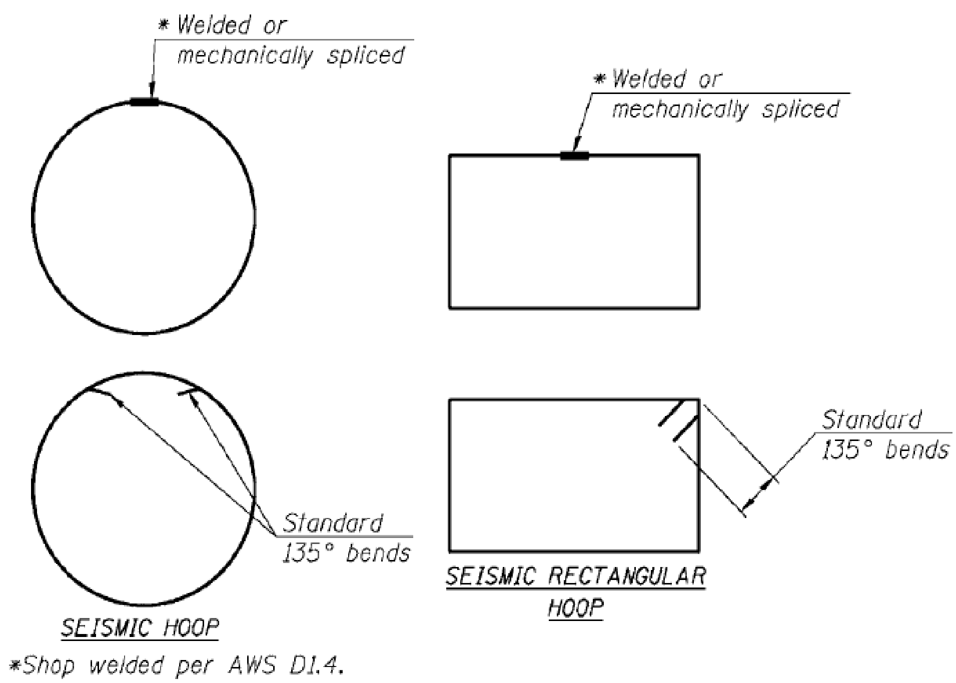


Figure 3.4: Illinois DOT Seismic Hoop Detail (Tobias et. al. 2008)

The requirement of the extension of transverse reinforcing into the bent cap beam and the drilled shaft was calculated. This requirement is not addressed in the Guide Specification for SDC B, but it is specified for SDC C and D. The extension is specifically addressed for SDC B in the LRFD Specification; therefore, that criterion was used here. The extension is an important part of the design. If the lateral reinforcing does not extend into both the bent cap beam and drilled shaft, a plane of weakness is formed at these joints. The column will be more likely to shear off at this point if the extension is not made. The extension is the larger of either 15 in. or one-half the column diameter.

3.3.10 Requirements outside the Plastic Hinge Region

For the design of the transverse reinforcing outside the plastic hinge zone, the LRFD Specification was used because the Guide Specification does not address the region outside the plastic hinge zone. This region encompasses the region in the column between the plastic hinge regions and the drilled shafts. All of these calculations can be viewed in Appendix A beginning on page 134. The region outside the plastic hinge zone was designed for the same shear force as the region inside the plastic hinge zone. The shear resistance of the steel reinforcement was calculated in a similar manner as in the Guide Specification. The shear resistance of the concrete was calculated in a little different manner. The concrete and steel reinforcing shear resistances were combined together, multiplied by a phi factor of 0.9, and checked to verify the combined value is greater than or equal to the shear force. If the resistance is less than the shear force, the spacing of the lateral reinforcing can be decreased, the reinforcing size can be increased or the column size can be increased.

The LRFD Specification has its own criteria for spacing outside the hinge zone. For the minimum transverse reinforcing, a requirement for the minimum transverse reinforcing is given in Article 5.8.2.5-1. The transverse reinforcing was then checked against the minimum reinforcing value and verified that it was greater than or equal to the minimum value. The maximum transverse reinforcing check in Article 5.8.2.7 has a few more steps than the minimum requirements. A shear stress on the concrete is first calculated, and the next step in the process is dependent on the shear stress. A program created in Mathcad to aid in this process can be seen in Appendix A on page 136. The maximum spacing, dependent on the shear stress, is either 24 in. or 12 in. After the maximum spacing is determined, it was checked to verify it supplies the strength needed in the design.

3.4 Design Process for the LRFD Specification

3.4.1 Major Differences from the Guide Specification

The initial steps in the seismic design of bridges in the LRFD Specification are similar to the Guide Specification; however, there are a few differences. As stated earlier, the primary difference is that the LRFD Specification is a forced based design and the Guide Specification is a displacement based design. The LRFD Specification also supplies a flow chart for seismic design. Instead of specifically stating the preliminary steps needed for the design, the flow chart instructs designers to do preliminary planning and design. It does not specify that an earthquake resisting system has to be identified or that liquefaction has to be checked. When selecting the Seismic Performance Zone, there is a slight difference in the upper limits for the 1-second period spectral acceleration range. The upper limits in the LRFD Specification are less than or equal to and not just less than like in the Guide Specification. Another difference is that the LRFD Specification can be used for the design of any classification of bridge. The differences after the initial design will be discussed later in this chapter.

3.4.2 Initial Steps for Design

Many of the initial steps for the LRFD Specification are similar to the Guide Specification. A new design worksheet was created in Mathcad for the LRFD Specification. The sheet is used in the same manner as for the Guide Specification. The first step was to input all the information about the bridge that was previously described. A response spectrum is generated in the same way as before. Instead of being called seismic design categories, the LRFD Specification refers to the design categories as Seismic Performance Zones (SPZ). Also, instead of using letters, the SPZs are numbers, beginning with 1 and ending with 4. Therefore, SPZ 2 is equivalent to SDC B.

After the seismic performance zone is determined, the response modification factors, R, for the structure were chosen. The LRFD Specification recognizes it is uneconomical to design a bridge to resist large earthquakes elastically; therefore, columns are assumed to deform inelastically where seismic forces exceed their design strength, which is established by dividing the elastically computed force effects by the appropriate R-factor. The R-factor for connections is smaller than those for substructure members in order preserve the integrity of the bridge is connections under extreme loads. Table 3.1 displays the R-factors that were used in the design of the structures for this project. As can be seen from the chart, the R-factor for the abutment to superstructure connection actually amplifies the design force.

Table 3.1: Response Modification Factors for LRFD Specifications

	Response Modification Factors
Multiple Column Bents	5.0
Connections: Superstructure to Abutment	0.8
Columns to Bent Cap	1.0
Column to Foundation	1.0

The uniform load method and the single-mode spectral analysis were also used in the analysis of the bridges. Since these methods have previously been described in the Guide Specification, they will not be discussed in detail here. After the analysis has been completed on the structure, two load combinations were created from the equivalent seismic loads that were determined during analysis. The load combinations are made up of the loads from both the transverse and longitudinal direction. The load combinations are shown in Equations 3.5 and 3.6. The maximum load combination was used in the design of the structures. The Mathcad worksheet was set up in a way that the elastic loads from SAP can be brought into the worksheet without any modification. An R equivalent value was created by dividing the largest load combination by the R-factor. The R equivalent value represents the greatest load combination from the equivalent seismic loads divided by the response modification factor. The R-equivalent value was created to avoid having to re-input the loads into SAP. Later, the shear forces from SAP were multiplied by the R-equivalent value in order to have the correct loading. This can be better explained by referring to the worksheet in Appendix B. The shear force for the drilled shafts was taken as twice as much as the columns which the drilled shafts were supporting. This can be done because the drilled shaft R-factor is half as much as the columns. These loads were used in design.

$$\text{Load Case 1} = \sqrt{(1.0\times p_{eTran})^2 + (0.3\times p_{eLong})^2} \qquad \textbf{Equation 3.9}$$

$$\text{Load Case 2} = \sqrt{(1.0\times p_{eLong})^2 + (0.3\times p_{eTran})^2} \qquad \textbf{Equation 3.10}$$

3.4.3 Column Design

One of the first steps in the seismic design of the structure was to calculate the minimum support length. The minimum support length was calculated in the same manner as in the Guide Specification. The span length, column height and angle of skew are still the controlling factors in the calculation for the minimum support length.

After the minimum support length was calculated, the minimum and maximum amount longitudinal reinforcing was checked. The area of the longitudinal reinforcing was calculated. Programs were set up in the worksheet to check the minimum and maximum longitudinal reinforcing requirements. According to Article 5.10.11.3, the minimum longitudinal reinforcing is 0.01 times the gross area of the column, and the maximum longitudinal reinforcing is 0.06 times the gross area of the column. Also, the flexural resistance of the column needs be checked. This can be done by using any kind of column design program that creates a column interaction diagram. For this project, PCA Column was used to develop column interaction diagrams. All critical load combinations were checked to ensure they fell within the interaction diagram.

The next step in the design process was to design the transverse reinforcing in the end regions. The initial input for the program includes the shear and axial load for the column, column diameter, phi factor for shear, spacing of the transverse reinforcing, area and diameter of the transverse reinforcing, concrete cover, hoop diameter and longitudinal bars diameter. For a nonprestressed section, the LRFD Specification allows β and θ to be 2.0 and 45 degrees, respectively. β is the factor indicating the ability of diagonally cracked concrete to transmit tension and shear, and θ is the angle of inclination of diagonal compressive stresses (AASHTO 2007). These factors were used to calculate the shear capacity of the concrete. The effective shear depth of the column was calculated. A program was created in Mathcad to determine the allowable shear resistance of the concrete. The checks the program makes can be seen in Appendix B on page 188. The shear capacity of the concrete is dependent on f'_c, β, column diameter, effective shear depth, gross area of the column and the minimum axial compressive load, which is the reason the minimum axial load was calculated in the beginning of the worksheet. After the concrete shear capacity is calculated, the shear reinforcement capacity was calculated. The shear reinforcement capacity is based on the area of reinforcing, reinforcing yield stress, shear depth, θ, and the spacing of the transverse reinforcing. The equations for the concrete and reinforcing shear capacity are shown in Equations 3.11 and 3.12, respectively. Equations 3.13, 3.14, and 3.15 show the calculation for the d_v variable. The shear reinforcement capacity and concrete shear capacity were summed together, multiplied by the phi factor for shear, and checked to verify the capacity is greater than the demand.

$$V_c = 0.0316 \times \beta \times f'_c \times b_v \times d_v \qquad \textbf{Equation 3.11}$$

$$V_s = \frac{2 \times A_{sp} \times \sqrt{f_y} \times d_v \times \cot(\theta)}{s} \qquad \textbf{Equation 3.12}$$

$$d_v = 0.9 * d_e \qquad \textbf{Equation 3.13}$$

$$d_e = \frac{b_v}{2} + \frac{D_r}{\Pi} \qquad \textbf{Equation 3.14}$$

$$D_r = b_v - Cover - D_{sp} - \frac{d_{bl}}{2} \qquad \textbf{Equation 3.15}$$

The length of the plastic hinge region was then calculated. Once again, a program was created in the Mathcad worksheet to calculate the length of the plastic hinge region. The plastic hinge region is dependent on the column diameter and the height. According to Article 5.10.11.4.1c, the largest of the following options determines the plastic hinge region: the column diameter, 1/6 times the column height in inches, or 18 in. According to the LRFD Specification, the spacing in the plastic hinge region is the smallest of either ¼ the column diameter or 4 in. The shear strength of the reinforcing and concrete were again checked against the shear force to verify they were still greater than or equal to the load. Also as described in the Guide Specification, an extension of the transverse reinforcing into the bent cap beam and drilled shaft is required. The requirements for this extension are the same as before.

For the transverse reinforcing within the plastic hinge region, the hoops must be detailed to be seismic hoops. The requirements, which are described in the Guide Specification, to be a seismic hoop are the same as in the LRFD Specification. However, there are a few additional requirements in the plastic hinge region. There is a minimum required volumetric ratio of the seismic hoop reinforcing. The volumetric ratio must be greater than or equal to 0.12 times the compressive stress of the concrete divided by the yield stress of the reinforcing bars. Equation 3.16 shows the formula for the volumetric ratio, and Equation 3.17 is the equation the volumetric ratio is compared against. If this requirement is failed, then the spacing of the lateral reinforcing can be decreased, the area of the lateral reinforcing can be increased, or the diameter of the column can be decreased.

$$\rho_s = \frac{4 \times A_{sp}}{s \times D'} \qquad \textbf{Equation 3.16}$$

$$z = 0.12 \times \frac{f'_c}{f_y} \qquad \textbf{Equation 3.17}$$

The LRFD Specification does not allow lap splices in longitudinal reinforcement in the plastic hinge region. In the design and construction process, it is often desirable to lap longitudinal reinforcement with dowels at the column base; however, this is undesirable for seismic performance. The splice occurs in a potential plastic hinge region where requirements for bond is critical, and lapping the main reinforcement will tend to concentrate plastic deformation close to the base and reduce the effective plastic hinge length as a result of stiffening over the splice length.

3.4.4 Requirements outside the Plastic Hinge Zone

The requirements outside the plastic hinge zone are the same as described in the Guide Specification; therefore, please refer back the Guide Specification process for this information.

3.5 Connection Design

3.5.1 Connection between Substructure and Superstructure

For the connection between the substructure and superstructure, the same design process was used for the LRFD Specification and Guide Specification. The Guide Specification does not address the connection of the girders to the bent cap beam or abutment. The LRFD Specification and AISC Steel Construction Manual were used as the standards for design. ALDOT has standard clip angle details that have been used for this connection. Their current connection uses an L6x6x1/2x12 connected to the bent cap beam with an anchor bolt, either 1.25 in. or 1.5 in. in diameter, and two precast screw inserts in the girder. Figure 3.3 shows the current connection used by ALDOT. After designing a few of the bridges, it became evident the current connection would have to change. The same angle size of 6x6 was still kept, but the length and thickness of the angle had to increased, along with the number of anchor bolts in certain situations. Also, instead of using the precast screw inserts, it was decided to use a through bolt in the girder. This will provide much more strength than the precast inserts. The new connection is displayed in Figure 3.4.

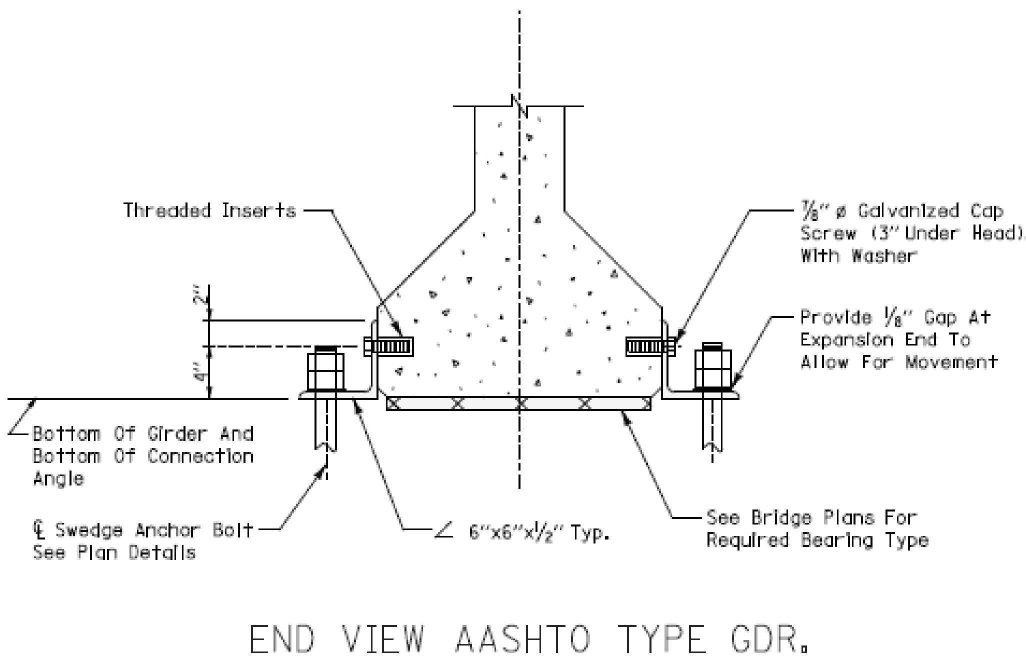


Figure 3.2: Standard Specification Connection used by ALDOT (Taken from ALDOT Standard Details Standard Drawing I-131 Sheet 7 of 8)

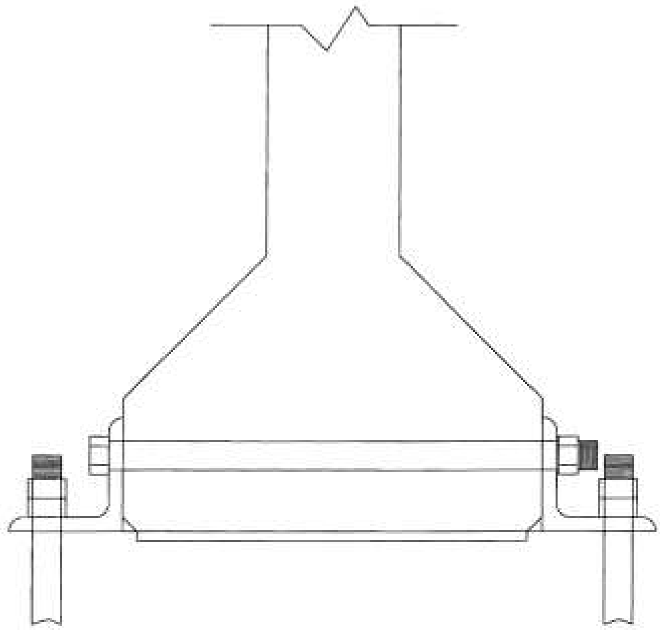


Figure 3.3: Modified Substructure to Superstructure Connection

Several pieces of information must be input in the beginning of the worksheet for the connection. The entire shear force for the bent is evenly distributed among the girders and their connections. The Mathcad sheet has the resistance factors from the LRFD Specification that are relevant to the connection design. An ASTM A307 Grade C bolt was used. The ultimate tensile stress, F_u, for this bolt is 58 ksi. The angle properties that were input are yield stress, ultimate stress, thickness, height, width and length, and the height above the bevel, which is the k-value in the AISC Steel Manual. The distance from the vertical leg to the center of the hole of the through bolt was used in calculating the bolt tension. The hole diameter is 0.25 in. larger than the bolt diameter. This is staying consistent with ALDOT's previous details. The block shear length and block shear width for the angle was calculated. The distances from the center of the bolt to the edge of the angle and to the toe of the fillet were determined.

After the initial information is input into the worksheet, the design of the connection can begin. The calculations for the connection can be seen in Appendix A beginning on page 151. The shear force for the angle is calculated by dividing the entire shear force for the bent or abutment by the number of connections. Each girder has two clip angles. The shear force for the angle was used as the shear force for the bolt if there is only one bolt. If there is more than one bolt, the shear force for the angle needs to be divided by the number of bolts. To simplify the design process, it was assumed that the anchor bolts and through bolts have the same diameter and are the same material. The first design check was the shear resistance of the bolt. The bolt shear resistance was calculated and compared to the applied shear force. If the shear resistance was too low, the diameter could be increased, the grade can be changed, or the number can be increased.

The next check was the bearing resistance of both slotted and standard holes. The bearing strength for standard holes is dependent on the bolt diameter, angle thickness, and ultimate stress. The bearing strength for slotted holes is dependent on the ultimate stress of the angle, angle thickness, and clear distance between the bolt hole and the end of the member. These bearing strengths were checked against the bolt shear and verified that the bearing strength was greater than the shear per bolt.

The tensile strength of the bolt was then calculated. The shear force that the angle encounters was converted into a tensile force. It was assumed the shear force enters at the mid-height of the angle, and the tension force of the anchor bolt is located 4 in. away from where the moment is being summed. After the moments were summed, the tension force was determined. The tension resistance of the bolt is dependent on the area and the ultimate strength of the bolt. The tension was then checked to verify it was greater than or equal to the shear force per bolt.

The final check for the bolt is the combined tension and shear check. A program was created in the Mathcad worksheet to calculate the combined tension and shear resistance and be seen in Appendix A on page 154. The combined resistance is dependent on the shear resistance, the area, and tension capacity. The combined tension and shear resistance was checked against the shear force per bolt to ensure the resistance was greater than or equal to the shear force. If the connection fails, then increase the area of the bolt, change the grade of bolt, or increase the number of bolts.

All of the strength checks for the angle come out of the AISC Steel Manual. The first check made for the angle strength was the block shear check. The block shear length and width were calculated by hand and input into the program in the initial steps. Since the tensile stress is uniform, according the AISC Manual the shear lag factor for this situation is 1.0 (AISC 2005). The block shear equations in the worksheet need to be changed as the number of bolt holes within the angle changes. A program was created to verify that the block shear resistance was greater than or equal to the shear per angle.

Next, the member was checked to ensure it had sufficient tensile strength. According to the AISC Manual, the shear lag factor for single angles with 2 or 3 fasteners per line in the direction of the loading is 0.6; therefore, it is conservative to use this factor even if there is one bolt. The net tensile and effective areas were calculated. The effective area was used to calculate the tensile resistance of the angle. It was verified that the tensile resistance was greater than or equal to the shear force per angle.

The angle was also checked for bending strength. A SAP model was created of the angle, and a shear force was applied at the bolt location in the top leg. The critical section for bending in the angle is a found just above the bevel. The k distance in the AISC Manual is the distance above the bevel (AISC 2005). The moment was determined at that point, and then compared to the moment resistance of the angle. The moment resistance was calculated by determining the plastic section modulus for the angle and multiplying it by the yield stress of the angle and the flexural phi factor. The bending strength is dependent on the length and thickness of the angle.

The last design check made for the connection was the shear resistance of the angle. The shear resistance calculation is dependent on the yield stress of the angle and area of the angle. The calculated shear resistance was compared to the shear force per angle and verified it is greater than or equal to the applied shear.

3.5.2 Connection of Drilled Shaft

The drilled shaft is designed in the same way as a section outside the plastic hinge zone. The drilled shaft is a capacity protected member, which means that it must remain elastic. In the Guide Specification design, the drilled shaft is designed for the overstrength moment capacity or the elastic force in the column, whichever method is chosen by the designer. According to the LRFD Specification, seismic forces for foundations, other than pile bents and retaining walls, shall be determined by dividing the elastic seismic forces by half the R-factor. Therefore, the drilled shafts of the structures in this project were divided by R/2. Also as addressed earlier, there must be an extension of the plastic hinge reinforcing a certain distance into the drilled shaft.

3.6 Conclusion

A description of the design processes used for the design of the three bridges chosen for this study is provided in this chapter. As can be seen, the design process for the bridges in Alabama in SDC B is a more in depth process than the previous design process in the Standard Specification. However, the worksheets developed for these design processes help in the seismic design of the bridges.

Chapter 4 : Bridge Design

4.1 Introduction

The three bridges chosen for this study are described in this chapter. Following the description, the design according to the Guide Specification and LRFD Specification are detailed. The transverse reinforcing and the superstructure to substructure connection saw the most changes. The design worksheets for the bridges can be seen in the Appendix A, B, D, E, G, and H.

4.2 Oseligee Creek Bridge

4.2.1 Description of the Bridge

Oseligee Creek Bridge consists of three 80 ft, simple spans. The concrete bridge is 240-ft long and 32-ft 9-in. wide. The substructure has a 7-in. thick concrete deck supported by four AASHTO Type III girders that are equally spaced at 8-ft 4-in. on center. Eight inch thick web walls are located at the abutments, bents, and at mid-span. The bridge has two bents which are made of a 4 ft x 5 ft x 30 ft cap beam, two 42-in. diameter circular concrete columns, and two 42-in. diameter circular drilled shafts. The abutments, which are 3 ft x 3.5 ft x 35 ft, are supported by two 42-in. diameter circular drilled shafts. The columns and drilled shafts have 6 in. of cover. The columns have an average above ground height of 9 ft. Currently, the column and drilled shaft longitudinal reinforcing is (12) - #11 bars, and the transverse reinforcing is #5 hoops at 12 in. o.c. It was assumed that 4,000 psi concrete and 60,000 psi reinforcing was used in the design and construction. A 3-D model of the bridge is shown in Figure 4.1.

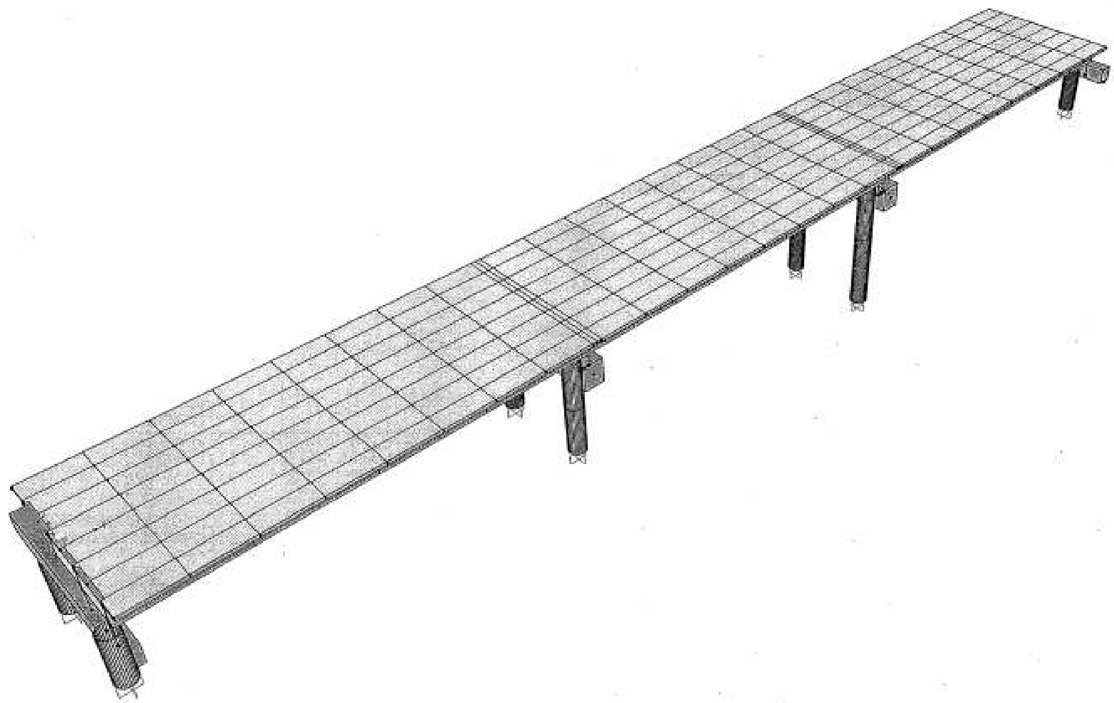


Figure 4.1: 3-D SAP Model of Oseligee Creek Bridge

4.2.2 Results from Guide Specification Design

One of the first steps required to design this bridge was to create a bridge model in SAP. From the plans, a model was created. Both the uniform load method and single-mode spectral analysis were done on this bridge. The full results from those analysis and design can be seen in Appendix A beginning on page 111. The uniform load was applied to the bridge model as described earlier. The maximum deflections in the longitudinal and transverse directions from the unit uniform load were 1.671 in. and 3.228 in., respectively. The design maximum deflections in the longitudinal and transverse directions were 0.643 in. and 1.009 in., respectively. The maximum deflections from the single-mode spectral method were somewhat lower in the longitudinal direction and slightly lower in the transverse direction. The SAP model produced a period of 0.394 sec in the transverse direction and 0.441 sec in the longitudinal direction. The response spectrum of the bridge in Figure 4.2 shows that the periods fall on the horizontal section. As can be seen by these deflections, this is a very stiff bridge.

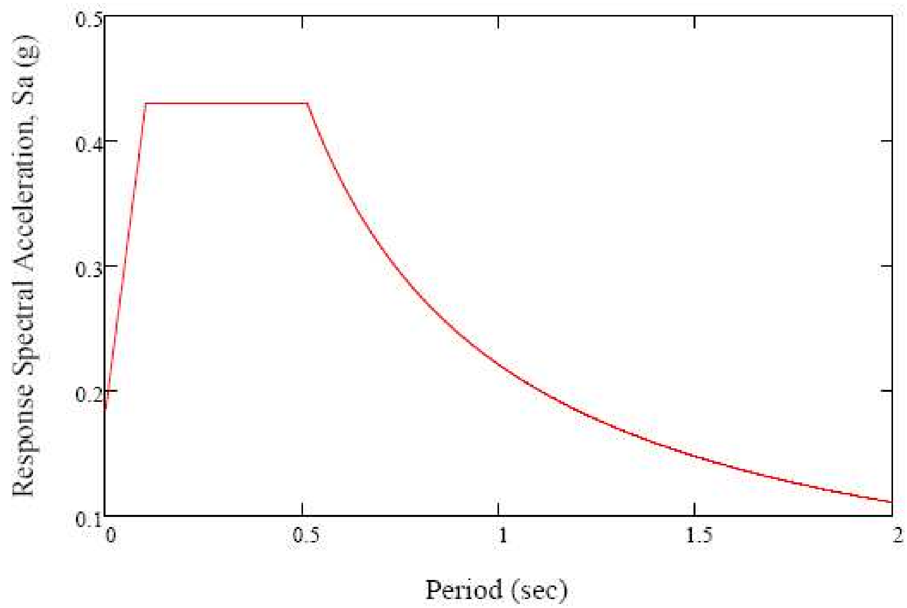


Figure 4.2: Response Spectrum of Oseligee Creek Bridge

The displacement capacity of the columns had to be verified. When the simplified equations were checked for the above ground height, approximately 9 ft, the columns failed the capacity equations. After discussing this with ALDOT, it was decided to allow hinging to occur below ground. The point of fixity was assumed at the rock line. After allowing below ground hinging, Bent 3 satisfied the simplified equations; however, Bent 2 still did not satisfy the equations because of the diameter-to-length ratio was too large. It was determined that in order to satisfy the equations with a 42 in. diameter column the clear column height would have to be 20 ft. The Bent 2 clear column height was 18 ft, and Bent 3 column height was nearly 26 ft. According to the Guide Specification, a pushover analysis can be done to verify the displacement capacity of the bridge. The bridge satisfied the pushover analysis, and these results can be seen in Table 4.1. Figure 4.3 shows a pushover curve diagram of one of the load cases created in SAP. From the location of the displacement demand on the curve, it can be seen that the demand displacement is still in the curves elastic portion. After the displacement analysis was completed the minimum support length was calculated. The minimum support lengths for the bents and abutments were all approximately 17.5 in.

Table 4.1: Pushover Analysis Results

Load Case	Demand (in.)	Capacity (in.)	Check
Bent 2 Transverse Direction	0.96	2.75	OK
Bent 2 Longitudinal Direction	1.24	2.12	OK
Bent 3 Transverse Direction	1.06	3.61	OK
Bent 3 Longitudinal Direction	1.14	4.63	OK

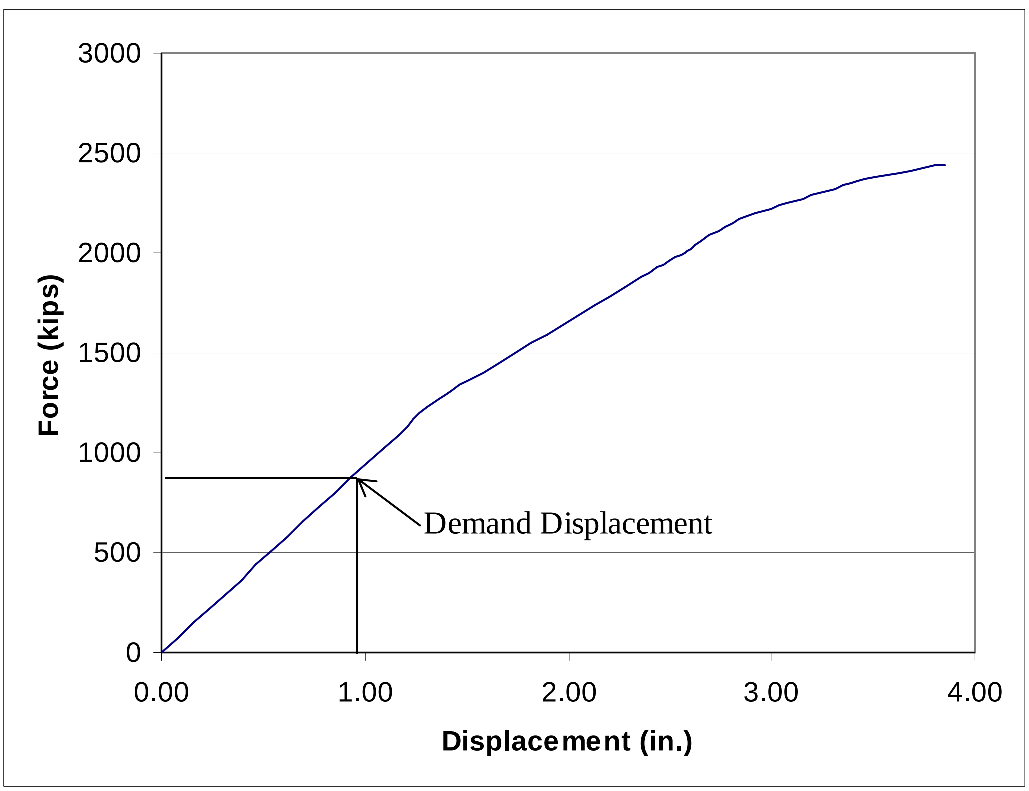


Figure 4.3: Oseligee Creek Bridge Pushover Curve for Load Case Bent 2 Transverse Direction

For this particular bridge, it was decided that the overstrength moment capacity would be used in the design. The overstrength capacity may be a little conservative;

however, it was important to complete a design with this approach because this method is preferred by the Guide Specification. Interaction diagrams were generated for the columns and drilled shafts to verify their capacity. The interaction diagrams can be seen in Appendix C starting on page 224. All longitudinal reinforcing for the columns and drilled shafts proved to be sufficient.

After calculating the seismic forces, the plastic hinge length was calculated. Both Bent 2 and Bent 3 were controlled by 1.5 times the column diameter or 63 in. The transverse reinforcing in the plastic hinge region is #5 hoops at 6 in. o.c. and was detailed as seismic hoops. The hoop spacing was controlled by the maximum allowed by the Guide Specification. The minimum transverse reinforcement requirement was satisfied. The maximum and minimum longitudinal reinforcement requirements were satisfied by the reinforcing of (12)-#11 bars. The extension of the transverse reinforcing into the cap beam and the drilled shaft was 21 in. which was controlled by half the column diameter.

The region outside the plastic hinge was designed according to the LRFD Specification because the Guide Specification does not address this region. In the current design, the hoop spacing was set at 12 in. on center. That was used as the maximum spacing outside the plastic hinge zone. For both Bent 2 and 3, #5 hoops at 12 in. on center. were sufficient to provide the required strength. This confinement steel configuration was sufficient in satisfying the minimum and maximum requirements.

The connection between the superstructure and substructure was also affected by the new design forces. The angles used in the design are ASTM A36 steel, and the bolts are ASTM A307 Grade C. The standard holes were ¼-in. larger than the anchor or through bolt diameter, and the slotted hole was ¼-in. wider than the anchor bolt diameter and six inches long. The length of the slotted hole was maintained at six inches. If the connection required expansion, it was redesigned as an expansion connection. Both abutments allowed for expansion. The clip angle at the abutments was an L6x6x3/4x12 with one 1.25-in. diameter anchor bolt and through bolt. The detail for this connection is displayed in Figure 4.4. For Bent 2, the side span girder connection was designed for expansion. The other set of girders for Bent 2 and all the girders supported by Bent 3 have fixed connections. For the fixed connection, an L6x6x7/8x16 was used. The expansion connection used a 20-in. long angle. The angle was connected to the substructure with two 1.25-in. diameter anchor bolts and through bolts. These details are shown in Figure 4.5. The length of the angle was controlled by the spacing of the bolt holes, and the thickness of the angle was controlled by the angle's bending strength. The combined tension and shear check was what drove the need for two through bolts and anchor bolts.

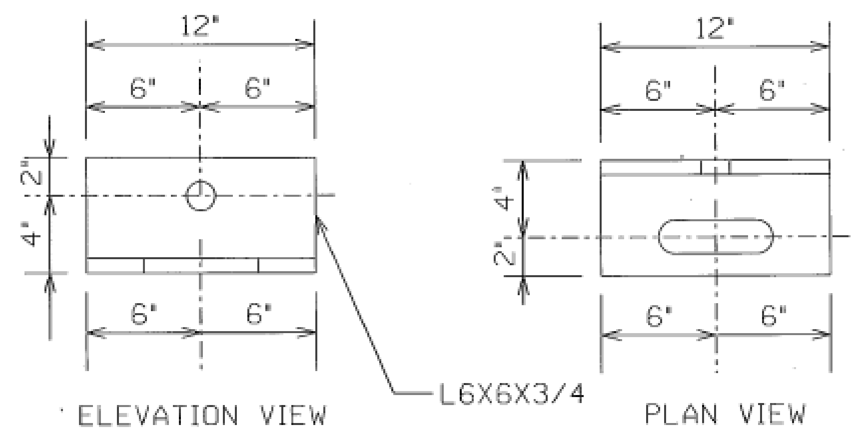


Figure 4.4: Oseligee Creek Bridge Abutment Connection (Expansion)

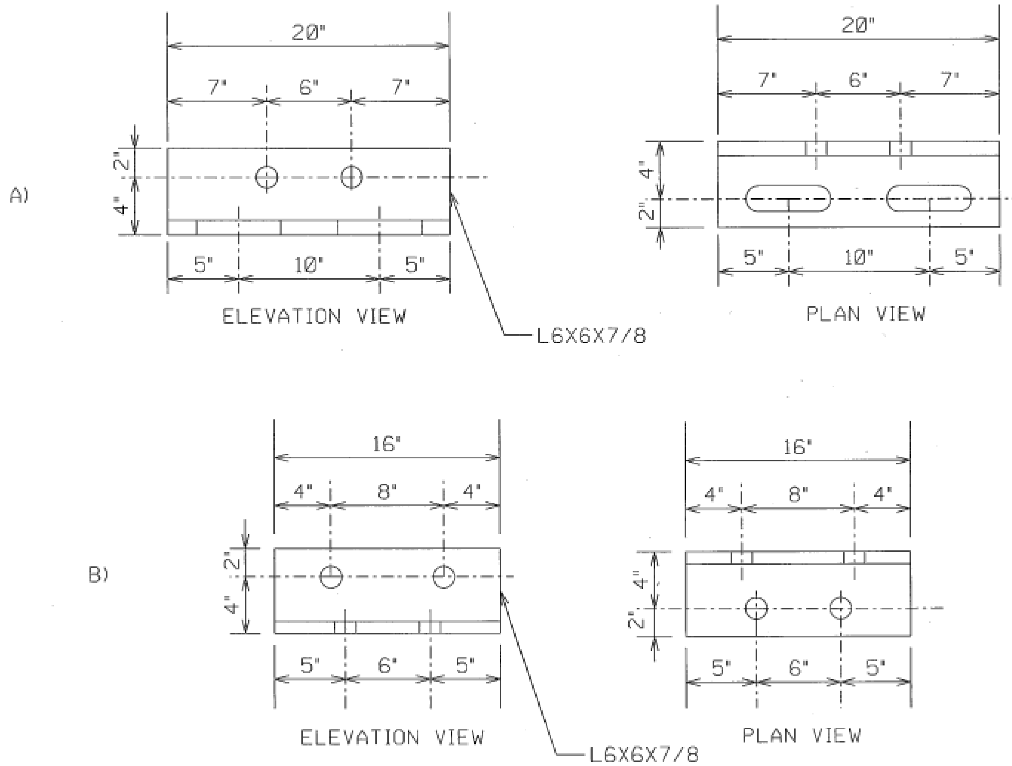


Figure 4.5: Oseligee Creek Bridge Bent 2 & 3 Connections
A) Expansion B) Fixed

4.3.2 Results from LRFD Specification Design

The same bridge model that was created for the Guide Specification was used. The same analysis procedures were used for both specifications. Therefore, the unit deflections were the same. All the analysis and design can be seen in Appendix B starting on page 167. The equivalent seismic loads for both the transverse and longitudinal direction were 0.255 kips per inch. Since the LRFD Specification does not have an amplification factor for structures with short periods, the maximum deflections in the transverse and longitudinal direction were 0.823 in. and 0.426 in., respectively. Since the equivalent seismic loads were the same, the load combination, as described in Equations 3.9 and 3.10, for both cases was 0.266 kip per inch.

The next step in the design process was to input the shear forces and axial forces from the SAP model into the worksheet, and then convert those loads into the design loads. After the loads were calculated, the minimum support lengths were calculated, and they were the same as for the Guide Specification. The minimum and maximum longitudinal reinforcing check in the columns was satisfied with (12)-#11 bars. The interaction diagrams used in the Guide Specification were also used in the LRFD Specification design. All of the controlling load combinations fell within the interaction diagrams. The columns in both Bents 2 and 3 had more than adequate shear strength for the design. The length of the plastic hinge zone was controlled by the column diameter for Bent 2, which is 42 in, and controlled by 1/6 the column height for Bent 3. The spacing within the plastic hinge was controlled by the maximum spacing set by the LRFD Specification or 4 in. The plastic hinge reinforcing was #5 hoops at 4 in. on center. The transverse reinforcement volumetric ratio was satisfied by this spacing. The column extension in the drilled shaft and bent cap beam was 21 in., which was controlled by half the diameter of the column.

The region outside the plastic hinge was designed in the same manner as the Guide Specification, but different loads were used in the LRFD Specification design. Outside the plastic hinge the transverse reinforcing was #5 hoops at 12 in. on center. This is the same spacing as in the Guide Specification; therefore, all the requirements for this region were met.

The connection between the girders and abutments or cap beams underwent some changes in the redesign. The expansion connection at the abutments was an L6x6x3/4x12. The thickness of the angle was governed by the angle bending strength.

The connections at the bents were L6x6x1/2x12. The connections at the abutments and bents required one 1.5-in. diameter anchor bolt and through bolt. The connection details for the abutment and bents are displayed in Figures 4.6 and 4.7.

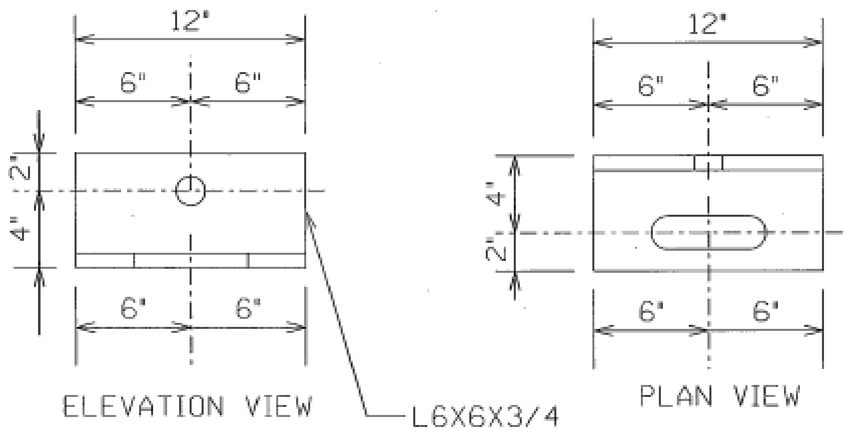


Figure 4.6: Oseligee Creek Bridge Abutment Connection (Expansion)

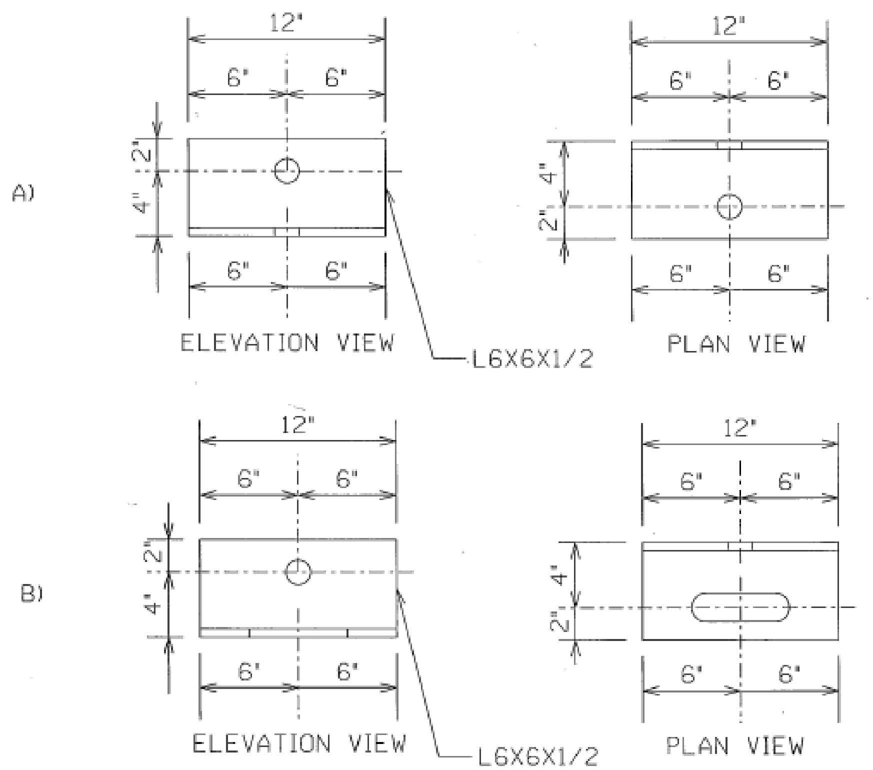


Figure 4.7: Oseligee Creek Bridge Bent 2 & 3 Connections
A) Fixed B) Expansion

4.2.3 Comparison of Standard, Guide, and LRFD Specifications

After all the design calculations were completed, there were a few differences between the codes. There was increase in the amount of hoops in the columns. This increase came from having a plastic hinge zone. The change in hoops can be seen in Tables 4.2 and 4.3. Figures 4.8, 4.9, 4.10 and 4.11 show the changes in column design. The figures compare the Standard Specification with the Guide Specification and the LRFD Specification. As can be seen from the table and figures, the LRFD Specification increased the amount of hoops the most. Even though the length of the plastic hinge zone is longer for the Guide Specification the larger spacing requires fewer hoops than the LRFD Specification.

Table 4.2: Oseligee Creek Bridge Bent 2 Design Changes

Category	**Standard Specification**	**Guide Specification**	**LRFD Specification**
Stirrup Size	#5	#5	#5
Stirrup Spacing Outside Plastic Hinging Region	12 in. o.c.	12 in. o.c.	12 in. o.c.
Stirrup Spacing Inside Plastic Hinging Region	12 in. o.c.	6 in. o.c.	4 in. o.c.
Length of Plastic Hinge Region	0	63 in.	42 in.
Number of Stirrups per Column	34	51	60
% Increase in Stirrups	0	50%	77%

Table 4.3: Oseligee Creek Bridge Bent 3 Design Changes

Category	Standard Specification	Guide Specification	LRFD Specification
Stirrup Size	#5	#5	#5
Stirrup Spacing Outside Plastic Hinging Region	12 in. o.c.	12 in. o.c.	12 in. o.c.
Stirrup Spacing Inside Plastic Hinging Region	12 in. o.c.	6 in. o.c.	4 in. o.c.
Length of Plastic Hinge Region	0	63 in.	52 in.
Number of Stirrups per Column	34	51	62
% Increase in Stirrups	0	50%	82%

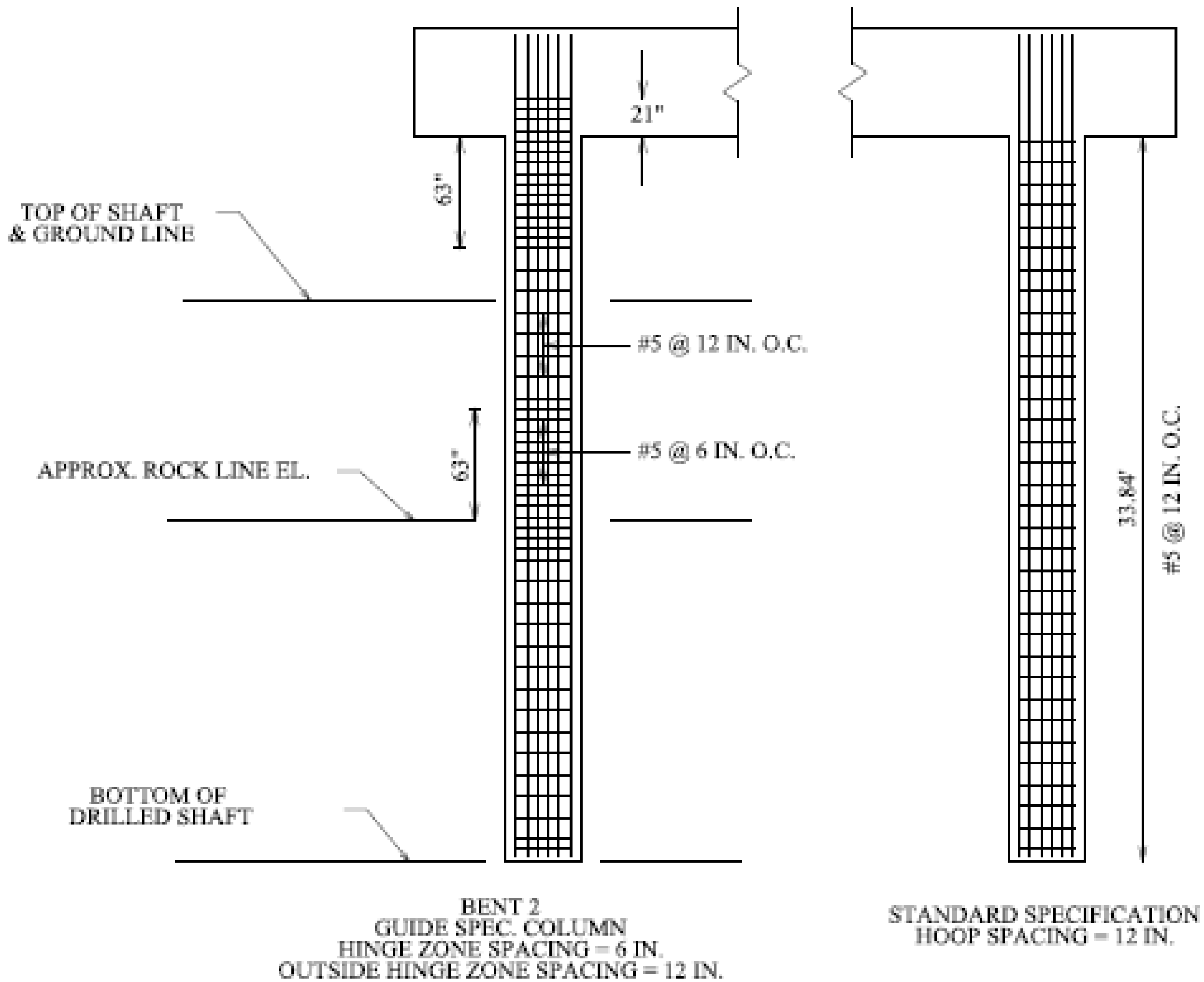


Figure 4.8: Oseligee Creek Bridge Bent 2 Guide Specification vs. Standard Specification

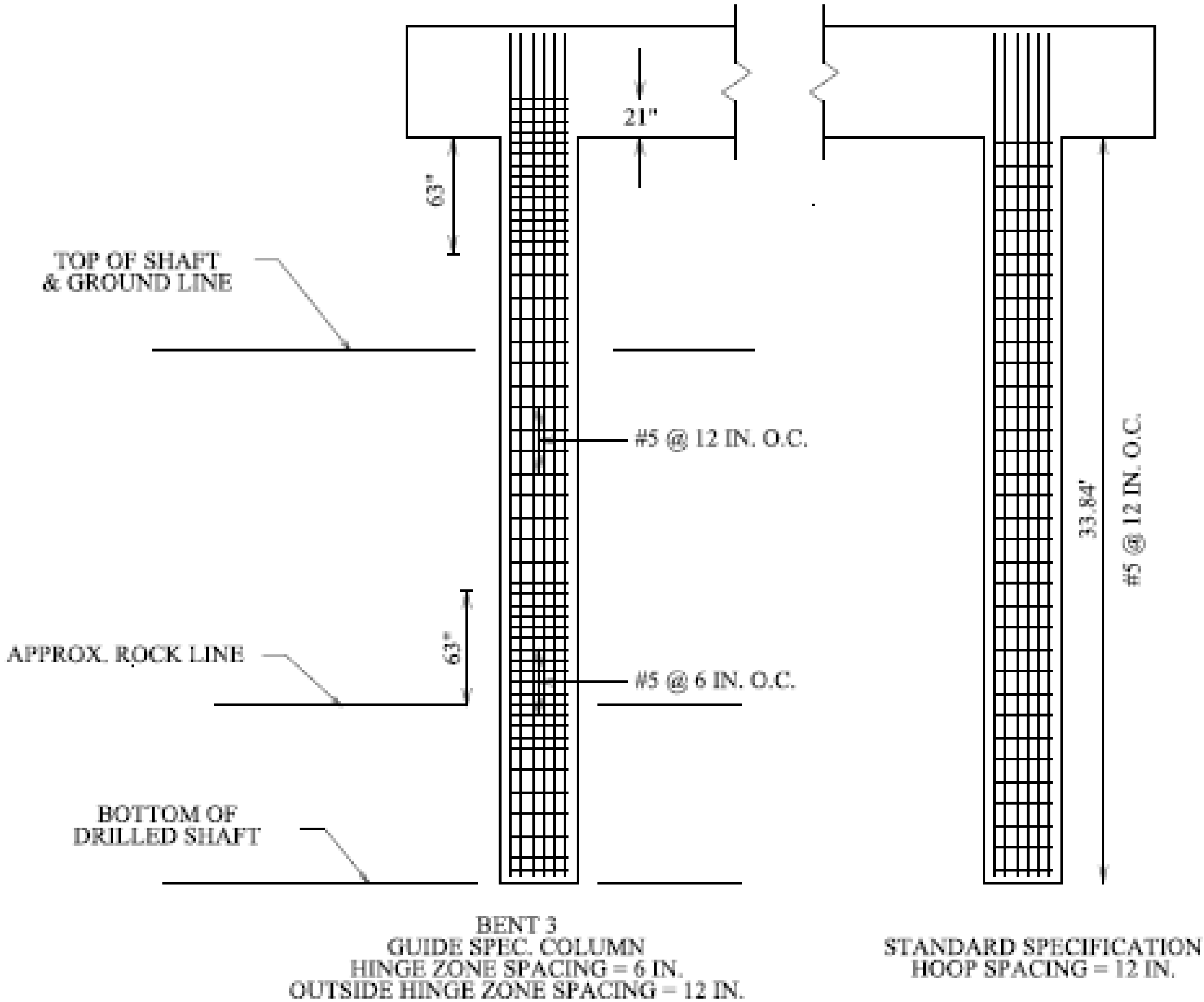


Figure 4.9: Oseligee Creek Bridge Bent 3 Guide Specification vs. Standard Specification

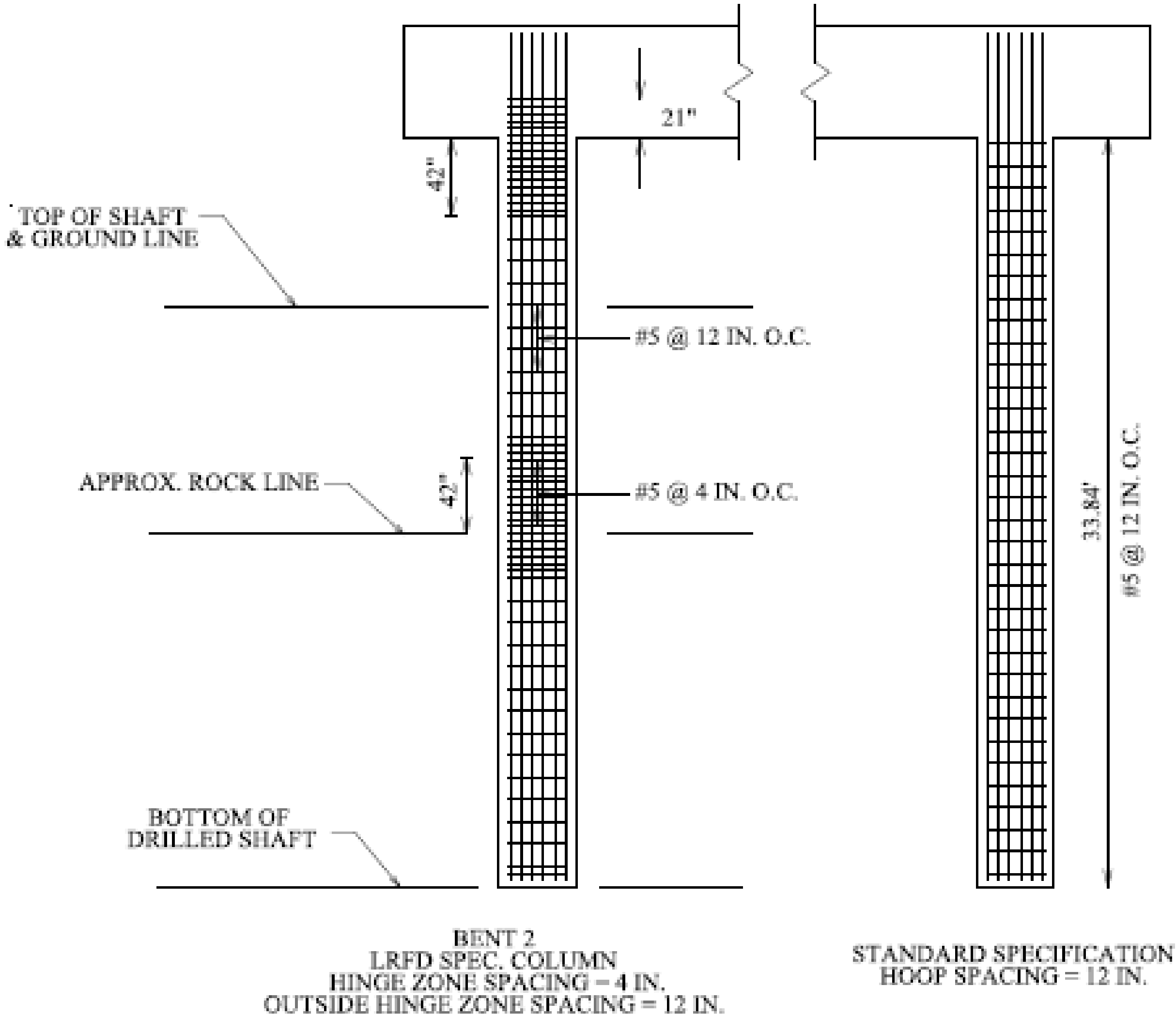


Figure 4.10: Oseligee Creek Bridge Bent 2 LRFD Specification vs. Standard Specification

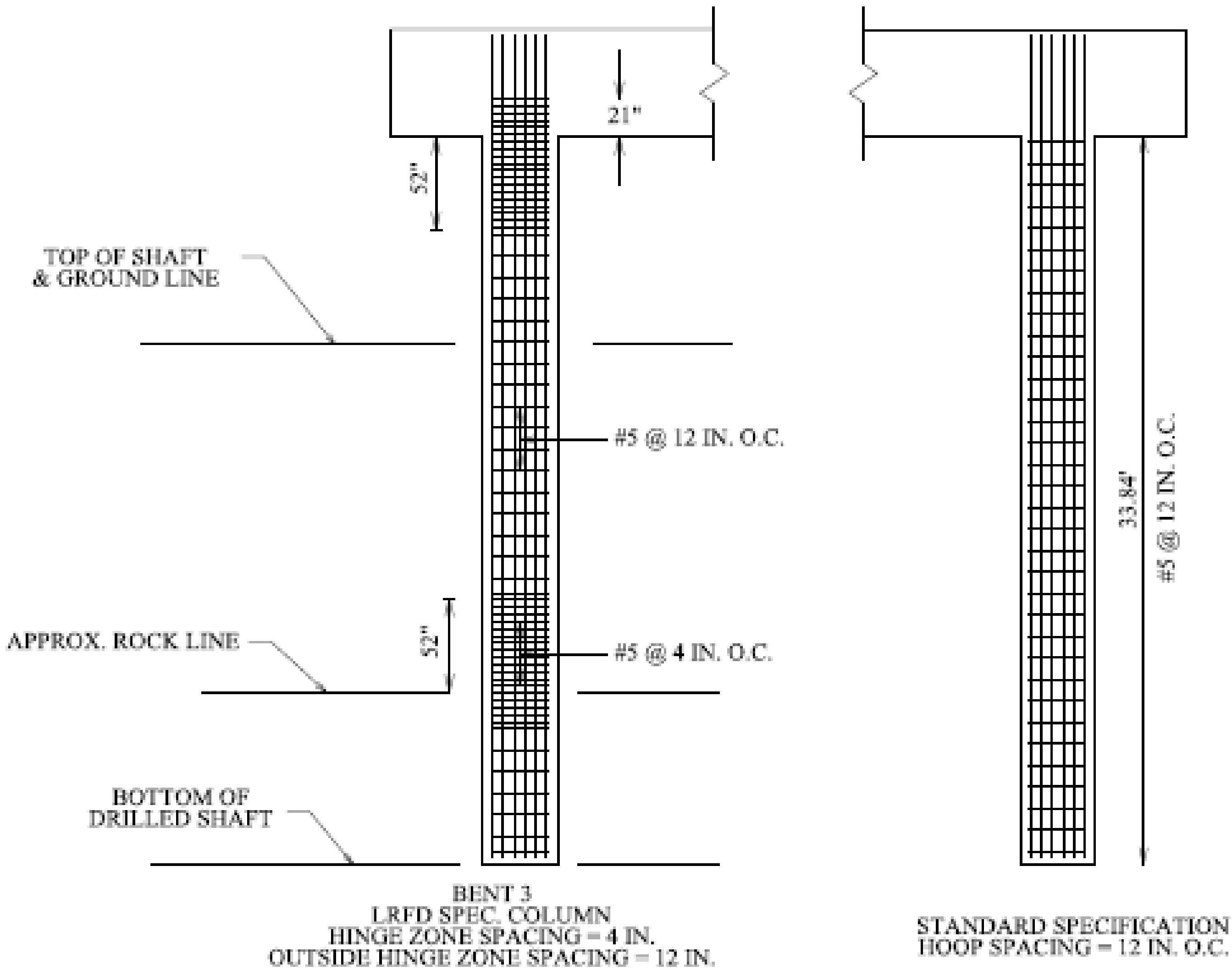


Figure 4.11: Oseligee Creek Bridge Bent 3 LRFD Specification vs. Standard Specification

The connection between the substructure and superstructure experienced some expected changes. Table 4.4 is the best way to see the differences in the design. If two numbers are in one cell, then the smaller number is for the fixed connection and the larger number is for the expansion connection. Instead of having the threaded inserts, a through bolt was used in the design. For the Guide Specification design, the number of anchor bolts was increased at Bents 2 and 3. The Standard Specification and LRFD Specification had the same connections at Bents 2 and 3. The thickness of the angle had to be increased at the connection between the abutments and the superstructure for both the LRFD Specification and Guide Specification. The same size anchor bolt was used for all three specifications.

Table 4.4: Oseligee Creek Bridge Connection Design Changes

Category	Standard Specification	Guide Specification	LRFD Specification
BENT 2 & 3			
Angle Thickness (in.)	0.5	0.875	0.5
Angle Length (in.)	12	16 or 20	12
Bolt Diameter (in.)	1.25	1.25	1.25
Number of bolts/angle	1	2	1
ABUTMENT			
Angle Thickness (in.)	0.5	0.75	0.75
Angle Length (in.)	12	12	12
Bolt Diameter (in.)	1.25	1.25	1.5
Number of bolts/angle	1	1	1

4.3 Little Bear Creek Bridge

4.3.1 Description of the Bridge

Little Bear Creek Bridge is a three span concrete prestressed I-girder bridge with 85-ft long side spans and a 130-ft middle span. The total length and width of the bridge is 300 ft and 42 ft 9 in., respectively. The short span superstructure consists of a 7-in. thick concrete deck supported by six AASHTO Type III girders spaced equally. The short spans have 8 in. thick web walls located at the abutments, bents and mid-span. The long span superstructure consists of a 7-in. thick concrete deck supported by six BT-72 girders equally spaced. The long span has 8-in. thick web walls at the bents and quarter points of the span. There are two bents which consist of a cap beam and two 54-in. diameter circular columns which are supported by 60-in. diameter circular drilled shafts. The bent cap beam allows for the change in size for the girders and has a total depth of 9 ft 4 in., width of 5 ft and length of 40 ft. Currently, the columns and drilled shafts longitudinal reinforcing is (24)-#11 bars. The transverse reinforcing in the columns and drilled shafts are #5 hoops at 12 in. on center and #6 hoops at 12 in. on center, respectively. The columns have 3 in. of cover, and the drilled shafts have 6 in. of cover. Bent 2 has an above ground height of 12 ft, and Bent 3 has an above ground height of 16 ft 8 in. Abutment 1, which is 3 ft x 3.5 ft x 45 ft, is supported by three 42-in. diameter circular drilled shafts. Abutment 2 has the same dimension as Abutment 1, but it is supported by a spread footing. In Figure 4.12, a 3-D model of the bridge is displayed.

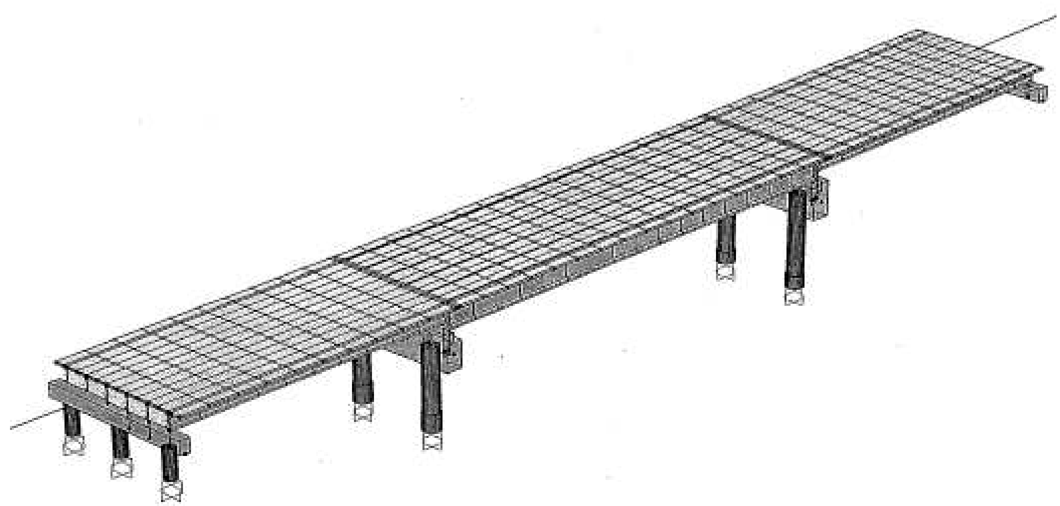


Figure 4.12: 3-D SAP Model of Little Bear Creek Bridge

4.3.2 Results from Guide Specification Design

In order for the analysis of the structure to begin, a model was created in SAP 2000 Bridge Modeler. Since one abutment of the bridge is supported by a spread footing and not drilled shafts, the abutment's contribution to the earthquake resisting system was neglected; therefore, it was only restrained in the vertical direction and allowed to freely move in the longitudinal and transverse directions. When the uniform load was applied to the structure, the maximum deflection was found at the abutment with no drilled shafts. For all the calculations for this bridge refer to Appendix D beginning on page 236. The maximum deflections in the transverse and longitudinal directions from the unit uniform load were 5.263 in. and 0.647 in., respectively. After applying the equivalent seismic load, the maximum deflections from the uniform load method were 0.448 in. longitudinally and 1.486 in. transversely. The SAP model produced a period of 0.546 sec transversely and 0.441 sec in the longitudinal direction. The response spectrum of the bridge in Figure 4.13 shows both periods on the horizontal portion of the graph, with the transverse period close to the edge of the horizontal portion.

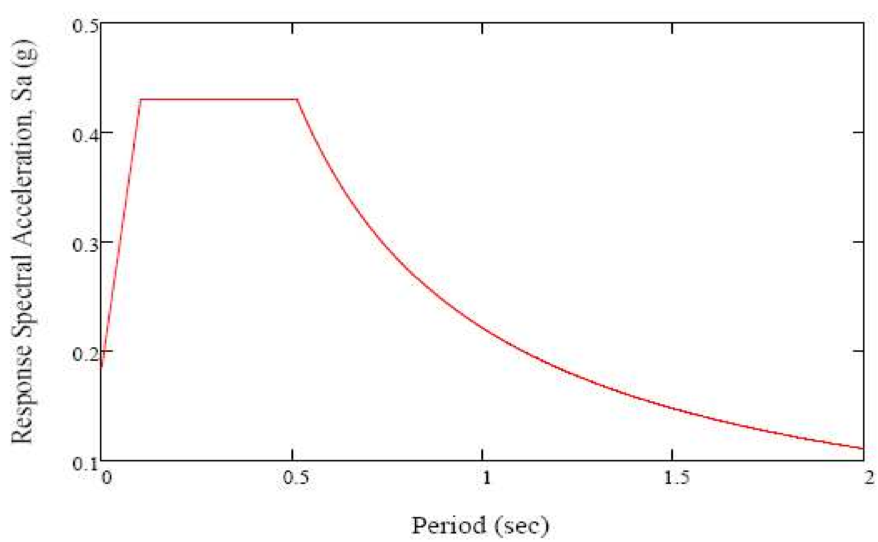


Figure 4.13: Response Spectrum for Little Bear Creek Bridge

The next step was to check the displacement demand and capacity of the columns. The deflections of the bents were taken from the SAP model and used for the displacement demand. Since the columns for both of the bents are short and stiff, the bents failed the requirements of the simplified equations; therefore, a pushover analysis was done on the bridge with the SAP. The results from the analysis can be viewed in Table 4.5, and a sample of a pushover curve is displayed in Figure 4.14. Since the demand displacement is at the point of transition from elastic to plastic, it is practical to design for the elastic forces. The minimum support length ranged from 16.3 in. to 18 in.

Table 4.5: Pushover Analysis Results

Load Case	Demand (in)	Capacity (in)	Check
Bent 2 Transverse Direction	0.69	2.80	OK
Bent 2 Longitudinal Direction	0.35	1.52	OK
Bent 3 Transverse Direction	3.30	13.21	OK
Bent 3 Longitudinal Direction	0.50	2.51	OK

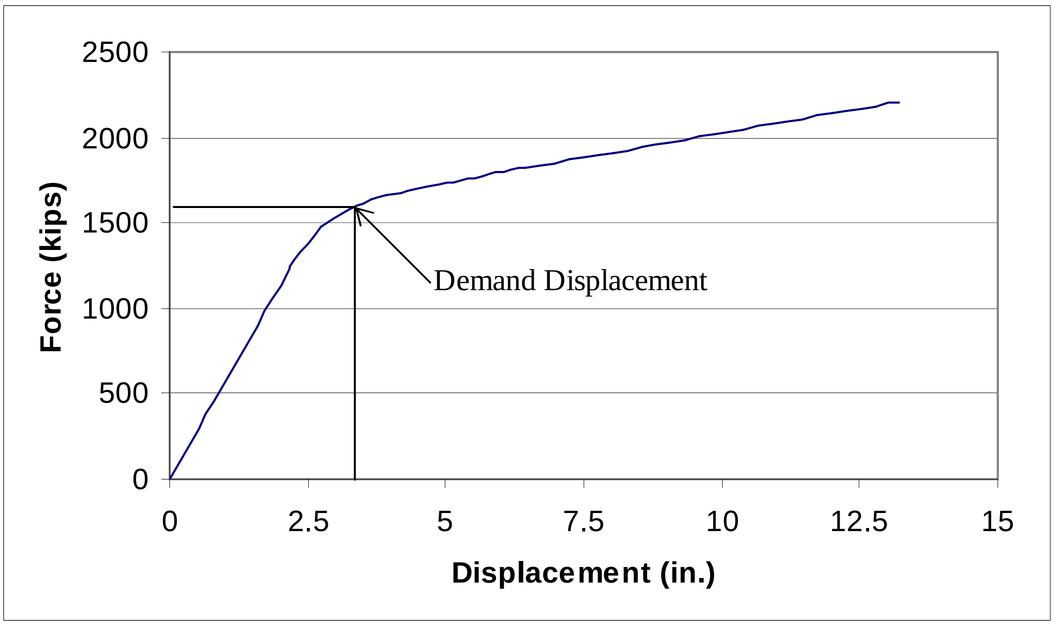


Figure 4.14: Little Bear Creek Bridge Pushover Curve for Load Case Bent 3 Transverse Direction

After the displacement demand was satisfied, the next step in the process was to design the column for shear strength. Interaction diagrams were created for the columns and drilled shafts of each bent and can be viewed in Appendix F starting on page 354. For the columns in Bent 3 the longitudinal reinforcing had to be increased from (24)-#11 bars to (28)-#11 bars. This increase was because of the uplift force from the seismic load. Because the dead load was so small and the column moment was still high; the loading fell outside the interaction diagram. The drilled shaft for Bent 3 had the same problem, and the longitudinal reinforcing had to be increased to (32)-#11 bars. The longitudinal reinforcing in the abutment drilled shafts, the columns of Bent 2, and drilled shafts of Bent 2 satisfied the flexural demands. After determining the moment capacity of the columns, it was determined that it was not practical to design for the column capacity, but to design the bridge for the elastic forces from a linear elastic seismic analysis. The linear elastic forces were determined from the moment and shear diagrams provided by the SAP model. The shear force from the SAP model is for the unit loading; therefore, it must be converted into a design load by multiplying the unit load by the equivalent seismic load.

The plastic hinge length for the both Bents 2 and 3 was calculated. Both plastic hinge regions were controlled by 1.5 times the column diameter or 81 in. The programs created in the worksheet were used to calculate the shear strength of the concrete and the transverse reinforcing. The column sections proved to have adequate shear strength. The maximum and minimum longitudinal reinforcing requirements were satisfied by both bents. The transverse reinforcing in the plastic hinge region for both bents were #5 hoops at 6 in. on center. The spacing was controlled by the maximum established by the Guide

Specification. The extension of lateral reinforcing into the drilled shaft and bent cap beam is 27 in., which was controlled by ½ the column diameter.

Next, the spacing of the transverse reinforcement outside the plastic hinge region was designed. The spacing was originally 12 in. on center, but it was changed to 10.5 in. on center because the column did not meet the minimum transverse reinforcing requirements. The column shear strength in this region was sufficient to resist the column shear force.

The drilled shaft design was done in the same way as the region outside the plastic hinge. The transverse reinforcement in the drilled shafts for the bents is #6 hoops at 12 in. on center. The shear strength of the concrete and hoops in the drilled shafts for both bents was greater than the shear demand. For Bents 2 and 3, the minimum and maximum transverse reinforcement requirements were met. The abutment drilled shaft shear strength was sufficient to resist the applied shear force. The transverse reinforcing in the abutment drilled shaft is #5 hoops at 12 in. o.c. The maximum and minimum transverse reinforcing requirements were also satisfied.

The connection between the substructure and superstructure saw the effects of the increased shear forces. An L6x6x1x12 was used for the connection at the bents, but the location of the bolt holes in the angle was modified. One 1.5-inch diameter through bolt and anchor bolt were used in the connection at the bents. The connection at the abutments is an L6x6x1x20, which is connected to the structure with two 1.5-in. diameter anchor bolts and through bolts. The details for the connections are displayed in Figures 4.15, 4.16, and 4.17. The expansion connections are at the abutments and the side span for Bent 2. The rest of the connections are fixed. The through bolt hole in the angle is located

4 in. from the bottom of the angle for the Type III girders and 3.5 in. from the bottom of the angle for the BT-72 girders. The diameter of the bolt was controlled by the bolt shear strength. The thickness of the angle was controlled by the angle bending strength.

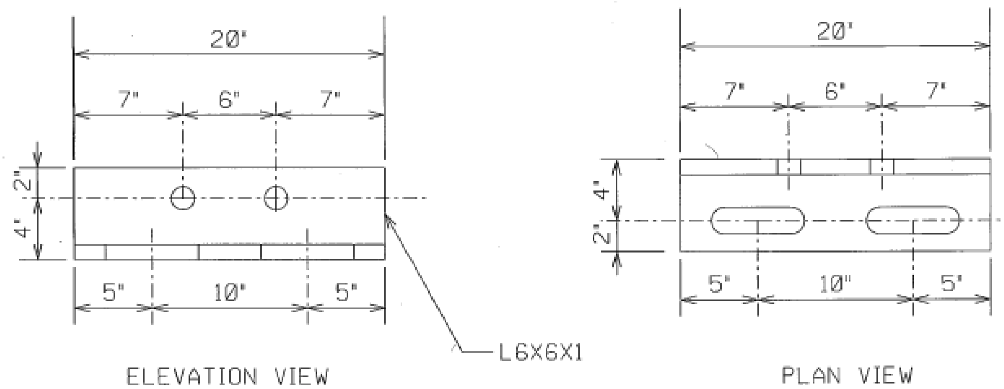


Figure 4.15: Little Bear Creek Abutment Connection (Expansion)

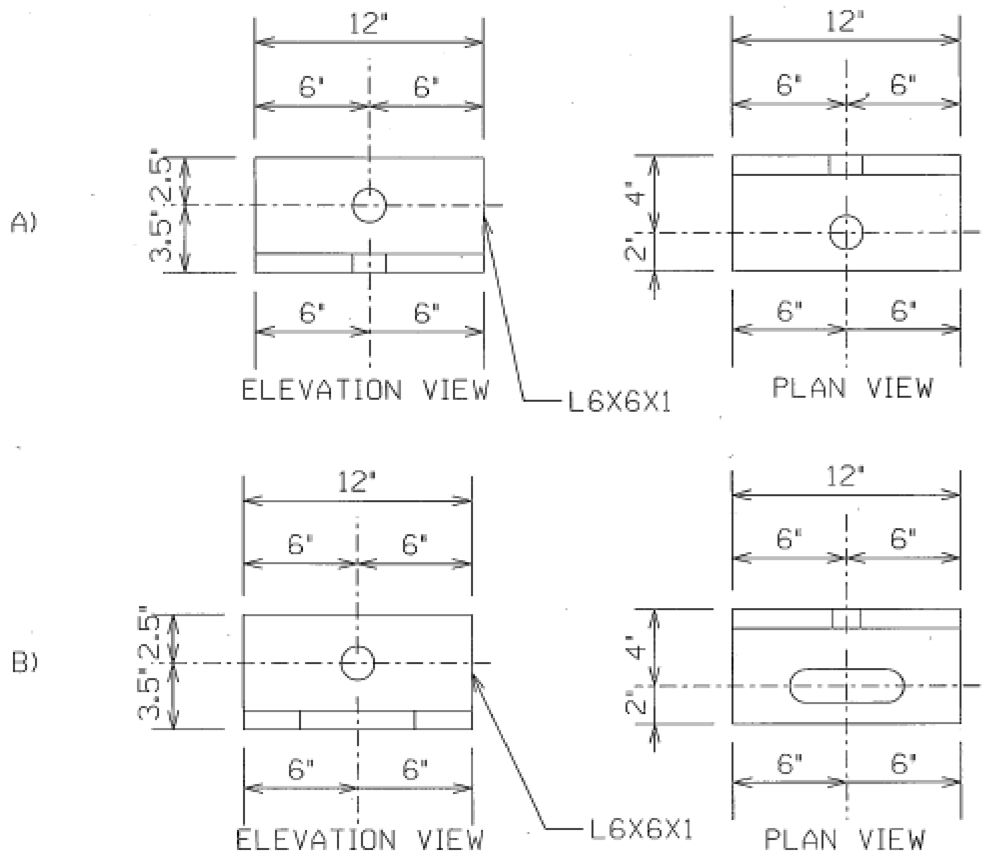


Figure 4.16: Little Bear Creek Bent 2 & 3 Connections for Bulb Tee Girders
A) Fixed B) Expansion

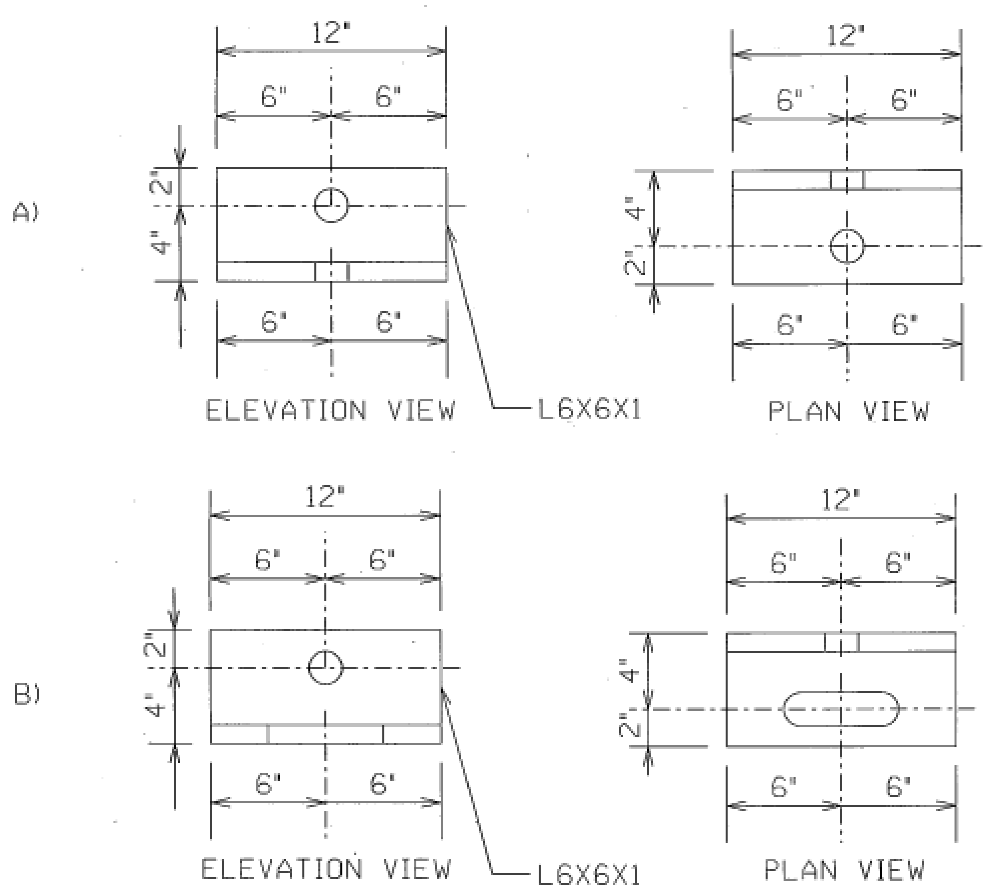


Figure 4.17: Little Bear Creek Bent 2 & 3 Connections for Type III Girders
A) Fixed B) Expansion

4.3.2 Results from LRFD Specification Design

The bridge model created for the Guide Specification was also used for the LRFD Specification design. For the analysis and design calculations, refer to Appendix E starting on page 295. The equivalent seismic loads were 0.378 kips per inch in the longitudinal direction and 0.282 kips per inch in the transverse direction. The design maximum deflections in the longitudinal and transverse direction are 0.244 in. and 1.485 in., respectively. The largest factored load for all load combinations that include the equivalent seismic forces were 0.387 kips per inch. This load was used in forming the R-equivalent values for this bridge. For the LRFD Specification, the loads in the columns and drilled shafts were input into the worksheet, and there they were converted into the design loads on the structure.

After the loads had been calculated, the column design was adjusted from its previous design. The minimum and maximum longitudinal reinforcing was satisfied for both bents. The increase in the longitudinal reinforcing in the columns and drilled shafts in Bent 3 were still needed in the LRFD Specification design. The length of the plastic hinge region for Bents 2 and 3 was 54 in., which is controlled by the column diameter. The design of hoops in this region was #5 hoops at 3 in. on center. The maximum spacing allowed by the specification is 4 in. on center; however, the required volumetric ratio required 3 in. on center. The extension of the reinforcing in the plastic hinge region is the same as the Guide Specification, which is 27 in.

The design process for the transverse reinforcing outside the plastic hinge region was the same as in the Guide Specification. Even though the design loads were different, the same transverse reinforcing design was a product of the design calculations for the LRFD Specification.

The connection between the substructure and superstructure was a component of the bridge that was influenced by the increase in seismic loads. The connection between the bents was an L6x6x1/2x12 with one 1.25-inch diameter anchor bolt and through bolt. The connection details are shown in Figures 4.18, 4.19 and 4.20. The connection at the abutments is larger than at the bents. An L6x6x1x20 was used for the abutment connections. Two 1.5-inch diameter anchor bolts and through bolts were used to connect the angle to the structure. The diameter of the bolts was controlled by the combination of shear and tension check. The thickness of the angle was governed by the angle bending strength.

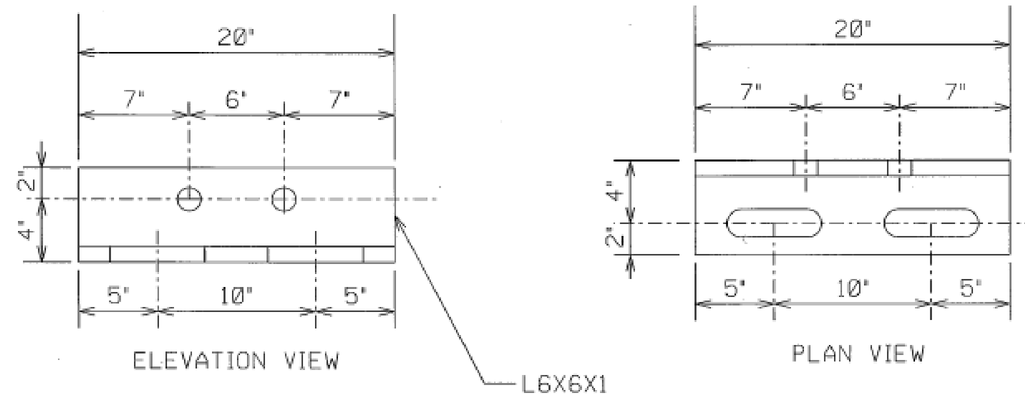


Figure 4.18: Little Bear Creek Abutment Connection for Type III Girders

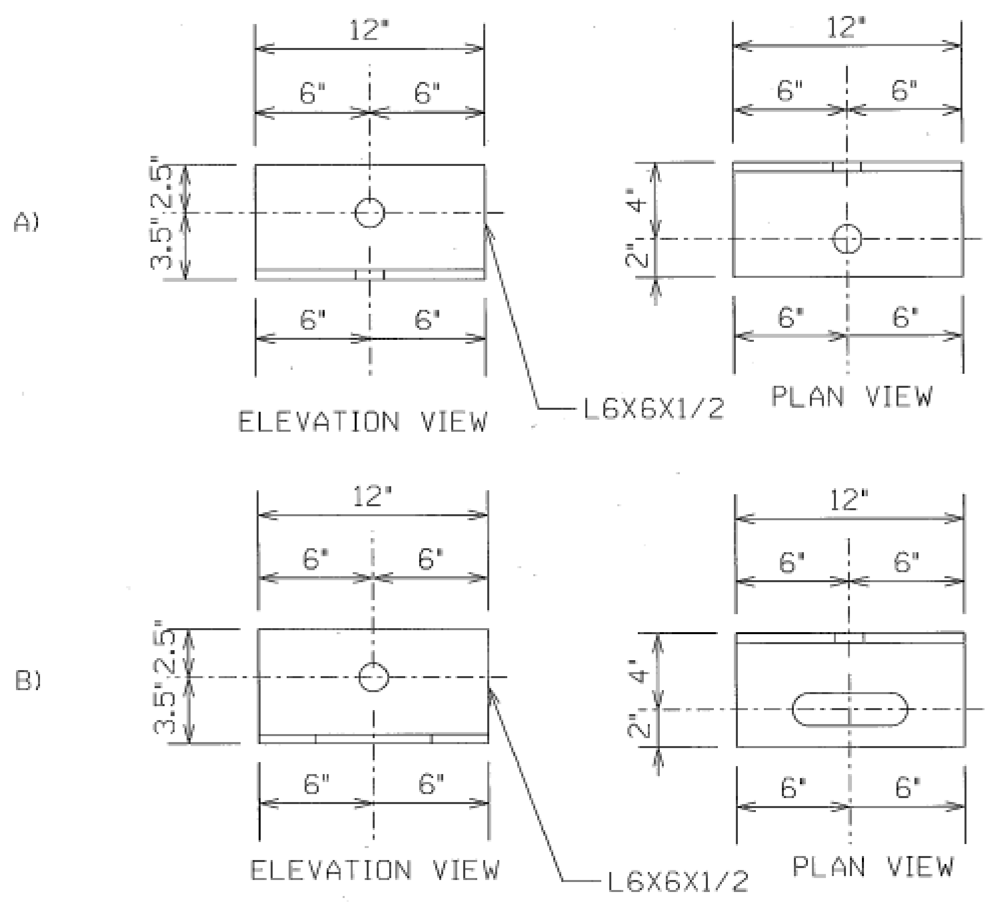


Figure 4.19: Little Bear Creek Bent 2 & 3 Connections for Bulb Tee Girders
A) Fixed B) Expansion

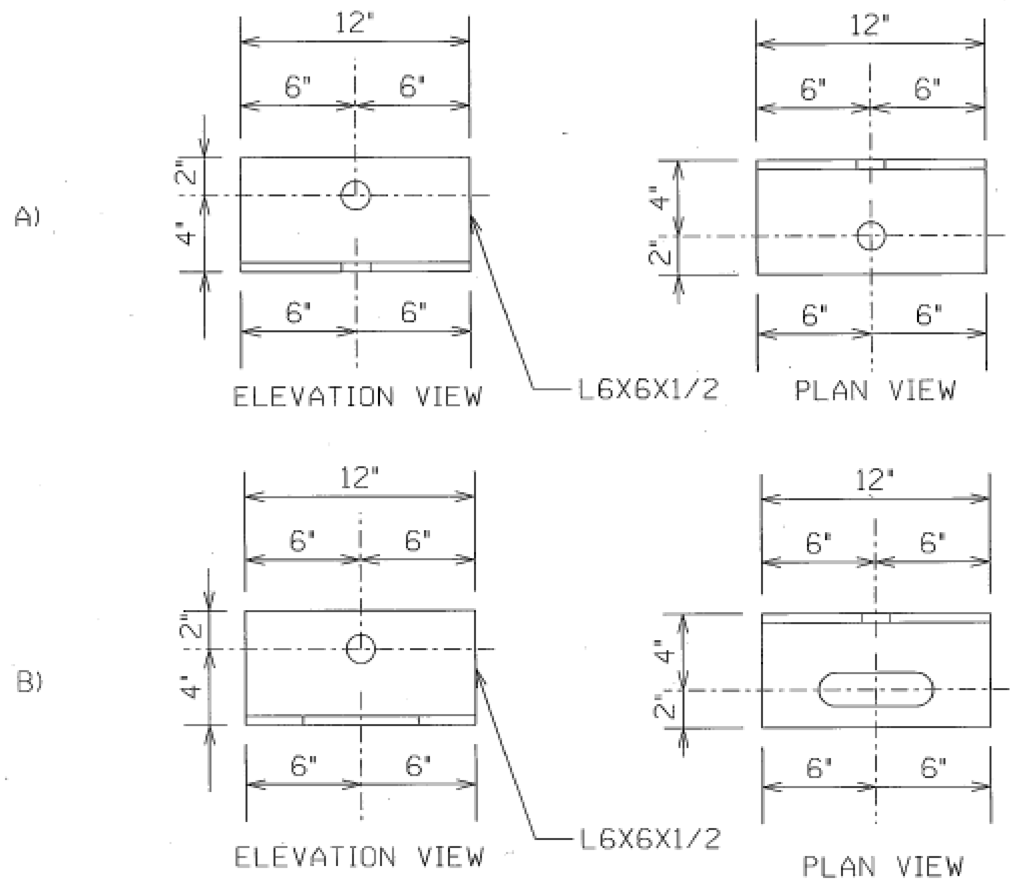


Figure 4.20: Little Bear Creek Bent 2 & 3 Connections for Type III Girders
A) Fixed B) Expansion

4.3.3 Comparison of Standard, Guide, and LRFD Specifications

When all the design calculations were done, several differences between the three specifications were apparent. There was an increase in hoops in the columns and drilled shafts from the Standard Specification. The differences in the number and spacing of the hoops for the columns and drilled shafts can be seen in Tables 4.6, 4.7, 4.8 and 4.9. Elevation views of the bents are shown in Figures 4.21, 4.22, 4.23 and 4.24. These figures visually show the increase in hoops for the columns and drilled shafts. As can be seen in the tables, the LRFD Specification required a greater increase in the number of hoops. The stirrups within the plastic hinge regions must be detailed as seismic hoops, which is a change from the Standard Specification. The increase of transverse reinforcing in the drilled shafts can be attributed to the extension of the plastic hinge zone. Outside the plastic hinge region, the spacing of the hoops had to be changed to 10.5 in. for the Guide Specification and the LRFD Specification because the Standard Specification spacing of 12 in. was not meeting the minimum amount of transverse reinforcing.

Table 4.6: Little Bear Creek Bridge Column Design Changes for Bent 2

Category	Standard Specification	Guide Specification	LRFD Specification
Stirrup Size	#5	#5	#5
Stirrup Spacing Outside Plastic Hinging Region	12 in. o.c.	10.5 in. o.c.	10.5 in. o.c.
Stirrup Spacing Inside Plastic Hinging Region	12 in. o.c.	6 in. o.c.	3 in. o.c.
Length of Plastic Hinge Region	0	81 in.	54 in.
Number of Stirrups per Column	12	29	49
% Increase in Stirrups	0	142%	308%

Table 4.7: Little Bear Creek Bridge Column Design Changes for Bent 3

Category	Standard Specification	Guide Specification	LRFD Specification
Stirrup Size	#5	#5	#5
Stirrup Spacing Outside Plastic Hinging Region	12 in. o.c.	10.5 in. o.c.	10.5 in. o.c.
Stirrup Spacing Inside Plastic Hinging Region	12 in. o.c.	6 in. o.c.	3 in. o.c.
Length of Plastic Hinge Region	0	81 in.	54 in.
Number of Stirrups per Column	17	36	56
% Increase in Stirrups	0	112%	230%

Table 4.8: Little Bear Creek Bridge Drilled Shaft Design Changes for Bent 2

Category	Standard Specification	Guide Specification	LRFD Specification
Stirrup Size	#6	#6	#6
Stirrup Spacing Outside Zone	12 in. o.c.	12 in. o.c.	12 in. o.c.
Stirrup Spacing in Extension Zone	12 in. o.c.	6 in. o.c.	3 in. o.c.
Length Extension Zone	0	27 in.	27 in.
Number of Stirrups per Drilled Shaft	13	16	20
% Increase in Stirrups	0	23%	54%

Table 4.9: Little Bear Creek Bridge Drilled Shaft Design Changes for Bent 3

Category	Standard Specification	Guide Specification	LRFD Specification
Stirrup Size	#6	#6	#6
Stirrup Spacing Outside Zone	12 in. o.c.	12 in. o.c.	12 in. o.c.
Stirrup Spacing in Extension Zone	12 in. o.c.	6 in. o.c.	3 in. o.c.
Length Extension Zone	0	27 in.	27 in.
Number of Stirrups per Drilled Shaft	12	15	20
% Increase in Stirrups	0	25%	67%

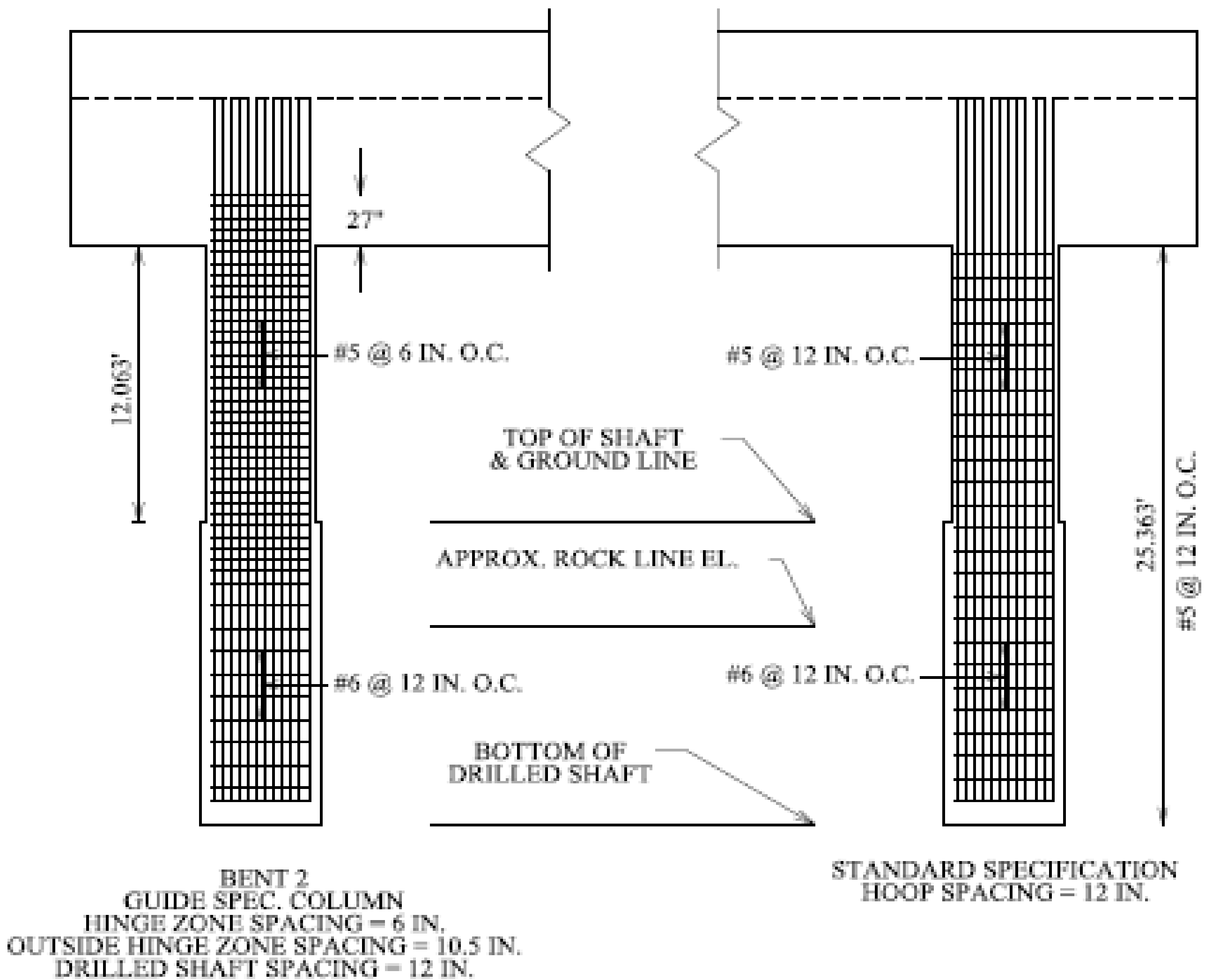


Figure 4.21: Little Bear Creek Bridge Bent 2 Guide Specification vs. Standard Specification

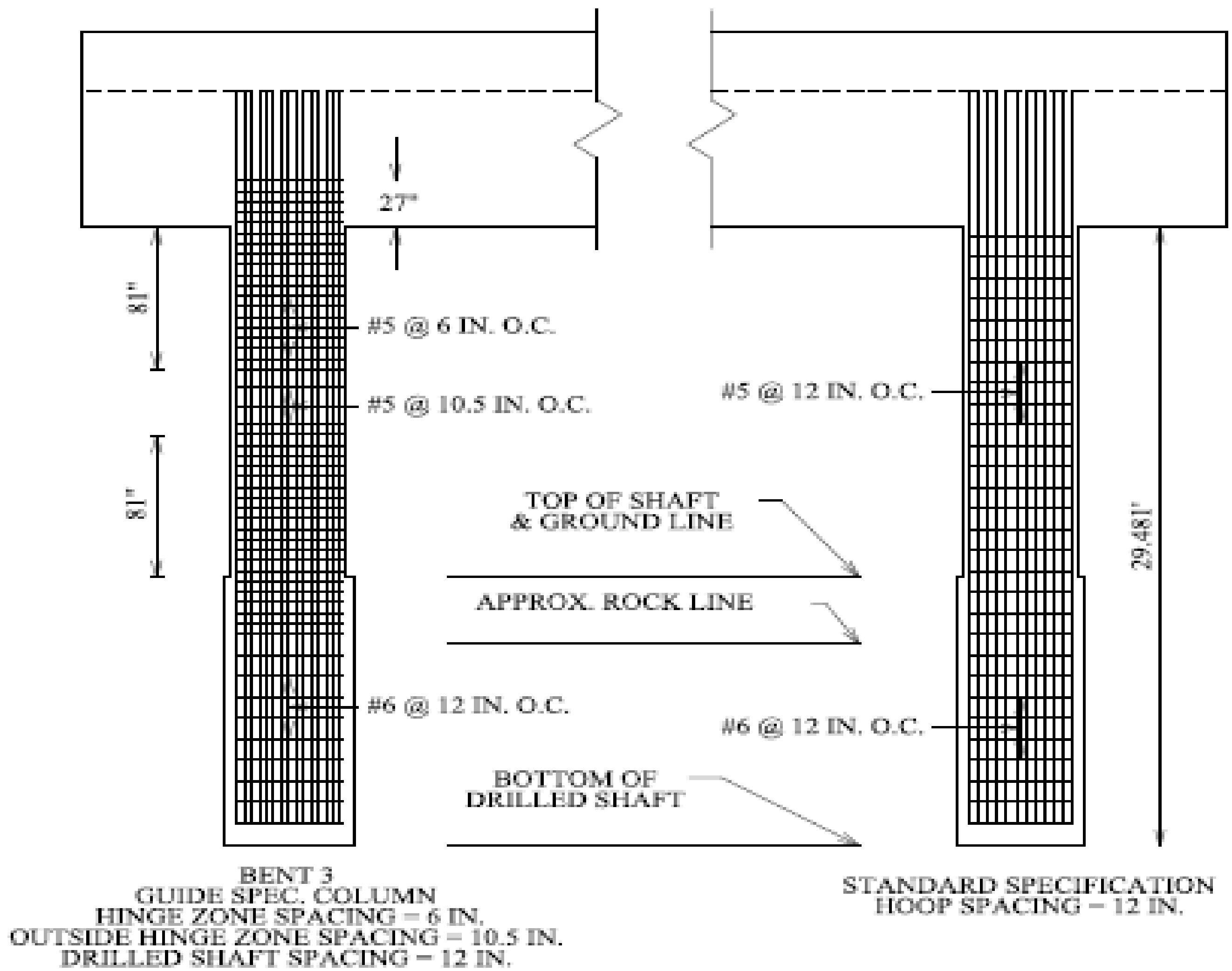


Figure 4.22: Little Bear Creek Bridge Bent 3 Guide Specification vs. Standard Specification

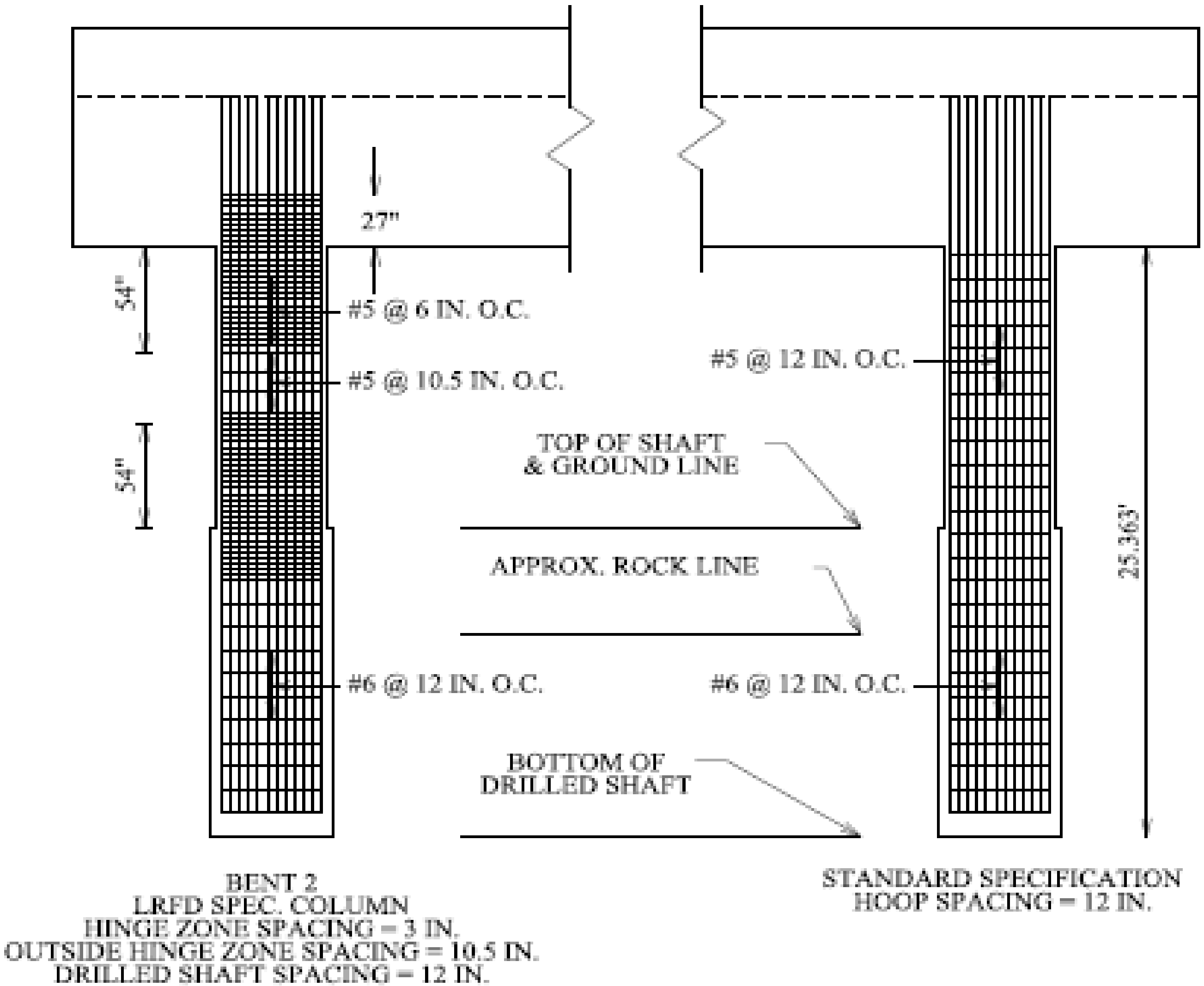


Figure 4.23: Little Bear Creek Bridge Bent 2 LRFD Specification vs. Standard Specification

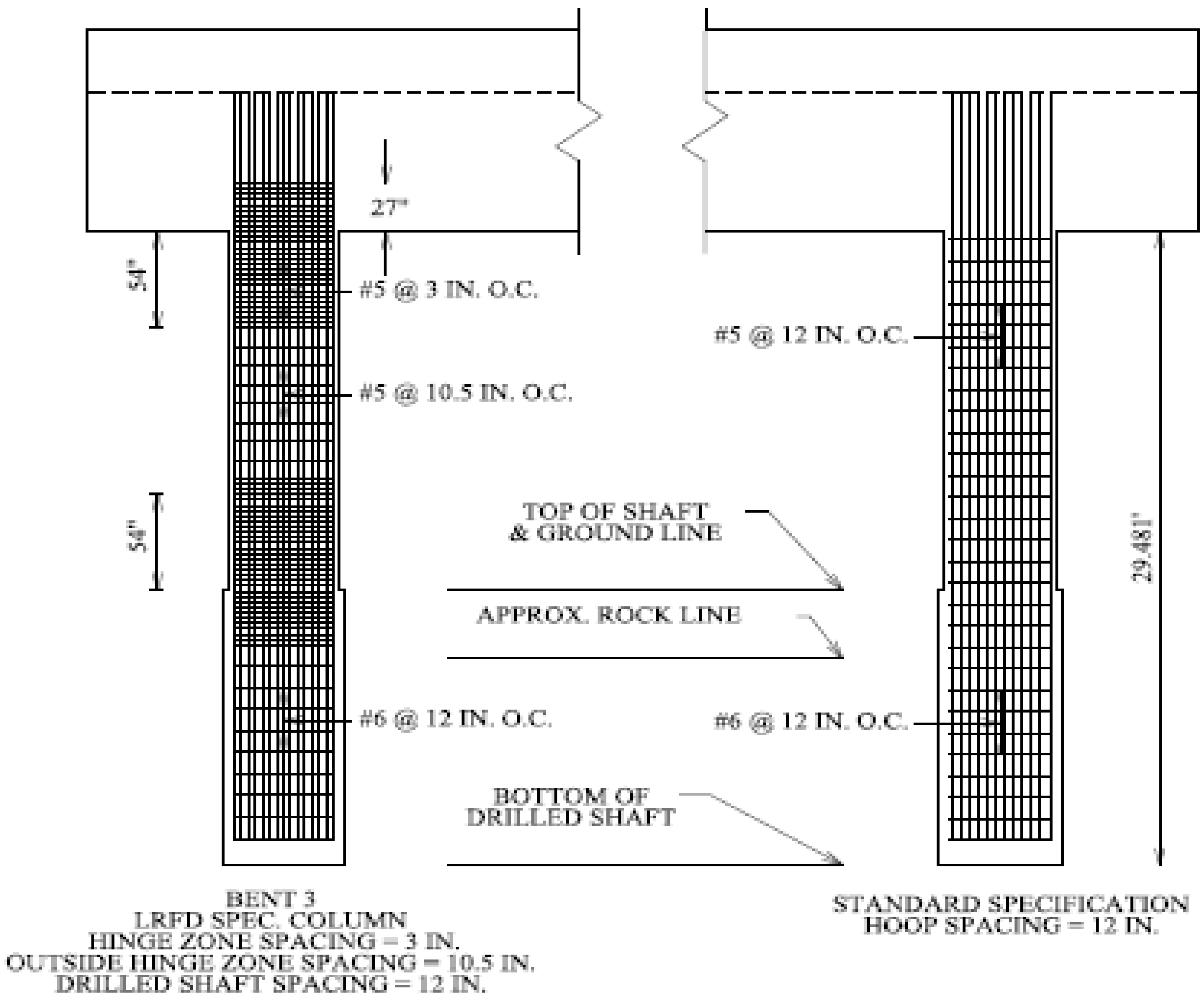


Figure 4.24: Little Bear Creek Bridge Bent 3 LRFD Specification vs. Standard Specification

There were also differences with connections between the superstructure and substructure. The differences are listed in Table 4.10. According to the standard details ALDOT is currently using, the AASHTO Type III Girder is connected with an L6x6x1/2x12 clip angle with a 1.25-in. diameter anchor bolt. The Bulb Tee Type Girder is connected with an L6x6x1/2x12 clip angle with a 1.5-in. diameter anchor bolt. For both the Guide and LRFD Specification, the same connection for the AASHTO Type III Girder and the BT-72 Type Girder can be used other than the location of the bolt holes in the vertical leg of the angle. For the Guide Specification, the angle thickness was increased and the diameters of the anchor and through bolt were increased. This connection was used for both the bents and abutments. The same connection as the Standard Specification AASHTO Type III Girder was used for the bents in the LRFD Specification design; however for the abutments, the angle thickness had to be increased, the length was increased, and the bolt diameters were increased.

Table 4.10: Little Bear Creek Bridge Connection Design Changes

Category	Standard Specification (Type III)	Standard Specification (BT-72)	Guide Specification	LRFD Specification
BENT 2 & 3				
Angle Thickness (in.)	0.5	0.5	1	0.5
Angle Length (in.)	12	12	12	12
Bolt Diameter (in.)	1.25	1.5	1.5	1.25
Number of bolts/angle	1	1	1	1
ABUTMENT				
Angle Thickness (in.)	0.5	----	1	1
Angle Length (in.)	12	----	20	20
Bolt Diameter (in.)	1.25	----	1.5	1.5
Number of bolts/angle	1	----	2	2

4.4 Scarham Creek Bridge

4.4.1 Description of the Bridge

The Scarham Creek Bridge is a four equal span concrete prestressed I-girder bridge. The span length is 130 ft, and the total width of the bridge is 42 ft 9 in. The girders for all four spans are BT-72 Girders. The superstructure is a 7-inch thick concrete slab supported by six girders equally spaced along the width. Eight-inch thick web walls are located at the abutments, bents and the quarter points of the bridge. The bridge consists of three frame bents containing two circular columns supported by circular drilled shafts and a horizontal strut. The three bents have significantly different heights. The above ground heights for the columns in Bent 2 are 25 ft and 34 ft. For Bent 3, the above ground column heights are 59 ft and 55 ft. The above ground heights of the columns for Bent 4 are 32 ft and 25 ft. The cap beam for all three bents is 5.5 ft x 7.5 ft x 40 ft. For Bents 2 and 4, the columns are 60 in. in diameter which are supported by 66-in. diameter drilled shafts. The columns and drilled shafts for Bents 2 and 4 are reinforced longitudinally with (24)-#11 bars and transversely with #6 hoops at 12 in. on center. For Bent 3, the column diameter is 72 in., and the drilled shaft diameter is 78 in. The columns and drilled shafts for Bent 3 are reinforced longitudinally with (32)-#11 bars and transversely with #6 hoops at 6 in. on center. The columns and drilled shafts have 3 in. and 6 in. of cover, respectively. The dimensions of the struts for Bents 2 and 4 are 3.5 ft x 6 ft x 19 ft, and the top of the strut is located 12 ft below the bottom of the cap beam. The strut for Bents 2 and 4 are reinforced longitudinally with (8)-#11 bars both top and bottom and (20)-#5 bars spaced at 6 in. on center along the sides and reinforced transversely with #5 hoops at 12 in. on center. The dimensions of the strut for Bent 3 are 3.5 ft x 10 ft x 18 ft., and the top the strut is located 25 ft below the bottom of the cap beam. For Bent 3, the strut is reinforced longitudinally with (8)-#11 bars both top and bottom and (36)-#5 bars spaced at 6 in. on center along the sides and reinforced transversely with #5 hoops at 12 in. on center. Both abutment beams are supported by three 42-in. diameter drilled shafts. The abutment beam dimensions are 2 ft 11 in. x 4 ft x 55 ft. The drilled shafts supporting the abutments are reinforced in the longitudinal direction with (16)-#11 bars and reinforced in the transverse direction with #5 hoops at 12 in. on center. Figure 4.25 shows a 3-D model of Scarham Creek Bridge.

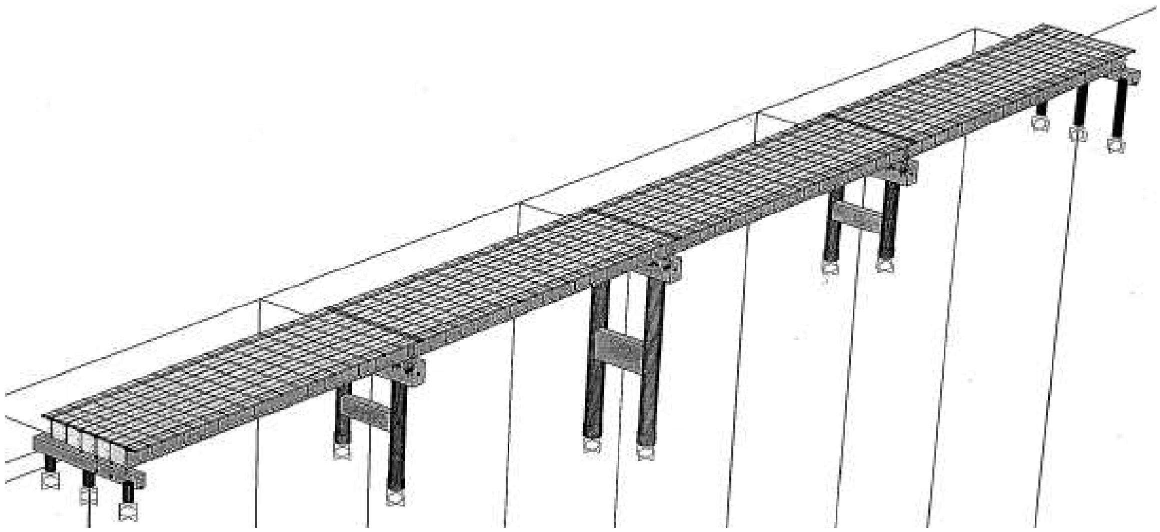


Figure 4.25: 3-D SAP Model of Scarham Creek Bridge

4.4.2 Results from Guide Specification

After all the initial design steps had been taken, a bridge model was created in SAP. All of the calculations and inputs for the bridge design can be seen in Appendix G beginning on page 374. A unit uniform load was applied to the structure which produced a maximum deflection longitudinally of 0.382 in. and transversely of 4.330 in. The equivalent seismic loads for the longitudinal and transverse directions are 0.502 kips per inch and 0.359 kips per inch, respectively. After multiplying the unit maximum deflections by p_e/p_o, the design maximum deflections were 0.384 in. in the longitudinal direction and 1.553 in. in the transverse direction. From the SAP model, the transverse and longitudinal periods were 0.583 sec and 0.448 sec, respectively. From Figure 4.26, it is seen the longitudinal period falls within the horizontal region of the response spectrum, but the transverse period falls just outside the horizontal region.

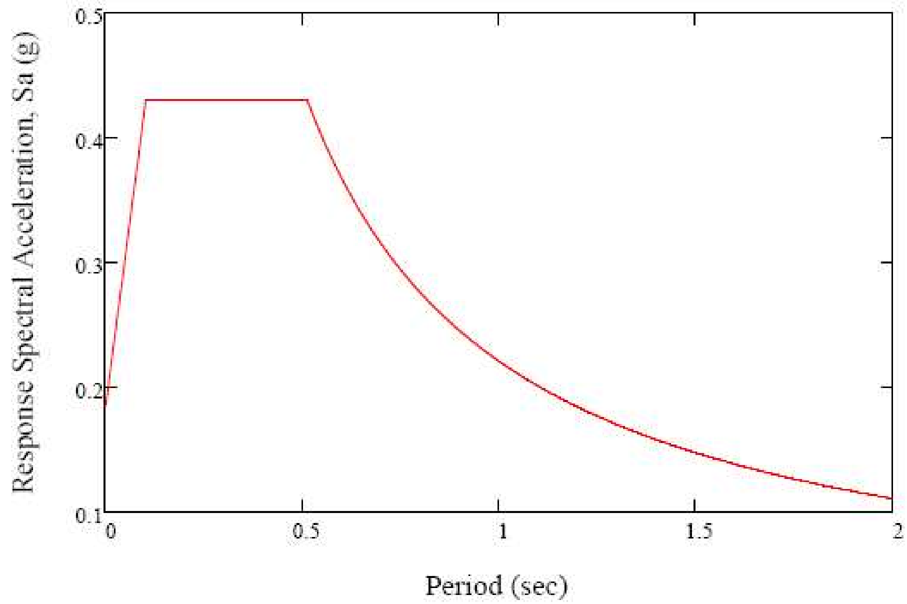


Figure 4.26: Response Spectrum for Scarham Creek Bridge

Since the bridge consists of frame bents, the simplified equations for displacement capacity do not apply to this type of bridge; therefore, it was necessary to do a pushover analysis. The struts were allowed to hinge at the plastic hinge lengths, which were calculated later in the design process. This relieves some of the flexibility demand of the columns and drilled shafts. Once again, SAP was used for pushover analysis, and the pushover analysis results can be seen in Table 4.11. Figure 4.27 shows a one of the pushover curves created by SAP. The pushover curve shows that the demand is well within the elastic range. As can be seen from the table, the bridge had plenty of ductility and satisfied the displacement demand.

Table 4.11: Pushover Analysis Results for Scarham Creek

Load Case	Demand (in.)	Capacity (in.)	Check
Bent 2 Transverse Direction	2.44	9.77	OK
Bent 2 Longitudinal Direction	0.55	2.20	OK
Bent 3 Transverse Direction	6.90	25.64	OK
Bent 3 Longitudinal Direction	0.87	3.57	OK
Bent 4 Transverse Direction	2.87	11.47	OK
Bent 4 Longitudinal Direction	0.62	2.64	OK

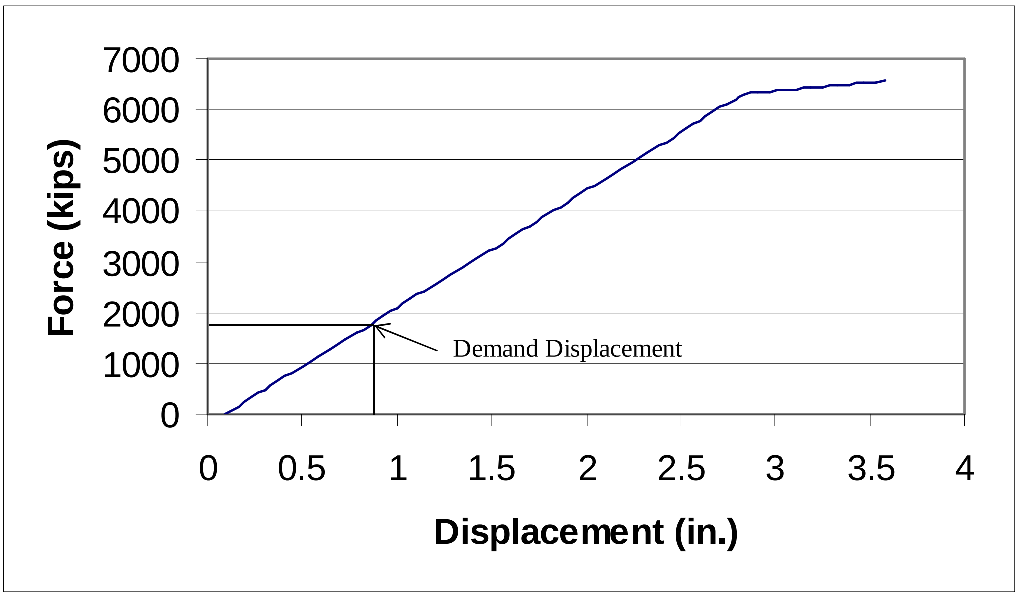


Figure 4.27: Scarham Creek Bridge Pushover Curve for Load Case Bent 3 Longitudinal Direction

When the analysis was complete, the columns and drilled shafts of the bents were designed. The minimum support lengths ranged from 20 in. to 23 in. It was decided that it was practical to design this bridge for the overstrength moment capacity loads instead of the linear elastic loads because the difference between the two loadings was insignificant. From the interaction diagrams, which can be viewed in Appendix I starting on page 579, the moment capacity of the columns was calculated. Then, the shear forces were calculated from the moment capacities.

After doing the design calculations, Bents 2 and 4 resulted in the same design; therefore, they will be discussed together. The plastic hinge regions were 90 in., which was controlled by 1.5 times the column diameter. The shear strength of the column was greater than the shear demand. The original longitudinal column reinforcing remained the same. Both the minimum and maximum reinforcement requirements were satisfied. The transverse reinforcing in the plastic hinge region is #6 hoops at 6 in. on center. This value was controlled by the maximum allowed spacing. The minimum transverse reinforcement requirement was satisfied by this spacing. The extension of the hoops into the cap beam and drilled shaft was 30 in., which was controlled by ½ of the column diameter.

The regions outside the plastic hinge zone for Bents 2 and 4 were designed according to LRFD Specification. The combined concrete and reinforcement strength was greater than the applied shear. The transverse reinforcing for this region is #6 hoops at 12 in. on center. The maximum spacing could have been larger for seismic design; however, it was assumed the original spacing of 12 in. was required for the strength design. The minimum transverse reinforcement requirement was also met by this configuration.

Bent 3 contains larger columns and drilled shafts and is taller than the other two, so the design was different. The length of the plastic hinge was controlled by 1.5 times the column diameter or 108 in. The column's shear capacity was greater than the shear force. The transverse reinforcement for this column was #6 hoops at 6 in on center. and was detailed as seismic hoops. The longitudinal reinforcing of (32)-#11 bars satisfied the minimum and maximum reinforcement checks. The extension into the cap beam and drilled shaft was 36 in., which was controlled by ½ the column diameter.

The region outside the plastic hinge zone was similar to the design in Bents 2 and 4. The transverse reinforcing in this region was #6 hoops at 6 in. on center. The current design had the spacing of the transverse reinforcement set at 6 in.; therefore, this was used as the maximum spacing because the strength demand was not known. The hoops satisfied the maximum and minimum spacing.

The drilled shafts were designed in the same manner as the region outside the plastic hinge. For Bents 2 and 4, the transverse reinforcement was #6 hoops at 12 in. on center. Both of these drilled shafts satisfied the minimum and maximum checks. For Bent 3, the confinement steel was #6 hoops at 6 in. on center. The drilled shaft reinforcing for Bent 3 met all the transverse reinforcement requirements.

The diameter of the drilled shaft and longitudinal reinforcing in the abutment drilled shafts were increased in order to supply enough flexural and axial strength. The diameter was increased from 42 in. to 54 in., and the longitudinal reinforcing was increased to (24)-#11 bars from (16)-#11 bars. This was determined from the interaction diagrams. The transverse reinforcement for the drilled shafts is #5 hoops at 10 in. on center. The spacing was controlled by the minimum transverse reinforcement requirement.

The same design checks that were made for the columns were done in the strut design. The extension of the transverse reinforcing in the plastic hinge zone in the columns was not done. The struts were the first elements to hinge, and then the columns hinged at the bottom and top. The columns remained elastic at the column to strut connection. No hinging occurred at the connection of the strut and column; therefore, this is why the extension was not needed. Since the strut depths were so large, the entire length was designed as a plastic hinge zone. The plastic hinge regions for Bents 2 and 4 and Bent 3 are 108 in. and 180 in., respectively. The plastic hinge lengths were controlled by 1.5 times the strut depth. The transverse reinforcing for Bents 2 and 3 is #5 hoops at 4 in. on center. For Bent 3, the lateral reinforcing was #6 hoops at 3.5 in. on center. The hoop spacing was controlled by the minimum amount of transverse reinforcing. To satisfy minimum longitudinal requirements, the side reinforcing in Bent 3 was increased from #5 bars to #8 bars.

The increase of shear force on the structure affected the connection between the substructure and superstructure. The same connections were used at Bents 2 and 4. The expansion connections for the bridge were at the abutments and the left set of girders for Bents 2 and 3, and the rest were fixed. For the expansion connection at Bent 2, the angle was an L6x6x1x20. The fixed connection at Bent 2 and 4 was an L6x6x1x16. Both the angles were connected to the structure with two 1.5-inch diameter anchor bolts and through bolts. The Bent 3 connection was an L6x6x1x12, and it was connected with one 1.5-inch diameter anchor bolt and through bolt. Abutment 1 used an L6x6x1x20 and was connected to the superstructure and substructure with two 1.5-inch diameter anchor bolts and through bolts. Abutment 5 used an L6x6x3/4x12 and was connected to the structure with one 1.25-inch anchor bolt and through bolt. The details of the connection can be seen in Figures 4.28, 4.29 and 4.30. The angle thickness was controlled by bending strength. The angle length for the connection was governed by the location and sizes of the bolt holes. The number of anchor bolts and size of anchor bolts were controlled by the shear strength of the bolt and the combination of tension and shear.

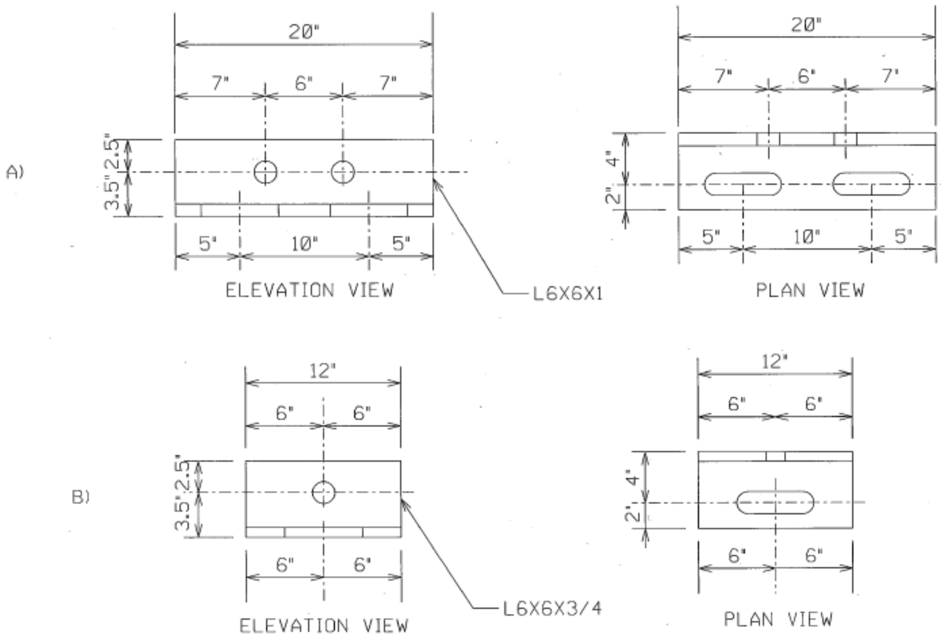


Figure 4.28: Scarham Creek Abutment Connections
A) Abutment 1(Expansion) B) Abutment 5 (Expansion)

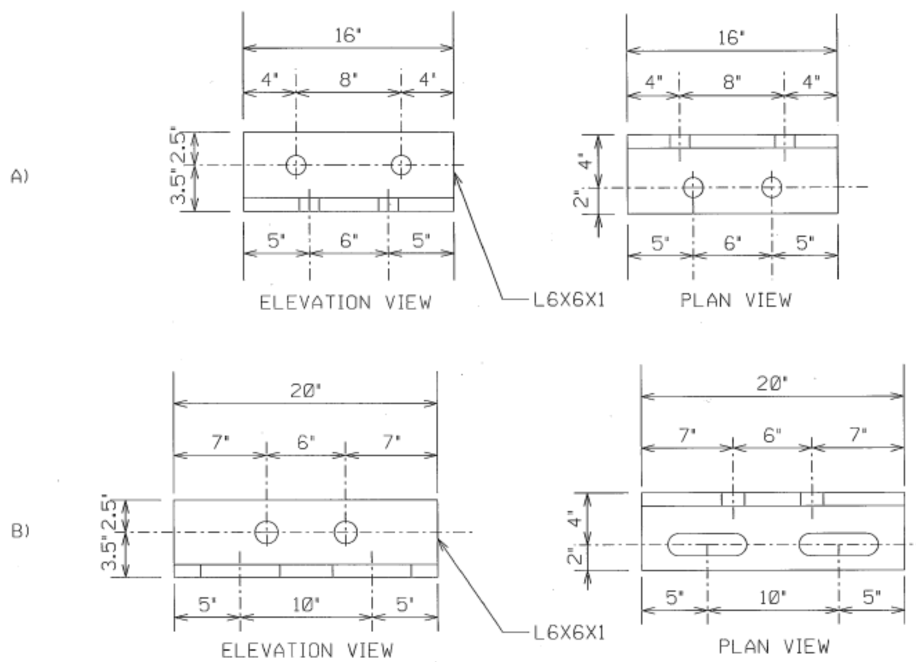


Figure 4.29: Scarham Creek Bents 2 & 4 Connections
A) Fixed B) Expansion

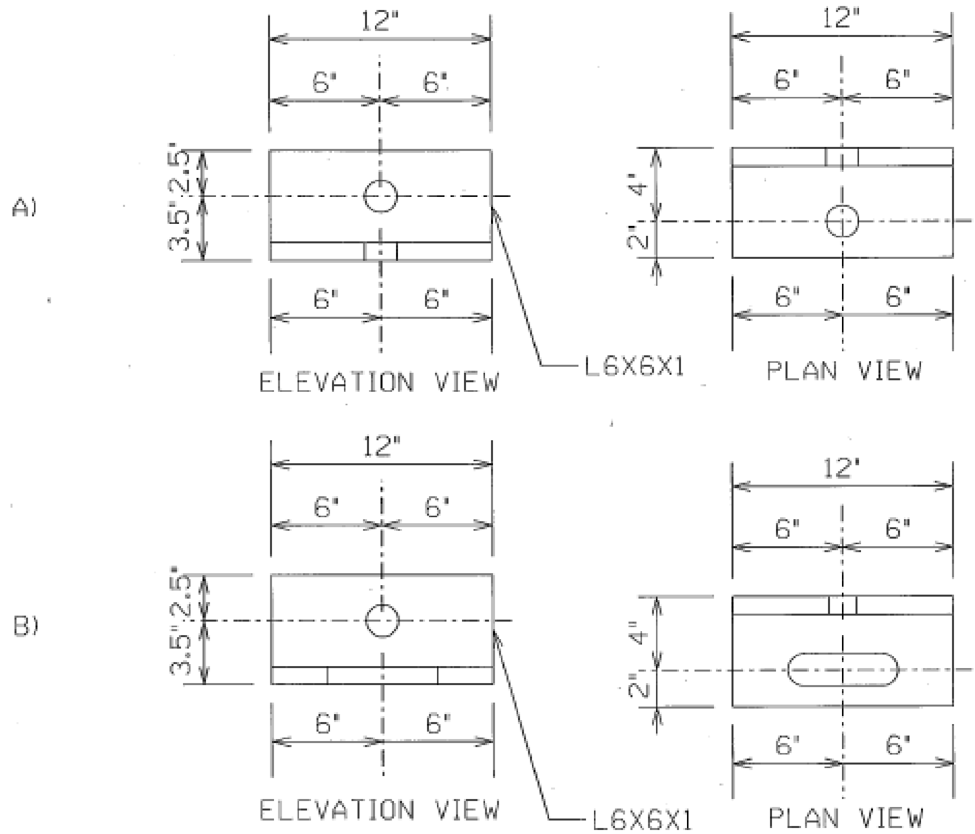


Figure 4.30: Scarham Creek Bents 3 Connection
A) Fixed B) Expansion

4.4.3 Results from LRFD Specification

Many of the initial steps taken in the Guide Specification design were also done in the LRFD Specification design. The same bridge model was used for this design method. The calculations and inputs for this bridge can be seen in Appendix H beginning on page 479. The design maximum deflections in the longitudinal and transverse direction were 0.192 in. and 1.553 in., respectively. The same response modification factors that were used for the previous two bridges were used for this bridge. The equivalent seismic loads for the structure were the same as in the Guide Specification. After combining the equivalent seismic loading, the maximum equivalent force was 0.513 kip per inch. This force was divided by the response modification factors to determine the R-equivalent values. The linear elastic loads were brought into the design worksheet from SAP, and then modified to be used as the design loads.

After inputting the loads into the worksheet, the design of the columns and drilled shafts was done. The minimum support lengths for the bridge range from 20 in. to 23 in. Bents 2 and 4 resulted in the same design; therefore, they will be described together. The longitudinal reinforcing for the columns was (24)-#11 bars. The maximum and minimum requirements were satisfied by the reinforcement. The interaction diagrams were used in the verification of the flexural strength of the columns and drilled shafts. The combination of the concrete shear strength and the reinforcement shear strength was greater than the applied shear force. The plastic hinge region for Bent 2 was 68 in; while, the plastic hinge region for Bent 4 is 64 in. The plastic hinge regions were controlled by 1/6 the column height. The transverse reinforcement in the plastic hinge region for the columns #6 hoops at 4 in. on center. The spacing was controlled by the maximum allowed by the LRFD Specification. The extension of the transverse reinforcing into the drilled shaft and cap beam was 30 in., which was controlled by half the column diameter. The lateral reinforcement satisfied the volumetric ratio for hoop reinforcing.

The columns and drilled shafts in Bent 3 are larger than those in Bents 2 and 4. The longitudinal reinforcement of (32)-#11 bars for Bent 3 met the requirements for maximum and minimum longitudinal reinforcing. The interaction diagrams were used to verify the flexural resistance of the column and drilled shafts. The column’s shear resistance was greater than the applied shear force. The transverse reinforcement in the columns is #6 hoops at 3 in. on center. The spacing was controlled by the required volumetric ratio of seismic hoop reinforcing. The plastic hinge region for this bent is 118 in. long, and like Bents 2 and 4, the plastic hinge region was controlled by 1/6 the column height. The extension of the transverse reinforcing into the bent cap beam and drilled shaft was 36 in., which was controlled by ½ the diameter of the column.

The column design outside the plastic hinge region and the drilled shaft design was the same process as described in the Guide Specification design. The LRFD Specification resulted in the same design.

The strut design was done in a similar manner as the columns; however since the struts are rectangular, some of the requirements changed. The struts plastic hinge regions for Bents 2 and 4 and Bent 3 are 72 in. and 120 in., respectively. These regions were controlled by the depth of the member. For Bents 2 and 4, the transverse reinforcing was #6 hoops at 3.5 in. on center. The lateral reinforcing for Bent 3 was #7 hoops at 3.5 in. on center. The spacing was governed by the minimum requirement. The spacing outside the plastic hinge region for Bents 2 and 4 was #6 hoops at 12 in. on center. The longitudinal reinforcing was the same as for the Guide Specification design. All the maximum and minimum reinforcing requirements were satisfied in the strut design.

The connection between the substructure and superstructure was influenced by the change in seismic loads. The same type of bolts and angles that were used in the Guide Specification were used in the LRFD Specification design. An L6x6x5/8x12 was used for the connection of all the bents with one 1.25-inch diameter anchor bolt and through bolt. The details for this connection are displayed in Figure 4.31. The connections at the abutments were larger than at the bents. Abutment 1 used an L6x6x1x26 and was connected to the structure with three 1.5-inch diameter anchor bolts and through bolts.

Abutment 5 was connected with an L6x6x7/8x20 and was connected to the structure with two 1.5-inch diameter anchor bolts and through bolts. Figure 4.32 shows the connection details. The combination of shear and tension controlled the design of the anchor bolts and through bolts for Abutment 1. The number of bolts for Abutment 5 was controlled by the shear force. The angle thickness for both abutments was controlled by bending strength. The angle lengths were governed by the spacing of the bolt holes.

A)

B)

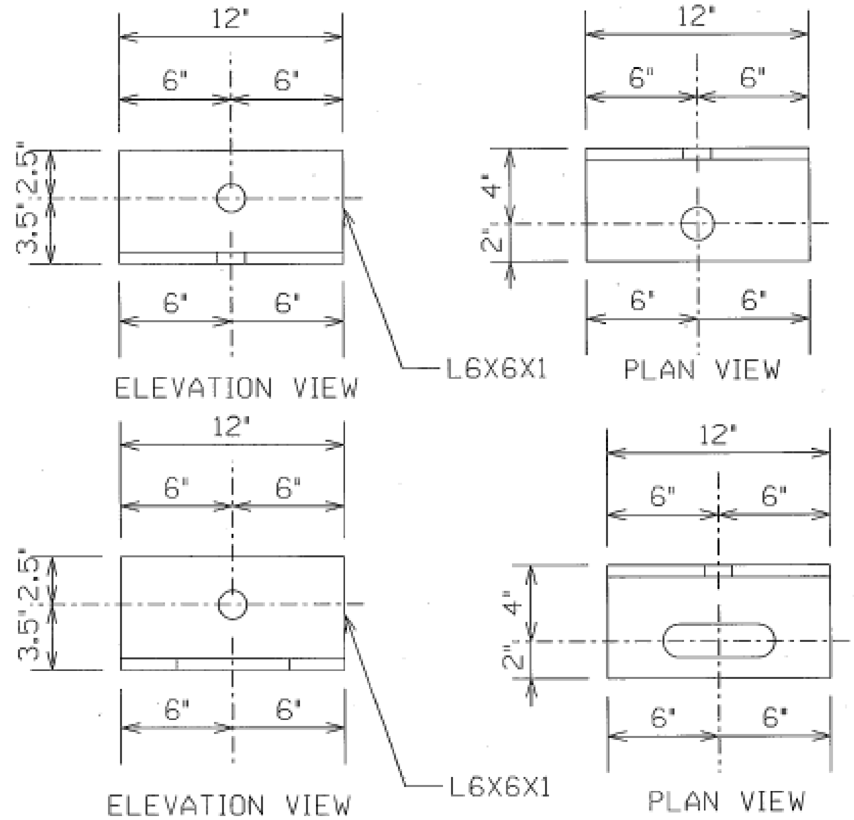


Figure 4.31: Scarham Creek Bent 2, 3 & 4 Connections
A) Fixed B) Expansion

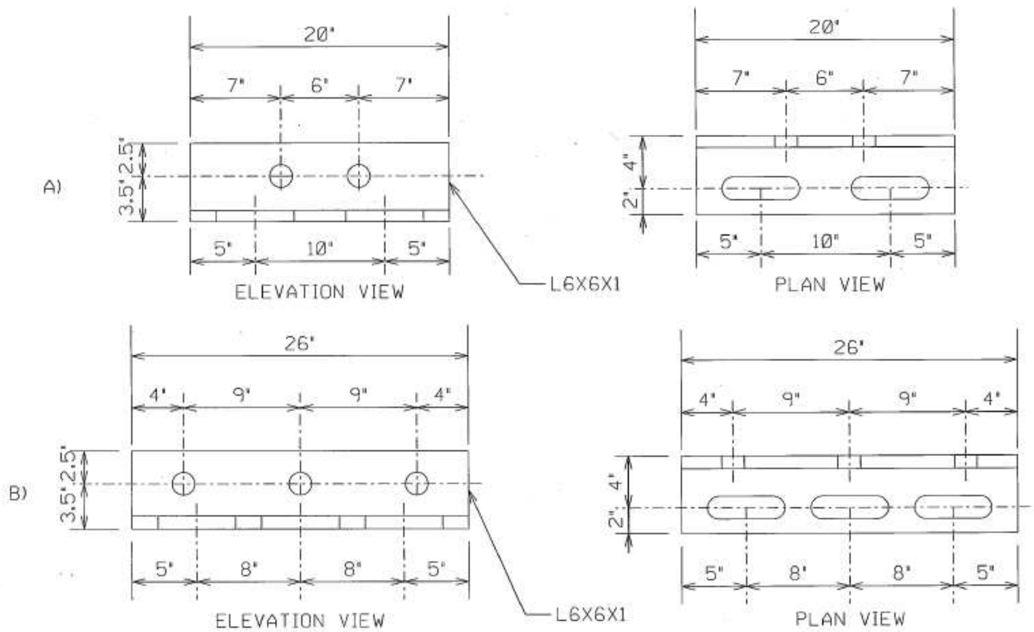


Figure 4.32: Scarham Creek Abutment Connections
A) Abutment 5 (Expansion) B) Abutment 1 (Expansion)

4.4.4 Comparison of Standard, Guide, and LRFD Specifications

The comparison of the three specifications allows the differences within the designs to be easily seen. An increase in hoops in the columns and drilled shafts from the Standard Specification was a major change in the designs. The differences in the number and spacing of the hoops can be seen in Tables 4.12, 4.13 and 4.14. The increase in the hoops for the struts can be seen in Tables 4.15 and 4.16. Since the stirrup bar diameter changed in the strut design, the percent increase is given as the percent increase of area of transverse reinforcing. Elevation views of the bents are shown in Figures 4.33-4.41. The drawings of the bents give a better visual of what the new required bent design. For Bent 3, the plastic hinge region in the LRFD Specification design is longer than the plastic hinge region in the Guide Specification. The other two bents plastic hinge regions are longer in the Guide Specification design versus the LRFD Specification design. With the spacing of 3 in. within the plastic hinge region, the LRFD Specification increased the hoops more than the Guide Specification.

Table 4.12: Scarham Creek Bridge Bent 2 Design Changes

Category	**Standard Specification**	**Guide Specification**	**LRFD Specification**
Stirrup Size	#6	#6	#6
Stirrup Spacing Outside Plastic Hinging Region	12 in. o.c.	12 in. o.c.	12 in. o.c.
Stirrup Spacing Inside Plastic Hinging Region	12 in. o.c.	6 in. o.c.	4 in. o.c.
Length of Plastic Hinge Region	0	90 in.	68 in.
Number of Stirrups for Both Columns	87	133	159
Increase in Stirrups	0%	53%	83%

Table 4.13: Scarham Creek Bridge Bent 3 Design Changes

Category	**Standard Specification**	**Guide Specification**	**LRFD Specification**
Stirrup Size	#6	#6	#6
Stirrup Spacing Outside Plastic Hinging Region	6 in. o.c.	6 in. o.c.	6 in. o.c.
Stirrup Spacing Inside Plastic Hinging Region	6 in. o.c.	6 in. o.c.	3 in. o.c.
Length of Plastic Hinge Region	0	108 in.	118 in.
Number of Stirrups for Both Columns	290	290	397
Increase in Stirrups	0%	0%	37%

Table 4.14: Scarham Creek Bridge Bent 4 Design Changes

Category	Standard Specification	Guide Specification	LRFD Specification
Stirrup Size	#6	#6	#6
Stirrup Spacing Outside Plastic Hinging Region	12 in. o.c.	12 in. o.c.	12 in. o.c.
Stirrup Spacing Inside Plastic Hinging Region	12 in. o.c.	6 in. o.c.	4 in. o.c.
Length of Plastic Hinge Region	0	90 in.	64 in.
Number of Stirrups for Both Columns	83	133	159
Increase in Stirrups	0%	60%	92%

Table 4.15: Scarham Bridge Strut 2 and 4 Design Changes

Category	Standard Specification	Guide Specification	LRFD Specification
Stirrup Size	#5	#5	#6
Stirrup Spacing Outside Plastic Hinging Region	12 in. o.c.	12 in. o.c.	12 in. o.c.
Stirrup Spacing Inside Plastic Hinging Region	12 in. o.c.	4 in. o.c.	3.5 in. o.c.
Length of Plastic Hinge Region	0	108 in.	72 in.
Number of Stirrups for Both Columns	20	57	66
Increase Area of Stirrups	0%	185%	368%

Table 4.16: Scarham Bridge Strut 3 Design Changes

Category	Standard Specification	Guide Specification	LRFD Specification
Stirrup Size	#5	#6	#7
Stirrup Spacing Outside Plastic Hinging Region	12 in. o.c.	12 in. o.c.	12 in. o.c.
Stirrup Spacing Inside Plastic Hinging Region	12 in. o.c.	3.5 in. o.c.	3.5 in. o.c.
Length of Plastic Hinge Region	0	180 in.	120 in.
Number of Stirrups for Both Columns	19	62	62
Increase Area of Stirrups	0%	363%	532%

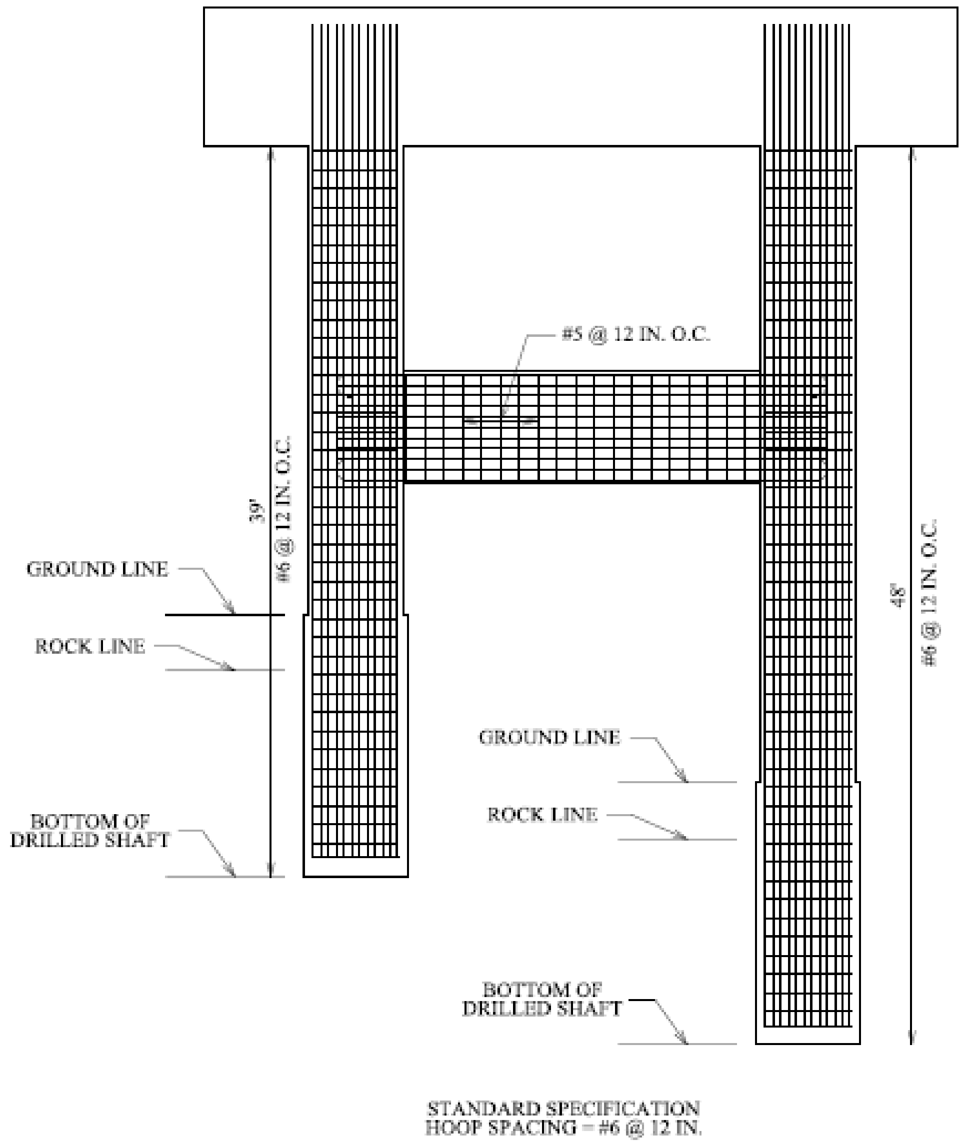


Figure 4.33: Scarham Creek Bridge Bent 2 Standard Specification

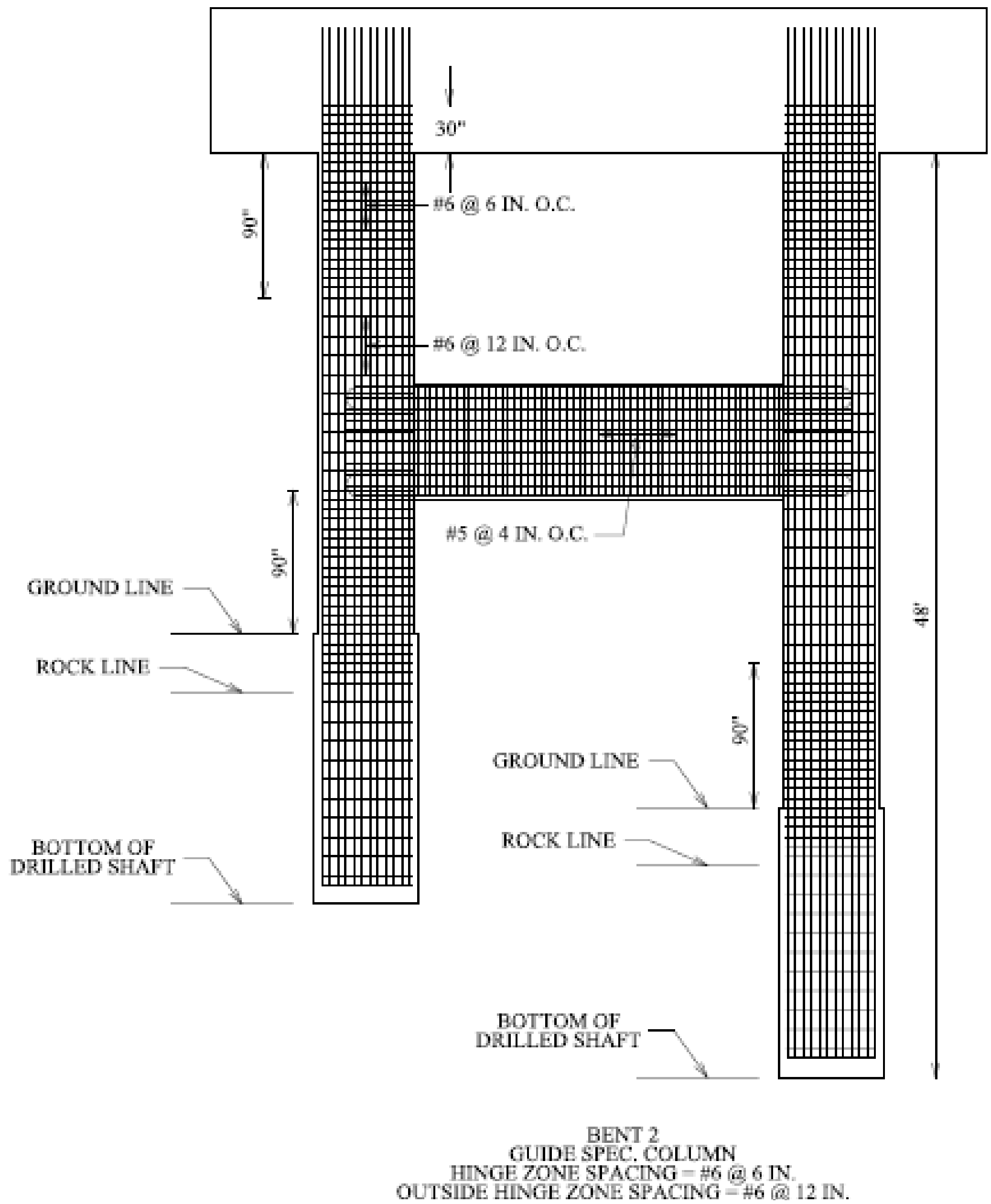


Figure 4.34: Scarham Creek Bridge Bent 2 Guide Specification

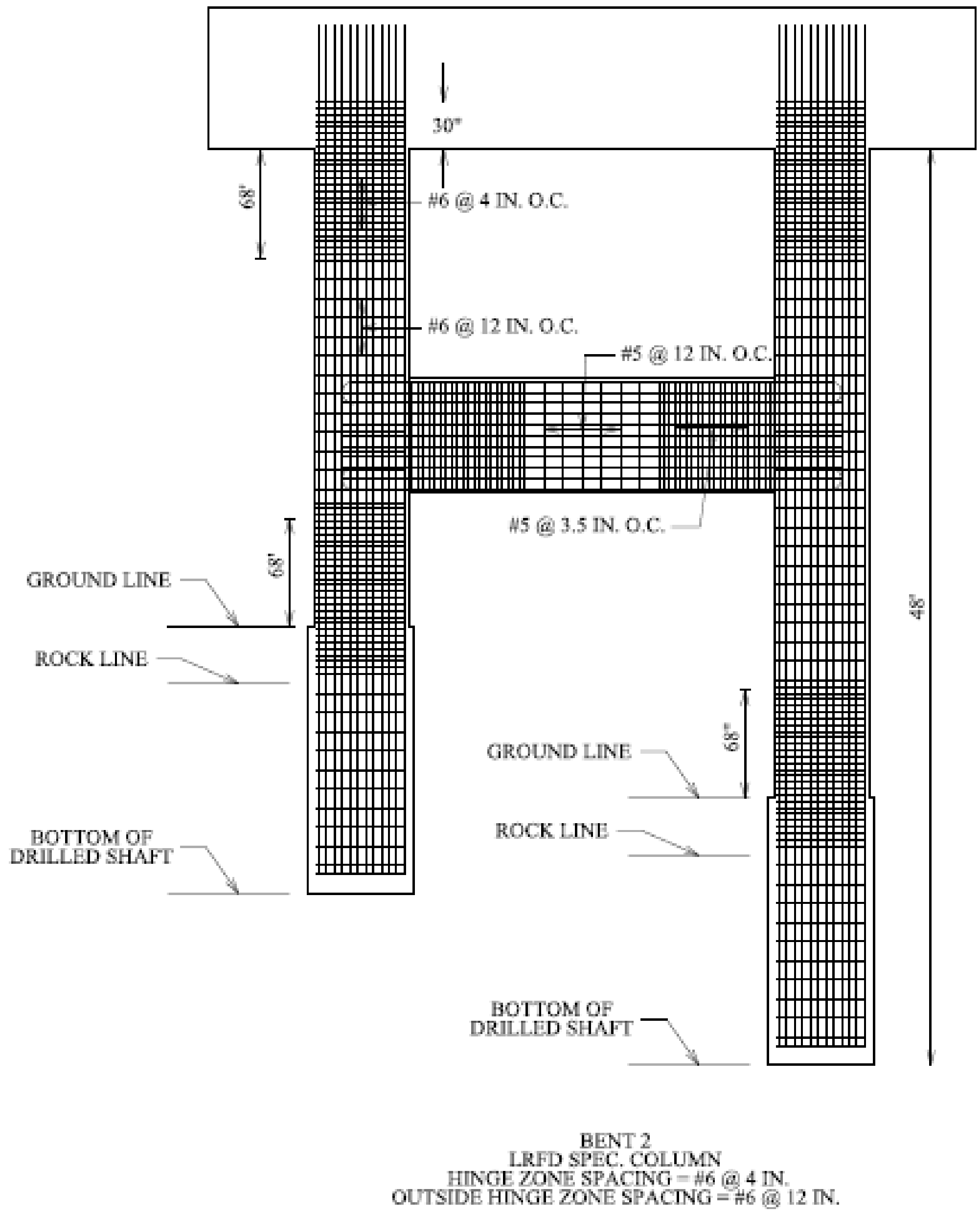


Figure 4.35: Scarham Creek Bridge Bent 2 LRFD Specification

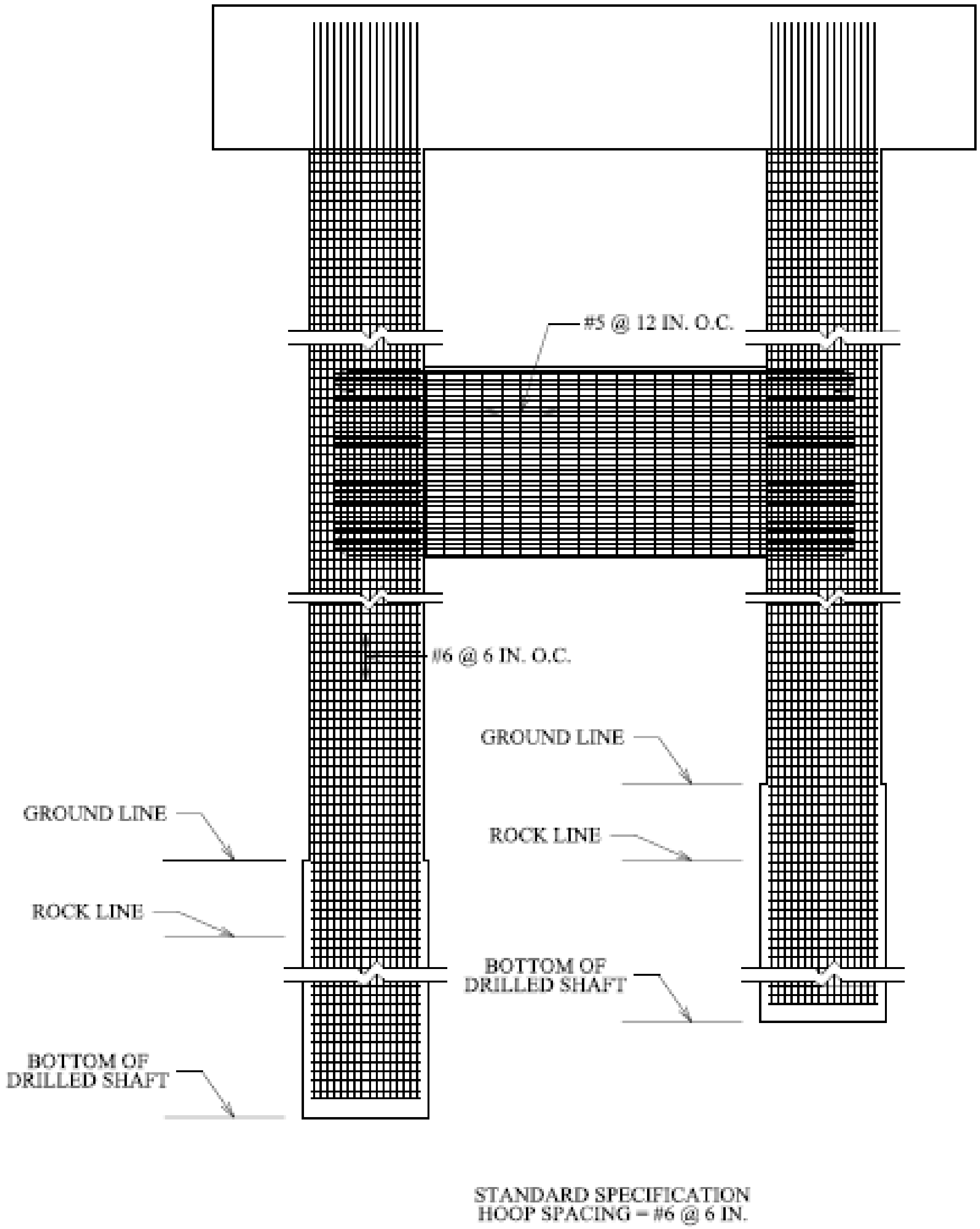


Figure 4.36: Scarham Creek Bridge Bent 3 Standard Specification

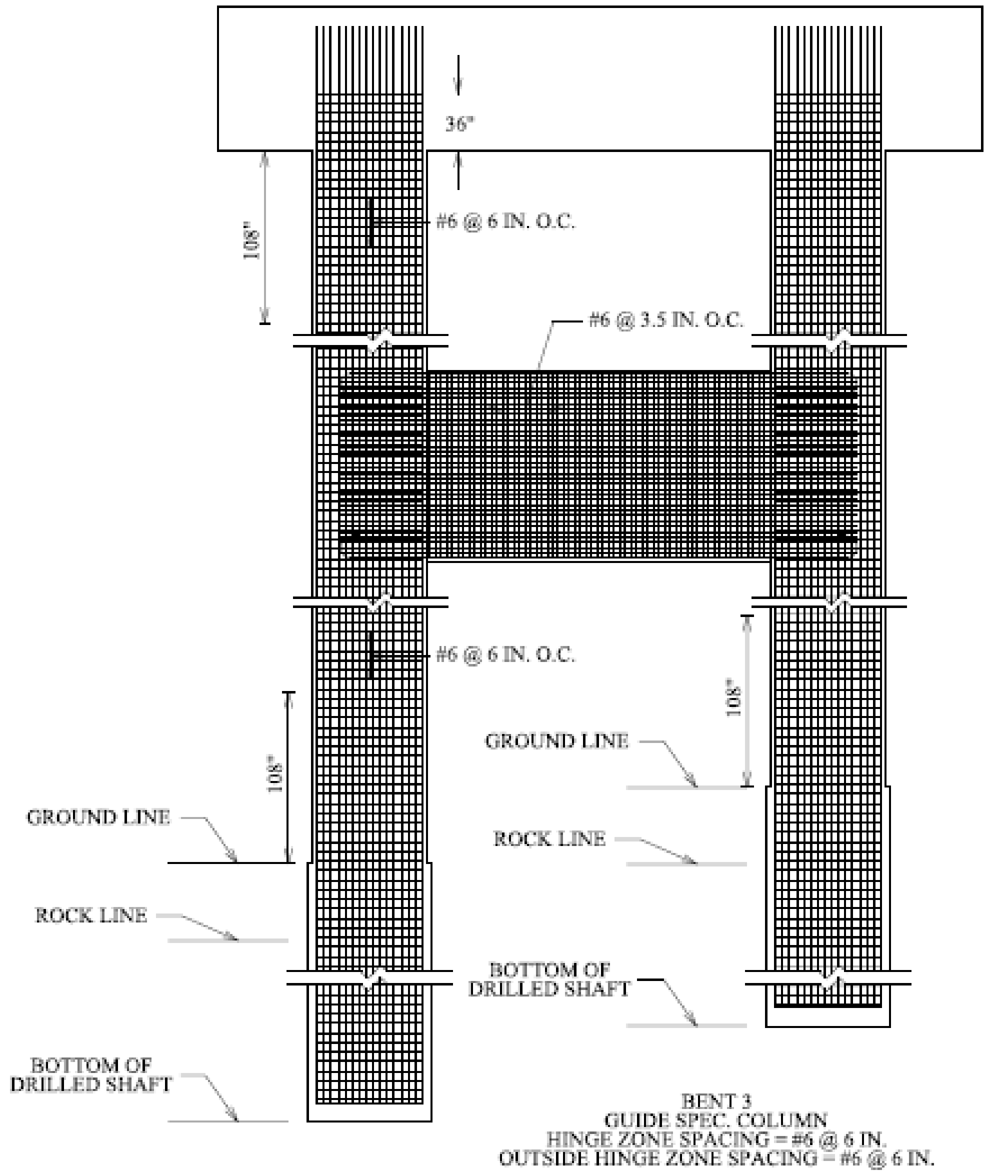


Figure 4.37: Scarham Creek Bridge Bent 3 Guide Specification

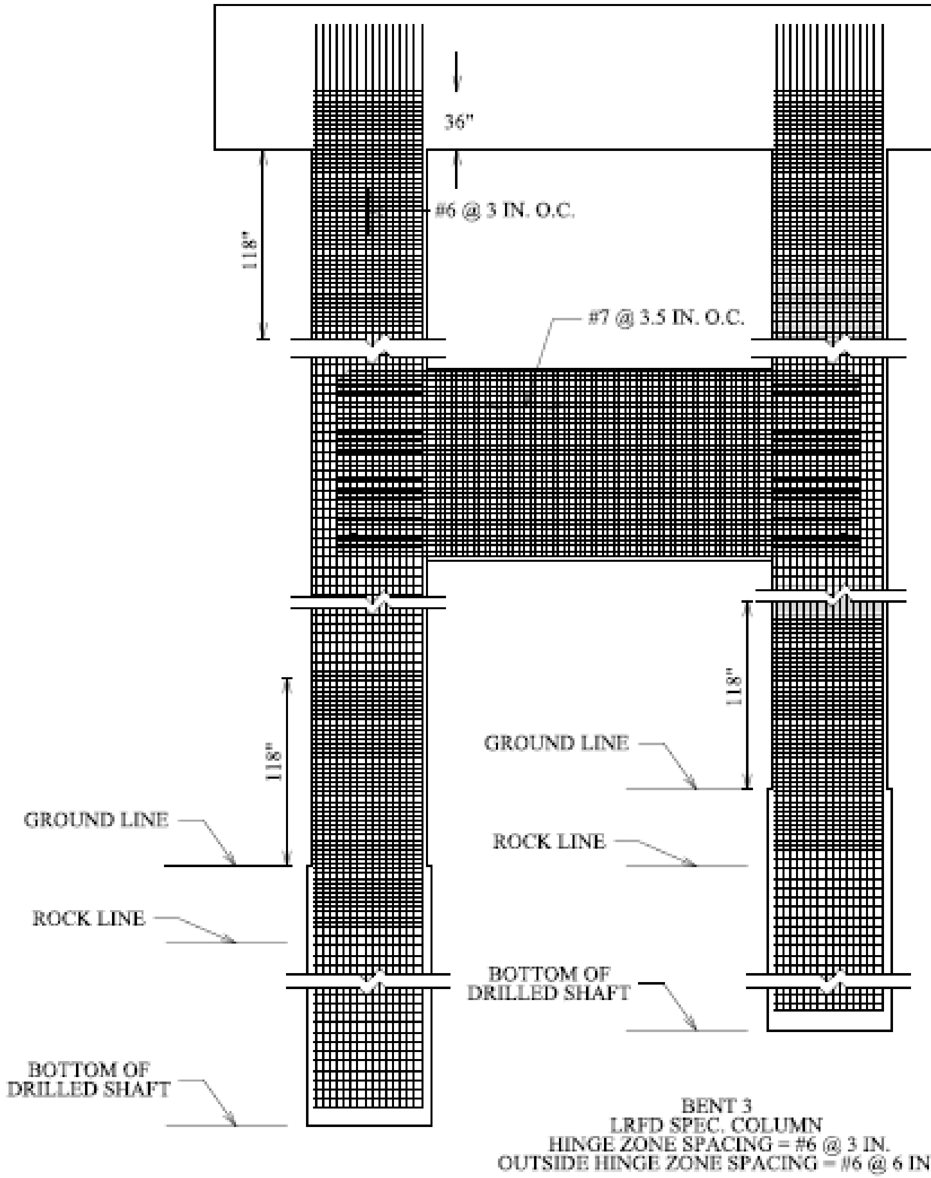


Figure 4.38: Scarham Creek Bridge Bent 3 LRFD Specification

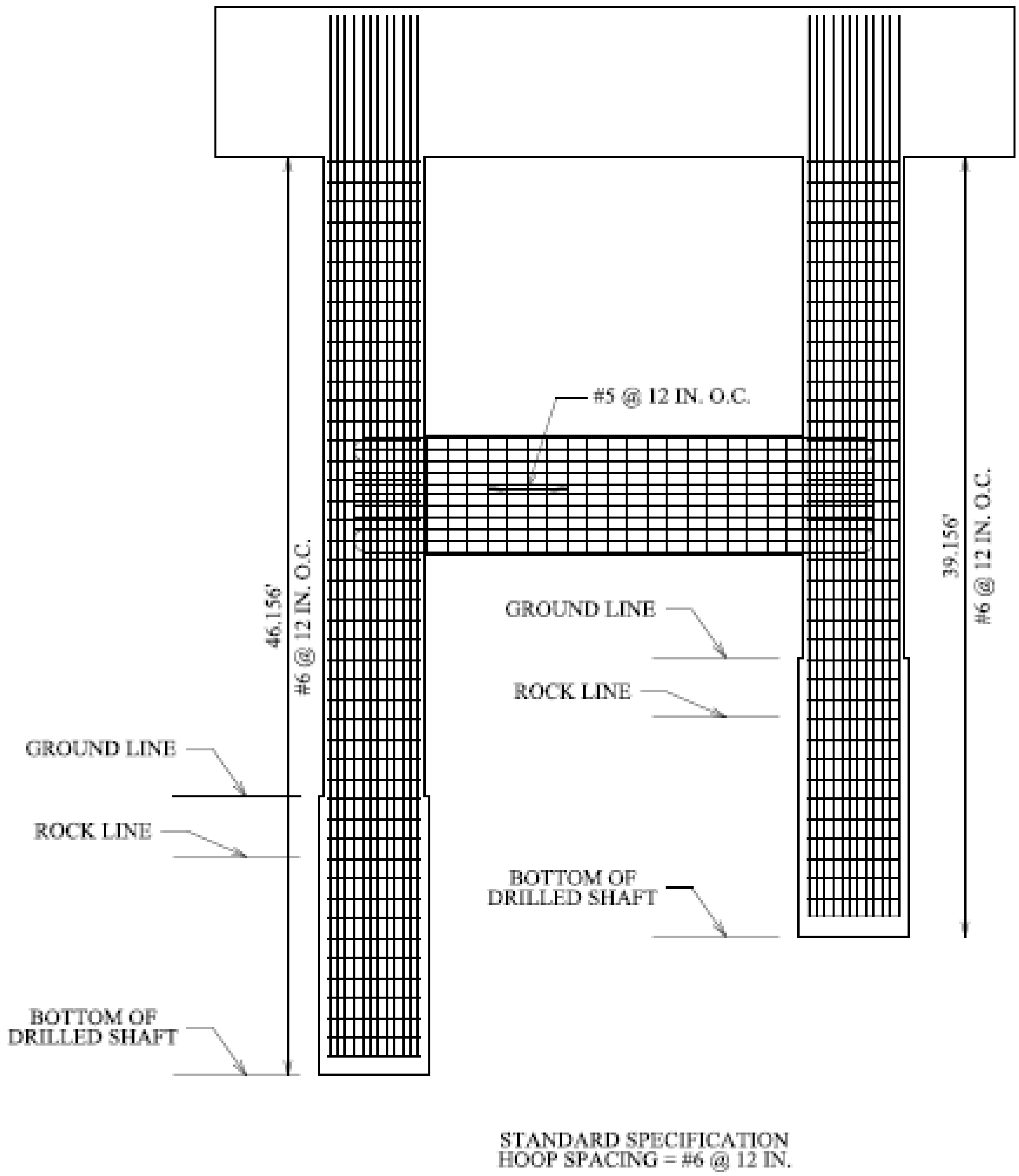


Figure 4.39: Scarham Creek Bridge Bent 4 Standard Specification

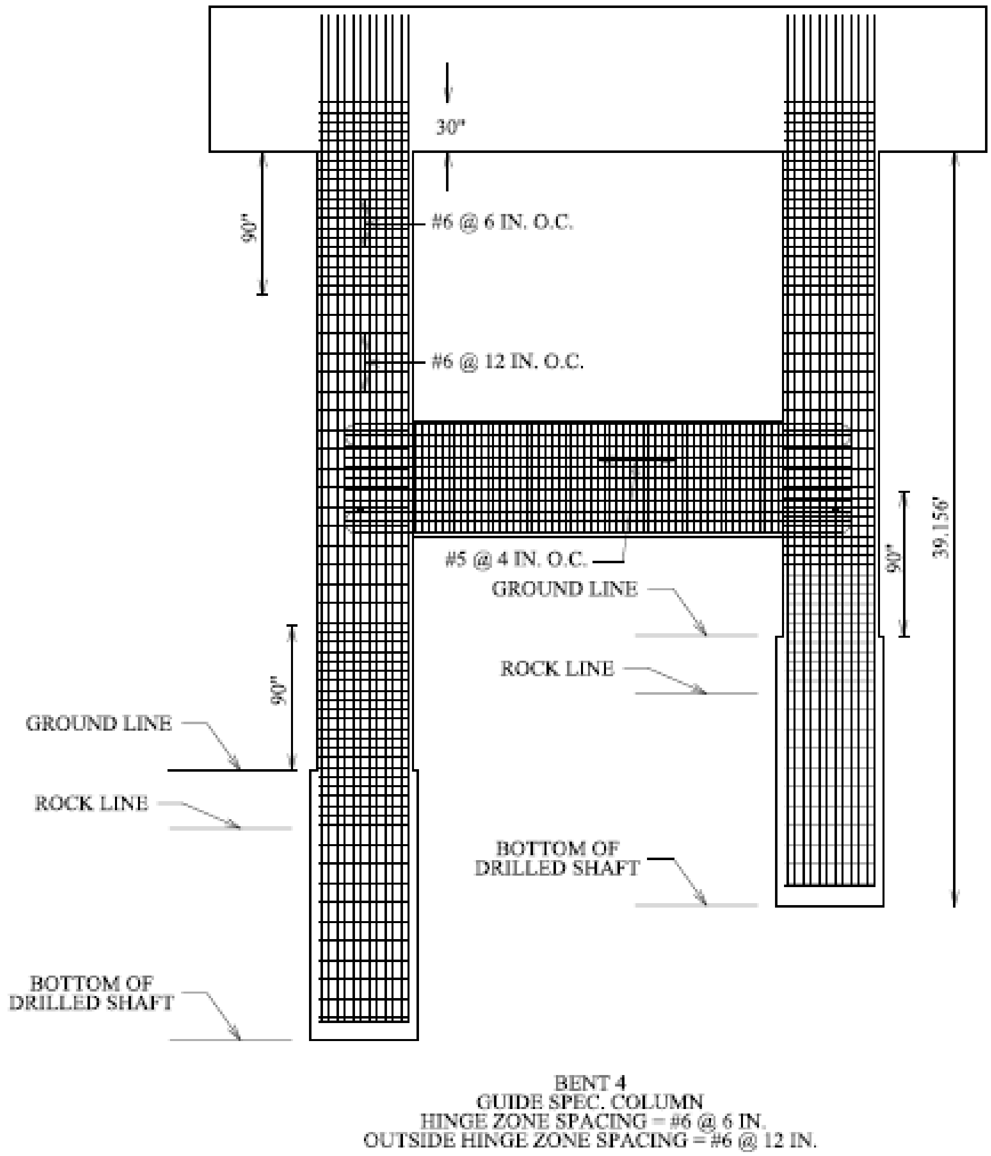


Figure 4.40: Scarham Creek Bridge Bent 4 Guide Specification

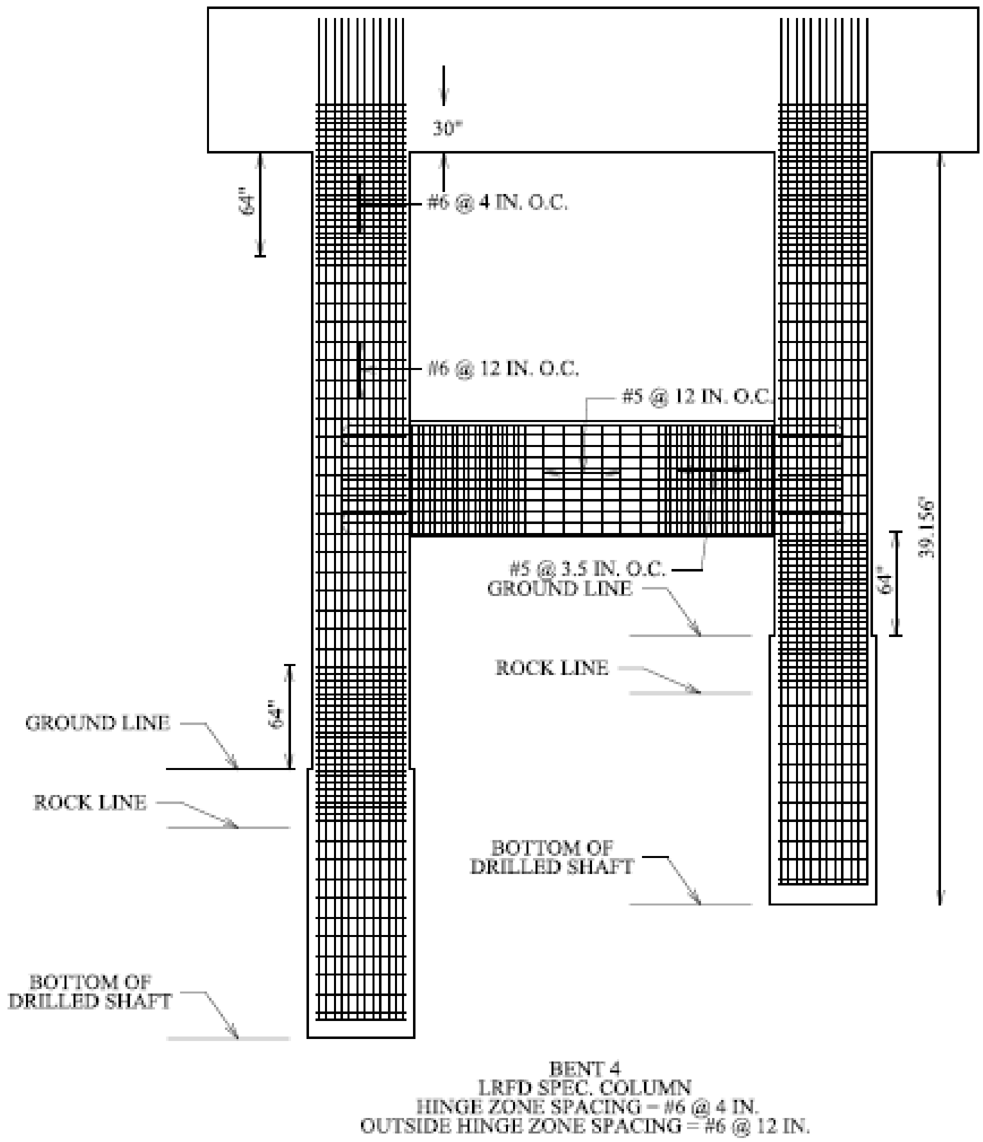


Figure 4.41: Scarham Creek Bridge Bent 4 LRFD Specification

The connection between the superstructure and substructure was another area that changes required. The changes made to the connection are displayed in Table 4.17. As can be seen, the lengths and thicknesses of the angles were increased. Also, the number of anchor bolts and through bolts was increased in some cases. The increase in angle thickness was due to the shear force the angle was resisting. The increase in length at the abutments was needed for the increase in flexural resistance and need to accommodate the slotted holes for the expansion connections.

Table 4.17: Scarham Creek Bridge Connection Changes

Category	**Standard Specification**	**Guide Specification**	**LRFD Specification**
BENT 2			
Angle Thickness (in.)	0.5	1	0.625
Angle Length (in.)	12	16 or 20	12
Bolt Diameter (in.)	1.5	1.5	1.25
Number of bolts/angle	1	2	1
BENT 3			
Angle Thickness (in.)	0.5	1	0.625
Angle Length (in.)	12	12	12
Bolt Diameter (in.)	1.5	1.5	1.25
Number of bolts/angle	1	1	1
BENT 4			
Angle Thickness (in.)	0.5	1	0.625
Angle Length (in.)	12	16 or 20	12
Bolt Diameter (in.)	1.5	1.5	1.25
Number of bolts/angle	1	2	1
ABUTMENT 1			
Angle Thickness (in.)	0.5	1	1
Angle Length (in.)	12	20	26
Bolt Diameter (in.)	1.5	1.5	1.5
Number of bolts/angle	1	2	3
ABUTMENT 5			
Angle Thickness (in.)	0.5	0.75	0.875
Angle Length (in.)	12	12	20
Bolt Diameter (in.)	1.5	1.25	1.5
Number of bolts/angle	1	1	2

4.5 Conclusion

The design of all three bridges was detailed in this chapter. With the increase in seismic design forces, the changes in design were seen in the transverse reinforcing and the connection between the substructure and superstructure. The design proved that there are some consistencies in the design of the bridges, but there are some differences that prohibit a standard connection being designated for all the bridges.

One of the reasons for this project was to update the design of three bridges from the Standard Specification Design to the AASHTO LRFD Bridge Design Specifications. From this project, some valuable information was determined about the design of concrete bridges in SDC B. If the bridges are being designed according to the Guide Specification, then there are a few things that can be concluded. The plastic hinge region will be controlled by 1.5 times the column diameter. The extension of transverse reinforcing into the cap beam and the drilled shaft will be controlled by ½ the column diameter. The spacing of the transverse reinforcement will be controlled by the maximum spacing allowed by the Guide Specification which is 6 in. on center. The three parameters listed were true for the three bridges that were investigated. The hoop spacing in the strut was controlled by the minimum transverse reinforcement ratio because of large member size. The consistency in the design process will help to simplify design.

When bridges are being designed according to the LRFD Specification, there seemed to be more inconsistency. The transverse reinforcement spacing within the plastic hinge region was controlled by the maximum of 4 in., and in another case, it was controlled by the required volumetric ratio of seismic hoop reinforcing. For Little Bear Creek Bridge, the plastic hinge region length was controlled by the column diameter, but for the Scarham Creek Bridge, 1/6 the height the column controlled the plastic hinge region length. For Oseligee Creek Bridge, the plastic hinge length in one column was controlled by its diameter and the other was controlled by 1/6 the column’s height. One thing that can be concluded is the extension of the transverse reinforcement into the drilled shaft and bent cap beam will be controlled by ½ the column diameter. The strut's confinement steel spacing was controlled by the volumetric ratio of transverse reinforcing. The inconsistencies in design can be further investigated by designing more bridges.

In order to improve in the seismic design of bridges, a few changes can be made by ALDOT. To meet the simplified deflection equations, the flexibility in the columns needs to increased which can be done by decreasing the column's diameter to length ratio. This would allow the simplified equations to be used and eliminate the need of a pushover analysis. Also in order to ensure hinging at the column and drilled shaft connection, the drilled shafts diameter should be larger than the column's diameter which will provide for the capacity protection needed in the drilled shaft. When designing struts for bridges, the flexibility of the strut should be considered. A strut with a large area, requires a lot of transverse and longitudinal reinforcing. In order for the bridges in Alabama to be more efficiently seismically designed, bridges need to be more flexible to meet the new design codes without sacrificing gravity load and stability requirements.

One of ALDOT's goals was to determine if it was possible to design a standard, econonical connection for all standard concrete bridges in Alabama. It was determined that will not be possible for concrete bridges in SDC B. When one of the parameters tested changes, there is enough of a change in the design that makes it hard to develop a connection that will work in all situations. The design worksheet created will ease the process of seismically designing concrete bridges in SDC B. As was seen in the three bridges designed, there are inconsistencies in the design of the connections and transverse reinforcing. It may be possible to have a certain group of bridges that can be designed in the same way if they have similar column heights, span lengths, and material, but to have one standard design for the whole state does not seem realistic for concrete bridges in SDC B.

Chapter 5 : Conclusions

The case study of the three bridges was done to update the seismic design of bridges in the state of Alabama. The bridges chosen were designed for the worst seismic hazard in Alabama. The main objectives of this project were to determine the effects of the LRFD seismic provisions on the design and detailing and to determine if typical, economically feasible details can be utilized for all the selected bridges.

5.1 Summary of Conclusions

The differences in the three specifications were sometimes significant. The specifications have developed rapidly over the recent years and are still continuously being changed. The seismic analysis and design has significantly increased from what was required by the Standard Specification for Alabama. The major difference between the LRFD Specification and Guide Specification is that the Guide Specification is a displacement-based design and the LRFD Specification is a force-based design, which is the same approach as the Standard Specification. The design procedures for the LRFD Specification and the Guide Specification were described in this work. A worksheet was created for both approaches to aid in the seismic design of concrete bridges in SDC B.

From the analysis and design of the three bridges, valuable information was discovered. From the design results, there is to be more consistency in the Guide Specification design of the columns and drilled shaft. With the Guide Specification, the plastic hinge region, extension of transverse reinforcing, and transverse reinforcing spacing within the hinge zone was controlled by the same parameters. The extension of the transverse reinforcement was the only consistent design change in the LRFD Specification design. The plastic hinge region was dependent on the column's height and diameter. Also, the transverse reinforcement was governed by both the maximum spacing and minimum amount of transverse reinforcement requirements. The only consistency in the connection design of the superstructure and substructure was the change of the precast screw inserts to a through bolt. In most of the connections, the angle size, anchor bolt size and number of anchor bolts increased. The increase in size was due to the increase in applied shear force.

ALDOT's goal was to evaluate if typical details could be created for the worst seismic scenario in Alabama. After the three cases studies were completed, it proves not to be possible. There seems to be too many variables that affect the connection design and plastic hinge region. However, bridges could be grouped based on certain criteria, such as span length, column height and column diameter, and perhaps a certain detail for each grouping could be determined. This is something that should be further investigated. Until further investigation, the worksheet created should be useful to and with the design of concrete bridges in SDC B.

5.2 Future Needs and Recommended Future Work

Different kinds of bridges should be investigated to evaluate the impact of LRFD's seismic requirements for Alabama's bridges. The list below contains some topics that could be further evaluated.

1. Design for concrete bridges in SDC A,
2. Design for steel bridges in SDC A and B,

3. Seismic Design for Critical and Essential bridges,
4. The soil-structure interaction,
5. Investigate deep foundation issues (Driven piles versus Drilled shafts), and
6. Connection Development.

The seismic design of bridges is a complex process. It is important to investigate the listed items in order to be fully prepared to design all bridges for Alabama for the seismic requirements of AASHTO LRFD. ALDOT is investigating a new way to make the substructure to superstructure connection, but the connection designed for this study is sufficient for the time being. Currently, ALDOT is making several changes in their design process to meet the new requirements. The changes in seismic design discussed in this thesis are a step in the direction of having Alabama bridges be design according to LRFD requirements.

Works Cited

AISC. *Steel Construction Manual Thirteenth Edition.*United States of America: American Institute of Steel Construction, 2005.

AASHTO. *AASHTO Guide Specifications for LRFD Seismic Bridge Design.* Washington D.C.: American Associtation of State Highway and Transportation Officials, 2008.

AASHTO. *AASHTO LRFD Bridge Design Specifications.* Washington D.C.: American Associtation of State Highway and Transportation Officials, 2007.

AASHTO. *Standard Specification for Highway Bridges 17th Edtion.* Washington D.C.: American Associtation of State Highway and Transportation Officials, 2002.

Computer & Structures, Inc. *SAP2000 User's Guide.* Berkeley, CA: Computer & Structures, Inc., 2007.

Parametric Technology Corportaion. *User's Guide Mathcad 14.* Needham, MA: Parametric Technology Corporation, 2007.

PCA. *PCA Column.* Portland Cement Association, 2004.

Tobias, Daniel, Ralph E. Anderson, Chad E. Hodel, William M. Kramer, Riyad M. Wahab, and Richard J. Chaput. "Overview of Earthquake Resisting System

Design and Retrofit Strategy for Bridges in Illinois." *Practice Periodical on Structural Design and Construction* 13, no. 3, 2008, 147-158.

Widjana, Matius A. “The Influence of the Recommended LRFD Guidelines for the Seismic Design of Highway Bridges on Virginia Bridges.” Virginia Polytechnic Institute and State University, 2003.

Appendix A: Oseligee Creek Bridge Guide Specification Design

Designer: Paul Coulston
Project Name: Oseligee Creek Bridge
Job Number:
Date: 11/2/2010

ORIGIN := 1

Description of worksheet: This worksheet is a seismic bridge design worksheet for the **AASHTO Guide Specification for LRFD Seismic Bridge Design**. All preliminary design should already be done for non-seismic loads.

Project Known Information

Location: Chambers County
Zip Code or Coordinates: 35.0069 N 88.2025 W

Superstructure Type: AASTHO I girders
Substructure Type: Circular columns supported on drilled shafts
Abutment Type: Abutment beam supported on drilled shafts

Note: *Input all of the below information.*

$fc := 4000 \quad psi$

$fye := 60000 \quad psi$

$\rho_{conc} := 0.08681 \quad \frac{lb}{in^3}$

$g := 386.4 \quad \frac{in}{s^2}$

Length of Bridge (ft)	$L := 240$	ft
Span (ft)	$Span := 80$	ft
Deck Thickness (in)	$t_{deck} := 7$	in
Deck Width (ft)	$DeckWidth := 32.75$	ft
I-Girder X-Sectional Area (in^2)	$IGirderArea := 559.5$	in^2
Guard Rail Area (in^2)	$GuardRailArea := 310$	in^2
Bent Volume (ft^3)	$BentVolume := 5\cdot 4\cdot 30 = 600$	ft^3
Column Diameter (in)	$Columndia := 42$	in
Drilled Shaft Diameter (in)	$DSdia := 42$	in
Drilled Shaft Abutment Diameter (in)	$DSabutdia := 42$	in
Average Column Height (ft)	$ColumnHeight := 22$	ft

Column Area (in^2)	$Acolumn := \frac{Columndia^2 \cdot \pi}{4} = 1.385 \times 10^3$	in^2
Drilled Shaft Area (in^2)	$Adrilledshaft := \frac{DSdia^2 \pi}{4} = 1.385 \times 10^3$	in^2
Drilled Shaft Abutment Area (in^2)	$Adsabut := \frac{DSabutdia^2 \cdot \pi}{4} = 1.385 \times 10^3$	in^2

Note: These are variables that were easier to input in ft and then convert to inches.

$Span := Span \cdot 12 = 960$ in

$L := L \cdot 12 = 2.88 \times 10^3$ in

$DeckWidth := DeckWidth \cdot 12 = 393$ in

$BentVolume := BentVolume \cdot 12^3 = 1.037 \times 10^6$ in^3

$ColumnHeight := ColumnHeight \cdot 12 = 264$ in

Steps for Seismic Design

Article 3.1: The Guide Specification only applies to the design of **CONVENTIONAL** BRIDGES.

Article 3.2: Bridges are design for the life safety performance objective.

Article 6.2: Requires a subsurface investigation take place.

Article 6.8 and C6.8: Liquefaction Design Requirements - A liquefaction assessment should be made if loose to very loose sands are present to a sufficient extent to impact the bridge stability and A_s is greater than or equal to 0.15.

Article 3.3: The type of Earthquake Resisting System (ERS) should be considered. This is not a requirement as in SDC C and D, but should be considered. A Type 1 ERS has a ductile substructure and essentially elastic superstructure.

Type of Bridge: TYPE 1

***Article 3.4*: Determine Design Response Spectrum**

Note: AASHTO Guide Specifications for LRFD Seismic Bridge Design is accompanied with a program that has the Seismic Hazard maps. This program will calculate several of the variables that are needed for the analysis.

1) *Article 3.4.2.1*:Determine the Site Class. Table 3.4.2.1-1

INPUT Site Class: **D**

2) Enter maps and find PGA, S_s,and S_1. Then enter those values in their respective spot. Also, the the Guide Specification is accompanied with a cd that contains a program that will find these values for the designer.

$PGA := 0.116 \ g$

INPUT $S_s := 0.272 \ g$

$S_1 := 0.092 \ g$

3) *Article 3.4.2.3:* Site Coefficients. From the PGA,S_s, and S_1 values and site class choose F_{PGA}, F_a, and F_v. Note: straight line interpolation is permitted.

INPUT

$F_{PGA} := 1.57$ Table 3.4.2.3-1

$F_a := 1.58$ Table 3.4.2.3-1

$F_v := 2.4$ Table 3.4.2.3-2

Eq. 3.4.1-1 $A_s := F_{PGA} \cdot PGA = 0.182 \ g$ A_s : Acceleration Coefficient

Eq. 3.4.1-2 $SDS := F_a \cdot S_s = 0.43 \ g$ S_{DS} = Short Period Acceleration Coefficient

Eq. 3.4.1-3 $SD1 := F_v \cdot S_1 = 0.221 \ g$ S_{D1} = 1-sec Period Acceleration Coefficient

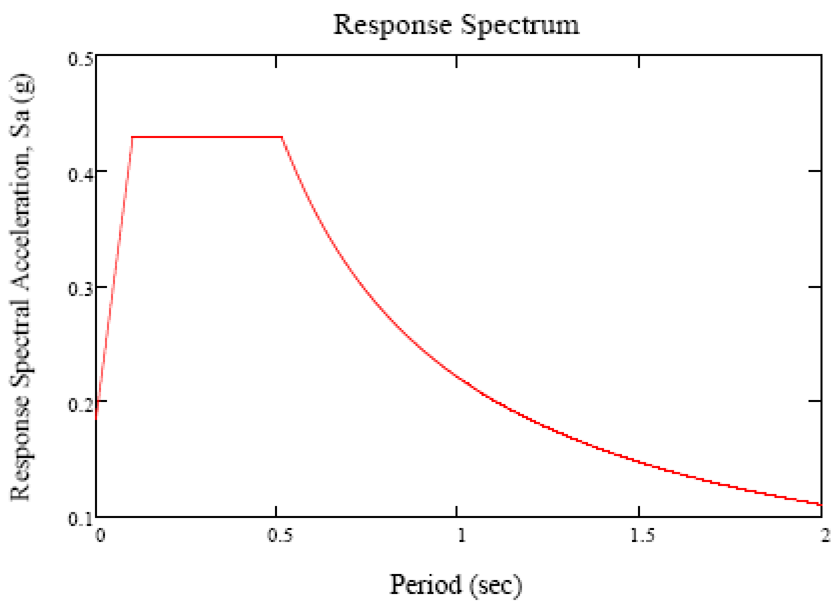
4) Creating a Response Spectrum

Note: The DesignSpectrum Code creates a Response Spectrum for the Bridge. At this time the period of the bridge is unknown; therefore, the Sa value cannot be calculated.

$Tmax := 2 \ s \qquad Dt := 0.001 \ s$

$$DesignSpectrum(SDS, SD1, A_s, Tmax, Dt) := \left| \begin{array}{l} T_s \leftarrow \dfrac{SD1}{SDS} \\ T_o \leftarrow 0.2 \cdot T_s \\ n_{max} \leftarrow \dfrac{Tmax}{Dt} \\ \text{for } i \in 1 .. n_{max} \\ \quad \left| \begin{array}{l} T_i \leftarrow Dt \cdot i \\ a_i \leftarrow (SDS - A_s) \cdot \dfrac{Dt \cdot i}{T_o} + A_s \ \text{ if } Dt \cdot i < T_o \\ a_i \leftarrow SDS \ \text{ if } Dt \cdot i \ge T_o \wedge Dt \cdot i \le T_s \\ a_i \leftarrow \dfrac{SD1}{Dt \cdot i} \ \text{ if } Dt \cdot i > T_s \end{array} \right. \\ R \leftarrow augment(T, a) \\ R \end{array} \right.$$

$BridgeSpectrum := DesignSpectrum(SDS, SD1, A_s, Tmax, Dt)$



***Article 3.5*: Selection of Seismic Design Category**

SD1 = 0.221 g   From Table 3.5-1 Choose SDC

```
SDCprogram(SD1) := | for c ∈ SD1
                   |   | c ← "A"  if SD1 < 0.15
                   |   | c ← "B"  if SD1 ≥ 0.15 ∧ SD1 < 0.3
                   |   | c ← "C"  if SD1 ≥ 0.3 ∧ SD1 < 0.5
                   |   | c ← "D"  if SD1 ≥ 0.5
                   | Rs ← c
                   | c
```

SDC := SDCprogram(SD1) = "B"

Displacement Demand Analysis Δ_D

Figure 1.3-2 Demand Analysis Flowchart

***Article 4.2*: Selection of Analysis Procedure**

This is a function of the SDC and the regularity of the bridge.

Procedure 1 = Equivalent Static Method

Article 4.3.3: Displacement Magnification for Short-Period Structures

$u_d := 2$ for SDC B

$$\text{Rdprogram}(T, SDS, SD1, u_d) := \left| \begin{array}{l} Ts \leftarrow \dfrac{SD1}{SDS} \\ Tb \leftarrow 1.25 \cdot Ts \\ x \leftarrow \left(1 - \dfrac{1}{u_d}\right) \cdot \dfrac{Tb}{T} + \dfrac{1}{u_d} \\ y \leftarrow 1.0 \\ a \leftarrow x \ \text{ if } \ \dfrac{Tb}{T} > 1.0 \\ a \leftarrow y \ \text{ if } \ \dfrac{Tb}{T} \le 1.0 \\ a \end{array} \right.$$

Note: This Rd value will be calculated when the period of the structure is known. This factor will amplify the displacement demand.

***Article 5.4*: Analytical Procedure 1 (Equivalent Static Analysis)**

Note: There are two methods that can be used according to this procedure. The Uniform Load Method is suitable for regular bridges that respond principally in their fundamental mode of vibration. The Single Mode Spectral Method may be a better method if there is a major change in the spans, stiffness of the piers, etc.
The Uniform Load Method is simpler and less time consuming and will give accurate results, and this is the reason it has been chosen in this design.

<u>Uniform Load Method</u>

Step 1: Build a bridge model
Step 2: Apply a uniform load of Po = 1.0 kip/in. in both the longitudinal and transverse direction. Also, the uniform load can be converted into point loads and applied as joint loads in SAP. Calculate the static displacement for both directions. In SAP, tables of the displacements can be exported to EXCEL, and the MAX Function can be used to find the maximum displacement.

Step 3: Calculate the bridge lateral stiffness, K, and total weight, W.

$$p_o := 1.0 \quad \frac{kip}{in}$$

INPUT

$$\upsilon_{smaxLong} := 1.671281 \quad in$$

$$\upsilon_{smaxTran} := 3.228449 \quad in$$

Eq. C5.4.2-1 $$K_{Long} := \frac{p_o \cdot L}{\upsilon_{smaxLong}} = 1.723 \times 10^3 \qquad \frac{kip}{in}$$

Eq. C5.4.2-2 $$K_{Tran} := \frac{p_o \cdot L}{\upsilon_{smaxTran}} = 892.069 \qquad \frac{kip}{in}$$

INPUT: Multiplying factors

$$W := \frac{\rho_{conc} \cdot \begin{pmatrix} L \cdot t_{deck} \cdot DeckWidth + 2 \cdot BentVolume + 4 \cdot Acolumn \cdot ColumnHeight \ldots \\ + L \cdot 4 \cdot IGirderArea + 2 \cdot GuardRailArea \cdot L \end{pmatrix}}{1000}$$

$$W = 1709.336 \qquad kips$$

Step 4: Calculate the period, T_m.

Eq. C5.4.2-3 $$T_{mLong} := 2\pi \cdot \sqrt{\frac{W}{K_{Long} \cdot g}} = 0.318 \qquad s$$

Step 5: Calculate equivalent static earthquake loading p_e.

$$acc(SDS, SD1, T_{mLong}, A_s) := \left| \begin{array}{l} T_s \leftarrow \dfrac{SD1}{SDS} \\ T_o \leftarrow 0.2 \cdot T_s \\ \text{for } a \in T_{mLong} \\ \quad \left| \begin{array}{l} a \leftarrow (SDS - A_s) \cdot \dfrac{T_{mLong}}{T_o} + A_s \;\text{ if } T_{mLong} < T_o \\ a \leftarrow SDS \;\text{ if } T_{mLong} \geq T_o \wedge T_{mLong} \leq T_s \\ a \leftarrow \dfrac{SD1}{T_{mLong}} \;\text{ if } T_{mLong} > T_s \end{array} \right. \\ Ra \leftarrow a \\ a \end{array} \right.$$

$$Sa_{Long} := acc(SDS, SD1, T_{mLong}, A_s) = 0.43$$

Eq. C5.4.2-4 $$p_{eLong} := \frac{Sa_{Long} \cdot W}{L} = 0.255 \qquad \frac{kip}{in}$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$Rd_{Long} := Rdprogram(T_{mLong}, SDS, SD1, u_d) = 1.509$$

$$v_{smaxLong} := Rd_{Long} \cdot \frac{p_{eLong}}{p_o} \cdot \upsilon_{smaxLong} = 0.643 \qquad in$$

Repeat Steps 4, 5, and 6 for transverse loading.

Step 4: Calculate the period, T_m.

Eq. C5.4.2-3 $$T_{mTran} := 2\pi \cdot \sqrt{\frac{W}{K_{Tran} \cdot g}} = 0.442 \qquad s$$

Step 5: Calculate equivalent static earthquake loading p_e.

$$Sa_{Tran} := acc\left(SDS, SD1, T_{mTran}, A_s\right) = 0.43$$

Eq. C5.4.2-4 $$p_{eTran} := \frac{Sa_{Tran} \cdot W}{L} = 0.255 \qquad \frac{kip}{in}$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$Rd_{Tran} := Rdprogram\left(T_{mTran}, SDS, SD1, u_d\right) = 1.226$$

$$v_{smaxTran} := Rd_{Tran} \cdot \frac{p_{eTran}}{p_o} \cdot \upsilon_{smaxTran} = 1.009 \qquad in$$

Single-Mode Spectral Method

This procedure is not specifically addressed in the Guide Specifications. The Guide Spec. refers you to the AASHTO LRFD Bridge Design Specifications.

Article 4.7.4.3.2: Single-Mode Spectral Method

Step 1: Build a bridge model
Step 2: Apply a uniform load of Po = 1.0 in both the longitudinal and transverse direction. Calculate the static displacement for both directions.

Step 3: Calculate factors α, β , and γ.

Note: The Deflection equations come from analysis of the SAP model. The displacement is taken at the joints along the length of the bridge and input into an Excel Worksheet. Then a graph is created of the displacements along the length of the bridge. A best fit line is plotted, and that is the equation that is shown below.

$$v_{stran}(x) := -1\cdot 10^{-6}\cdot x^2 + 0.0034\cdot x - 0.2945 \qquad v_{slong}(x) := -2\cdot 10^{-8}\cdot x^2 + 6\cdot 10^{-5}\cdot x + 1.5856$$

C4.7.4.3.2b-1 $\alpha_{Tran} := \int_0^L v_{stran}(x)\,dx$ $\qquad \alpha_{Long} := \int_0^L v_{slong}(x)\,dx$

C4.7.4.3.2b-2 $\beta_{Tran} := \int_0^L \frac{W}{L} v_{stran}(x)\,dx$ $\qquad \beta_{Long} := \int_0^L \frac{W}{L}\cdot v_{slong}(x)\,dx$

C4.7.4.3.2b-3 $\gamma_{Tran} := \int_0^L \frac{W}{L}\cdot v_{stran}(x)^2\,dx = 6.739\times 10^3$ $\qquad \gamma_{Long} := \int_0^L \frac{W}{L}\cdot v_{slong}(x)^2\,dx$

α = Displacement along the length

β = Weight per unit length * Displacement

γ = Weight per unit length * Displacement^2

Step 4: Calculate the Period of the Bridge

Eq. 4.7.4.3.2b-4 $T_{mTran1} := 2\pi\cdot\sqrt{\frac{\gamma_{Tran}}{p_o\cdot g\cdot \alpha_{Tran}}} = 0.361$ s

Eq. 4.7.4.3.2b-4 $T_{mLong1} := 2\pi\cdot\sqrt{\frac{\gamma_{Long}}{p_o\cdot g\cdot \alpha_{Long}}} = 0.313$ s

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smLong} := acc\left(SDS, SD1, T_{mLong1}, A_s\right) = 0.43$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

Eq. C4.7.4.3.2b-5 $$PeLong(x) := \frac{\beta_{Long} \cdot C_{smLong}}{\gamma_{Long}} \cdot \frac{W}{L} \cdot v_{slong}(x)$$

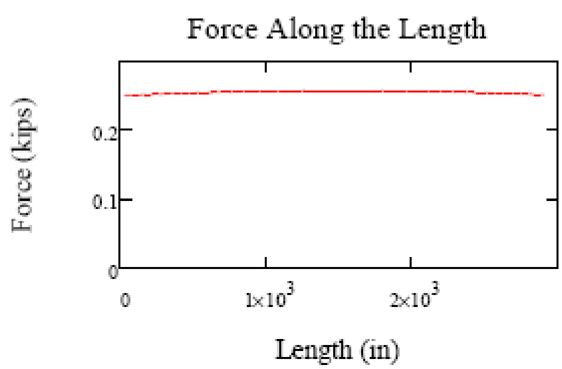
$$PeLong(x) \rightarrow 0.0000094657649785618823161 \cdot x + -3.1552549928539607720e\text{-}9 \cdot x^2 + 0.25014861583346201$$

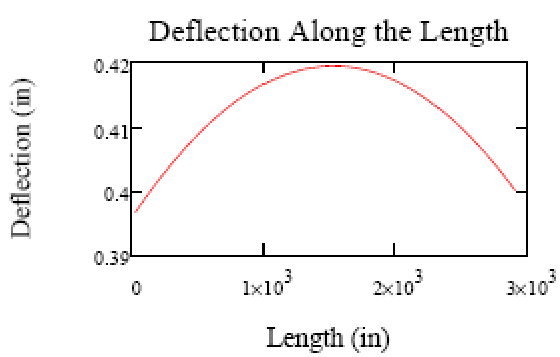
$$dW := \frac{L}{100}$$

$$i := 1 .. 101$$

$$Pelong_i := PeLong[(i-1) \cdot dW] \qquad \delta long_i := v_{slong}[(i-1)dW]$$

$$\Delta long_i := Pelong_i \cdot \delta long_i$$





Maximum Deflection

$$\max(\Delta long) = 0.419 \quad in$$

NOTE: Repeat Steps 5 and 6 for Transverse Direction.

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smTran} := acc\left(SDS, SD1, T_{mTran1}, A_s\right) = 0.43$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

Eq. C4.7.4.3.2b-5 $$PeTran(x) := \frac{\beta_{Tran} \cdot C_{smTran}}{\gamma_{Tran}} \cdot \frac{W}{L} \cdot v_{stran}(x)$$

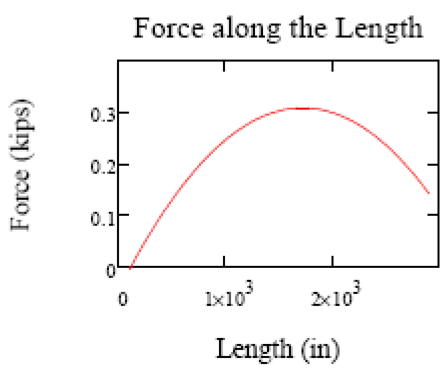
$$PeTran(x) \rightarrow 0.00040401585387083704979 \cdot x + -1.1882819231495207347e\text{-}7 \cdot x^2 - 0.034994902636753385636$$

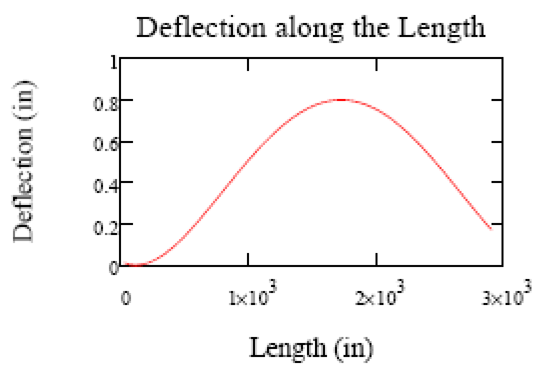
$$dL := \frac{L}{100}$$

$$i := 1..101$$

$$Petran_i := PeTran[(i-1) \cdot dL] \qquad \delta tran_i := v_{stran}[(i-1)dL]$$

$$\Delta tran_i := Petran_i \cdot \delta tran_i$$





Maximum Deflection

$\max(\Delta tran) = 0.801$ in

***Article 5.6*: Effective Section Properties**

Note: Use 0.7*Ig for ductile reinforced concrete members.
Refer to the charts on page 5-20 of the Guide Specification if a more precise value is desired.

***Article 5.2:* Abutment Modeling**

Note: This is taken care of in the SAP model.

***Article 5.3*: Foundations Modeling**

Note: Since in SDC B, Foundation Modeling Methods I can be used.

FMM is dependent on the type of foundation.

For bridges with Pile Bent/Drilled Shaft the depth of fixity can be estimated.
Since details regarding reinforcing are not known, reduce the stiffness of the drilled shafts to **one half** the uncracked section.

Note: Special provisions need to be considered if Liquefaction is present. (Article 6.8)

Article 4.4: Combination of Orthogonal Seismic Displacement Demands

$$LoadCase1 := \sqrt{\left(1 \cdot v_{smaxLong}\right)^2 + \left(0.3 \cdot v_{smaxTran}\right)^2} = 0.711 \quad in$$

$$LoadCase2 := \sqrt{\left(1 \cdot v_{smaxTran}\right)^2 + \left(0.3 \cdot v_{smaxLong}\right)^2} = 1.028 \quad in$$

COLUMN DESIGN

***Article 4.8*: Displacement Demand/Capacity**

Note: If the column height is different for each bent, a capacity check needs to be made at each bent.

Displacement Demand/Capacity for the Bents $\Delta_D < \Delta_C$

BENT 2

Note: The displacement demand is taken as the bent displacement. This can be found by using the SAP Bridge model that was created.

Input $\Delta_{Dlong} := 1.3462$ in $\Delta_{DLong} := Rd_{Long} \cdot \Delta_{Dlong} \cdot p_{eLong} = 0.518$ in

$\Delta_{Dtran} := 2.0805$ in $\Delta_{DTran} := Rd_{Tran} \cdot \Delta_{Dtran} \cdot p_{eTran} = 0.65$ in

Input $H_o := 18$ ft

Input $B_o := \frac{Columndia}{12}$ ft

$\Lambda := 2$ Fixed and top and bottom

Eq. 4.8.1-3 $x := \frac{\Lambda \cdot B_o}{H_o} = 0.389$

Eq. 4.8.1-1 $\Delta_C := 0.12 \cdot H_o \cdot (-1.27 \cdot \ln(x) - 0.32) = 1.9$ in

$0.12 \cdot H_o = 2.16$ in

$$CheckLimit(\Delta_C) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } \Delta_C \ge 0.12 \cdot H_o \\ a \leftarrow \text{"FAILURE"} \quad \text{if } \Delta_C < 0.12 \cdot H_o \end{array} \right.$$

$CheckLimit(\Delta_C) = \text{"FAILURE"}$

$$\mathrm{CheckCapacity}(\Delta_C, \Delta_D) := \begin{array}{|l} c \leftarrow \text{"OK"} \quad \text{if } \Delta_C \geq \Delta_D \\ c \leftarrow \text{"FAILURE"} \quad \text{if } \Delta_C < \Delta_D \end{array}$$

$$\mathrm{CheckCapacity}(\Delta_C, \Delta_{DLong}) = \text{"OK"}$$

$$\mathrm{CheckCapacity}(\Delta_C, \Delta_{DTran}) = \text{"OK"}$$

BENT 3

INPUT

$\Delta_{Dlong} := 1.4373$ in $\qquad \Delta_{DLong} := Rd_{Long} \cdot \Delta_{Dlong} \cdot p_{eLong} = 0.553$ in

$\Delta_{Dtran} := 2.9001$ in $\qquad \Delta_{DTran} := Rd_{Tran} \cdot \Delta_{Dtran} \cdot p_{eTran} = 0.65$ in

INPUT $\quad H_o := 25.834$ ft

INPUT $\quad B_o := \dfrac{\mathrm{Columndia}}{12}$ ft

$\Lambda := 2$ Fixed and top and bottom

Eq. 4.8.1-3 $\qquad x := \dfrac{\Lambda \cdot B_o}{H_o} = 0.271$

Eq. 4.8.1-1 $\qquad \Delta_C := 0.12 \cdot H_o \cdot (-1.27 \cdot \ln(x) - 0.32) = 4.149$ in

$0.12 \cdot H_o = 3.1$ in

$$\mathrm{CheckLimit}(\Delta_C) := \begin{array}{|l} a \leftarrow \text{"OK"} \quad \text{if } \Delta_C \geq 0.12 \cdot H_o \\ a \leftarrow \text{"FAILURE"} \quad \text{if } \Delta_C < 0.12 \cdot H_o \end{array}$$

$$\mathrm{CheckLimit}(\Delta_C) = \text{"OK"}$$

$$\mathrm{CheckCapacity}(\Delta_C, \Delta_D) := \begin{array}{|l} c \leftarrow \text{"OK"} \quad \text{if } \Delta_C \geq \Delta_D \\ c \leftarrow \text{"FAILURE"} \quad \text{if } \Delta_C < \Delta_D \end{array}$$

$$\mathrm{CheckCapacity}(\Delta_C, \Delta_{DLong}) = \text{"OK"}$$

$$\mathrm{CheckCapacity}(\Delta_C, \Delta_{DTran}) = \text{"OK"}$$

NOTE: IF THE SIMPLIFIED EQUATIONS ABOVE DO NOT WORK, A PUSHOVER ANALYSIS OF THE BRIDGE CAN BE DONE TO VERIFY THE DISPLACEMENT CAPACITY. In SAP 2000, there is an earthquake design program that allows a pushover analysis to be done by setting the SDC to D. Be sure to amplify the demand values by the appropriate Rd value. List the results below to verify that the Displacement Capacity is sufficient. The Demand Displacement must be multiplied by pe/po. The below chart was created in Excel and then brought into Mathcad.

GenDispl	Demand (in)	Capacity (in)	Check
_GD_TR1_DESIGN	0.9609	2.753592	OK
_GD_LG1_DESIGN	1.243476	2.11728	OK
_GD_TR2_DESIGN	1.058172	3.612048	OK
_GD_LG2_DESIGN	1.1439	4.627332	OK

***Article 4.12*: Minimum Support Length Requirements**

Note: May need to add more calculations if column heights are different at the bents.

N (in) = Minimum support length measured normal to the bridge

L (ft) = Length of bridge to adjacent expansion joint or end of the bridge

H (ft) = average height of columns supporting bridge deck for abutments
for columns and piers = column height

S (Degree) = angle of skew

Abutment Support Length Requirement

INPUT $\text{Span}_{\text{abutment}} := \frac{\text{Span}}{12} = 80$ ft $\quad H_{\text{abutment}} := \frac{\text{ColumnHeight}}{12} = 22$ ft

Note: The $\text{Span}_{\text{abutment}}$ is divided by number of spans and inches.

$\text{Skew}_{\text{abutment}} := 0$ Degrees

Eq. 4.12.2-1

$$\text{Nabutment} := 1.5\cdot\left(8 + 0.02\text{Span}_{\text{abutment}} + 0.08H_{\text{abutment}}\right)\cdot\left(1 + 0.000125\text{Skew}_{\text{abutment}}{}^{2}\right) = 17.04 \quad \text{in}$$

Bent Support Length Requirement

BENT 2

INPUT $Span_{Bent} := \frac{Span}{1 \cdot 12} = 80$ ft

Note: The $Span_{abutment}$ is divided by number of spans and inches.

INPUT $H_{Bent} := 18$ ft *INPUT*: Column Height for this Bent

INPUT $Skew_{Bent} := 0$ Degrees

Eq. 4.12.2-1

$$N_{Bent} := 1.5 \cdot (8 + 0.02 Span_{Bent} + 0.08 H_{Bent}) \cdot \left(1 + 0.000125 Skew_{Bent}^{2}\right) = 16.56 \quad \text{in}$$

BENT 3

INPUT $Span_{Bent} := \frac{Span}{1 \cdot 12} = 80$ ft

Note: The $Span_{abutment}$ is divided by number of spans and inches.

INPUT $H_{Bent} := 25.834$ ft *INPUT*: Column Height for this Bent

INPUT $Skew_{Bent} := 0$ Degrees

Eq. 4.12.2-1

$$N_{Bent} := 1.5 \cdot (8 + 0.02 Span_{Bent} + 0.08 H_{Bent}) \cdot \left(1 + 0.000125 Skew_{Bent}^{2}\right) = 17.5 \quad \text{in}$$

***Article 4.14*: Superstructure Shear Keys**

$$V_{ok} := 2 \cdot V_{n}$$

Note: This does not apply to this bridge.

Figure 1.3-5 SDC B Detailing

Decide what Type of bridge designing.

Structure Type: **Type 1**

***Article 8.3*: Determine Flexure and Shear Demands**

***Article 8.5*: Plastic Moment Capacity**

Note: Article 8.5 refers the designer back to Article 4.11.1-4.

BENT 2 DESIGN

***Article 4.11.1-4*: Steps to find Moment Capacity, Shear Capacity, and Axial Force**

Note: Use some kind of software to find the Moment Capacity of the Column. PCA Column was used to create an Interaction Diagram and to calculate the Moment Capacity. The shear for the bent was found by knowing the Moment.

INPUT $M_p := 29299200 \quad lb \cdot in$

INPUT $Fixity := 216\,in$

Note: Fixity is the point of fixity for the column/drilledshaft.

$$V_p := \frac{2 \cdot M_p}{Fixity \cdot 1000} = 271.289 \quad kips \qquad V_{pBent2} := 2 \cdot V_p = 542.578 \quad kips$$

Note: If the decision is made to design for **Elastic Forces** then the above variables need to be override. This can be done by simply changing the Vp variable to the elastic force from SAP2000 that has been multiplied by pe/po.

Note: Pu is the combination of Elastic Axial force from the earthquake and dead load.

INPUT $P_u := 520000 \quad lb$

Article 8.6: Shear Demand and Capacity for Ductile Concrete Members

Note: It is recommended to use the plastic hinging forces whenever practical.

$V_u := V_p \qquad \phi_s := 0.9$

***Article 4.11.6*: Analytical Plastic Hinge Length**

Note: For reinforced concrete columns framing into a footing, an integral bent cap, an oversized shaft, or cased shaft.

<u>*INPUT*</u> $d_{bl} := 1.41 \quad in$ $\qquad$ d_{bl}: Diameter of Longitudinal Bar

Eq. 4.11.6-1

$$\text{PlasticHinge}(\text{Fixity}, \text{fye}, d_{bl}) := \left| \begin{array}{l} lp \leftarrow 0.08 \cdot \text{Fixity} + 0.15 \cdot \frac{\text{fye}}{1000} \cdot d_{bl} \\ m \leftarrow 0.03 \cdot \frac{\text{fye}}{1000} \cdot d_{bl} \\ a \leftarrow lp \ \text{ if } \ lp \geq m \\ a \leftarrow m \ \text{ if } \ lp < m \\ a \end{array} \right.$$

$$L_p := \text{PlasticHinge}(\text{Fixity}, \text{fye}, d_{bl}) = 29.97 \quad in$$

***Article 4.11.7*: Reinforced Concrete Column Plastic Hinge Region**

Note: y is the region of column with a moment demand exceeding 75% of the maximum plastic moment. From the SAP model, find the location at which the moment demand is 0.75*Mp. The 0.75*Mp value should be divided by p_{eTran} to take into account the model loads have not been multiplied by p_{eTran}. The location will also need to be <u>**INPUT**</u> into the PlasticHingeRegion program in inches.

$$Mp75 := 0.75 \cdot M_p = 2.197 \times 10^7 \quad lb \cdot in$$

$$\text{PlasticHingeRegion}(L_p, \text{Columndia}) := \left| \begin{array}{l} z \leftarrow 1.5 \cdot \text{Columndia} \\ x \leftarrow L_p \\ y \leftarrow 0 \\ a \leftarrow \max(z, x, y) \end{array} \right.$$

Note: Input the arguments into the program. Most likely the column diameter is the only variable that has changed.

INPUT $Lpr := PlasticHingeRegion(L_p, Columndia) = 63$ in

***Article 8.6.2*: Concrete Shear Capacity**

$Ag := Acolumn$

Eq. 8.6.2-2 $Ae := 0.8 \cdot Ag = 1.108 \times 10^3$ in^2

$\mu_D := 2$ Specified in Article 8.6.2 of Guide Spec.

INPUT $s := 6$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $Asp := .31$ in^2 Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.625$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 6$ in Cover: Concrete cover for the Column (in)

INPUT $Dprime := 30$ in Dprime: Diameter of spiral or hoop for circular columns (in)

Eq. 8.6.2-7 $\rho_s := \frac{4 \cdot Asp}{s \cdot Dprime} = 6.889 \times 10^{-3}$

$fyh := \frac{fye}{1000} = 60$ ksi

Eq. 8.6.2-6 $StressCheck(\rho_s, fyh) := \left| \begin{array}{l} fs \leftarrow \rho_s \cdot fyh \\ a \leftarrow fs \text{ if } fs \le 0.35 \end{array} \right.$

$fs := StressCheck(\rho_s, fyh) = 0.413$

Eq. 8.6.2-5

$$\alpha program(fs, \mu_D) := \begin{array}{|l} \alpha prime \leftarrow \frac{fs}{0.15} + 3.67 - \mu_D \\ a \leftarrow 0.3 \ \text{ if } \ \alpha prime \le 0.3 \\ a \leftarrow \alpha prime \ \text{ if } \ \alpha prime > 0.3 \wedge \alpha prime < 3 \\ a \leftarrow 3 \ \text{ if } \ \alpha prime \ge 3 \\ a \end{array}$$

$$\alpha Prime := \alpha program(fs, \mu_D) = 3$$

If Pu is Compressive

Eq. 8.6.2-3

$$vcprogram(\alpha Prime, fc, P_u, Ag) := \begin{array}{|l} vc \leftarrow 0.032 \cdot \alpha Prime \cdot \left(1 + \frac{P_u}{2Ag \cdot 1000}\right) \cdot \sqrt{\frac{fc}{1000}} \\ min1 \leftarrow 0.11 \sqrt{\frac{fc}{1000}} \\ min2 \leftarrow 0.047 \alpha Prime \cdot \sqrt{\frac{fc}{1000}} \\ minimum \leftarrow \min(min1, min2) \\ a \leftarrow vc \ \text{ if } \ vc \le minimum \\ a \leftarrow minimum \ \text{ if } \ vc > minimum \\ a \end{array}$$

Eq. 8.6.2-4

If Pu is **NOT** Compressive VC = 0

Note: If Pu is not compressive, will have to manually input 0 for vc. Just input it below the vc:=vcprogram and the variable will assume the new value.

$$vc := vcprogram(\alpha Prime, fc, P_u, Ag) = 0.22 \quad ksi$$

$$Vc := vc \cdot Ae = 243.838 \quad kips$$

***Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity**

INPUT $n := 2$ n: number of individual interlocking spiral or hoop core sections

Eq. 8.6.3-1
Eq. 8.6.4-1

$$\text{vsprogram}(n, Asp, fyh, Dprime, s, fc, Ae) := \left| \begin{array}{l} vs \leftarrow \frac{\pi}{2} \cdot \left(\frac{n\,Asp \cdot fyh \cdot Dprime}{s} \right) \\ maxvs \leftarrow 0.25 \cdot \sqrt{\frac{fc}{1000}} \cdot Ae \\ a \leftarrow vs \ \text{ if } \ vs \le maxvs \\ a \leftarrow maxvs \ \text{ if } \ vs > maxvs \\ a \end{array} \right.$$

$$Vs := \text{vsprogram}(n, Asp, fyh, Dprime, s, fc, Ae) = 292.168 \quad \text{kips}$$

Eq. 8.6.1-2

$$\phi Vn := \phi_s \cdot (Vs + Vc) = 482.405 \quad \text{kips}$$

$$\text{ShearCheck}(\phi Vn, V_u) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \ \text{ if } \ \phi Vn \ge V_u \\ a \leftarrow \text{"FAILURE"} \ \text{ if } \ \phi Vn < V_u \\ a \end{array} \right.$$

$$\text{Shearcheck} := \text{ShearCheck}(\phi Vn, V_u) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

***Article 8.6.5*: Minimum Shear Reinforcement**

For Circular Columns

Eq. 8.6.5-1

$$\text{mintranprogram}(\rho_s) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \ \text{ if } \ \rho_s \ge 0.003 \\ a \leftarrow \text{"Increase Shear Reinforcing Ratio"} \ \text{ if } \ \rho_s < 0.003 \\ a \end{array} \right.$$

$$\text{CheckTransverse} := \text{mintranprogram}(\rho_s) = \text{"OK"}$$

Note: If the minimum shear reinforcement program responses "Increase Shear Reinforcing Ratio", it is recommended to <u>decrease the spacing (s)</u> or <u>increase the area of the shear reinforcement (Asp)</u> in the inputs.

***Article 8.8*: Longitudinal and Lateral Reinforcement Requirements**

***Article 8.8.1*: Maximum Longitudinal Reinforcement**

INPUT $A_{bl} := 1.56 \quad in^2$

INPUT $NumberBars := 12$

$A_{long} := NumberBars \cdot A_{bl} = 18.72 \quad in^2$

Eq. 8.8.1-1
$$\rho program(A_{long}, Ag) := \begin{array}{|l} a \leftarrow \text{"OK"} \quad \text{if } A_{long} \le 0.04 \cdot Ag \\ a \leftarrow \text{"Section Over Reinforced"} \quad \text{if } A_{long} > 0.04 \cdot Ag \\ a \end{array}$$

$$ReinforcementRaitoCheck := \rho program(A_{long}, Ag) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either <u>increase the section size (Ag)</u> or <u>decrease the longitudinal reinforcing (Abl and NumberBars)</u> in the inputs.

***Article 8.8.2*: Minimum Longitudinal Reinforcement**

For Columns in SDC B and C:

Eq. 8.8.2-1
$$minAlprogram(A_l, Ag) := \begin{array}{|l} a \leftarrow \text{"OK"} \quad \text{if } A_{long} \ge 0.007 \cdot Ag \\ a \leftarrow \text{"Increase Longitudinal Reinforcing"} \quad \text{if } A_{long} < 0.007 \cdot Ag \\ a \end{array}$$

$$MinimumA_l := minAlprogram(A_{long}, Ag) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars</u> in the inputs.

***Article 8.8.9*: Requirements for Lateral Reinforcement for SDCs B,C, and D**

These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension **of NOT** less than **6*d_b** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Minimum Size of Lateral Reinforcement

#4 bars for #9 or smaller longitudinal bars
#5 bars for #10 or larger longitudinal bars
#5 bars for bundled longitudinal bars

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

Shall Not Exceed the Smallest of:

$$\text{Spacingprogram}(\text{Columndia}, d_{bl}) := \left| \begin{array}{l} q \leftarrow \left(\frac{1}{5}\right)\text{Columndia} \\ r \leftarrow 6 \cdot d_{bl} \\ t \leftarrow 6 \\ a \leftarrow \min(q, r, t) \\ a \end{array} \right.$$

$$\text{MaximumSpacing} := \text{Spacingprogram}(\text{Columndia}, d_{bl}) = 6 \qquad \text{in}$$

$$\text{SpacingCheck(MaximumSpacing}, s) := \left| \begin{array}{l} a \leftarrow s \ \ \text{if} \ \ s \le \text{MaximumSpacing} \\ a \leftarrow \text{MaximumSpacing} \ \ \text{if} \ \ s > \text{MaximumSpacing} \\ a \end{array} \right.$$

$$\text{FINALSPACING} := \text{SpacingCheck(MaximumSpacing}, s) = 6 \quad \text{in}$$

$$\text{scheck} := \text{ShearCheck(MaximumSpacing}, s) = \text{"OK"}$$

Note: If scheck returns "Failure", increase the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.3 (LRFD SPEC.): Column Connections

Note: This needs to be done whenever the column dimension changes. The spacing in the hinge region shall continue into the drilled shaft or cap beam the Extension length.

$$\text{ExtensionProgram}(d) := \left| \begin{array}{l} z \leftarrow 15 \\ x \leftarrow \frac{1}{2} \cdot d \\ a \leftarrow \max(z, x) \\ a \end{array} \right.$$

INPUT $\text{Extension} := \text{ExtensionProgram(Columndia)} = 21 \quad \text{in}$

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.
Refer to the **AASHTO LRFD Bridge Design Specifications**.

***5.8.3.3* Nominal Shear Resistance**

$V_p = 271.289$ kips

INPUT $\text{spaceNOhinge} := 12$ in

INPUT $\text{bv} := \text{Columndia}$

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 34.67 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 32.036 \quad \text{in}$$

$$dv := 0.9 \cdot de = 28.832 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 153.065 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 89.38 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 218.2 \quad \text{kips}$$

$$\text{ShearCheck}\left(\phi Vn, V_u\right) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } \phi Vn \geq V_u \\ a \leftarrow \text{"FAILURE"} \quad \text{if } \phi Vn < V_u \\ a \end{array} \right.$$

$$\text{Shearcheck} := \text{ShearCheck}\left(\phi Vn, V_p\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1

$$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.531 \qquad in^2$$

$$Av := 2 \cdot Asp = 0.62 \qquad in^2$$

$$TranCheck(Avmin, Av) := \left| \begin{array}{l} a \leftarrow \text{"Decrease Spacing or Increase Bar Size"} \quad \text{if } Avmin > Av \\ a \leftarrow \text{"OK"} \quad \text{if } Avmin \le Av \\ a \end{array} \right.$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 Maximum Spacing of Transverse Reinforcement

Eq. 5.8.2.9-1

$$vu := \frac{V_u}{\phi_s \cdot bv \cdot dv} = 0.249 \qquad ksi$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2

$$spacingProgram(Vu, dv, fc) := \left| \begin{array}{l} v \leftarrow 0.125 \cdot \frac{fc}{1000} \\ q \leftarrow 0.8 \cdot dv \\ r \leftarrow 0.4 \cdot dv \\ z \leftarrow q \quad \text{if } q \le 24 \\ z \leftarrow 24 \quad \text{if } q > 24 \\ t \leftarrow r \quad \text{if } r \le 12 \\ t \leftarrow 12 \quad \text{if } r > 12 \\ a \leftarrow z \quad \text{if } Vu < v \\ a \leftarrow t \quad \text{if } Vu \ge v \\ a \end{array} \right.$$

$$MaxSpacing := spacingProgram(vu, dv, fc) = 23.066 \qquad in$$

$$\text{Spacecheck}(\text{MaxSpacing}, s) := \left| \begin{array}{l} a \leftarrow s \ \text{ if } \ s \le \text{MaxSpacing} \\ a \leftarrow \text{MaxSpacing} \ \text{ if } \ s > \text{MaxSpacing} \\ a \end{array} \right.$$

$$\text{MAXSPACING} := \text{Spacecheck}(\text{MaxSpacing}, \text{spaceNOhinge}) = 12 \qquad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MAXSPACING}, \text{spaceNOhinge}) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

BENT 3 DESIGN

***Article 4.11.1-4*: Steps to find Moment Capacity, Shear Capacity, and Axial Force**

Note: Use some kind of software to find the Moment Capacity of the Column. PCA Column was used to create an Interaction Diagram and to calculate the Moment Capacity. The shear for the bent was found by knowing the Moment.

INPUT $M_p := 2929920 \quad \text{lb}\cdot\text{in}$

INPUT $\text{Fixity} := 310\ \text{in}$

Note: Fixity is the point of fixity for the column/drilledshaft.

$$V_p := \frac{2 \cdot M_p}{\text{Fixity} \cdot 1000} = 18.903 \quad \text{kips} \qquad V_{pBent3} := 2 \cdot V_p = 37.805 \quad \text{kips}$$

Note: If the decision is made to design for **ELASTIC FORCES** then the above variables need to be override. This can be done by simply changing the Vp variable to the elastic force from SAP2000 that has been multiplied by pe/po.

Note: Pu is the combination of Elastic Axial force from the earthquake and dead load.

INPUT $P_u := 417000 \quad \text{lb}$

Article 8.6: Shear Demand and Capacity for Ductile Concrete Members

Note: It is recommended to use the plastic hinging forces whenever practical.

$V_u := V_p \qquad \phi_s := 0.9$

Article 4.11.6: **Analytical Plastic Hinge Length**

Note: For reinforced concrete columns framing into a footing, an integral bent cap, an oversized shaft, or cased shaft.

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of Longitudinal Bar

$L_p := \text{PlasticHinge}(\text{Fixity}, \text{fye}, d_{bl}) = 37.49$ in

Article 4.11.7: **Reinforced Concrete Column Plastic Hinge Region**

Note: y is the region of column with a moment demand exceeding 75% of the maximum plastic moment. From the SAP model, find the location at which the moment demand is 0.75*Mp. The 0.75*Mp value should be divided by p_{eTran} to take into account the model loads have not been multiplied by p_{eTran}. The location will also need to be **INPUT** into the PlasticHingeRegion program in inches.

$Mp75 := 0.75 \cdot M_p = 2.197 \times 10^6$ lb·in

$$\text{PlasticHingeRegion}(L_p, \text{Columndia}) := \left| \begin{array}{l} z \leftarrow 1.5 \cdot \text{Columndia} \\ x \leftarrow L_p \\ y \leftarrow 0 \\ a \leftarrow \max(z, x, y) \end{array} \right.$$

Note: Input the arguments into the program. Most likely the column diameter is the only variable that has changed.

INPUT $\text{Lpr} := \text{PlasticHingeRegion}(L_p, \text{Columndia}) = 63$ in

***Article 8.6.2*: Concrete Shear Capacity**

$Ag := Acolumn$

Eq. 8.6.2-2 $Ae := 0.8 \cdot Ag = 1.108 \times 10^{3}$ in^{2}

$\mu_D := 2$ Specified in Article 6.8.2 Guide Spec.

INPUT $s := 6$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $Asp := 0.31$ in^{2} Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.625$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 6$ in Cover: Concrete cover for the Column (in)

INPUT $Dprime := 30$ in Dprime: Diameter of spiral or hoop for circular columns (in)

Eq. 8.6.2-7 $\rho_s := \frac{4 \cdot Asp}{s \cdot Dprime} = 6.889 \times 10^{-3}$

$fyh := \frac{fye}{1000} = 60$ ksi

Eq. 8.6.2-6 $fs := StressCheck(\rho_s, fyh) = 0.413$

Eq. 8.6.2-5 $\alpha Prime := \alpha program(fs, \mu_D) = 3$

If Pu is Compressive

Eq. 8.6.2-4

If Pu is **NOT** Compressive $VC = 0$

Note: If Pu is not compressive, will have to manually input 0 for vc. Just input it below the vc:=vcprogram and the variable will assume the new value.

$vc := vcprogram(\alpha Prime, fc, P_u, Ag) = 0.22$ ksi

$Vc := vc \cdot Ag = 304.797$ kips

***Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity**

INPUT $n := 2$ n: number of individual interlocking spiral or hoop core sections

Eq. 8.6.3-1
Eq. 8.6.4-1 $Vs := vsprogram(n, Asp, fyh, Dprime, s, fc, Ae) = 292.168$ kips

Eq. 8.6.1-2 $\phi Vn := \phi_s \cdot (Vs + Vc) = 537.269$ kips

$Shearcheck := ShearCheck(\phi Vn, V_u) = \text{"OK"}$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

***Article 8.6.5*: Minimum Shear Reinforcement**

For Circular Columns

Eq. 8.6.5-1 $CheckTransverse := mintranprogram(\rho_s) = \text{"OK"}$

Note: If the minimum shear reinforcement program responses "Increase Shear Reinforcing Ratio", it is recommended to <u>decrease the spacing (s)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

***Article 8.8*: Longitudinal and Lateral Reinforcement Requirements**

***Article 8.8.1*: Maximum Longitudinal Reinforcement**

INPUT $A_{bl} := 1.56$ in^2

INPUT $NumberBars := 12$

$A_{long} := NumberBars \cdot A_{bl} = 18.72$ in^2

Eq. 8.8.1-1 $ReinforcementRaitoCheck := \rho program(A_{long}, Ag) = \text{"OK"}$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either <u>increase the section size (Ag)</u> or <u>decrease the longitudinal reinforcing (Abl and NumberBars)</u> in the inputs.

***Article 8.8.2*: Minimum Longitudinal Reinforcement**

For Columns in SDC B and C:

Eq. 8.8.2-1 $\text{MinimumA}_l := \text{minAlprogram}(A_{long}, Ag) = \text{"OK"}$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars)</u> in the inputs.

***Article 8.8.9*: Requirements for Lateral Reinforcement for SDCs B,C, and D**

These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension **of NOT** less than **6*d_b** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Minimum Size of Lateral Reinforcement

#4 bars for #9 or smaller longitudinal bars
#5 bars for #10 or larger longitudinal bars
#5 bars for bundled longitudinal bars

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

Shall Not Exceed the Smallest of:

$$\text{MaximumSpacing} := \text{Spacingprogram}(\text{Columndia}, d_{bl}) = 6 \quad \text{in}$$

$$\text{FINALSPACING} := \text{SpacingCheck}(\text{MaximumSpacing}, s) = 6 \quad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MaximumSpacing}, s) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.3 (LRFD SPEC.): Column Connections

Note: This needs to be done whenever the column dimension changes. The spacing in the hinge region shall continue into the drilled shaft or cap beam the Extension length.

INPUT $\text{Extension} := \text{ExtensionProgram}(\text{Columndia}) = 21$ in

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.
Refer to the **AASHTO LRFD Bridge Design Specifications**.

***5.8.3.3* Nominal Shear Resistance**

$V_p = 18.903$ kips

INPUT $\text{spaceNOhinge} := 12$ in

INPUT $\text{bv} := \text{Columndia}$

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$\beta := 2.0$

$\theta := \frac{\pi}{180} \cdot 45 = 0.785$ rad

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$\text{Dr} := \text{bv} - \text{Cover} - \text{Dsp} - \frac{d_{bl}}{2} = 34.67$ in

Eq. C5.8.2.9-2 $\text{de} := \frac{\text{bv}}{2} + \frac{\text{Dr}}{\pi} = 32.036$ in

$\text{dv} := 0.9 \cdot \text{de} = 28.832$ in

Eq. 5.8.3.3-3 $V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 153.065$ kips

Eq. 5.8.3.3-4 $V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 89.38$ kips

$\phi V_n := (V_c + V_s) \cdot \phi_s = 218.2$ kips

$Shearcheck := ShearCheck(\phi Vn, V_p) = \text{"OK"}$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.531$ in^2

$Av := 2 \cdot Asp = 0.62$ in^2

$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

***5.8.2.7* Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $\quad vu := \dfrac{V_u}{\phi_s \cdot bv \cdot dv} = 0.017 \quad$ ksi

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $\quad MaxSpacing := spacingProgram(vu, dv, fc) = 23.066 \quad$ in

$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \quad$ in

$scheck := ShearCheck(MAXSPACING, spaceNOhinge) =$ "OK"

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT DESIGN

Article 6.5: Drilled Shafts

NOTE: The guide specification states that the drilled shafts shall conform to the requirements of columns in SDC B, C, or D as applicable. Also, there are special provisions regarding liquefaction that needs to be investigated if this is a concern for a certain bridge.

Since the hinging will not occur in the drilled shaft, the drilled shaft will be design using the column design from the LRFD Specification.

ABUTMENT DRILLED SHAFT

Nominal Shear Resistance for members OUTSIDE Plastic Hinge Region.
Refer to the AASHTO LRFD Bridge Design Specifications.

***5.8.3.3* Nominal Shear Resistance**

INPUT $\quad V_p := 133 \quad$ kips $\quad$ NOTE: Abutment Drilled shaft is being designed for Elastic Force.

$V_u := V_p$

INPUT $\quad spaceNOhinge := 12 \quad$ in

INPUT $\quad Asp := 0.31 \quad in^2$

INPUT $\quad Cover := 6 \quad in^2$

INPUT $\quad bv := DSdia$

INPUT $\quad Dsp := 0.625 \quad in$

INPUT $\quad d_{bl} := 1.41 \quad in$

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad rad$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 34.67 \quad in$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 32.036 \quad in$$

$$dv := 0.9 \cdot de = 28.832 \quad in$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 153.065 \quad kips$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 89.38 \quad kips$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 218.2 \quad kips$$

$$Shearcheck := ShearCheck\left(\phi Vn, V_p\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $\quad Avmin := 0.0316 \cdot \sqrt{\dfrac{fc}{1000}} \cdot \dfrac{bv \cdot spaceNOhinge}{\dfrac{fye}{1000}} = 0.531 \quad in^2$

$Av := 2 \cdot Asp = 0.62 \quad in^2$

MinimumTran := TranCheck(Avmin, Av) = "OK"

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $\quad vu := \dfrac{V_u}{\phi_s \cdot bv \cdot dv} = 0.122 \quad ksi$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $\quad$ MaxSpacing := spacingProgram(vu, dv, fc) = 23.066 in

MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 in

scheck := ShearCheck(MAXSPACING, spaceNOhinge) = "OK"

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

BENT 2 DRILLED SHAFT

Nominal Shear Resistance for members OUTSIDE Plastic Hinge Region.
Refer to the AASHTO LRFD Bridge Design Specifications.

5.8.3.3 **Nominal Shear Resistance**

INPUT $\quad V_p := \dfrac{V_{pBent2}}{2} = 271.289 \quad kips$

$V_u := V_p$

INPUT $\text{spaceNOhinge} := 12$ in

INPUT $\text{Asp} := 0.31$ in^2

INPUT $\text{Cover} := 6$ in^2

INPUT $\text{bv} := \text{DSdia}$

INPUT $\text{Dsp} := 0.625$ in

INPUT $d_{bl} := 1.41$ in

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$\text{Dr} := \text{bv} - \text{Cover} - \text{Dsp} - \frac{d_{bl}}{2} = 34.67 \quad \text{in}$$

Eq. C5.8.2.9-2 $$\text{de} := \frac{\text{bv}}{2} + \frac{\text{Dr}}{\pi} = 32.036 \quad \text{in}$$

$$\text{dv} := 0.9 \cdot \text{de} = 28.832 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{\text{fc}}{1000}} \text{bv} \cdot \text{dv} = 153.065 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2\text{Asp} \cdot \frac{\text{fye}}{1000} \text{dv} \cdot \cot(\theta)}{\text{spaceNOhinge}} = 89.38 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 218.2 \quad \text{kips}$$

$$\text{Shearcheck} := \text{ShearCheck}\left(\phi\text{Vn}, V_p\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.531 \qquad in^2$$

$$Av := 2 \cdot Asp = 0.62 \qquad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{V_u}{\phi_s \cdot bv \cdot dv} = 0.249 \qquad ksi$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $$MaxSpacing := spacingProgram(vu, dv, fc) = 23.066 \qquad in$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \qquad in$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

BENT 3 DRILLED SHAFT

Nominal Shear Resistance for members OUTSIDE Plastic Hinge Region.
Refer to the AASHTO LRFD Bridge Design Specifications.

5.8.3.3 **Nominal Shear Resistance**

<u>*INPUT*</u> $$V_p := \frac{V_{pBent3}}{2} = 18.903 \qquad kips$$

$$V_u := V_p$$

INPUT $\text{spaceNOhinge} := 12$ in

INPUT $\text{Asp} := 0.31$ in^2

INPUT $\text{Cover} := 6$ in^2

INPUT $\text{bv} := \text{DSdia}$

INPUT $\text{Dsp} := 0.625$ in

INPUT $d_{bl} := 1.41$ in

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$\text{Dr} := \text{bv} - \text{Cover} - \text{Dsp} - \frac{d_{bl}}{2} = 34.67 \quad \text{in}$$

Eq. C5.8.2.9-2 $$\text{de} := \frac{\text{bv}}{2} + \frac{\text{Dr}}{\pi} = 32.036 \quad \text{in}$$

$$\text{dv} := 0.9 \cdot \text{de} = 28.832 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{\text{fc}}{1000}} \text{bv} \cdot \text{dv} = 153.065 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2\text{Asp} \cdot \frac{\text{fye}}{1000} \text{dv} \cdot \cot(\theta)}{\text{spaceNOhinge}} = 89.38 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 218.2 \quad \text{kips}$$

$$\text{Shearcheck} := \text{ShearCheck}\left(\phi\text{Vn}, V_p\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.531 \qquad in^2$$

$$Av := 2 \cdot Asp = 0.62 \qquad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

***5.8.2.7* Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{V_u}{\phi_s \cdot bv \cdot dv} = 0.017 \qquad ksi$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $$MaxSpacing := spacingProgram(vu, dv, fc) = 23.066 \qquad in$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \qquad in$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

CONNECTION DESIGN FOR BENT/ABUTMENT TO GIRDER

Bent 3 Connection Design

Note: Also use for Bent 2 connection.

INPUT $Vcolbent := V_{pBent2} = 542.578$

INPUT $Ngirderperbent := 8$ Ngirderbent = Number of girders per bent

Article 6.5.4.2: Resistance Factors

$\phi_t := 0.8$	Tension for A307
$\phi_s := 0.75$	Shear for A307
$\phi_{bs} := 0.80$	Block Shear
$\phi_{bb} := 0.80$	Bolts Bearing
$\phi_{sc} := 0.85$	Shear Connectors
$\phi_f := 1.00$	Flexure
$\phi_{sangle} := 1.00$	Shear for the Angle

For Type III Girders

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.25$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT $Fy := 36$ ksi Fy = Yield Stress of the Angle

INPUT $Fu := 58$ ksi Fu = Ultimate Stress of the Angle

INPUT $t := 0.875$ in t = Thickness of Angle

INPUT $h := 6$ in h = Height of the Angle

INPUT	$w := 6$	in	w = Width of the Angle
INPUT	$l := 16$	in	l = Length of the Angle
INPUT	$k := 1.375$	in	k = Height of the Bevel
INPUT	$distanchorhole := 4$	in	distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.
INPUT	$diahole := 1.75$	in	diahole = Diameter of bolt hole
INPUT	$BLSHlength := 11$	in	BLSHlength = Block Shear Length
INPUT	$BLSHwidth := 2$	in	BLSHwidth = Block Shear Width
INPUT	$Ubs := 1.0$		Ubs = Shear Lag Factor for Block Shear
INPUT	$a := 2$	in	a = Distance from the center of the bolt to the edge of plate
INPUT	$b := 3.5$	in	b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent}{2Ngirderperbent} = 33.911 \qquad \text{kips}$$

Shear Force per Bolt

$n := 2$ n = Number of bolts

$$Vbolt := \frac{Vangle}{n} = 16.956 \qquad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\ 2}}{4} = 1.227 \qquad \text{in}^2$$

Eq. 6.13.2.12-1 $\phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 25.624$ kips

$Shearcheck := ShearCheck(\phi sRn, Vbolt) = \text{"OK"}$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 152.25$ kips

For Slotted Holes

INPUT $Lc := 2$ in Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\phi bbRns := Lc \cdot t \cdot Fub = 101.5$ kips

$$Bearingcheck := ShearCheck(\phi bbRn, Vbolt) = \text{"OK"}$$

$$Bearingscheck := ShearCheck(\phi bbRns, Vbolt) = \text{"OK"}$$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 8.478 \quad \text{kips}$$

Eq. 6.13.2.10.2-1 $\phi tTn := \phi_t \cdot 0.76 \cdot A_b \cdot Fub = 43.275$ kips

$$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$Pu := Vbolt$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2

$$CombinedProgram(Pu, A_b, Fub, \phi sRn, \phi_s) := \begin{array}{|l} t \leftarrow 0.76 \cdot A_b \cdot Fub \\ r \leftarrow 0.76 \cdot A_b \cdot Fub \cdot \sqrt{1 - \left(\frac{Pu}{\phi sRn}\right)^2} \\ a \leftarrow t \ \text{ if } \frac{Pu}{\left(\frac{\phi sRn}{\phi_s}\right)} \le 0.33 \\ a \leftarrow r \ \text{ if } \frac{Pu}{\left(\frac{\phi sRn}{\phi_s}\right)} > 0.33 \\ a \end{array}$$

$Tn_{combined} := CombinedProgram(Pu, A_b, Fub, \phi sRn, \phi_s) = 40.557$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 32.446$ kips

$Combinedcheck := ShearCheck(\phi tTn_{combined}, Vbolt) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$Agv := t \cdot BLSHlength = 9.625$ in^2

$Anv := t \cdot (BLSHlength - 1.5 \cdot diahole) = 7.328$ in^2

$Ant := t \cdot (BLSHwidth - 1.5 \cdot diahole) = -0.547$ in^2

Note this is for if there are two through bolts in the upper leg.

(J4-5)

$$BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) := \begin{array}{|l} b \leftarrow 0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant \\ c \leftarrow 0.6Fy \cdot Agv + Ubs \cdot Fu \cdot Ant \\ a \leftarrow b \ \text{ if } b \le c \\ a \leftarrow c \ \text{ if } b > c \\ a \end{array}$$

$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 176.181$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 140.945$ kips

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = "OK"$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$ — Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 3.719$ in^2

(D3-1) $Ae := Ant \cdot Ut = 2.231$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 103.53$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) = "OK"$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

INPUT $Muangle := 74.26$ kip·in

Note: This is assuming we need the same size anchor bolt in the top as we do the bottom.

$$Zx := \frac{1 \cdot (t)^2}{4} = 3.063 \quad in^3$$

$\phi fMn := \phi_f \cdot Fy \cdot Zx = 110.25$ kip·in

Had to increase the thickness of the angle to 7/8 in. or can increase the length.

$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = "OK"$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$Cv := 1.0$

$Aw := t \cdot w = 5.25$ in^2

(G2-1) $\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 113.4$ kips

$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Bent 2 Expansion Connection Design

INPUT $Vcolbent := V_{pBent2} = 542.578$

INPUT $Ngirderperbent := 8$ Ngirderbent = Number of girders per bent

Article 6.5.4.2: Resistance Factors

$\phi_t := 0.8$	Tension for A307
$\phi_s := 0.75$	Shear for A307
$\phi_{bs} := 0.80$	Block Shear
$\phi_{bb} := 0.80$	Bolts Bearing
$\phi_{sc} := 0.85$	Shear Connectors
$\phi_f := 1.00$	Flexure
$\phi_{sangle} := 1.00$	Shear for the Angle

For Type III Girders

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.25$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT $Fy := 36$ ksi Fy = Yield Stress of the Angle

INPUT $Fu := 58$ ksi Fu = Ultimate Stress of the Angle

INPUT $t := 0.875$ in t = Thickness of Angle

INPUT $h := 6$ in h = Height of the Angle

INPUT $w := 6$ in w = Width of the Angle

INPUT $l := 20$ in l = Length of the Angle

INPUT $k := 1.375$ in k = Height of the Bevel

INPUT $distanchorhole := 4$ in distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT $diahole := 1.75$ in diahole = Diameter of bolt hole

INPUT $SlottedHole := 6$ in SlottedHole = Length of slotted hole

INPUT $BLSHlength := 15$ in BLSHlength = Block Shear Length

INPUT $BLSHwidth := 2$ in BLSHwidth = Block Shear Width

INPUT $Ubs := 1.0$ Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent}{2Ngirderperbent} = 33.911 \quad \text{kips}$$

Shear Force per Bolt

$n := 2$ n = Number of bolts

$$Vbolt := \frac{Vangle}{n} = 16.956 \quad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 1.227 \quad \text{in}^2$$

Eq. 6.13.2.12-1 $\phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 25.624$ kips

$Shearcheck := ShearCheck(\phi sRn, Vbolt) = \text{"OK"}$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 152.25$ kips

For Slotted Holes

INPUT $Lc := 2$ in Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\phi bbRns := Lc \cdot t \cdot Fub = 101.5$ kips

$Bearingcheck := ShearCheck(\phi bbRn, Vbolt) = \text{"OK"}$

$Bearingscheck := ShearCheck(\phi bbRns, Vbolt) = \text{"OK"}$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 8.478 \quad \text{kips}$$

Eq. 6.13.2.10.2-1 $\phi tTn := \phi_t \cdot 0.76 \cdot A_b \cdot Fub = 43.275$ kips

$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$Pu := Vbolt$

$Tn_{combined} := CombinedProgram(Pu, A_b, Fub, \phi sRn, \phi_s) = 40.557$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 32.446$ kips

$Combinedcheck := ShearCheck(\phi tTn_{combined}, Vbolt) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$Agv := t \cdot BLSHlength = 13.125$ in^2

$Anv := t \cdot \left(BLSHlength - 1.5 \cdot \frac{SlottedHole}{2}\right) = 9.188$ in^2

Note this is for if there are two through bolts in the upper leg.

$Ant := t \cdot (BLSHwidth - 1.5 \cdot diahole) = -0.547$ in^2

$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 251.781$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 201.425$ kips

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 3.719$ in^2

(D3-1) $Ae := Ant \cdot Ut = 2.231$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 103.53$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

Note: This is assuming we need the same size anchor bolt in the top as we do the bottom.

INPUT $\quad Muangle := 74.26 \ \ kip \cdot in$

$$Zx := \frac{1 \cdot (t)^2}{4} = 3.828 \qquad in^3$$

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 137.813 \qquad kip \cdot in$$

Had to increase the thickness of the angle to 7/8 in. or can increase the length.

$$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t \cdot w = 5.25 \qquad in^2$$

(G2-1) $\quad \phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 113.4 \qquad kips$

$$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Abutment to Girder Connection

INPUT $Vcolbent := 102$ kips

INPUT $Ngirderperbent := 4$ Ngirderbent = Number of girders per bent

For Type III Girders

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.25$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT $Fy := 36$ ksi Fy = Yield Stress of the Angle

INPUT $Fu := 58$ ksi Fu = Ultimate Stress of the Angle

INPUT $t := 0.75$ in t = Thickness of Angle

INPUT $h := 6$ in h = Height of the Angle

INPUT $w := 6$ in w = Width of the Angle

INPUT $l := 12$ in l = Length of the Angle

INPUT $k := 1.25$ in k = Height of the Bevel

INPUT $distanchorhole := 4$ in distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT $diahole := 1.5$ in diahole = Diameter of bolt hole

INPUT $SlottedHole := 6$ in SlottedHole = Length of Slotted hole

INPUT $BLSHlength := 6$ in BLSHlength = Block Shear Length

INPUT $BLSHwidth := 2$ in BLSHwicth = Block Shear Width

INPUT $Ubs := 1.0$ Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent}{2Ngirderperbent} = 12.75 \qquad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\ 2}}{4} = 1.227 \qquad in^2$$

Eq. 6.13.2.12-1 $\quad \phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 25.624 \qquad \text{kips}$

Note: This is checking to verify that the anchor bolt has enough shear strength.

$$Shearcheck := ShearCheck(\phi sRn, Vangle) = \text{"OK"}$$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\quad \phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 130.5 \qquad \text{kips}$

For Slotted Holes

Eq. 6.13.2.9.3 $\quad \phi bbRns := 2.0 \cdot Dia_b \cdot t \cdot Fub = 108.75 \qquad \text{kips}$

$$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$$

$$Bearingscheck := ShearCheck(\phi bbRns, Vangle) = \text{"OK"}$$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 3.188 \qquad \text{kips}$$

Eq. 6.13.2.10.2-1 $\phi tTn := 0.76 \cdot A_b \cdot Fub = 54.094$ kips

$$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$$Pu := Vangle$$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2 $Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 46.922$ kips

$$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 37.538 \qquad \text{kips}$$

$$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vangle\right) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$Agv := t \cdot BLSHlength = 4.5$ in^2

$Anv := t \cdot \left(BLSHlength - 0.5 \cdot \frac{SlottedHole}{2}\right) = 3.375$ in^2

Note this is for if there are one through bolts in the upper leg.

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 0.938$ in^2

(J4-5) $Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 151.575$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 121.26$ kips

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 3.375$ in^2

(D3-1) $Ae := Ant \cdot Ut = 2.025$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 93.96$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

INPUT $Muangle := 44.52 \quad kip \cdot in$

Note: This is assuming we need the same size anchor bolt in the top as we do the bottom.

$$Zx := \frac{1 \cdot (t)^2}{4} = 1.688 \qquad in^3$$

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 60.75 \qquad kip \cdot in$$

Had to increase the thickness of the angle to 3/4 in. or can increase the length.

$$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t \cdot w = 4.5 \qquad in^2$$

(G2-1) $$\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 97.2 \qquad kips$$

$$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Appendix B: Oseligee Creek Bridge LRFD Specification Design

Designer: Paul Coulston
Project Name: Oseligee Creek Bridge
Job Number:
Date: 11/2/2010

ORIGIN := 1

Description of worksheet: This worksheet is a seismic bridge design worksheet for the **AASHTO LRFD Bridge Design Specification**. All preliminary design should already be done for non-seismic loads.

Project Known Information

Location: Chambers County
Zip Code or Coordinates: 35.0069 N 88.2025 W

Superstructure Type: AASTHO I girders
Substructure Type: Circular columns supported on drilled shafts
Abutment Type: Abutment beam supported on drilled shafts

Note: ***Input** all of the below information.*

$fc := 4000$ psi

$fye := 60000$ psi

$\rho_{conc} := 0.08681 \ \frac{lb}{in^3}$

$g := 386.4 \ \frac{in}{s^2}$

Length of Bridge (ft)	$L := 240$	ft
Span (ft)	$Span := 80$	ft
Deck Thickness (in)	$t_{deck} := 7$	in
Deck Width (ft)	$DeckWidth := 32.75$	ft
I-Girder X-Sectional Area (in^2)	$IGirderArea := 559.5$	in^2
Guard Rail Area (in^2)	$GuardRailArea := 310$	in^2
Bent Volume (ft^3)	$BentVolume := 5 \cdot 4 \cdot 30 = 600$	ft^3
Column Diameter (in)	$Columndia := 42$	in
Drilled Shaft Diameter (in)	$DSdia := 42$	in
Drilled Shaft Abutment Diameter (in)	$DSabutdia := 42$	in
Average Column Height (ft)	$ColumnHeight := 22$	ft

Column Area (in^2) $$Acolumn := \frac{Columndia^2 \cdot \pi}{4} = 1.385 \times 10^3 \quad in^2$$

Drilled Shaft Area (in^2) $$Adrilledshaft := \frac{DSdia^2 \pi}{4} = 1.385 \times 10^3 \quad in^2$$

Drilled Shaft Abutment Area (in^2) $$Adsabut := \frac{DSabutdia^2 \cdot \pi}{4} = 1.385 \times 10^3 \quad in^2$$

Note: These are variables that were easier to input in ft and then convert to inches.

$Span := Span \cdot 12 = 960$ in

$L := L \cdot 12 = 2.88 \times 10^3$ in

$DeckWidth := DeckWidth \cdot 12 = 393$ in

$BentVolume := BentVolume \cdot 12^3 = 1.037 \times 10^6$ in^3

$ColumnHeight := ColumnHeight \cdot 12 = 264$ in

Steps for Seismic Design

Use Appendix A3 Seismic Design Flowcharts

Description of Difference from Guide Specification
The LRFD Specification is a force based approach to design verses the Guide Specification which is a displacement based approach. The LRFD Specification has an Response Modification Factor (R) that will be used in calculating the loads applied to the structure.

Article 3.10.1: Earthquake Effects - This is just the applicability of the Specifications.

***Article 3.10.2*: Determine Design Response Spectrum**

Note: AASHTO Guide Specifications for LRFD Seismic Bridge Design is accompanied with a program that has the Seismic Hazard maps. This program will calculate several of the variables that are needed for the analysis.

1) *Article 3.10.3.1*:Determine the Site Class.

INPUT Site Class: **D**

2) Enter maps and find PGA, S_s,and S_1. Then enter those values in their respective spot.

$PGA := 0.116\ g$

INPUT $S_s := 0.272\ g$

$S_1 := 0.092\ g$

3) *Article 3.10.3.2:* Site Coefficients. From the PGA,S_s, and S_1 values and site class choose F_{PGA}, F_a, and F_v. Note: straight line interpolation is permitted.

$F_{PGA} := 1.57$

INPUT $F_a := 1.58$

$F_v := 2.4$

$A_s := F_{PGA} \cdot PGA = 0.182\ g$ — A_s : Acceleration Coefficient

$SDS := F_a \cdot S_s = 0.43\ g$ — S_{DS} = Short Period Acceleration Coefficient

$SD1 := F_v \cdot S_1 = 0.221\ g$ — S_{D1} = 1-sec Period Acceleration Coefficient

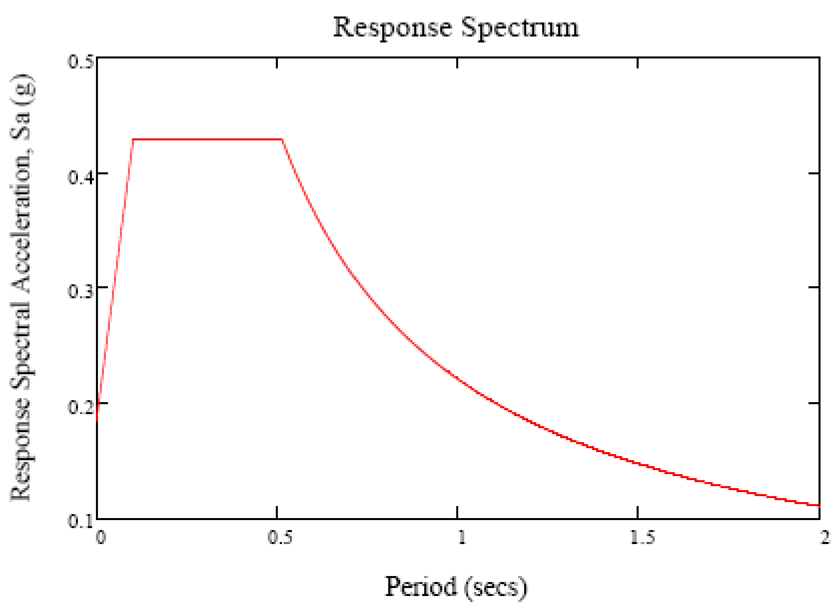
4) Creating a Response Spectrum

Note: The DesignSpectrum Code creates a Response Spectrum for the Bridge. At this time the period of the bridge is unknown; therefore, the Sa value cannot be calculated.

$Tmax := 2\ s \qquad Dt := 0.001\ s$

$$
\begin{array}{l}
DesignSpectrum(SDS, SD1, A_s, Tmax, Dt) := \left|
\begin{array}{l}
T_s \leftarrow \dfrac{SD1}{SDS} \\
T_o \leftarrow 0.2 \cdot T_s \\
n_{max} \leftarrow \dfrac{Tmax}{Dt} \\
\text{for } i \in 1 .. n_{max} \\
\quad \left| \begin{array}{l}
T_i \leftarrow Dt \cdot i \\
a_i \leftarrow (SDS - A_s) \cdot \dfrac{Dt \cdot i}{T_o} + A_s \ \text{ if } Dt \cdot i < T_o \\
a_i \leftarrow SDS \ \text{ if } Dt \cdot i \ge T_o \wedge Dt \cdot i \le T_s \\
a_i \leftarrow \dfrac{SD1}{Dt \cdot i} \ \text{ if } Dt \cdot i > T_s
\end{array} \right. \\
R \leftarrow augment(T, a) \\
R
\end{array} \right.
\end{array}
$$

$BridgeSpectrum := DesignSpectrum(SDS, SD1, A_s, Tmax, Dt)$



***Article 3.10.6*: Selection of Seismic Performance Zones**

$SD1 = 0.221 \quad g$ From Table 3.10.6-1 Choose SPZ

$$SDCprogram(SD1) := \left| \begin{array}{l} \text{for } c \in SD1 \\ \quad \left| \begin{array}{l} c \leftarrow \text{"1"} \ \text{ if } SD1 \le 0.15 \\ c \leftarrow \text{"2"} \ \text{ if } SD1 > 0.15 \wedge SD1 \le 0.3 \\ c \leftarrow \text{"3"} \ \text{ if } SD1 > 0.3 \wedge SD1 \le 0.5 \\ c \leftarrow \text{"4"} \ \text{ if } SD1 > 0.5 \end{array} \right. \\ Rs \leftarrow c \\ c \end{array} \right.$$

$SDC := SDCprogram(SD1) = \text{"2"}$

Article 3.10.5: Bridge Importance Category

Operational Classified: **Other bridges**

Article 3.10.7: Response Modification Factors

For Substructures: Table 3.10.7.1-1

INPUT Multiple Column Bents $R_{sub} := 5.0$

For Connections: Table 3.10.7.1-2

INPUT Superstructure to Abutment $R_{abutment} := 0.8$

INPUT Columns to Bent Cap $R_{columncap} := 1.0$

INPUT Column to foundation $R_{foundation} := 1.0$

Article 4.7.4.3: Multispan Bridges

Article 4.7.4.3.1 Selection of Method

Refer to Table 4.7.4.3.1-1 to select the required analysis procedure. This a function of seismic performance zone, regularity, and operational classification. For the worst case in Alabama, we can use either the Uniform Load Elastic Method or Single-Mode Elastic Method.

Note: There are two methods that can be used according to this procedure. The Uniform Load Method is suitable for regular bridges that respond principally in their fundamental mode of vibration. The Single Mode Spectral Method may be a better method if there is a major change in the spans, stiffness of the piers, etc.
The Uniform Load Method is simpler and less time consuming and will give accurate results, and this is the reason it has been chosen in this design.

Article 4.7.4.3.2c **Uniform Load Method**

Step 1: Build a bridge model
Step 2: Apply a uniform load of Po = 1.0 kip/in. in both the longitudinal and transverse direction. Also, the uniform load can be converted into point loads and applied as joint loads in SAP. Calculate the static displacement for both directions. In SAP, tables of the displacements can be exported to EXCEL, and the MAX Function can be used to find the maximum displacement.

Step 3: Calculate the bridge lateral stiffness, K, and total weight, W.

$$p_o := 1.0 \quad \frac{kip}{in}$$

INPUT

$$\upsilon_{smaxLong} := 1.671281 \quad in$$

$$\upsilon_{smaxTran} := 3.228449 \quad in$$

$$K_{Long} := \frac{p_o \cdot L}{\upsilon_{smaxLong}} = 1.723 \times 10^3 \qquad \frac{kip}{in}$$

$$K_{Tran} := \frac{p_o \cdot L}{\upsilon_{smaxTran}} = 892.069 \qquad \frac{kip}{in}$$

INPUT: Multiplying factors

$$W := \frac{\rho_{conc} \cdot \begin{pmatrix} L \cdot t_{deck} \cdot DeckWidth + 2 \cdot BentVolume + 4 \cdot Acolumn \cdot ColumnHeight \; ... \\ + L \cdot 4 \cdot IGirderArea + 2 \cdot GuardRailArea \cdot L \end{pmatrix}}{1000}$$

$$W = 1709.336 \qquad kips$$

Step 4: Calculate the period, T_m.

$$T_{mLong} := 2\pi \cdot \sqrt{\frac{W}{K_{Long} \cdot g}} = 0.318 \quad s$$

Step 5: Calculate equivalent static earthquake loading p_e.

$$acc(SDS, SD1, T_{mLong}, A_s) := \left| \begin{array}{l} T_s \leftarrow \frac{SD1}{SDS} \\ T_o \leftarrow 0.2 \cdot T_s \\ \text{for } a \in T_{mLong} \\ \quad \left| \begin{array}{l} a \leftarrow (SDS - A_s) \cdot \frac{T_{mLong}}{T_o} + A_s \ \text{ if } T_{mLong} < T_o \\ a \leftarrow SDS \ \text{ if } T_{mLong} \geq T_o \wedge T_{mLong} \leq T_s \\ a \leftarrow \frac{SD1}{T_{mLong}} \ \text{ if } T_{mLong} > T_s \end{array} \right. \\ Ra \leftarrow a \\ a \end{array} \right.$$

$$Csm_{Long} := acc(SDS, SD1, T_{mLong}, A_s) = 0.43$$

$$p_{eLong} := \frac{Csm_{Long} \cdot W}{L} = 0.255 \quad \frac{kip}{in}$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$v_{smaxLong} := \frac{p_{eLong}}{p_o} \cdot \upsilon_{smaxLong} = 0.426 \quad in$$

Repeat Steps 4, 5, and 6 for transverse loading.

Step 4: Calculate the period, T_m.

$$T_{mTran} := 2\pi \cdot \sqrt{\frac{W}{K_{Tran} \cdot g}} = 0.442 \quad s$$

Step 5: Calculate equivalent static earthquake loading p_e.

$$Csm_{Tran} := acc\left(SDS, SD1, T_{mTran}, A_s\right) = 0.43$$

$$p_{eTran} := \frac{Csm_{Tran} \cdot W}{L} = 0.255 \qquad \frac{kip}{in}$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$v_{smaxTran} := \frac{p_{eTran}}{p_o} \cdot \upsilon_{smaxTran} = 0.823 \qquad in$$

Single-Mode Spectral Method

Article 4.7.4.3.2: Single-Mode Spectral Method

Step 1: Build a bridge model
Step 2: Apply a uniform load of Po = 1.0 in both the longitudinal and transverse direction. Calculate the static displacement for both directions.
Step 3: Calculate factors α, β , and γ.

Note: The Deflection equations come from analysis of the SAP model. The displacement is taken at the joints along the length of the bridge and input into an Excel Worksheet. Then a graph is created of the displacements along the length of the bridge. A best fit line is plotted, and that is the equation that is shown below.

$$v_{stran}(x) := -1 \cdot 10^{-6} \cdot x^2 + 0.0034 \cdot x - 0.2945 \qquad v_{slong}(x) := -2 \cdot 10^{-8} \cdot x^2 + 6 \cdot 10^{-5} \cdot x + 1.5856$$

C4.7.4.3.2b-1 $\quad \alpha_{Tran} := \int_0^L v_{stran}(x)\,dx \qquad \alpha_{Long} := \int_0^L v_{slong}(x)\,dx$

C4.7.4.3.2b-2 $\quad \beta_{Tran} := \int_0^L \frac{W}{L} v_{stran}(x)\,dx \qquad \beta_{Long} := \int_0^L \frac{W}{L} \cdot v_{slong}(x)\,dx$

C4.7.4.3.2b-3 $\quad \gamma_{Tran} := \int_0^L \frac{W}{L} \cdot v_{stran}(x)^2\,dx = 6.739 \times 10^3 \qquad \gamma_{Long} := \int_0^L \frac{W}{L} \cdot v_{slong}(x)^2\,dx$

α = Displacement along the length

β = Weight per unit length * Displacement

γ = Weight per unit length * Displacement^2

Step 4: Calculate the Period of the Bridge

$$T_{mTran1} := 2\pi\cdot\sqrt{\frac{\gamma_{Tran}}{p_o\cdot g\cdot\alpha_{Tran}}} = 0.361 \qquad s$$

$$T_{mLong1} := 2\pi\cdot\sqrt{\frac{\gamma_{Long}}{p_o\cdot g\cdot\alpha_{Long}}} = 0.313 \qquad s$$

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smLong} := acc\left(SDS, SD1, T_{mLong1}, A_s\right) = 0.43$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$PeLong(x) := \frac{\beta_{Long}\cdot C_{smLong}}{\gamma_{Long}}\cdot\frac{W}{L}\cdot v_{slong}(x)$$

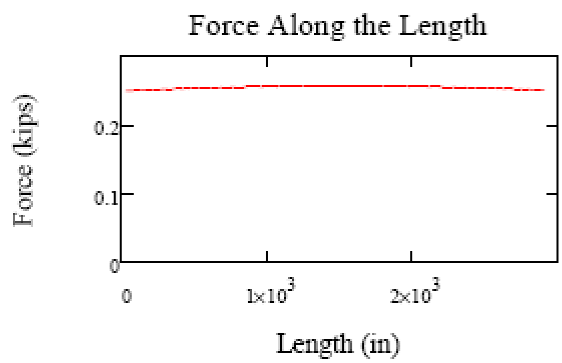
$$PeLong(x) \rightarrow 0.0000094657649785618823161\cdot x + -3.1552549928539607772e\text{-}9\cdot x^2 + 0.25014861583346201$$

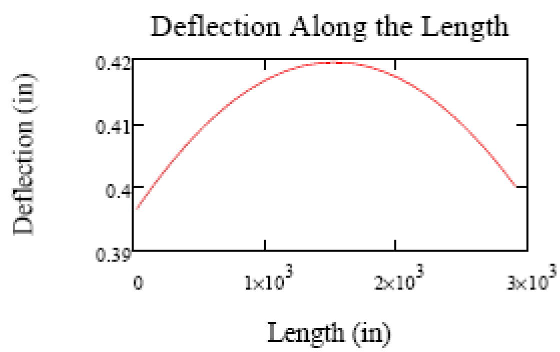
$$dW := \frac{L}{100}$$

$$i := 1\,..\,101$$

$$Pelong_i := PeLong[(i-1)\cdot dW] \qquad \delta long_i := v_{slong}[(i-1)dW]$$

$$\Delta long_i := Pelong_i\cdot\delta long_i$$





Maximum Deflection

$$\max(\Delta long) = 0.419 \quad \text{in}$$

NOTE: Repeat Steps 5 and 6 for Transverse Direction.

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smTran} := acc\left(SDS, SD1, T_{mTran1}, A_s\right) = 0.43$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$PeTran(x) := \frac{\beta_{Tran} \cdot C_{smTran}}{\gamma_{Tran}} \cdot \frac{W}{L} \cdot v_{stran}(x)$$

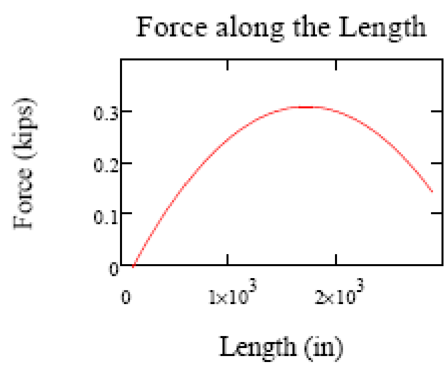
$$PeTran(x) \rightarrow 0.00040401585387083704979 \cdot x + -1.1882819231495207347e\text{-}7 \cdot x^2 - 0.034994902636753385636$$

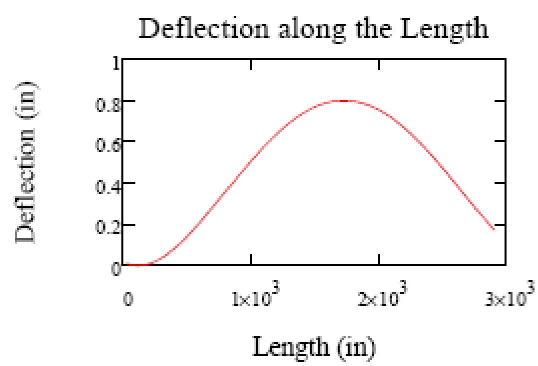
$$dL := \frac{L}{100}$$

$$i := 1..101$$

$$Petran_i := PeTran[(i-1) \cdot dL] \qquad \delta tran_i := v_{stran}[(i-1)dL]$$

$$\Delta tran_i := Petran_i \cdot \delta tran_i$$





Maximum Deflection

$\max(\Delta tran) = 0.801$ in

Article 3.10.8: Combination of Seismic Force Effects

$$\text{LoadCase1} := \sqrt{\left(1.0 \cdot p_{eTran}\right)^2 + \left(0.3 \cdot p_{eLong}\right)^2} = 0.266 \qquad \frac{kip}{in}$$

$$\text{LoadCase2} := \sqrt{\left(1.0 \cdot p_{eLong}\right)^2 + \left(0.3 \cdot p_{eTran}\right)^2} = 0.266 \qquad \frac{kip}{in}$$

Article 3.10.9.3: Determine Design Forces

$$\text{MaxLoadCase}(x,y) := \left| \begin{array}{l} a \leftarrow x \;\; \text{if} \;\; x \geq y \\ a \leftarrow y \;\; \text{if} \;\; y \geq x \\ a \end{array} \right.$$

$$\text{NominalForce} := \text{MaxLoadCase}(\text{LoadCase1}, \text{LoadCase2}) = 0.266 \qquad \frac{kip}{in}$$

Note: The Req values are factors that will be used to multiple loads that come out of SAP 2000. The NominalForce variable is truly just a factor also. It is easier to apply these factors to the loading for po than to change the loading in SAP 2000 because if an error is made in calculating pe then the loads will have to re-entered.

Multiple Column Bents $$\text{Req}_{substructure} := \frac{\text{NominalForce}}{R_{sub}} = 0.053$$

Note: Article 3.10.9.3 specifies that the Drilled shafts be designed for half of the R value. R/2 also must not be taken less than 1.

$$\text{Req}_{DrilledShafts} := \frac{\text{NominalForce}}{R_{sub} \cdot 0.5} = 0.107$$

Connections

Superstructure to Abutment $$\text{Req}_{subtoabutcon} := \frac{\text{NominalForce}}{R_{abutment}} = 0.333$$

Columns to Bent Cap $$\text{Req}_{coltocapcon} := \frac{\text{NominalForce}}{R_{columncap}} = 0.266$$

Column to foundation $$\text{Req}_{coltofoundcon} := \frac{\text{NominalForce}}{R_{foundation}} = 0.266$$

LOADS FOR DESIGN

NOTE: All the loads calculated in this section are for a single column or drilled shaft.

COLUMN SHEAR PROGRAM

$$\text{Shear}(\text{Vu}, \text{Reqsubstructure}) := \left| \begin{array}{l} a \leftarrow \text{Vu} \cdot \text{Reqsubstructure} \\ a \end{array} \right.$$

AXIAL LOAD PROGRAM

$$\text{PDEAD}(\text{Peq}, \text{Pd}, \text{Rsub}, \text{Reqsubstructure}) := \left| \begin{array}{l} a \leftarrow \left(\left| \text{Peq} \cdot \text{Reqsubstructure} - \frac{\text{Pd}}{\text{Rsub}} \right| \right) \\ a \end{array} \right.$$

Note: The axial load program calculates the minimum axial load on the column. This will needed later in the design process.

BENT 2

Note: Input the maximum shear value between all the columns in the bent from SAP which have not been multiplied by pe/po or divided by the R factor. This will be taken care of in one of the equations below.

INPUT $\text{Vu}_{\text{Bent2}} := 523$ kips

Note: The Axial load consists of both the Earthquake effect and dead load effect. It should be the minimum axial load the column will encounter during an earthquake.

INPUT $\text{Pueq}_{\text{Bent2}} := 1045$ kips

INPUT $\text{Pudead}_{\text{Bent2}} := 285$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $\text{Pumin}_{\text{Bent2}} := \text{PDEAD}\left(\text{Pueq}_{\text{Bent2}}, \text{Pudead}_{\text{Bent2}}, R_{\text{sub}}, \text{Req}_{\text{substructure}}\right) = 1.343$ kips

INPUT $\text{ColumnHeight}_{\text{Bent2}} := 18$ ft

Note: Input the correct variables into the below program to get the shear in the column or drilled shaft.

INPUT

$\text{Vucol}_{\text{Bent2}} := \text{Shear}\left(\text{Vu}_{\text{Bent2}}, \text{Req}_{\text{substructure}}\right) = 27.855$ kips

BENT 3

Note: Input the maximum shear value between all the columns in the bent from SAP which have not been multiplied by pe/po or divided by the R factor. This will be taken care in of one of the equations below.

INPUT $Vu_{Bent3} := 265$ kips

Note: The Axial load consists of both the Earthquake effect and dead load effect. It should be the minimum axial load the column will encounter during an earthquake.

INPUT $Pueq_{Bent3} := 540$ kips

INPUT $Pudead_{Bent3} := 292$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{Bent3} := PDEAD(Pueq_{Bent3}, Pudead_{Bent3}, R_{sub}, Req_{substructure}) = 29.639$ kips

INPUT $ColumnHeight_{Bent3} := 25.834$ ft

Note: Input the correct variables into the below program to get the shear in the column or drilled shaft.

INPUT

$Vucol_{Bent3} := Shear(Vu_{Bent3}, Req_{substructure}) = 14.114$ kips

DRILLED SHAFT 2

INPUT $Pueq_{DS2} := 1045$ kips

INPUT $Pudead_{DS2} := 285$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{DS2} := PDEAD(Pueq_{DS2}, Pudead_{DS2}, R_{sub}, Req_{DrilledShafts}) = 54.314$ kips

INPUT $ColumnHeight_{DS2} := 7.9$ ft

Note: The drilled shaft is design for an R/2; therefore, the shear in the column can be doubled to equal the design shear for the drilled shaft.

INPUT $Vu_{DS2} := Vucol_{Bent2} \cdot 2 = 55.71$ kips

DRILLED SHAFT 3

INPUT $Pueq_{DS3} := 540$ kips

INPUT $Pudead_{DS3} := 292$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{DS3} := PDEAD\left(Pueq_{DS3}, Pudead_{DS3}, R_{sub}, Req_{DrilledShafts}\right) = 0.879$ kips

INPUT $ColumnHeight_{DS3} := 8$ ft

Note: The drilled shaft is design for an R/2; therefore, the shear in the column can be doubled to equal the design shear for the drilled shaft.

INPUT $Vu_{DS3} := Vucol_{Bent3} \cdot 2 = 28.228$ kips

ABUTMENT DRILLED SHAFTS

Note: Input the loads from SAP which have not been multiplied by pe/po or divided by the R factor. This will be taken care of one of the equations below.

INPUT $Vu_{Abut} := 400$ kips

INPUT $ColumnHeight_{Abut} := 11.5$ ft

Note: Input the correct variables into the below program to get the shear in the column or drilled shaft.

INPUT

$Vu_{DSAbut} := Shear\left(Vu_{Abut}, Req_{subtoabutcon}\right) = 133.151$ kips

Article 4.7.4.4: Minimum Support Length Requirements

Note: May need to add more calculations if column heights are different at the bents.

N (in) = Minimum support length measured normal to the bridge

L (ft) = Length of bridge to adjacent expansion joint or end of the bridge

H (ft) = average height of columns supporting bridge deck for abutments
for columns and piers = column height

S (Degree) = angle of skew

Abutment Support Length Requirement

$Span_{abutment} := \frac{Span}{12} = 80$ ft $H_{abutment} := \frac{ColumnHeight}{12} = 22$ ft

Note: The $Span_{abutment}$ is divided by number of spans and inches.

$Skew_{abutment} := 0$ Degrees

$Nabutment := 1.5 \cdot (8 + 0.02Span_{abutment} + 0.08H_{abutment}) \cdot \left(1 + 0.000125Skew_{abutment}^{\ 2}\right) = 17.04$ in

Bent Support Length Requirement

BENT 2

INPUT $Span_{Bent} := \frac{Span}{12} = 80$ ft

Note: The $Span_{abutment}$ is divided by number of spans and inches.

INPUT $H_{Bent} := 18$ ft *INPUT*: Column Height for this Bent

INPUT $Skew_{Bent} := 0$ Degrees

$N_{Bent} := 1.5 \cdot (8 + 0.02Span_{Bent} + 0.08H_{Bent}) \cdot \left(1 + 0.000125Skew_{Bent}^{\ 2}\right) = 16.5\text{(}$ in

BENT 3

INPUT $Span_{Bent} := \frac{Span}{12} = 80$ ft

Note: The $Span_{abutment}$ is divided by number of spans and inches.

INPUT $H_{Bent} := 25.834$ ft *INPUT*: Column Height for this Bent

INPUT $Skew_{Bent} := 0$ Degrees

$$N_{Bent} := 1.5\cdot\left(8 + 0.02Span_{Bent} + 0.08H_{Bent}\right)\cdot\left(1 + 0.000125Skew_{Bent}^{\ 2}\right) = 17.5 \quad \text{in}$$

BENT 2 DESIGN

Article 5.10.11.3: Provisions for Seismic Design for Seismic Zone 2

Article 5.10.11.3: Longitudinal Reinforcement

INPUT $A_{longbar} := 1.56$ in^2

INPUT $N_{bars} := 12$

INPUT $Ads := Acolumn$

$$A_{longreinforcing} := A_{longbar}\cdot N_{bars} = 18.72 \quad in^2$$

Minimum Longitudinal Reinforcing Check

$$\text{Checkleastlongreinforcing}(Ag, Along) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } Along \geq Ag\cdot 0.01 \\ a \leftarrow \text{"Increase Longitudinal Reinforcing Ratio"} \quad \text{if } Along < 0.01\cdot Ag \\ a \end{array} \right.$$

$$\text{MinLongRatio} := \text{Checkleastlongreinforcing}\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars</u> in the inputs.

Maximum Longitudinal Reinforcing Check

$$\text{Checkmaxlongreinforcing}(Ag, Along) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } Along \le 0.06 \cdot Ag \\ a \leftarrow \text{"Decrease Longitudinal Reinforcing Ratio"} \quad \text{if } Along > 0.06 \cdot Ag \\ a \end{array} \right.$$

$$\text{MaxLongRatio} := \text{Checkmaxlongreinforcing}\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

Article 5.10.11.4.1b: Flexural Resistance

Check biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.9.4.
Flexural Resistance check can be done using some kind of Column Design Program. PCA Column was used for this project. After creating an interaction diagram, verify that all the critical load combinations fall within the diagram.

Article 5.10.11.4.1c: Column Shear and Transverse Reinforcement

Note: This is for the end regions of the column.

Article 5.8.3.3: Nominal Shear Resistance

$Vu := Vucol_{Bent2} = 27.855$ kips

$Pumin_{Bent2} = 1.343$ kips

INPUT	$bv := Columndia$		bv: effective width
INPUT	$\phi_s := 0.9$		
INPUT	$s := 4$	in	s: Spacing of hoops or pitch of spiral (in)
INPUT	$Asp := .31$	in^2	Asp: Area of spiral or hoop reinforcing (in^2)
INPUT	$Dsp := 0.625$	in	Dsp: Diameter of spiral or hoop reinforcing (in)
INPUT	$Cover := 6$	in	Cover: Concrete cover for the Column (in)
INPUT	$Dprime := 30$	in	Dprime: Diameter of spiral or hoop for circular columns (in)
INPUT	$d_{bl} := 1.41$	in	d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 34.67 \quad \text{in}$$

(Equation: C5.8.2.9-2) $de := \frac{bv}{2} + \frac{Dr}{\pi} = 32.036 \quad \text{in}$

$$dv := 0.9 \cdot de = 28.832 \quad \text{in}$$

Article 5.10.11.4.1c:

$$VcProgram(fc, \beta, bv, dv, Ag, Pu) := \left| \begin{array}{l} p \leftarrow Pu \\ v \leftarrow 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv \\ c \leftarrow 0.1 \cdot Ag \cdot \frac{fc}{1000} \\ x \leftarrow \frac{p \cdot v}{c} \\ a \leftarrow v \ \text{ if } \ p > c \\ a \leftarrow x \ \text{ if } \ p \leq c \\ a \end{array} \right.$$

Note: The area of the Column and the Axial Load for this column need to be input into the Vc equation that calls the program above.

$$V_c := VcProgram\left(fc, \beta, bv, dv, Acolumn, Pumin_{Bent2}\right) = 0.371 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{s} = 268.14 \quad \text{kips}$$

Eq. 5.8.3.3-1 $$\phi V_n := (V_c + V_s) \cdot \phi_s = 241.66 \quad \text{kips}$$

$$\text{ShearCheck}(\phi Vn, V_u) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } \phi Vn \geq V_u \\ a \leftarrow \text{"FAILURE"} \quad \text{if } \phi Vn < V_u \\ a \end{array} \right.$$

$$\text{Shearcheck} := \text{ShearCheck}(\phi V_n, Vu) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

Article 5.10.11.4.1c Length of End Region (Plastic Hinge Region)

End region is assumed to extend from the soffit of girders or cap beams at the top of the columns or from the top of foundations at the bottom of columns.

$$\text{EndRegionProgram}(d, H) := \left| \begin{array}{l} x \leftarrow d \\ y \leftarrow \frac{1}{6} H \cdot 12 \\ z \leftarrow 18 \\ a \leftarrow \max(x, y, z) \\ a \end{array} \right.$$

$$\text{LendgthEndRegion} := \text{EndRegionProgram}(\text{Columndia}, \text{ColumnHeight}_{Bent2}) = 42 \quad \text{in}$$

Article 5.10.11.4.3: Column Connections

Extension into Top and Bottom Connections

Note: This needs to be done whenever the column dimension changes.

$$\text{ExtensionProgram}(d) := \left| \begin{array}{l} z \leftarrow 15 \\ x \leftarrow \frac{1}{2} \cdot d \\ a \leftarrow \max(z, x) \\ a \end{array} \right.$$

$$\text{Extension} := \text{ExtensionProgram}(\text{Columndia}) = 21 \quad \text{in}$$

Article 5.10.11.4.1e: Spacing of Transverse Reinforcement for Confinement

Transverse Reinforcement for Confinement

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

Shall Not Exceed the Smallest of:

$$\text{Spacingprogram(Columndia)} := \left| \begin{array}{l} x \leftarrow \frac{1}{4} \cdot \text{Columndia} \\ y \leftarrow 4 \\ a \leftarrow \min(x, y) \\ a \end{array} \right.$$

$$\text{MaximumSpacing} := \text{Spacingprogram(Columndia)} = 4 \qquad \text{in}$$

$$\text{SpacingCheck(MaximumSpacing}, s) := \left| \begin{array}{l} a \leftarrow s \quad \text{if } s \le \text{MaximumSpacing} \\ a \leftarrow \text{MaximumSpacing} \quad \text{if } s > \text{MaximumSpacing} \\ a \end{array} \right.$$

$$\text{FINALSPACING} := \text{SpacingCheck(MaximumSpacing}, s) = 4 \qquad \text{in}$$

$$\text{scheck} := \text{ShearCheck(MaximumSpacing}, s) = \text{"OK"}$$

Note: If scheck returns "Failure", increase the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.1d: Transverse Reinforcement for Confinement at Plastic Hinges

Required Volumetric Ratio of Spiral or Seismic Hoop Reinforcing

$$\rho_s := \frac{4 \cdot Asp}{s \cdot Dprime} = 0.01$$

$$\text{RatioProgram}(fc, fy, \rho_s) := \left| \begin{array}{l} z \leftarrow 0.12 \cdot \dfrac{fc}{fy} \\ a \leftarrow \text{"OK"} \quad \text{if } \rho_s \geq z \\ a \leftarrow \text{"Increase Transverse Reinforcing Ratio"} \quad \text{if } \rho_s < z \\ a \end{array} \right.$$

$$\text{Check}\rho_s := \text{RatioProgram}(fc, fye, \rho_s) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Increase Transverse Reinforcing Ratio", it is recommended to <u>decrease the spacing (s)</u> or <u>increase the area of the transverse reinforcement (Asp)</u> in the inputs.

Note:These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension of **NOT** less than **6*d_b** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Article 5.10.11.4.1f: Splices

Lap Splices in longitudinal reinforcement shall not be used in plastic hinge region.

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.

5.8.3.3 **Nominal Shear Resistance**

INPUT $Vu_{sub} := Vucol_{Bent2} = 27.855$ kips

INPUT $spaceNOhinge := 12$ in

Note: β and θ come from Article 5.8.3.4.1

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 34.67 \quad \text{in}$$

Eq. C5.8.2.9-2

$$de := \frac{bv}{2} + \frac{Dr}{\pi} = 32.036 \quad \text{in}$$

$$dv := 0.9 \cdot de = 28.832 \quad \text{in}$$

Eq. 5.8.3.3-3

$$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 153.065 \quad \text{kips}$$

Eq. 5.8.3.3-4

$$V_s := \frac{Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 44.69 \quad \text{kips}$$

$$\phi V_n := (V_c + V_s) \cdot \phi_s = 177.979 \quad \text{kips}$$

$$\text{ShearCheck}(\phi Vn, V_u) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } \phi Vn \ge V_u \\ a \leftarrow \text{"FAILURE"} \quad \text{if } \phi Vn < V_u \\ a \end{array} \right.$$

$$\text{Shearcheck} := \text{ShearCheck}(\phi V_n, Vu_{sub}) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$\text{Avmin} := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot \text{spaceNOhinge}}{\frac{fye}{1000}} = 0.531 \quad in^2$$

$$\text{Av} := 2 \cdot \text{Asp} = 0.62 \quad in^2$$

$$\text{TranCheck}(\text{Avmin}, \text{Av}) := \left| \begin{array}{l} a \leftarrow \text{"Decrease Spacing or Increase Bar Size"} \quad \text{if Avmin} > \text{Av} \\ a \leftarrow \text{"OK"} \quad \text{if Avmin} \le \text{Av} \\ a \end{array} \right.$$

$$\text{MinimumTran} := \text{TranCheck}(\text{Avmin}, \text{Av}) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

***5.8.2.7* Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $vu := \frac{Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 0.026$ ksi

$$\text{spacingProgram}(Vu, dv, fc) := \left| \begin{array}{l} v \leftarrow 0.125 \cdot \frac{fc}{1000} \\ q \leftarrow 0.8 \cdot dv \\ r \leftarrow 0.4 \cdot dv \\ z \leftarrow q \ \text{ if } q \le 24 \\ z \leftarrow 24 \ \text{ if } q > 24 \\ t \leftarrow r \ \text{ if } r \le 12 \\ t \leftarrow 12 \ \text{ if } r > 12 \\ a \leftarrow z \ \text{ if } Vu < v \\ a \leftarrow t \ \text{ if } Vu \ge v \\ a \end{array} \right.$$

$\text{MaxSpacing} := \text{spacingProgram}(vu, dv, fc) = 23.066$ in

$$\text{Spacecheck}(\text{MaxSpacing}, s) := \left| \begin{array}{l} a \leftarrow s \ \text{ if } s \le \text{MaxSpacing} \\ a \leftarrow \text{MaxSpacing} \ \text{ if } s > \text{MaxSpacing} \\ a \end{array} \right.$$

$\text{MAXSPACING} := \text{Spacecheck}(\text{MaxSpacing}, \text{spaceNOhinge}) = 12$ in

$\text{scheck} := \text{ShearCheck}(\text{MAXSPACING}, \text{spaceNOhinge}) = \text{"OK"}$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

BENT 3 DESIGN

Article 5.10.11.3: Provisions for Seismic Design for Seismic Zone 2

Article 5.10.11.3: Longitudinal Reinforcement

INPUT $A_{longbar} := 1.56 \quad in^2$

INPUT $N_{bars} := 12$

INPUT $Ads := Acolumn$

$$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 18.72 \quad in^2$$

Minimum Longitudinal Reinforcing Check

$$MinLongRatio := Checkleastlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Maximum Longitudinal Reinforcing Check

$$MaxLongRatio := Checkmaxlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

Article 5.10.11.4.1b: Flexural Resistance

Check biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.9.4.
Flexural Resistance check can be done using some kind of Column Design Program. PCA Column was used for this project. After creating an interaction diagram, verify that all the critical load combinations fall within the diagram.

Article 5.10.11.4.1c: Column Shear and Transverse Reinforcement

Note: This is for the end regions of the column.

Article 5.8.3.3: Nominal Shear Resistance

$Vucol_{Bent3} = 14.114$ kips

$Pumin_{Bent3} = 29.639$ kips

INPUT	$bv := Columndia$		bv: effective width
INPUT	$\phi_s := 0.9$		
INPUT	$s := 4$	in	s: Spacing of hoops or pitch of spiral (in)
INPUT	$Asp := .31$	in^2	Asp: Area of spiral or hoop reinforcing (in^2)
INPUT	$Dsp := 0.625$	in	Dsp: Diameter of spiral or hoop reinforcing (in)
INPUT	$Cover := 6$	in	Cover: Concrete cover for the Column (in)
INPUT	$Dprime := 30$	in	Dprime: Diameter of spiral or hoop for circular columns (in)
INPUT	$d_{bl} := 1.41$	in	d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 34.67 \quad \text{in}$$

Eq. C5.8.2.9-2

$$de := \frac{bv}{2} + \frac{Dr}{\pi} = 32.036 \quad \text{in}$$

$$dv := 0.9 \cdot de = 28.832 \quad \text{in}$$

Article 5.10.11.4.1c: Column Shear and Transverse Reinforcement

INPUT

$$V_c := \text{VcProgram}\left(fc, \beta, bv, dv, Acolumn, Pumin_{Bent3}\right) = 8.186 \quad \text{kips}$$

$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{s} = 268.14 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 248.694 \quad \text{kips}$$

$$\text{Shearcheck} := \text{ShearCheck}\left(\phi V_n, Vu\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

Length of End Region (Plastic Hinge Region)

End region is assumed to extend from the soffit of girders or cap beams at the top of the columns or from the top of foundations at the bottom of columns.

$$\text{LendgthEndRegion} := \text{EndRegionProgram}\left(\text{Columndia}, \text{ColumnHeight}_{Bent3}\right) = 51.668 \quad \text{in}$$

Article 5.10.11.4.3: Column Connections

Extension into Top and Bottom Connections

Note: This needs to be done whenever the column dimension changes.

$$\text{Extension} := \text{ExtensionProgram}(\text{Columndia}) = 21 \quad \text{in}$$

Article 5.10.11.4.1e: Spacing of Transverse Reinforcement for Confinement

Transverse Reinforcement for Confinement

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

$$\text{MaximumSpacing} := \text{Spacingprogram}(\text{Columndia}) = 4 \qquad \text{in}$$

$$\text{FINALSPACING} := \text{SpacingCheck}(\text{MaximumSpacing}, s) = 4 \qquad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MaximumSpacing}, s) = \text{"OK"}$$

Note: If scheck returns "Failure", increase the spacing of <u>shear reinforcing spacing (s)</u>. The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.1d: Transverse Reinforcement for Confinement at Plastic Hinges

Required Volumetric Ratio of Spiral or Seismic Hoop Reinforcing

$$\rho_s := \frac{4 \cdot \text{Asp}}{s \cdot \text{Dprime}} = 0.01$$

$$\text{Check}\rho_s := \text{RatioProgram}\left(\text{fc}, \text{fye}, \rho_s\right) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Increase Transverse Reinforcing Ratio", it is recommended to <u>decrease the spacing (s)</u> or <u>increase the area of the transverse reinforcement (Asp)</u> in the inputs.

Note:These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension of **NOT** less than 6*d_b at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Article 5.10.11.4.1f: Splices

Lap Splices in longitudinal reinforcement shall not be used in plastic hinge region.

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.

5.8.3.3 **Nominal Shear Resistance**

INPUT $Vu_{sub} := Vucol_{Bent3} = 14.114$ kips

INPUT $spaceNOhinge := 12$ in

Note: β and θ come from Article 5.8.3.4.1

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 34.67 \quad in$$

Eq. C5.8.2.9-2

$$de := \frac{bv}{2} + \frac{Dr}{\pi} = 32.036 \quad in$$

$$dv := 0.9 \cdot de = 28.832 \quad in$$

Eq. 5.8.3.3-3

$$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 153.065 \quad kips$$

Eq. 5.8.3.3-4

$$V_s := \frac{Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 44.69 \quad kips$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 177.979 \quad kips$$

$$Shearcheck := ShearCheck\left(\phi V_n, Vu_{sub}\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1

$$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.531 \quad in^2$$

$$Av := 2 \cdot Asp = 0.62 \quad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $vu := \dfrac{Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 0.013$ ksi

$MaxSpacing := spacingProgram(vu, dv, fc) = 23.066$ in

$$Spacecheck(MaxSpacing, s) := \begin{array}{|l} a \leftarrow s \ \text{ if } \ s \le MaxSpacing \\ a \leftarrow MaxSpacing \ \text{ if } \ s > MaxSpacing \\ a \end{array}$$

$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12$ in

$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT DESIGN

DRILLED SHAFT 2

Article 5.13.4.6.2b: Cast-in-place Piles

INPUT $A_{longbar} := 1.56$ in^2

INPUT $N_{bars} := 12$

INPUT $Ads := Adrilledshaft$

$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 18.72$ in^2

Note: Article 5.13.4.6.2b only has a provision for the minimum amount of longitudinal reinforcing. This minimum reinforcing check only applies to the **FIRST 1/3*PILE LENGTH OR 8 FT**.

CHECK: There also needs be a check made if the column Diameter is less than 24.0 in. For these drilled shafts, spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at pitch not exceeding 9.0 in., except that the pitch shall not exceed 4.0 in. within a length below the pile cap reinforcement of not less than 2.0 ft or 1.5 pile diameters, whichever is greater.

Minimum Longitudinal Reinforcing Check (First 1/3*Pile Length or 8 ft)

$$\text{Checkleastlongreinforcing}(Ag, Along) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } Along \geq Ag \cdot 0.005 \\ a \leftarrow \text{"Increase Longitudinal Reinforcing Ratio"} \quad \text{if } Along < 0.005 \cdot Ag \\ a \end{array} \right.$$

$$\text{MinLongRatio} := \text{Checkleastlongreinforcing}\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars</u> in the inputs.

Article 5.10.11.3: Longitudinal Reinforcement

Minimum Longitudinal Reinforcing Check

$$\text{MinLongRatio} := \text{Checkleastlongreinforcing}\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars</u> in the inputs.

Maximum Longitudinal Reinforcing Check

$$\text{MaxLongRatio} := \text{Checkmaxlongreinforcing}\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either <u>increase the section size (Ag)</u> or <u>decrease the longitudinal reinforcing (Abl and NumberBars)</u> in the inputs.

5.8.3.3 **Nominal Shear Resistance**

<u>*INPUT*</u> $Vu_{sub} := Vu_{DS2} = 55.71$ kips

<u>*INPUT*</u> $spaceNOhinge := 12$ in s: Spacing of hoops or pitch of spiral (in)

<u>*INPUT*</u> $bv := DSdia$ bv: effective width

<u>*INPUT*</u> $Asp := .31$ in^2 Asp: Area of spiral or hoop reinforcing (in^2)

<u>*INPUT*</u> $Dsp := 0.625$ in Dsp: Diameter of spiral or hoop reinforcing (in)

<u>*INPUT*</u> $Cover := 6$ in Cover: Concrete cover for the Column (in)

<u>*INPUT*</u> $Dprime := 30$ in Dprime: Diameter of spiral or hoop for circular columns (in)

<u>*INPUT*</u> $d_{bl} := 1.41$ in d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 34.67 \quad \text{in}$$

Eq. C5.8.2.9-2
$$de := \frac{bv}{2} + \frac{Dr}{\pi} = 32.036 \quad \text{in}$$

$$dv := 0.9 \cdot de = 28.832 \quad \text{in}$$

Eq. 5.8.3.3-3
$$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 153.065 \quad \text{kips}$$

Eq. 5.8.3.3-4
$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 89.38 \quad \text{kips}$$

$$\phi V_n := (V_c + V_s)\cdot\phi_s = 218.2 \qquad \text{kips}$$

$$\text{Shearcheck} := \text{ShearCheck}(\phi V_n, \text{Vu}) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$\text{Avmin} := 0.0316\cdot\sqrt{\frac{\text{fc}}{1000}}\cdot\frac{\text{bv}\cdot\text{spaceNOhinge}}{\frac{\text{fye}}{1000}} = 0.531 \qquad \text{in}^2$$

$$\text{Av} := 2\cdot\text{Asp} = 0.62 \qquad \text{in}^2$$

$$\text{MinimumTran} := \text{TranCheck}(\text{Avmin}, \text{Av}) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$\text{vu} := \frac{\text{Vu}_{\text{sub}}}{\phi_s\cdot\text{bv}\cdot\text{dv}} = 0.051 \qquad \text{ksi}$$

$$\text{MaxSpacing} := \text{spacingProgram}(\text{vu}, \text{dv}, \text{fc}) = 23.066 \qquad \text{in}$$

$$\text{MAXSPACING} := \text{Spacecheck}(\text{MaxSpacing}, \text{spaceNOhinge}) = 12 \qquad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MAXSPACING}, \text{spaceNOhinge}) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of transverse reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT 3

Article 5.13.4.6.2b: Cast-in-place Piles

INPUT $A_{longbar} := 1.56 \quad in^2$

INPUT $N_{bars} := 12$

INPUT $Ads := Adrilledshaft$

$$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 18.72 \quad in^2$$

Note: Article 5.13.4.6.2b only has a provision for the minimum amount of longitudinal reinforcing. This minimum reinforcing check only applies to the **FIRST 1/3*PILE LENGTH OR 8 FT**.

CHECK: There also needs be a check made if the column Diameter is less than 24.0 in. For these drilled shafts, spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at pitch not exceeding 9.0 in., except that the pitch shall not exceed 4.0 in. within a length below the pile cap reinforcement of not less than 2.0 ft or 1.5 pile diameters, whichever is greater.

Minimum Longitudinal Reinforcing Check (First 1/3*Pile Length or 8 ft)

$$MinLongRatio := Checkleastlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Article 5.10.11.3: Longitudinal Reinforcement

Minimum Longitudinal Reinforcing Check

$$MinLongRatio := Checkleastlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Maximum Longitudinal Reinforcing Check

$$MaxLongRatio := Checkmaxlongreinforcing(Ads, A_{longreinforcing}) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

5.8.3.3 **Nominal Shear Resistance**

INPUT $Vu_{sub} := Vu_{DS3} = 28.228$ kips

INPUT $spaceNOhinge := 12$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $bv := DSdia$ bv: effective width

INPUT $Asp := .31$ in^2 Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.625$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 6$ in Cover: Concrete cover for the Column (in)

INPUT $Dprime := 30$ in Dprime: Diameter of spiral or hoop for circular columns (in)

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 34.67 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 32.036 \quad \text{in}$$

$$dv := 0.9 \cdot de = 28.832 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316\cdot\beta\cdot\sqrt{\frac{fc}{1000}}bv\cdot dv = 153.065 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp\cdot\frac{fye}{1000}dv\cdot\cot(\theta)}{spaceNOhinge} = 89.38 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right)\cdot\phi_s = 218.2 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right)\cdot\phi_s = 218.2 \quad \text{kips}$$

$$Shearcheck := ShearCheck\left(\phi V_n, Vu\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316\cdot\sqrt{\frac{fc}{1000}}\cdot\frac{bv\cdot spaceNOhinge}{\frac{fye}{1000}} = 0.531$$

$$Av := 2\cdot Asp = 0.62 \quad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $vu := \dfrac{Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 0.026$ ksi

$MaxSpacing := spacingProgram(vu, dv, fc) = 23.066$ in

$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12$ in

$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$

Note: If scheck returns "Failure", change the spacing of transverse reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT ABUTMENT

Article 5.13.4.6.2b: Cast-in-place Piles

INPUT $A_{longbar} := 1.56$ in^2

INPUT $N_{bars} := 12$

INPUT $Ads := Adsabut$

$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 18.72$ in^2

Note: Article 5.13.4.6.2b only has a provision for the minimum amount of longitudinal reinforcing. This minimum reinforcing check only applies to the **FIRST 1/3*PILE LENGTH OR 8 FT**.

CHECK: There also needs be a check made if the column Diameter is less than 24.0 in. For these drilled shafts, spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at pitch not exceeding 9.0 in., except that the pitch shall not exceed 4.0 in. within a length below the pile cap reinforcement of not less than 2.0 ft or 1.5 pile diameters, whichever is greater.

Minimum Longitudinal Reinforcing Check (First 1/3*Pile Length or 8 ft)

$MinLongRatio := Checkleastlongreinforcing(Ads, A_{longreinforcing}) = "OK"$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars</u> in the inputs.

Article 5.10.11.3: Longitudinal Reinforcement

Minimum Longitudinal Reinforcing Check

$MinLongRatio := Checkleastlongreinforcing(Ads, A_{longreinforcing}) = "OK"$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars</u> in the inputs.

Maximum Longitudinal Reinforcing Check

$MaxLongRatio := Checkmaxlongreinforcing(Ads, A_{longreinforcing}) = "OK"$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either <u>increase the section size (Ag)</u> or <u>decrease the longitudinal reinforcing (Abl and NumberBars)</u> in the inputs.

5.8.3.3 **Nominal Shear Resistance**

<u>*INPUT*</u> $Vu_{sub} := Vu_{DSAbut} = 133.151$ kips

<u>*INPUT*</u> $spaceNOhinge := 12$ in — s: Spacing of hoops or pitch of spiral (in)

<u>*INPUT*</u> $bv := DSabutdia$ — bv: effective width

<u>*INPUT*</u> $Asp := .31$ in^2 — Asp: Area of spiral or hoop reinforcing (in^2)

<u>*INPUT*</u> $Dsp := 0.625$ in — Dsp: Diameter of spiral or hoop reinforcing (in)

<u>*INPUT*</u> $Cover := 6$ in — Cover: Concrete cover for the Column (in)

<u>*INPUT*</u> $Dprime := 30$ in — Dprime: Diameter of spiral or hoop for circular columns (in)

<u>*INPUT*</u> $d_{bl} := 1.41$ in — d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 34.67 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 32.036 \quad \text{in}$$

$$dv := 0.9 \cdot de = 28.832 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 153.065 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 89.38 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 218.2 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 218.2 \quad \text{kips}$$

$$Shearcheck := ShearCheck\left(\phi V_n, Vu\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.531$$

$$Av := 2 \cdot Asp = 0.62 \qquad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 0.122 \qquad ksi$$

$$MaxSpacing := spacingProgram(vu, dv, fc) = 23.066 \qquad in$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \qquad in$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of transverse reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

Connection Design for Girder to Bent Cap

INPUT $Vcolbent := Vucol_{Bent2} = 27.855$

INPUT $Ngirderperbent := 12$

Article 6.5.4.2: Resistance Factors

$\phi_t := 0.8$	Tension for A307
$\phi_s := 0.75$	Shear for A307
$\phi_{bs} := 0.80$	Block Shear
$\phi_{bb} := 0.80$	Bolts Bearing
$\phi_{sc} := 0.85$	Shear Connectors
$\phi_f := 1.00$	Flexure
$\phi_{sangle} := 1.00$	Shear for the Angle

For Type III Girders

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.25$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT $Fy := 36$ ksi Fy = Yield Stress of the Angle

INPUT $Fu := 58$ ksi Fu = Ultimate Stress of the Angle

INPUT $t := 0.5$ in t = Thickness of Angle

INPUT $h := 6$ in h = Height of the Angle

INPUT $w := 6$ in w = Width of the Angle

INPUT $l := 12$ in — l = Length of the Angle

INPUT $k := 1.00$ in — k = Height of the Bevel

INPUT $distanchorhole := 4$ in — distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT $diahole := 1.5$ in — diahole = Diameter of bolt hole

INPUT $BLSHlength := 6$ in — BLSHlength = Block Shear Length

INPUT $BLSHwidth := 2$ in — BLSHwicth = Block Shear Width

INPUT $Ubs := 1.0$ — Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in — a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in — b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent \cdot 2}{2Ngirderperbent} = 2.321 \quad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 1.227 \quad in^2$$

Eq. 6.13.2.12-1 $\phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 25.624$ kips

Note: This is checking to verify that the anchor bolt has enough shear strength.

$$Shearcheck := ShearCheck(\phi sRn, Vangle) = \text{"OK"}$$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 87$ kips

For Slotted Holes

INPUT $Lc := 2$ in Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\phi bbRns := Lc \cdot t \cdot Fub = 58$ kips

$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$

$Bearingscheck := ShearCheck(\phi bbRns, Vangle) = \text{"OK"}$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 0.58 \quad \text{kips}$$

Eq. 6.13.2.10.2-1 $\phi tTn := \phi_t \cdot 0.76 \cdot A_b \cdot Fub = 43.275$ kips

$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$Pu := Vangle$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2

$$CombinedProgram(Pu, A_b, Fub, \phi sRn, \phi_s) := \left| \begin{array}{l} t \leftarrow 0.76 \cdot A_b \cdot Fub \\ r \leftarrow 0.76 \cdot A_b \cdot Fub \cdot \sqrt{1 - \left(\frac{Pu}{\phi sRn}\right)^2} \\ a \leftarrow t \ \text{ if } \frac{Pu}{\left(\frac{\phi sRn}{\phi_s}\right)} \le 0.33 \\ a \leftarrow r \ \text{ if } \frac{Pu}{\left(\frac{\phi sRn}{\phi_s}\right)} > 0.33 \\ a \end{array} \right.$$

$Tn_{combined} := CombinedProgram(Pu, A_b, Fub, \phi sRn, \phi_s) = 54.094$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 43.275$ kips

$Combinedcheck := ShearCheck(\phi tTn_{combined}, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

Note this is for if there are one through bolts in the upper leg.

$Agv := t \cdot BLSHlength = 3$ in^2

$Anv := t \cdot (BLSHlength - 0.5 \cdot diahole) = 2.625$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 0.625$ in^2

(J4-5)

$$BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) := \left| \begin{array}{l} b \leftarrow 0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant \\ c \leftarrow 0.6Fy \cdot Agv + Ubs \cdot Fu \cdot Ant \\ a \leftarrow b \ \text{ if } b \le c \\ a \leftarrow c \ \text{ if } b > c \\ a \end{array} \right.$$

$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 101.05$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 80.84$ kips

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 2.25$ in^2

(D3-1) $Ae := Ant \cdot Ut = 1.35$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 62.64$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

INPUT $Muangle := 6.64$ kip·in

$Zx := \frac{1 \cdot (t)^2}{4} = 0.75$ in^3

$\phi fMn := \phi_f \cdot Fy \cdot Zx = 27$ kip·in

$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$Cv := 1.0$

$Aw := t \cdot w = 3$ in^2

(G2-1) $\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 64.8$ kips

$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) =$ "OK"

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Expansion Connection Design for Girder to Bent Cap

INPUT $Vcolbent := Vucol_{Bent2} = 27.855$

INPUT $Ngirderperbent := 12$

Article 6.5.4.2: Resistance Factors

$\phi_t := 0.8$	Tension for A307
$\phi_s := 0.75$	Shear for A307
$\phi_{bs} := 0.80$	Block Shear
$\phi_{bb} := 0.80$	Bolts Bearing
$\phi_{sc} := 0.85$	Shear Connectors
$\phi_f := 1.00$	Flexure
$\phi_{sangle} := 1.00$	Shear for the Angle

For Type III Girders

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT	$Fub := 58$	ksi	
INPUT	$Dia_b := 1.25$	in	
INPUT	$Ns := 1$		Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT	$Fy := 36$	ksi	Fy = Yield Stress of the Angle
INPUT	$Fu := 58$	ksi	Fu = Ultimate Stress of the Angle
INPUT	$t := 0.5$	in	t = Thickness of Angle
INPUT	$h := 6$	in	h = Height of the Angle
INPUT	$w := 6$	in	w = Width of the Angle

INPUT $l := 12$ in l = Length of the Angle

INPUT $k := 1.00$ in k = Height of the Bevel

INPUT $distanchorhole := 4$ in distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT $diahole := 1.5$ in diahole = Diameter of bolt hole

INPUT $SlottedHole := 6$ in SlottedHole = Length of slotted hole

INPUT $BLSHlength := 6$ in BLSHlength = Block Shear Length

INPUT $BLSHwidth := 2$ in BLSHwicth = Block Shear Width

INPUT $Ubs := 1.0$ Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent \cdot 2}{2Ngirderperbent} = 2.321 \qquad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot {Dia_b}^2}{4} = 1.227 \qquad \text{in}^2$$

Eq. 6.13.2.12-1 $\phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 25.624$ kips

Note: This is checking to verify that the anchor bolt has enough shear strength.

$$Shearcheck := ShearCheck(\phi sRn, Vangle) = \text{"OK"}$$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 87$ kips

For Slotted Holes

INPUT $Lc := 2$ in Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\phi bbRns := Lc \cdot t \cdot Fub = 58$ kips

$\text{Bearingcheck} := \text{ShearCheck}(\phi bbRn, \text{Vangle}) = \text{"OK"}$

$\text{Bearingscheck} := \text{ShearCheck}(\phi bbRns, \text{Vangle}) = \text{"OK"}$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{\text{Vangle} \cdot 1}{\text{distanchorhole}} = 0.58 \quad \text{kips}$$

Eq. 6.13.2.10.2-1 $\phi tTn := \phi_t \cdot 0.76 \cdot A_b \cdot Fub = 43.275$ kips

$\text{Tensioncheck} := \text{ShearCheck}(\phi tTn, Tu) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$Pu := Vangle$

$Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 54.094$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 43.275$ kips

$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vangle\right) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

Note this is for if there are one through bolts in the upper leg.

$Agv := t \cdot BLSHlength = 3$ in^2

$Anv := t \cdot \left(BLSHlength - 0.5 \cdot \frac{SlottedHole}{2}\right) = 2.25$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 0.625$ in^2

$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 101.05$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 80.84$ kips

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 2.25$ in^2

(D3-1) $Ae := Ant \cdot Ut = 1.35$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 62.64$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

INPUT $Muangle := 6.64$ $kip \cdot in$

$$Zx := \frac{1 \cdot (t)^2}{4} = 0.75 \qquad in^3$$

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 27 \qquad kip \cdot in$$

$$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t \cdot w = 3 \qquad in^2$$

(G2-1) $$\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 64.8 \qquad kips$$

$$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Connection Design for Girder to Abutment

For Type III Girders

INPUT $Vcolbent := Vu_{DSAbut}$

INPUT $Ngirderperbent := 6$

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.5$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT $Fy := 36$ ksi Fy = Yield Stress of the Angle

INPUT $Fu := 58$ ksi Fu = Ultimate Stress of the Angle

INPUT $t := 0.75$ in t = Thickness of Angle

INPUT $h := 6$ in h = Height of the Angle

INPUT $w := 6$ in w = Width of the Angle

INPUT $l := 12$ in l = Length of the Angle

INPUT $k := 1.25$ in k = Height of the Bevel

INPUT $distanchorhole := 4$ in distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT $diahole := 1.75$ in diahole = Diameter of bolt hole

INPUT $SlottedHole := 6$ in SlottedHole = Length of slotted hole

INPUT $BLSHlength := 6$ in BLSHlength = Block Shear Length

INPUT $BLSHwidth := 2$ in BLSHwicth = Block Shear Width

INPUT $Ubs := 1.0$ Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent \cdot 2}{2Ngirderperbent} = 22.192 \qquad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 1.767 \qquad \text{in}^2$$

Eq. 6.13.2.12-1 $\quad \phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 36.898 \qquad \text{kips}$

Note: This is checking to verify that the anchor bolt has enough shear strength.

$$Shearcheck := ShearCheck(\phi sRn, Vangle) = \text{"OK"}$$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\quad \phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 156.6 \qquad \text{kips}$

For Slotted Holes

INPUT $\quad Lc := 2 \quad \text{in} \quad$ Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\quad \phi bbRns := Lc \cdot t \cdot Fub = 87 \qquad \text{kips}$

$$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$$

$$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 5.548 \qquad \text{kips}$$

Eq. 6.13.2.10.2-1 $$\phi tTn := 0.76 \cdot A_b \cdot Fub = 77.896 \qquad \text{kips}$$

$$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$$Pu := Vangle$$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2 $$Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 62.233 \qquad \text{kips}$$

$$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 49.786 \qquad \text{kips}$$

$$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vangle\right) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$$Agv := t \cdot BLSHlength = 4.5 \qquad in^2$$

$$Anv := t \cdot \left(BLSHlength - 0.5 \cdot \frac{SlottedHole}{2}\right) = 3.375 \qquad in^2$$

$$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 0.844 \qquad in^2$$

(J4-5) $$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 146.137 \qquad \text{kips}$$

$$\phi bsRn := \phi_{bs} \cdot Rn = 116.91 \qquad \text{kips}$$

$$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 3.188 \quad in^2$

(D3-1) $Ae := Ant \cdot Ut = 1.912 \quad in^2$

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 88.74 \quad kips$

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

INPUT $Muangle := 44.1 \quad kip \cdot in$

$$Zx := \frac{1 \cdot (t)^2}{4} = 1.688 \quad in^3$$

$\phi fMn := \phi_f \cdot Fy \cdot Zx = 60.75 \quad kip \cdot in$

$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$Cv := 1.0$

$Aw := t \cdot w = 4.5 \quad in^2$

(G2-1) $\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 97.2 \quad kips$

$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Appendix C: Interaction Diagrams of Osiligee Creek Bridge

Designer: PJC
Project Name: OSILIGEE CREEK
Job Number:
Date: 12/8/2010

Moment Capacity and Interaction Diagram Data

Column or Drilled Shaft: BENT 2 COLUMN

INPUTS

Column Diameter	42	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	#11	
Number of Longitudinal Reinforcing Bars	12	
Transverse Reinforcing Bar Size	#5	
Spacing of Transverse Reinforcing Bars	12	in
Column Height	18	ft
Cover	6	in

H (NUMBER OF COLS/BENT)

Dead Load Reactions

PuDead	300	kips
MuDead	22.4	kip*ft

Enter into Column Design Software (PCA COLUMN)
Finding Moment Capacity of the column.

Mp = ϕMp/0.9 → 1630 kip*ft

Vpcol = 2*Mp/Heightcolumn → 181 kips

Vpbent = 2*Vpcol 362 kips

Create SAP model of Bent
Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Fuoverturning = 284 kips

Pu = PuDead +/- Fuoverturning 584 or 16 kips

Re-enter into Column Design Software (PCA COLUMN)

Mp = ϕMp/0.9 → 1744 kip*ft

Vpcol = 2*Mp/Heightcolumn → 194 kips

Vpbent = 2*Vpcol 387 kips

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

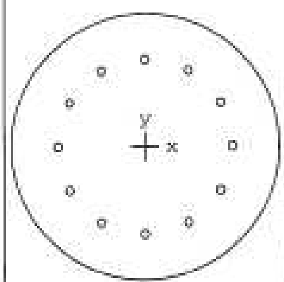
Check = [(Vpbent2 - Vpbent1) / Vpbent2] * 100% 6.5% OK

Check Other Seismic Load Cases

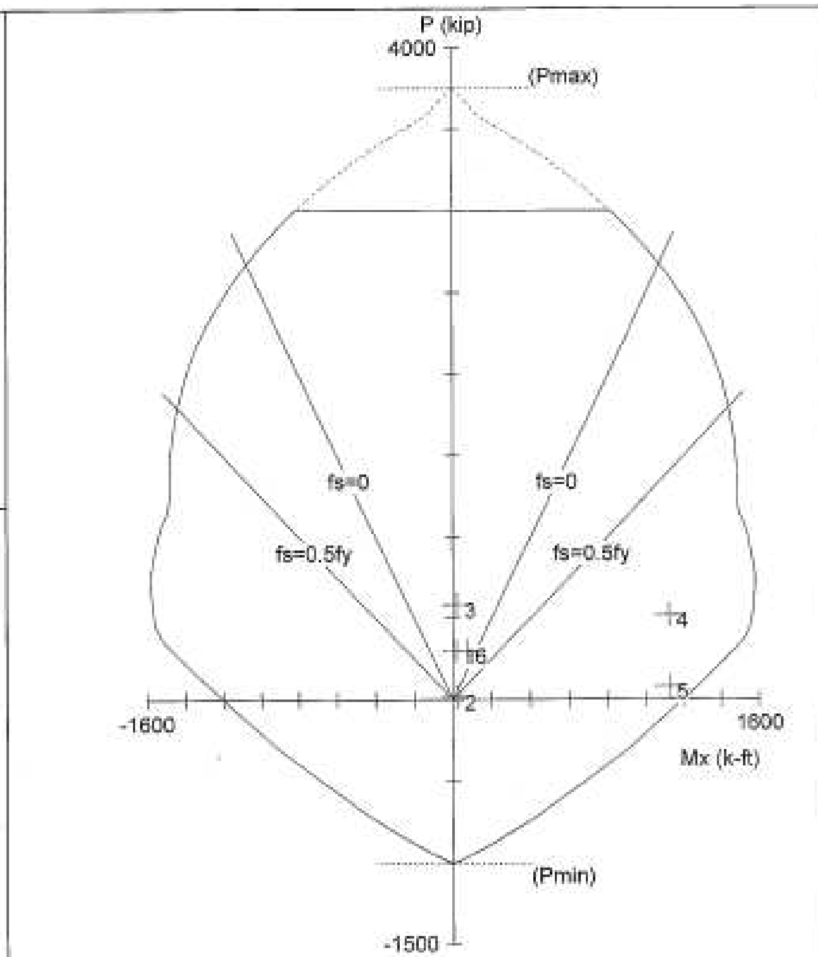
Mupotrans	4325	kip*ft	petran =	0.355
Futran	866	kips	pelong =	0.355
Mupolong	217	kip*ft		
Pulong	0.31	kips		

Pu = PuDead +/- Pufran → 520 or 80 kips (220)
Mu = MuDead + Mupotran → 1125 kip*ft

Pu = PuDead +/- Pulong → 2.. or 1.. kips
Mu = MuDead + Mupolong → 77.8 kip*ft



42 in diam.



Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/02/10

Time: 13:54:47

pcaColumn v3.64. Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59

File: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\Oseligee Cre...\Bent2ColumnDesign.col

Project: Oseligee Creek

Column: Bent 2		Engineer: PJC	
f'c = 4 ksi	fy = 60 ksi	Ag = 1385.44 in^2	12 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 18.72 in^2	Rho = 1.35%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 152745 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 152745 in^4
Beta1 = 0.85		Clear spacing = 5.73 in	Clear cover = 6.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

01:54 PM

```
0000000     00000     00000     00000     00000   00
00     00  00    00  00    00  00    00  00    00  00
00     00  00    00  00    00  00    00  00    00  00
00     00  00        00    00  00        00    00  00
00     00  00        0000000   00        00    00  00
0000000    00    00  00    00  00    00  00    00  00
00         00    00  00    00  00    00  00    00  00
00          00000    00    00   00000     00000   00000  (TM)
```

Computer program for the Strength Design of Reinforced Concrete Sections

01:54 PM

General Information:
```
====================
   File Name: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\Oseligee
Creek\PCA
Column\Bent2ColumnDesign.col
   Project:  Oseligee Creek
   Column:   Bent 2                    Engineer: PJC
   Code:     ACI 318-02                Units: English

   Run Option: Investigation           Slenderness: Not considered
   Run Axis:   X-axis                  Column Type: Structural
```

Material Properties:
```
====================
   f'c  = 4 ksi                        fy  = 60 ksi
   Ec   = 3605 ksi                     Es  = 29000 ksi
   Ultimate strain = 0.003 in/in
   Beta1 = 0.85
```

Section:
```
========
   Circular:   Diameter = 42 in

   Gross section area, Ag =  1385.44 in^2
   Ix =  152745 in^4                   Iy =  152745 in^4
   Xo =  0 in                          Yo =  0 in
```

Reinforcement:
```
==============
   Rebar Database: ASTM A615
```

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

```
   Confinement: Tied; #3 ties with #10 bars,  #4 with larger bars.
   phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.65

   Layout: Circular
   Pattern: All Sides Equal  (Cover to transverse reinforcement)
   Total steel area, As = 18.72 in^2 at 1.35%
   12 #11   Cover = 6 in
```

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)

No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	300.0	22.4	1467.2	65.498
2	16.0	22.4	1214.3	54.210
3	584.0	22.4	1570.6	70.116
4	520.0	1125.0	1564.1	1.390
5	80.0	1125.0	1272.6	1.131
6	300.0	77.8	1467.2	18.858

*** Program completed as requested! ***

Designer: P J C
Project Name: OSPLIGER CREEK
Job Number:
Date: 11/2/2010

Moment Capacity and Interaction Diagram Data

Column or Drilled Shaft: BENT 2 COLUMNS

INPUTS

Column Diameter	42	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	12	
Number of Lonitudinal Reinforcing Bars	# 11	
Transverse Reinforcing Bar Size	# 5	
Spacing of Transverse Reinforcing Bars	12	in
Column Height	25.8	ft
Cover	6	in

Dead Load Reactions

PuDead	316	kips
MuDead	9.7	kip*ft

Enter into Column Design Software (PCA COLUMN)
Finding Moment Capacity of the column.

Mp = ϕMp/0.9 → 1644 kip*ft

Vpcol = 2*Mp/Heightcolumn → 127.5 kips

Vpbent = 2*Vpcol 255 kips

Create SAP model of Bent
Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Puoverturning = 255 kips

Pu = PuDead +/- Puoverturning 571 or 61 kips

Re-enter into Column Design Software (PCA COLUMN)

Mp = ϕMp/0.9 → 1744 kip*ft

Vpcol = 2*Mp/Heightcolumn → 135 kips

Vpbent = 2*Vpcol 270 kips

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

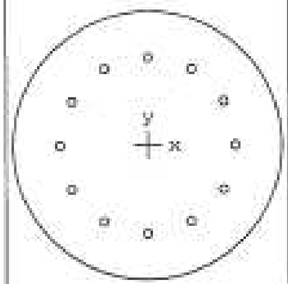
Check = ((Vpbent2 − Vpbent1) / Vpbent2) * 100% 5.55 % OK

Check Other Seismic Load Cases

Mupotrans	2955	kip*ft	petran =	0.255
Putran	397	kips	pelong =	0.255
Mupolong	124	kip*ft		
Pulong	0.232	kips		

Pu = PuDead +/- Putran → 417 or 215 kips
Mu = MuDead + Mupotran → 702.8 kip*ft

Pu = PuDead +/- Pulong → 316 or 316 kips
Mu = MuDead + Mupolong → 41.22 kip*ft



42 in diam.

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

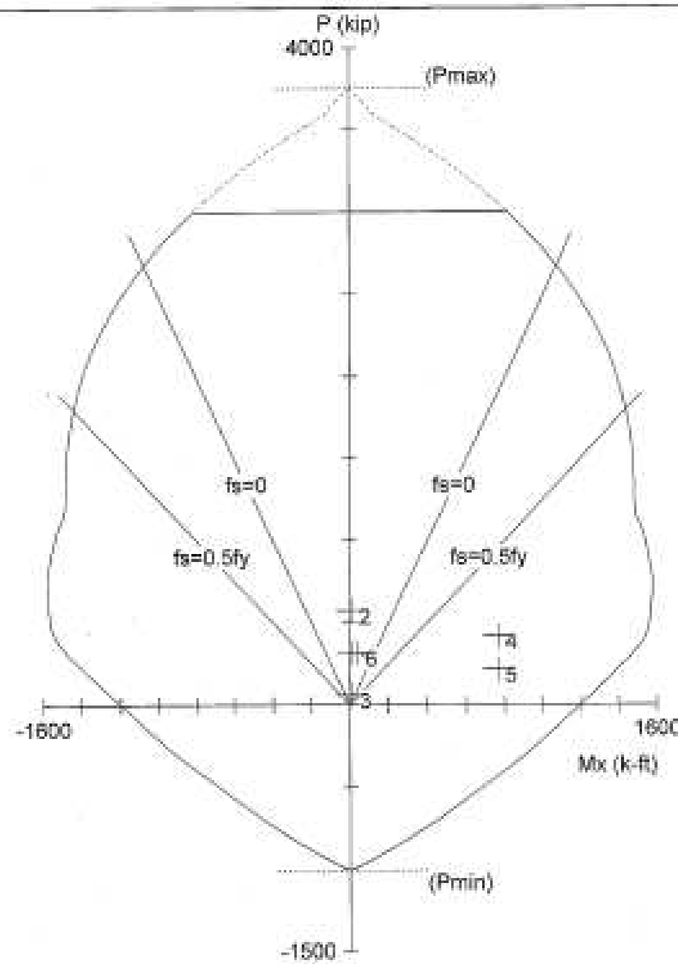
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/02/10

Time: 14:05:07



pcaColumn v3.64. Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59

File: untitled.col

Project: Oseligee Creek

Column: Bent 3		Engineer: PJC	
f'c = 4 ksi	fy = 60 ksi	Ag = 1385.44 in^2	12 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 18.72 in^2	Rho = 1.35%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 152745 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 152745 in^4
Beta1 = 0.85		Clear spacing = 5.73 in	Clear cover = 6.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

```
OOOOOOO   OOOOO     OOOOO     OOOOO     OOOOO   OO
OO   OO  OO   OO  OO   OO  OO   OO  OO   OO  OO
OO   OO  OO   OO  OO   OO  OO   OO  OO   OO  OO
OO   OO  OO       OO   OO  OO       OO   OO  OO
OO   OO  OO       OOOOOOO  OO       OO   OO  OO
OOOOOOO  OO   OO  OO   OO  OO   OO  OO   OO  OO
OO       OO   OO  OO   OO  OO   OO  OO   OO  OO
OO        OOOOO   OO   OO   OOOOO    OOOOO   OOOOO  (TM)
```

Computer program for the Strength Design of Reinforced Concrete Sections

General Information:

File Name: untitled.col
Project: Oseligee Creek
Column: Bent 3
Code: ACI 318-02
Engineer: PJC
Units: English

Run Option: Investigation
Run Axis: X-axis
Slenderness: Not considered
Column Type: Structural

Material Properties:

f'c = 4 ksi
Ec = 3605 ksi
Ultimate strain = 0.003 in/in
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

Section:

Circular: Diameter = 42 in

Gross section area, Ag = 1385.44 in^2
Ix = 152745 in^4
Xo = 0 in
Iy = 152745 in^4
Yo = 0 in

Reinforcement:

Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Circular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area, As = 18.72 in^2 at 1.35%
12 #11 Cover = 6 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)

No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	316.0	9.7	1480.7	152.651
2	571.0	9.7	1569.4	161.795
3	61.0	9.7	1256.0	129.482
4	417.0	763.2	1549.1	2.030
5	215.0	763.2	1393.6	1.826
6	316.0	41.3	1480.7	35.835

*** Program completed as requested! ***

Designer: PSC
Project Name: [illegible] CREEK
Job Number:
Date: 11 / 3 / 2009

Moment Capacity and Interaction Diagram Data
Column or Drilled Shaft: ABUTMENT DRILLED SHAFT

INPUTS

Column Diameter	42	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	#8 / 16	
Number of Longitudinal Reinforcing Bars	# / 11	
Transverse Reinforcing Bar Size	#5	
Spacing of Transverse Reinforcing Bars	6	in
Column Height	8	ft
Cover	6	in

Dead Load Reactions

PuDead	151	kips
MuDead	1008	kip*ft

Enter into Column Design Software (PCA COLUMN)
Finding Moment Capacity of the column.

Mp = φMp/0.9 → ______ kip*ft

Vpcol = 2*Mp/Heightcolumn → ______ kips

Vpbent = 2*Vpcol ______ kips

Create SAP model of Bent
Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Puoverturning = ______ kips

Pu = PuDead +/- Puoverturning ______ or ______ kips

Re-enter into Column Design Software (PCA COLUMN)

Mp = φMp/0.9 → ______ kip*ft

Vpcol = 2*Mp/Heightcolumn → ______ kips

Vpbent = 2*Vpcol ______ kips

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

Check = (|Vpbent2 - Vpbent1| / Vpbent2) * 100% ______

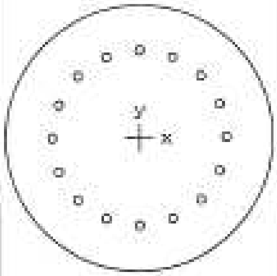
Check Other Seismic Load Cases

Mupotrans	1075	kip*ft	potran =	0.255
Putran	487	kips	polong =	0.255
Mupolong	1516	kip*ft		
Pulong	13.7	kips		

Pu = PuDead +/- Putran → 326 or 24 kips
Mu = MuDead + Mupotran → 1433 kip*ft

Pu = PuDead +/- Pulong → 154 or 148 kips
Mu = MuDead + Mupolong → 1246 kip*ft

DUE TO SMALL AXIAL LOAD IN DRILLED SHAFTS, INCREASE LONGITUDINAL REINFORCING # 16 & #11 BARS



42 in diam.

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

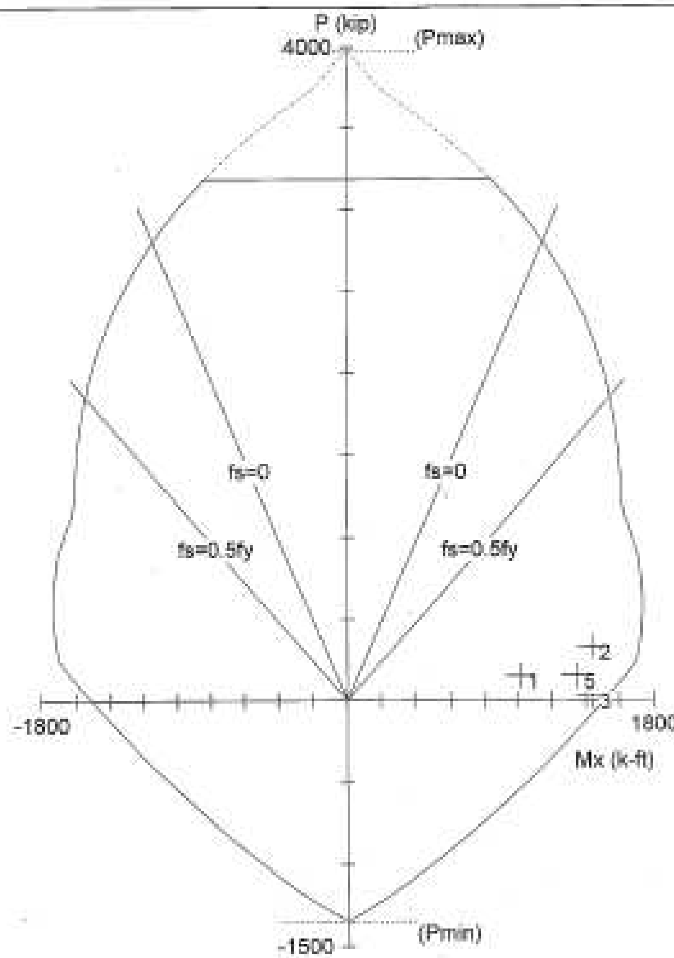
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/02/10

Time: 14:26:51



pcaColumn v3.64. Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59

File: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\Oseligee ...\AbutmentColumnDesign.col

Project: Oseligee Creek

Column: Abutment		Engineer: PJC	
f'c = 4 ksi	fy = 60 ksi	Ag = 1385.44 in^2	16 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 24.96 in^2	Rho = 1.80%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 152745 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 152745 in^4
Beta1 = 0.85		Clear spacing = 3.97 in	Clear cover = 6.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

02:25 PM

```
0000000    00000    00000    00000    00000   00
00    00  00   00  00   00  00   00  00   00  00
00    00  00   00  00   00  00   00  00   00  00
00    00  00       00   00  00       00   00  00
00    00  00       0000000  00       00   00  00
0000000   00   00  00   00  00   00  00   00  00
00        00   00  00   00  00   00  00   00  00
00         00000   00   00   00000    00000   00000  (TM)
```

===
Computer program for the Strength Design of Reinforced Concrete Sections
===

Licensee stated above acknowledges that Portland Cement Association (PCA) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the pcaColumn(tm) computer program. Furthermore, PCA neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the pcaColumn(tm) program.Although PCA has endeavored to produce pcaColumn(tm) error free, the program is not and can't be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensees. Accordingly, PCA disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the pcaColumn(tm) program.

02:25 PM

General Information:
====================
File Name: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\Oseligee Creek\PCA Column\AbutmentColumnDesign.col
Project: Oseligee Creek
Column: Abutment
Code: ACI 318-02
Engineer: BJC
Units: English

Run Option: Investigation
Run Axis: X-axis
Slenderness: Not considered
Column Type: Structural

Material Properties:
====================
f'c = 4 ksi
Ec = 3605 ksi
Ultimate strain = 0.003 in/in
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

Section:
========
Circular: Diameter = 42 in

Gross section area, Ag = 1385.44 in^2
Ix = 152745 in^4
Xo = 0 in
Iy = 152745 in^4
Yo = 0 in

Reinforcement:
==============
Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Circular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area, As = 24.96 in^2 at 1.80%
16 #11 Cover = 6 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
==

No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	151.0	1005.0	1632.9	1.625
2	326.0	1432.0	1714.2	1.197
3	24.0	1432.0	1526.8	1.066
4	154.0	1340.0	1635.5	1.220
5	148.0	1340.0	1630.5	1.217

*** Program completed as requested! ***

Appendix D: Little Bear Creek Bridge Guide Specification Design

Designer: Paul Coulston
Project Name: Little Bear Creek Bridge
Job Number:
Date: 7/22/2010

$ORIGIN := 1$

Description of worksheet: This worksheet is a seismic bridge design worksheet for the **AASHTO Guide Specifications for LRFD Seismic Bridge Design**. All preliminary design should already be done for non seismic loads.

Project Known Information

Location: Franklin County
Zip Code or Coordinates: 35.0069 N 88.2025 W

Superstructure Type: AASTHO I girders
Substructure Type: Circular columns supported on drilled shafts
Abutment Type: Abutment beam supported on drilled shafts

Note: *Input all of the below information.*

$fc := 4000 \quad psi$

$fye := 60000 \quad psi$

$\rho_{conc} := 0.08681 \quad \frac{lb}{in^3}$

$g := 386.4 \quad \frac{in}{s^2}$

Length of Bridge (ft)	$L := 300$	ft
Short Span (ft)	$ShortSpan := 85$	ft
Long Span (ft)	$LongSpan := 130$	ft
Deck Thickness (in)	$t_{deck} := 7$	in
Deck Width (ft)	$DeckWidth := 42.75$	ft
I-Girder X-Sectional Area (in^2)	$IGirderArea := 559.5$	in^2
Bulb Girder X-Sectional Area (in^2)	$BulbGirderArea := 767$	in^2
Guard Rail Area (in^2)	$GuardRailArea := 310$	in^2
Bent Volume (ft^3)	$BentVolume := 40\cdot(7\cdot5 + 2.4\cdot2.5) = 1.64 \times 10^3$	ft^3
Column Diameter (in)	$Columndia := 54$	in
Drilled Shaft Diameter (in)	$DSdia := 60$	in
Drilled Shaft Abutment Diameter (in)	$DSabutdia := 42$	in

Average Column Height (ft) $ColumnHeight := 14.5$ ft

Column Area (in^2) $Acolumn := \frac{Columndia^2 \cdot \pi}{4} = 2.29 \times 10^3$ in^2

Drilled Shaft Area (in^2) $Adrilledshaft := \frac{DSdia^2 \pi}{4} = 2.827 \times 10^3$ in^2

Note: These are variables that were easier to input in ft and then convert to inches.

$ShortSpan := ShortSpan \cdot 12 = 1.02 \times 10^3$ in

$LongSpan := LongSpan \cdot 12 = 1.56 \times 10^3$ in

$L := L \cdot 12 = 3.6 \times 10^3$ in

$DeckWidth := DeckWidth \cdot 12 = 513$ in

$BentVolume := BentVolume \cdot 12^3 = 2.834 \times 10^6$ in^3

$ColumnHeight := ColumnHeight \cdot 12 = 174$ in

Steps for Seismic Design

Article 3.1: The Guide Specification only applies to the design of **CONVENTIONAL** BRIDGES.

Article 3.2: Bridges are design for the life safety performance objective.

Article 6.2: Requires a subsurface investigation take place.

Article 6.8 and C6.8: Liquefaction Design Requirements - A liquefaction assessment should be made if loose to very loose sands are present to a sufficient extent to impact the bridge stability and A_s is greater than or equal to 0.15.

Article 3.3: The type of Earthquake Resisting System (ERS) should be considered. This is not a requirement as in SDC C and D, but should be considered. A Type 1 ERS has a ductile substructure and essentially elastic superstructure.

Type of Bridge: TYPE 1

***Article 3.4*: Determine Design Response Spectrum**

Note: AASHTO Guide Specifications for LRFD Seismic Bridge Design is accompanied with a program that has the Seismic Hazard maps. This program will calculate several of the variables that are needed for the analysis.

1) *Article 3.4.2.1*:Determine the Site Class. Table 3.4.2.1-1

INPUT Site Class: **D**

2) Enter maps and find PGA, S_s,and S_1. Then enter those values in their respective spot. Also, the the Guide Specification is accompanied with a cd that contains a program that will find these values for the designer.

$PGA := 0.116 \ g$

INPUT $S_s := 0.272 \ g$

$S_1 := 0.092 \ g$

3) *Article 3.4.2.3:* Site Coefficients. From the PGA,S_s, and S_1 values and site class choose F_{PGA}, F_a, and F_v. Note: straight line interpolation is permitted.

$F_{PGA} := 1.57$ Table 3.4.2.3-1

INPUT $F_a := 1.58$ Table 3.4.2.3-1

$F_v := 2.4$ Table 3.4.2.3-2

Eq. 3.4.1-1 $A_s := F_{PGA} \cdot PGA = 0.182 \ g$ A_s : Acceleration Coefficient

Eq. 3.4.1-2 $SDS := F_a \cdot S_s = 0.43 \ g$ S_{DS} = Short Period Acceleration Coefficient

Eq. 3.4.1-3 $SD1 := F_v \cdot S_1 = 0.221 \ g$ S_{D1} = 1-sec Period Acceleration Coefficient

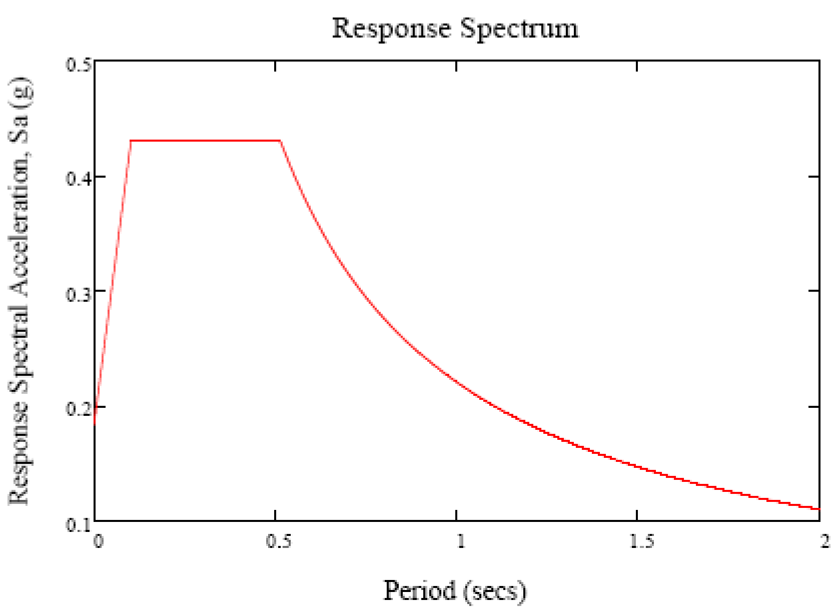
4) Creating a Response Spectrum

Note: The DesignSpectrum Code creates a Response Spectrum for the Bridge.
At this time the period of the bridge is unknown; therefore, the Sa value cannot be calculated.

$Tmax := 2\ s \qquad Dt := 0.001\ s$

$$DesignSpectrum(SDS, SD1, A_s, Tmax, Dt) := \left| \begin{array}{l} T_s \leftarrow \dfrac{SD1}{SDS} \\ T_o \leftarrow 0.2 \cdot T_s \\ n_{max} \leftarrow \dfrac{Tmax}{Dt} \\ \text{for } i \in 1..n_{max} \\ \quad \left| \begin{array}{l} T_i \leftarrow Dt \cdot i \\ a_i \leftarrow (SDS - A_s) \cdot \dfrac{Dt \cdot i}{T_o} + A_s \ \text{ if } Dt \cdot i < T_o \\ a_i \leftarrow SDS \ \text{ if } Dt \cdot i \ge T_o \wedge Dt \cdot i \le T_s \\ a_i \leftarrow \dfrac{SD1}{Dt \cdot i} \ \text{ if } Dt \cdot i > T_s \end{array} \right. \\ R \leftarrow augment(T, a) \\ R \end{array} \right.$$

$BridgeSpectrum := DesignSpectrum(SDS, SD1, A_s, Tmax, Dt)$



Article 3.5: **Selection of Seismic Design Category**

$SD1 = 0.221 \quad g$ From Table 3.5-1 Choose SDC

$$SDCprogram(SD1) := \left| \begin{array}{l} \text{for } c \in SD1 \\ \quad \left| \begin{array}{l} c \leftarrow \text{"A"} \quad \text{if } SD1 < 0.15 \\ c \leftarrow \text{"B"} \quad \text{if } SD1 \geq 0.15 \wedge SD1 < 0.3 \\ c \leftarrow \text{"C"} \quad \text{if } SD1 \geq 0.3 \wedge SD1 < 0.5 \\ c \leftarrow \text{"D"} \quad \text{if } SD1 \geq 0.5 \end{array} \right. \\ Rs \leftarrow c \\ c \end{array} \right.$$

$$SDC := SDCprogram(SD1) = \text{"B"}$$

Displacement Demand Analysis Δ_D

Figure 1.3-2 Demand Analysis Flowchart

Article 4.2: **Selection of Analysis Procedure**

This is a function of the SDC and the regularity of the bridge.

Procedure 1 = Equivalent Static Method

Article 4.3.3: Displacement Magnification for Short-Period Structures

$u_d := 2$ for SDC B

$$Rdprogram(T, SDS, SD1, u_d) := \left| \begin{array}{l} Ts \leftarrow \dfrac{SD1}{SDS} \\ Tb \leftarrow 1.25 \cdot Ts \\ x \leftarrow \left(1 - \dfrac{1}{u_d}\right) \cdot \dfrac{Tb}{T} + \dfrac{1}{u_d} \\ y \leftarrow 1.0 \\ a \leftarrow x \quad \text{if } \dfrac{Tb}{T} > 1.0 \\ a \leftarrow y \quad \text{if } \dfrac{Tb}{T} \leq 1.0 \\ a \end{array} \right.$$

Note: This Rd value will be calculated when the period of the structure is known. This factor will amplify the displacement demand.

***Article 5.4*: Analytical Procedure 1 (Equivalent Static Analysis)**

Note: There are two methods that can be used according to this procedure. The Uniform Load Method is suitable for regular bridges that respond principally in their fundamental mode of vibration. The Single Mode Spectral Method may be a better method if there is a major change in the spans, stiffness of the piers, etc.
The Uniform Load Method is simpler and less time consuming and will give accurate results, and this is the reason it has been chosen in this design.

Uniform Load Method

Step 1: Build a bridge model
Step 2: Apply a uniform load of Po = 1.0 kip/in. in both the longitudinal and transverse direction. Also, the uniform load can be converted into point loads and applied as joint loads in SAP. Calculate the static displacement for both directions. In SAP, tables of the displacements can be exported to EXCEL, and the MAX Function can be used to find the maximum displacement.

Step 3: Calculate the bridge lateral stiffness, K, and total weight, W.

$$p_o := 1.0 \quad \frac{kip}{in}$$

INPUT

$$\upsilon_{smaxLong} := 0.647204 \quad in$$

$$\upsilon_{smaxTran} := 5.263053 \quad in$$

Eq. C5.4.2-1 $$K_{Long} := \frac{p_o \cdot L}{\upsilon_{smaxLong}} = 5.562 \times 10^3 \qquad \frac{kip}{in}$$

Eq. C5.4.2-2 $$K_{Tran} := \frac{p_o \cdot L}{\upsilon_{smaxTran}} = 684.014 \qquad \frac{kip}{in}$$

INPUT: Multiplying factors

$$W := \frac{\rho_{conc} \cdot \left(\begin{array}{l} L \cdot t_{deck} \cdot DeckWidth + 2 \cdot BentVolume + 4 \cdot Acolumn \cdot ColumnHeight \ ... \\ + 2ShortSpan \cdot 6 \cdot IGirderArea + 6 \cdot LongSpan \cdot BulbGirderArea + 2 \cdot GuardRailArea \cdot L \end{array} \right)}{1000}$$

$$W = 3164.123 \qquad kips$$

Step 4: Calculate the period, T_m.

Eq. C5.4.2-3 $$T_{mLong} := 2\pi \cdot \sqrt{\frac{W}{K_{Long} \cdot g}} = 0.241 \quad s$$

Step 5: Calculate equivalent static earthquake loading p_e.

$$acc(SDS, SD1, T_{mLong}, A_s) := \left| \begin{array}{l} T_s \leftarrow \frac{SD1}{SDS} \\ T_o \leftarrow 0.2 \cdot T_s \\ \text{for } a \in T_{mLong} \\ \quad \left| \begin{array}{l} a \leftarrow (SDS - A_s) \cdot \frac{T_{mLong}}{T_o} + A_s \ \text{ if } T_{mLong} < T_o \\ a \leftarrow SDS \ \text{ if } T_{mLong} \geq T_o \wedge T_{mLong} \leq T_s \\ a \leftarrow \frac{SD1}{T_{mLong}} \ \text{ if } T_{mLong} > T_s \end{array} \right. \\ Ra \leftarrow a \\ a \end{array} \right.$$

$$Sa_{Long} := acc(SDS, SD1, T_{mLong}, A_s) = 0.43$$

Eq. C5.4.2-4 $$p_{eLong} := \frac{Sa_{Long} \cdot W}{L} = 0.378 \quad \frac{kip}{in}$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$Rd_{Long} := Rdprogram(T_{mLong}, SDS, SD1, u_d) = 1.832$$

$$v_{smaxLong} := Rd_{Long} \cdot \frac{p_{eLong}}{p_o} \cdot \upsilon_{smaxLong} = 0.448 \quad in$$

Repeat Steps 4, 5, and 6 for transverse loading.

Step 4: Calculate the period, T_m.

Eq. C5.4.2-3
$$T_{mTran} := 2\pi \cdot \sqrt{\frac{W}{K_{Tran} \cdot g}} = 0.687 \quad s$$

Step 5: Calculate equivalent static earthquake loading p_e.

$$Sa_{Tran} := acc(SDS, SD1, T_{mTran}, A_s) = 0.321$$

Eq. C5.4.2-4
$$p_{eTran} := \frac{Sa_{Tran} \cdot W}{L} = 0.282 \quad \frac{kip}{in}$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$Rd_{Tran} := Rdprogram(T_{mTran}, SDS, SD1, u_d) = 1$$

$$v_{smaxTran} := Rd_{Tran} \cdot \frac{p_{eTran}}{p_o} \cdot \upsilon_{smaxTran} = 1.486 \quad in$$

Single-Mode Spectral Method

This procedure is not specifically addressed in the Guide Specifications. The Guide Spec. refers you to the AASHTO LRFD Bridge Design Specifications.

Article 4.7.4.3.2: Single-Mode Spectral Method

Step 1: Build a bridge model
Step 2: Apply a uniform load of Po = 1.0 in both the longitudinal and transverse direction. Calculate the static displacement for both directions.

Step 3: Calculate factors α, β , and γ.

Note: The Deflection equations come from analysis of the SAP model. The displacement is taken at the joints along the length of the bridge and input into an Excel Worksheet. Then a graph is created of the displacements along the length of the bridge. A best fit line is plotted, and that is the equation that is shown below.

$v_{stran}(x) := 1\cdot 10^{-7}\cdot x^2 + 0.0017\cdot x + 0.3412$ $\qquad$ $v_{slong}(x) := -2\cdot 10^{-9}\cdot x^2 + 0.0001\cdot x + 0.2223$

C4.7.4.3.2b-1 $\qquad \alpha_{Tran} := \int_0^L v_{stran}(x)\,dx$ $\qquad \alpha_{Long} := \int_0^L v_{slong}(x)\,dx$

C4.7.4.3.2b-2 $\qquad \beta_{Tran} := \int_0^L \frac{W}{L} v_{stran}(x)\,dx$ $\qquad \beta_{Long} := \int_0^L \frac{W}{L}\cdot v_{slong}(x)\,dx$

C4.7.4.3.2b-3 $\qquad \gamma_{Tran} := \int_0^L \frac{W}{L}\cdot v_{stran}(x)^2\,dx = 6.102\times 10^4$ $\qquad \gamma_{Long} := \int_0^L \frac{W}{L}\cdot v_{slong}(x)^2\,dx$

α = Displacement along the length

β = Weight per unit length * Displacement

γ = Weight per unit length * Displacement^2

Step 4: Calculate the Period of the Bridge

Eq. 4.7.4.3.2b-4 $\qquad T_{mTran1} := 2\pi\cdot\sqrt{\frac{\gamma_{Tran}}{p_o\cdot g\cdot \alpha_{Tran}}} = 0.672$ $\qquad$ s

Eq. 4.7.4.3.2b-4 $\qquad T_{mLong1} := 2\pi\cdot\sqrt{\frac{\gamma_{Long}}{p_o\cdot g\cdot \alpha_{Long}}} = 0.194$ $\qquad$ s

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smLong} := acc\left(SDS, SD1, T_{mLong1}, A_s\right) = 0.43$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

Eq. C4.7.4.3.2b-5 $$PeLong(x) := \frac{\beta_{Long} \cdot C_{smLong}}{\gamma_{Long}} \cdot \frac{W}{L} \cdot v_{slong}(x)$$

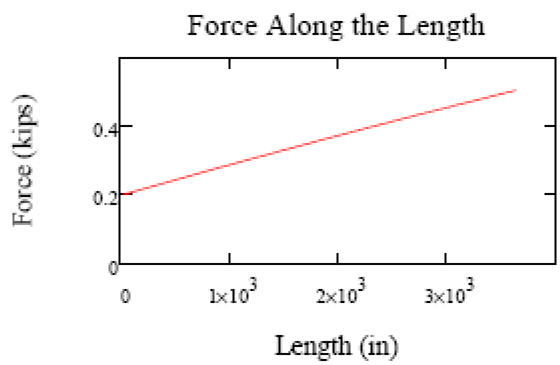
$PeLong(x) \rightarrow 0.000090517538799082222132 \cdot x + -1.8103507759816444426e\text{-}9 \cdot x^2 + 0.20122048875035977$

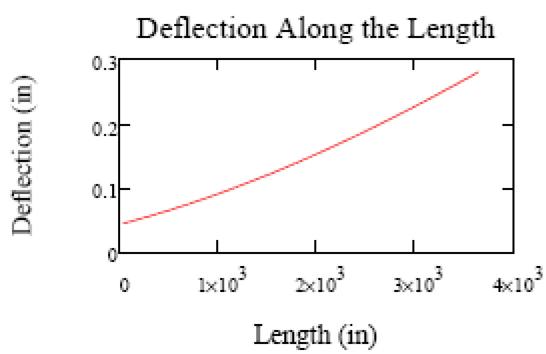
$dW := \frac{L}{100}$

$i := 1..101$

$Pelong_i := PeLong[(i-1) \cdot dW]$ $\delta long_i := v_{slong}[(i-1)dW]$

$\Delta long_i := Pelong_i \cdot \delta long_i$





Maximum Deflection

$\max(\Delta long) = 0.28$ in

NOTE: Repeat Steps 5 and 6 for Transverse Direction.

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smTran} := acc\left(SDS, SD1, T_{mTran1}, A_s\right) = 0.328$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

Eq. C4.7.4.3.2b-5 $$PeTran(x) := \frac{\beta_{Tran} \cdot C_{smTran}}{\gamma_{Tran}} \cdot \frac{W}{L} \cdot v_{stran}(x)$$

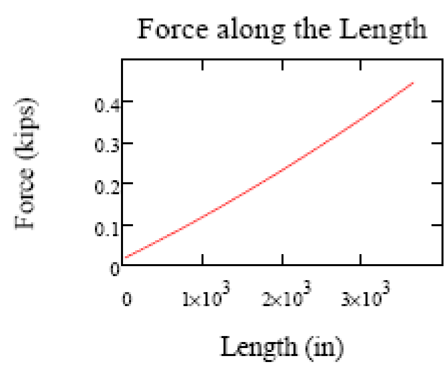
$$PeTran(x) \rightarrow 0.00009755430945480492 1037 \cdot x + 5.7384887914591130022e\text{-}9 \cdot x^2 + 0.019579723756458493563$$

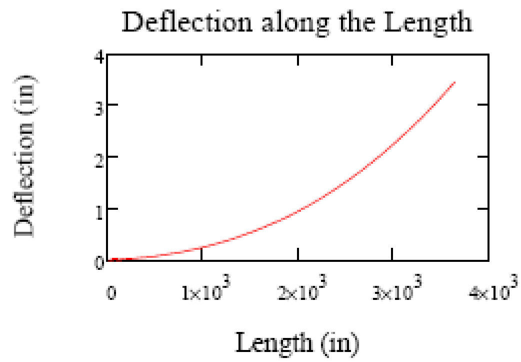
$$dL := \frac{L}{100}$$

$$i := 1..101$$

$$Petran_i := PeTran[(i-1) \cdot dL] \qquad \delta tran_i := v_{stran}[(i-1)dL]$$

$$\Delta tran_i := Petran_i \cdot \delta tran_i$$





Maximum Deflection

$\max(\Delta tran) = 3.453$ in

***Article 5.6*: Effective Section Properties**

Note: Use 0.7*Ig for ductile reinforced concrete members.
Refer to the charts on page 5-20 of the Guide Specification if a more precise value is desired.

***Article 5.2:* Abutment Modeling**

Note: This is taken care of in the SAP model.

***Article 5.3*: Foundations Modeling**

Note: Since in SDC B, Foundation Modeling Methods I can be used.

FMM is dependent on the type of foundation.

For bridges with Pile Bent/Drilled Shaft the depth of fixity can be estimated.
Since details regarding reinforcing are not known, reduce the stiffness of the drilled shafts to **one half** the uncracked section.

Note: Special provisions need to be considered if Liquefaction is present. (Article 6.8)

Article 4.4: Combination of Orthogonal Seismic Displacement Demands

$$LoadCase1 := \sqrt{\left(1\cdot v_{smaxLong}\right)^2 + \left(0.3\cdot v_{smaxTran}\right)^2} = 0.632 \quad in$$

$$LoadCase2 := \sqrt{\left(1\cdot v_{smaxTran}\right)^2 + \left(0.3\cdot v_{smaxLong}\right)^2} = 1.492 \quad in$$

COLUMN DESIGN

Article 4.8: **Displacement Demand/Capacity**

Note: If the column height is different for each bent, a capacity check needs to be made at each bent.

Displacement Demand/Capacity for the Bents $\Delta_D < \Delta_C$

BENT 2

Note: The displacement demand is taken as the bent displacement. This can be found by using the SAP Bridge model that was created.

Input

$\Delta_{Dlong} := 0.2566$ in $\Delta_{DLong} := Rd_{Long} \cdot \Delta_{Dlong} \cdot p_{eLong} = 0.178$ in

$\Delta_{Dtran} := 0.7953$ in $\Delta_{DTran} := Rd_{Tran} \cdot \Delta_{Dtran} \cdot p_{eTran} = 0.225$ in

Input $H_o := 12.063$ ft

Input $B_o := \dfrac{Columndia}{12}$ ft

$\Lambda := 2$ Fixed and top and bottom

Eq. 4.8.1-3 $x := \dfrac{\Lambda \cdot B_o}{H_o} = 0.746$

Eq. 4.8.1-1 $\Delta_C := 0.12 \cdot H_o \cdot (-1.27 \cdot \ln(x) - 0.32) = 0.075$ in

$0.12 \cdot H_o = 1.448$ in

$$CheckLimit(\Delta_C) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } \Delta_C \ge 0.12 \cdot H_o \\ a \leftarrow \text{"FAILURE"} \quad \text{if } \Delta_C < 0.12 \cdot H_o \end{array} \right.$$

$CheckLimit(\Delta_C) = \text{"FAILURE"}$

$$\text{CheckCapacity}(\Delta_C, \Delta_D) := \begin{cases} c \leftarrow \text{"OK"} & \text{if } \Delta_C \geq \Delta_D \\ c \leftarrow \text{"FAILURE"} & \text{if } \Delta_C < \Delta_D \end{cases}$$

$$\text{CheckCapacity}(\Delta_C, \Delta_{DLong}) = \text{"FAILURE"}$$

$$\text{CheckCapacity}(\Delta_C, \Delta_{DTran}) = \text{"FAILURE"}$$

BENT 3

INPUT $\Delta_{Dlong} := 0.3702$ in $\Delta_{DLong} := Rd_{Long} \cdot \Delta_{Dlong} \cdot p_{eLong} = 0.256$ in

$\Delta_{Dtran} := 2.2407$ in $\Delta_{DTran} := Rd_{Tran} \cdot \Delta_{Dtran} \cdot p_{eTran} = 0.225$ in

INPUT $H_o := 16.681$ ft

INPUT $B_o := \frac{\text{Columndia}}{12}$ ft

$\Lambda := 2$ Fixed and top and bottom

Eq. 4.8.1-3 $x := \frac{\Lambda \cdot B_o}{H_o} = 0.54$

Eq. 4.8.1-1 $\Delta_C := 0.12 \cdot H_o \cdot (-1.27 \cdot \ln(x) - 0.32) = 0.928$ in

$0.12 \cdot H_o = 2.002$ in

$$\text{CheckLimit}(\Delta_C) := \begin{cases} a \leftarrow \text{"OK"} & \text{if } \Delta_C \geq 0.12 \cdot H_o \\ a \leftarrow \text{"FAILURE"} & \text{if } \Delta_C < 0.12 \cdot H_o \end{cases}$$

$$\text{CheckLimit}(\Delta_C) = \text{"FAILURE"}$$

$$\text{CheckCapacity}(\Delta_C, \Delta_D) := \begin{cases} c \leftarrow \text{"OK"} & \text{if } \Delta_C \geq \Delta_D \\ c \leftarrow \text{"FAILURE"} & \text{if } \Delta_C < \Delta_D \end{cases}$$

$$\text{CheckCapacity}(\Delta_C, \Delta_{DLong}) = \text{"OK"}$$

$$\text{CheckCapacity}(\Delta_C, \Delta_{DTran}) = \text{"OK"}$$

NOTE: IF THE SIMPLIFIED EQUATIONS ABOVE DO NOT WORK, A PUSHOVER ANALYSIS OF THE BRIDGE CAN BE DONE TO VERIFY THE DISPLACEMENT CAPACITY. In SAP 2000, there is an earthquake design program that allows a pushover analysis to be done by setting the SDC to D. Be sure to amplify the demand values by the appropriate Rd value. List the results below to verify that the Displacement Capacity is sufficient. The Demand Displacement must be multiplied by pe/po. The below chart was created in Excel and then brought into Mathcad.

GenDispl	Demand (in)	Capacity (in)	Check
_GD_TR1_DReq11	0.808448256	2.801242	OK
_GD_LG1_DReq11	1.212062438	1.522609	OK
_GD_TR2_DReq11	3.883949328	13.210685	OK
_GD_LG2_DReq11	1.737604208	2.514419	OK

***Article 4.12*: Minimum Support Length Requirements**

Note: May need to add more calculations if column heights are different at the bents.

N (in) = Minimum support length measured normal to the bridge

L (ft) = Length of bridge to adjacent expansion joint or end of the bridge

H (ft) = average height of columns supporting bridge deck for abutments
for columns and piers = column height

S (Degree) = angle of skew

Abutment Support Length Requirement

INPUT $\quad Span_{abutment} := \frac{ShortSpan}{12} = 85$ ft $\qquad H_{abutment} := \frac{ColumnHeight}{12} = 14.5$ ft

Note: The $Span_{abutment}$ is divided by number of spans and inches.

$Skew_{abutment} := 0$ Degrees

Eq. 4.12.2-1

$$Nabutment := 1.5\cdot\left(8 + 0.02Span_{abutment} + 0.08H_{abutment}\right)\cdot\left(1 + 0.000125Skew_{abutment}^{2}\right) = 16.29 \quad \text{in}$$

Bent Support Length Requirement

BENT 2

INPUT $\quad Span_{Bent} := \frac{LongSpan}{1 \cdot 12} = 130 \quad ft$

Note: The $Span_{abutment}$ is divided by number of spans and inches.

INPUT $\quad H_{Bent} := 12.063 \quad ft$ *INPUT*: Column Height for this Bent

INPUT $\quad Skew_{Bent} := 0 \quad Degrees$

Eq. 4.12.2-1

$$N_{Bent} := 1.5 \cdot \left(8 + 0.02 Span_{Bent} + 0.08 H_{Bent}\right) \cdot \left(1 + 0.000125 Skew_{Bent}^{\;2}\right) = 17.348 \quad in$$

BENT 3

INPUT $\quad Span_{Bent} := \frac{LongSpan}{1 \cdot 12} = 130 \quad ft$

Note: The $Span_{abutment}$ is divided by number of spans and inches.

INPUT $\quad H_{Bent} := 16.681 \quad ft$ *INPUT*: Column Height for this Bent

INPUT $\quad Skew_{Bent} := 0 \quad Degrees$

Eq. 4.12.2-1

$$N_{Bent} := 1.5 \cdot \left(8 + 0.02 Span_{Bent} + 0.08 H_{Bent}\right) \cdot \left(1 + 0.000125 Skew_{Bent}^{\;2}\right) = 17.902 \quad in$$

***Article 4.14*: Superstructure Shear Keys**

$$V_{ok} := 2 \cdot V_n$$

Note: This does not apply to this bridge.

Figure 1.3-5 SDC B Detailing

Decide what Type of bridge designing.

Structure Type: **Type 1**

***Article 8.3*: Determine Flexure and Shear Demands**

***Article 8.5*: Plastic Moment Capacity**

Note: Article 8.5 refers the designer back to Article 4.11.1-4.

BENT 3 DESIGN

***Article 4.11.1-4*: Steps to find Moment Capacity, Shear Capacity, and Axial Force**

Note: Use some kind of software to find the Moment Capacity of the Column. PCA Column was used to create an Interaction Diagram and to calculate the Moment Capacity. The shear for the bent was found by knowing the Moment.

INPUT $M_p := 60000000 \quad lb \cdot in$

INPUT $Fixity := 200\ in$

Note: Fixity is the point of fixity for the column/drilledshaft.

$$V_p := \frac{2 \cdot M_p}{Fixity \cdot 1000} = 600 \quad kips \qquad V_{pBent3} := 2 \cdot V_p = 1.2 \times 10^3 \quad kips$$

Note: If the decision is made to design for **Elastic Forces** then the above variables need to be override. This can be done by simply changing the Vp variable to the elastic force from SAP2000 that has been multiplied by pe/po.

INPUT $V_p := 290 \quad kips \qquad V_{pBent3} := 2 \cdot V_p = 580 \quad kips$

Note: Pu is the combination of Elastic Axial force from the earthquake and dead load.

INPUT $P_u := 1284000 \quad lb$

Article 8.6: Shear Demand and Capacity for Ductile Concrete Members

Note: It is recommended to use the plastic hinging forces whenever practical.

$V_u := V_p \qquad \phi_s := 0.9$

Article 4.11.6: **Analytical Plastic Hinge Length**

Note: For reinforced concrete columns framing into a footing, an integral bent cap, an oversized shaft, or cased shaft.

<u>*INPUT*</u> $d_{bl} := 1.41$ in $\qquad$ d_{bl}: Diameter of Longitudinal Bar

Eq. 4.11.6-1

$$\text{PlasticHinge}(\text{Fixity}, \text{fye}, d_{bl}) := \left| \begin{array}{l} lp \leftarrow 0.08 \cdot \text{Fixity} + 0.15 \cdot \frac{\text{fye}}{1000} \cdot d_{bl} \\ m \leftarrow 0.03 \cdot \frac{\text{fye}}{1000} \cdot d_{bl} \\ a \leftarrow lp \ \text{ if } lp \geq m \\ a \leftarrow m \ \text{ if } lp < m \\ a \end{array} \right.$$

$L_p := \text{PlasticHinge}(\text{Fixity}, \text{fye}, d_{bl}) = 28.69$ in

Article 4.11.7: **Reinforced Concrete Column Plastic Hinge Region**

Note: y is the region of column with a moment demand exceeding 75% of the maximum plastic moment. From the SAP model, find the location at which the moment demand is 0.75*Mp. The 0.75*Mp value should be divided by p_{eTran} to take into account the model loads have not been multiplied by p_{eTran}. The location will also need to be **<u>INPUT</u>** into the PlasticHingeRegion program in inches.

$Mp75 := 0.75 \cdot M_p = 4.5 \times 10^7$ lb·in

$$\text{PlasticHingeRegion}(L_p, \text{Columndia}) := \left| \begin{array}{l} z \leftarrow 1.5 \cdot \text{Columndia} \\ x \leftarrow L_p \\ y \leftarrow 0 \\ a \leftarrow \max(z, x, y) \end{array} \right.$$

Note: Input the arguments into the program. Most likely the column diameter is the only variable that has changed.

INPUT $Lpr := PlasticHingeRegion(L_p, Columndia) = 81$ in

***Article 8.6.2*: Concrete Shear Capacity**

$Ag := Acolumn$

Eq. 8.6.2-2 $Ae := 0.8 \cdot Ag = 1.832 \times 10^{3}$ in^{2}

$\mu_D := 2$ Specified in Article 8.6.2 of Guide Spec.

INPUT $s := 6$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $Asp := .31$ in^{2} Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.625$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 3$ in Cover: Concrete cover for the Column (in)

INPUT $Dprime := 48$ in Dprime: Diameter of spiral or hoop for circular columns (in)

Eq. 8.6.2-7 $$\rho_s := \frac{4 \cdot Asp}{s \cdot Dprime} = 4.306 \times 10^{-3}$$

$$fyh := \frac{fye}{1000} = 60 \quad \text{ksi}$$

Eq. 8.6.2-6 $$StressCheck(\rho_s, fyh) := \left| \begin{array}{l} fs \leftarrow \rho_s \cdot fyh \\ a \leftarrow fs \ \text{ if } \ fs \le 0.35 \end{array} \right.$$

$$fs := StressCheck(\rho_s, fyh) = 0.258$$

Eq. 8.6.2-5 $\alpha program(fs, \mu_D) := \begin{array}{|l} \alpha prime \leftarrow \frac{fs}{0.15} + 3.67 - \mu_D \\ a \leftarrow 0.3 \ \text{ if } \ \alpha prime \le 0.3 \\ a \leftarrow \alpha prime \ \text{ if } \ \alpha prime > 0.3 \wedge \alpha prime < 3 \\ a \leftarrow 3 \ \text{ if } \ \alpha prime \ge 3 \\ a \end{array}$

$\alpha Prime := \alpha program(fs, \mu_D) = 3$

If Pu is Compressive

Eq. 8.6.2-3 $vcprogram(\alpha Prime, fc, P_u, Ag) := \begin{array}{|l} vc \leftarrow 0.032 \cdot \alpha Prime \cdot \left(1 + \frac{P_u}{2Ag \cdot 1000}\right) \cdot \sqrt{\frac{fc}{1000}} \\ min1 \leftarrow 0.11 \sqrt{\frac{fc}{1000}} \\ min2 \leftarrow 0.047 \alpha Prime \cdot \sqrt{\frac{fc}{1000}} \\ minimum \leftarrow min(min1, min2) \\ a \leftarrow vc \ \text{ if } \ vc \le minimum \\ a \leftarrow minimum \ \text{ if } \ vc > minimum \\ a \end{array}$

Eq. 8.6.2-4

If Pu is **NOT** Compressive $vc = 0$

Note: If Pu is not compressive, will have to manually input 0 for vc. Just input it below the vc:=vcprogram and the variable will assume the new value.

$vc := vcprogram(\alpha Prime, fc, P_u, Ag) = 0.22$ ksi

$Vc := vc \cdot Ae = 403.079$ kips

***Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity**

INPUT $n := 2$ n: number of individual interlocking spiral or hoop core sections

Eq. 8.6.3-1
Eq. 8.6.4-1

$$\mathrm{vsprogram}(n, Asp, fyh, Dprime, s, fc, Ae) := \left| \begin{array}{l} vs \leftarrow \frac{\pi}{2} \cdot \left(\frac{n\,Asp \cdot fyh \cdot Dprime}{s} \right) \\ maxvs \leftarrow 0.25 \cdot \sqrt{\frac{fc}{1000}} \cdot Ae \\ a \leftarrow vs \quad \text{if } vs \le maxvs \\ a \leftarrow maxvs \quad \text{if } vs > maxvs \\ a \end{array} \right.$$

$$Vs := \mathrm{vsprogram}(n, Asp, fyh, Dprime, s, fc, Ae) = 467.469 \quad \text{kips}$$

Eq. 8.6.1-2 $$\phi Vn := \phi_s \cdot (Vs + Vc) = 783.493 \quad \text{kips}$$

$$\mathrm{ShearCheck}(\phi Vn, V_u) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } \phi Vn \ge V_u \\ a \leftarrow \text{"FAILURE"} \quad \text{if } \phi Vn < V_u \\ a \end{array} \right.$$

$$Shearcheck := \mathrm{ShearCheck}(\phi Vn, V_u) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

***Article 8.6.5*: Minimum Shear Reinforcement**

For Circular Columns

Eq. 8.6.5-1 $$\mathrm{mintranprogram}(\rho_s) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } \rho_s \ge 0.003 \\ a \leftarrow \text{"Increase Shear Reinforcing Ratio"} \quad \text{if } \rho_s < 0.003 \\ a \end{array} \right.$$

$$CheckTransverse := \mathrm{mintranprogram}(\rho_s) = \text{"OK"}$$

Note: If the minimum shear reinforcement program responses "Increase Shear Reinforcing Ratio", it is recommended to <u>decrease the spacing (s)</u> or <u>increase the area of the shear reinforcement (Asp)</u> in the inputs.

Article 8.8: **Longitudinal and Lateral Reinforcement Requirements**

Article 8.8.1: **Maximum Longitudinal Reinforcement**

INPUT $A_{bl} := 1.56 \quad in^2$

INPUT $NumberBars := 28$

$A_{long} := NumberBars \cdot A_{bl} = 43.68 \quad in^2$

Eq. 8.8.1-1
$$\rho program(A_{long}, Ag) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } A_{long} \le 0.04 \cdot Ag \\ a \leftarrow \text{"Section Over Reinforced"} \quad \text{if } A_{long} > 0.04 \cdot Ag \\ a \end{array} \right.$$

$$ReinforcementRaitoCheck := \rho program(A_{long}, Ag) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

Article 8.8.2: **Minimum Longitudinal Reinforcement**

For Columns in SDC B and C:

Eq. 8.8.2-1
$$minAlprogram(A_l, Ag) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } A_{long} \ge 0.007 \cdot Ag \\ a \leftarrow \text{"Increase Longitudinal Reinforcing"} \quad \text{if } A_{long} < 0.007 \cdot Ag \\ a \end{array} \right.$$

$$MinimumA_l := minAlprogram(A_{long}, Ag) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Article 8.8.9: **Requirements for Lateral Reinforcement for SDCs B,C, and D**

These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension **of NOT** less than **6*d_b** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Minimum Size of Lateral Reinforcement

#4 bars for #9 or smaller longitudinal bars
#5 bars for #10 or larger longitudinal bars
#5 bars for bundled longitudinal bars

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

Shall Not Exceed the Smallest of:

$$\text{Spacingprogram}(\text{Columndia}, d_{bl}) := \left| \begin{array}{l} q \leftarrow \left(\frac{1}{5}\right)\text{Columndia} \\ r \leftarrow 6 \cdot d_{bl} \\ t \leftarrow 6 \\ a \leftarrow \min(q, r, t) \\ a \end{array} \right.$$

$$\text{MaximumSpacing} := \text{Spacingprogram}(\text{Columndia}, d_{bl}) = 6 \qquad \text{in}$$

$$\text{SpacingCheck}(\text{MaximumSpacing}, s) := \left| \begin{array}{l} a \leftarrow s \ \text{ if } \ s \le \text{MaximumSpacing} \\ a \leftarrow \text{MaximumSpacing} \ \text{ if } \ s > \text{MaximumSpacing} \\ a \end{array} \right.$$

$$FINALSPACING := SpacingCheck(MaximumSpacing, s) = 6 \quad in$$

$$scheck := ShearCheck(MaximumSpacing, s) = \text{"OK"}$$

Note: If scheck returns "Failure", increase the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.3 (LRFD SPEC.): Column Connections

Note: This needs to be done whenever the column dimension changes. The spacing in the hinge region shall continue into the drilled shaft or cap beam the Extension length.

$$ExtensionProgram(d) := \left| \begin{array}{l} z \leftarrow 15 \\ x \leftarrow \frac{1}{2} \cdot d \\ a \leftarrow \max(z, x) \\ a \end{array} \right.$$

INPUT $Extension := ExtensionProgram(Columndia) = 27 \quad in$

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.
Refer to the **AASHTO LRFD Bridge Design Specifications**.

***5.8.3.3* Nominal Shear Resistance**

$V_p = 290$ kips

INPUT $spaceNOhinge := 10.5 \quad in$

INPUT $bv := Columndia$

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$\beta := 2.0$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad rad$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 49.67 \qquad in$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 42.81 \qquad in$$

$$dv := 0.9 \cdot de = 38.529 \qquad in$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 262.986 \qquad kips$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 136.504 \qquad kips$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 359.541 \qquad kips$$

$$ShearCheck\left(\phi Vn, V_u\right) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } \phi Vn \geq V_u \\ a \leftarrow \text{"FAILURE"} \quad \text{if } \phi Vn < V_u \\ a \end{array} \right.$$

$$Shearcheck := ShearCheck\left(\phi Vn, V_p\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.597 \qquad in^2$$

$$Av := 2 \cdot Asp = 0.62 \qquad in^2$$

$$\text{TranCheck}(\text{Avmin}, \text{Av}) := \left| \begin{array}{l} a \leftarrow \text{"Decrease Spacing or Increase Bar Size"} \quad \text{if Avmin} > \text{Av} \\ a \leftarrow \text{"OK"} \quad \text{if Avmin} \le \text{Av} \\ a \end{array} \right.$$

$$\text{MinimumTran} := \text{TranCheck}(\text{Avmin}, \text{Av}) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $\quad \text{vu} := \dfrac{V_u}{\phi_s \cdot \text{bv} \cdot \text{dv}} = 0.155 \quad$ ksi

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2

$$\text{spacingProgram}(\text{Vu}, \text{dv}, \text{fc}) := \left| \begin{array}{l} v \leftarrow 0.125 \cdot \dfrac{\text{fc}}{1000} \\ q \leftarrow 0.8 \cdot \text{dv} \\ r \leftarrow 0.4 \cdot \text{dv} \\ z \leftarrow q \quad \text{if } q \le 24 \\ z \leftarrow 24 \quad \text{if } q > 24 \\ t \leftarrow r \quad \text{if } r \le 12 \\ t \leftarrow 12 \quad \text{if } r > 12 \\ a \leftarrow z \quad \text{if Vu} < v \\ a \leftarrow t \quad \text{if Vu} \ge v \\ a \end{array} \right.$$

$$\text{MaxSpacing} := \text{spacingProgram}(\text{vu}, \text{dv}, \text{fc}) = 24 \quad \text{in}$$

$$\text{Spacecheck}(\text{MaxSpacing}, s) := \left| \begin{array}{l} a \leftarrow s \quad \text{if } s \le \text{MaxSpacing} \\ a \leftarrow \text{MaxSpacing} \quad \text{if } s > \text{MaxSpacing} \\ a \end{array} \right.$$

$$\text{MAXSPACING} := \text{Spacecheck}(\text{MaxSpacing}, \text{spaceNOhinge}) = 10.5 \quad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MAXSPACING}, \text{spaceNOhinge}) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

BENT 2 DESIGN

***Article 4.11.1-4*: Steps to find Moment Capacity, Shear Capacity, and Axial Force**

Note: Use some kind of software to find the Moment Capacity of the Column. PCA Column was used to create an Interaction Diagram and to calculate the Moment Capacity. The shear for the bent was found by knowing the Moment.

INPUT $M_p := 59328000 \quad lb \cdot in$

INPUT $Fixity := 145\,in$

Note: Fixity is the point of fixity for the column/drilledshaft.

$$V_p := \frac{2 \cdot M_p}{Fixity \cdot 1000} = 818.317 \quad kips \qquad V_{pBent2} := 2 \cdot V_p = 1.637 \times 10^{3}\,kips$$

Note: If the decision is made to design for **ELASTIC FORCES** then the above variables need to be override. This can be done by simply changing the Vp variable to the elastic force from SAP2000 that has been multiplied by pe/po.

INPUT $V_p := 170 \quad kips \qquad V_{pBent2} := 2 \cdot V_p = 340 \quad kips$

Note: Pu is the combination of Elastic Axial force from the earthquake and dead load.

INPUT $P_u := 950000 \quad lb$

Article 8.6: Shear Demand and Capacity for Ductile Concrete Members

Note: It is recommended to use the plastic hinging forces whenever practical.

$V_u := V_p \qquad \phi_s := 0.9$

Article 4.11.6: **Analytical Plastic Hinge Length**

Note: For reinforced concrete columns framing into a footing, an integral bent cap, an oversized shaft, or cased shaft.

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of Longitudinal Bar

$$L_p := \text{PlasticHinge}(\text{Fixity}, \text{fye}, d_{bl}) = 24.29 \quad \text{in}$$

Article 4.11.7: **Reinforced Concrete Column Plastic Hinge Region**

Note: y is the region of column with a moment demand exceeding 75% of the maximum plastic moment. From the SAP model, find the location at which the moment demand is 0.75*Mp. The 0.75*Mp value should be divided by p_{eTran} to take into account the model loads have not been multiplied by p_{eTran}. The location will also need to be **INPUT** into the PlasticHingeRegion program in inches.

$$Mp75 := 0.75 \cdot M_p = 4.45 \times 10^7 \quad \text{lb} \cdot \text{in}$$

$$\text{PlasticHingeRegion}(L_p, \text{Columndia}) := \begin{array}{|l} z \leftarrow 1.5 \cdot \text{Columndia} \\ x \leftarrow L_p \\ y \leftarrow 0 \\ a \leftarrow \max(z, x, y) \end{array}$$

Note: Input the arguments into the program. Most likely the column diameter is the only variable that has changed.

INPUT $Lpr := \text{PlasticHingeRegion}(L_p, \text{Columndia}) = 81$ in

Article 8.6.2: **Concrete Shear Capacity**

$$Ag := \text{Acolumn}$$

Eq. 8.6.2-2 $Ae := 0.8 \cdot Ag = 1.832 \times 10^3 \quad \text{in}^2$

$\mu_D := 2$ Specified in Article 6.8.2 Guide Spec.

INPUT $s := 6$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $Asp := 0.31$ in^2 Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.625$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 3$ in Cover: Concrete cover for the Column (in)

INPUT $Dprime := 48$ in Dprime: Diameter of spiral or hoop for circular columns (in)

Eq. 8.6.2-7 $\rho_s := \frac{4 \cdot Asp}{s \cdot Dprime} = 4.306 \times 10^{-3}$

$fyh := \frac{fye}{1000} = 60$ ksi

Eq. 8.6.2-6 $fs := \text{StressCheck}(\rho_s, fyh) = 0.258$

Eq. 8.6.2-5 $\alpha Prime := \alpha program(fs, \mu_D) = 3$

If Pu is Compressive

Eq. 8.6.2-4

If Pu is NOT Compressive $VC = 0$

Note: If Pu is not compressive, will have to manually input 0 for vc. Just input it below the vc:=vcprogram and the variable will assume the new value.

$vc := \text{vcprogram}(\alpha Prime, fc, P_u, Ag) = 0.22$ ksi

$Vc := vc \cdot Ag = 503.849$ kips

***Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity**

INPUT $n := 2$ n: number of individual interlocking spiral or hoop core sections

Eq. 8.6.3-1
Eq. 8.6.4-1 $Vs := \text{vsprogram}(n, Asp, fyh, Dprime, s, fc, Ae) = 467.469$ kips

Eq. 8.6.1-2 $\phi Vn := \phi_s \cdot (Vs + Vc) = 874.186$ kips

$\text{Shearcheck} := \text{ShearCheck}(\phi Vn, V_u) = \text{"OK"}$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

***Article 8.6.5*: Minimum Shear Reinforcement**

For Circular Columns

Eq. 8.6.5-1 $\text{CheckTransverse} := \text{mintranprogram}(\rho_s) = \text{"OK"}$

Note: If the minimum shear reinforcement program responses "Increase Shear Reinforcing Ratio", it is recommended to decrease the spacing (s) or increase the area of the shear reinforcement (Asp) in the inputs.

***Article 8.8*: Longitudinal and Lateral Reinforcement Requirements**

***Article 8.8.1*: Maximum Longitudinal Reinforcement**

INPUT $A_{bl} := 1.56$ in^2

INPUT $\text{NumberBars} := 24$

$A_{long} := \text{NumberBars} \cdot A_{bl} = 37.44$ in^2

Eq. 8.8.1-1 $\text{ReinforcementRaitoCheck} := \rho\text{program}(A_{long}, Ag) = \text{"OK"}$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

***Article 8.8.2*: Minimum Longitudinal Reinforcement**

For Columns in SDC B and C:

Eq. 8.8.2-1 $\text{MinimumA}_l := \text{minAlprogram}(A_{long}, Ag) = \text{"OK"}$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars) in the inputs.

***Article 8.8.9*: Requirements for Lateral Reinforcement for SDCs B,C, and D**

These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension **of NOT** less than **6*d_b** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Minimum Size of Lateral Reinforcement

#4 bars for #9 or smaller longitudinal bars
#5 bars for #10 or larger longitudinal bars
#5 bars for bundled longitudinal bars

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

Shall Not Exceed the Smallest of:

$$\text{MaximumSpacing} := \text{Spacingprogram}\left(\text{Columndia}, d_{bl}\right) = 6 \qquad \text{in}$$

$$\text{FINALSPACING} := \text{SpacingCheck}(\text{MaximumSpacing}, s) = 6 \qquad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MaximumSpacing}, s) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.3 (LRFD SPEC.): Column Connections

Note: This needs to be done whenever the column dimension changes. The spacing in the hinge region shall continue into the drilled shaft or cap beam the Extension length.

INPUT $\text{Extension} := \text{ExtensionProgram}(\text{Columndia}) = 27 \qquad \text{in}$

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.
Refer to the **AASHTO LRFD Bridge Design Specifications**.

5.8.3.3 **Nominal Shear Resistance**

$V_p = 170$ kips

INPUT $spaceNOhinge := 10.5$ in

INPUT $bv := Columndia$

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180}\cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 49.67 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 42.81 \quad \text{in}$$

$$dv := 0.9\cdot de = 38.529 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316\cdot\beta\cdot\sqrt{\frac{fc}{1000}}\,bv\cdot dv = 262.986 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp\cdot\frac{fye}{1000}\,dv\cdot\cot(\theta)}{spaceNOhinge} = 136.504 \quad \text{kips}$$

$$\phi V_n := (V_c + V_s)\cdot\phi_s = 359.541 \quad \text{kips}$$

$$\text{Shearcheck} := \text{ShearCheck}(\phi Vn, V_p) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1

$$\text{Avmin} := 0.0316\cdot\sqrt{\frac{fc}{1000}}\cdot\frac{bv\cdot\text{spaceNOhinge}}{\frac{fye}{1000}} = 0.597 \quad in^2$$

$$\text{Av} := 2\cdot\text{Asp} = 0.62 \quad in^2$$

$$\text{MinimumTran} := \text{TranCheck}(\text{Avmin}, \text{Av}) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

***5.8.2.7* Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1

$$vu := \frac{V_u}{\phi_s\cdot bv\cdot dv} = 0.091 \quad \text{ksi}$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2

$$\text{MaxSpacing} := \text{spacingProgram}(vu, dv, fc) = 24 \quad \text{in}$$

$$\text{MAXSPACING} := \text{Spacecheck}(\text{MaxSpacing}, \text{spaceNOhinge}) = 10.5 \quad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MAXSPACING}, \text{spaceNOhinge}) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT DESIGN

Article 6.5: Drilled Shafts

NOTE: The guide specification states that the drilled shafts shall conform to the requirements of columns in SDC B, C, or D as applicable. Also, there are special provisions regarding liquefaction that needs to be investigated if this is a concern for a certain bridge.

Since the hinging will not occur in the drilled shaft, the drilled shaft will be design using the column design from the LRFD Specification.

DRILLED SHAFT 2

Nominal Shear Resistance for members outside Plastic Hinge Region.
Refer to the AASHTO LRFD Bridge Design Specifications.

5.8.3.3 **Nominal Shear Resistance**

INPUT $V_p := 170$ kips

$V_u := V_p$

INPUT $spaceNOhinge := 12$ in

INPUT $Asp := 0.44$ in^2

INPUT $Cover := 6$ in^2

INPUT $bv := DSdia$

INPUT $Dsp := 0.75$ in

INPUT $d_{bl} := 1.41$ in

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$\beta := 2.0$

$\theta := \frac{\pi}{180} \cdot 45 = 0.785$ rad

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 52.545 \quad in$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 46.726 \quad in$$

$$dv := 0.9 \cdot de = 42.053 \quad in$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 318.93 \quad kips$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 185.033 \quad kips$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 453.567 \quad kips$$

$$Shearcheck := ShearCheck\left(\phi Vn, V_p\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.758 \quad in^2$$

$$Av := 2 \cdot Asp = 0.88 \quad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{V_u}{\phi_s \cdot bv \cdot dv} = 0.075 \quad \text{ksi}$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \quad \text{in}$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \quad \text{in}$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT 3

Nominal Shear Resistance for members outside Plastic Hinge Region.
Refer to the AASHTO LRFD Bridge Design Specifications.

5.8.3.3 **Nominal Shear Resistance**

INPUT $V_p := 290$ kips

$V_u := V_p$

INPUT $spaceNOhinge := 12$ in

INPUT $Asp := 0.44$ in^2

INPUT $Cover := 6$ in^2

INPUT $bv := DSdia$

INPUT $Dsp := 0.75$ in

INPUT $d_{bl} := 1.41$ in

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 52.545 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 46.726 \quad \text{in}$$

$$dv := 0.9 \cdot de = 42.053 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 318.93 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 185.033 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 453.567 \quad \text{kips}$$

$$Shearcheck := ShearCheck\left(\phi Vn, V_u\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.758 \quad \text{in}^2$$

$$Av := 2 \cdot Asp = 0.88 \quad \text{in}^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

_5.8.2.7_ Maximum Spacing of Transverse Reinforcement

Eq. 5.8.2.9-1 $$vu := \frac{V_u}{\phi_s \cdot bv \cdot dv} = 0.128 \quad \text{ksi}$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \quad \text{in}$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \quad \text{in}$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT ABUTMENT

Nominal Shear Resistance for members outside Plastic Hinge Region.
Refer to the AASHTO LRFD Bridge Design Specifications.

_5.8.3.3_ Nominal Shear Resistance

INPUT $V_p := 140$ kips

$V_u := V_p$

INPUT $spaceNOhinge := 12$ in

INPUT $Asp := 0.31$ in^2

INPUT $bv := DSabutdia$

INPUT $Cover := 6$ in

INPUT $Dsp := 0.625$ in

INPUT $d_{bl} := 1.41$ in

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 34.67 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 32.036 \quad \text{in}$$

$$dv := 0.9 \cdot de = 28.832 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 153.065 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 89.38 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 218.2 \quad \text{kips}$$

$$Shearcheck := ShearCheck\left(\phi Vn, V_p\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.531 \quad \text{in}^2$$

$$Av := 2 \cdot Asp = 0.62 \quad \text{in}^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 Maximum Spacing of Transverse Reinforcement

Eq. 5.8.2.9-1 $vu := \frac{V_u}{\phi_s \cdot bv \cdot dv} = 0.128$ ksi

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $MaxSpacing := spacingProgram(vu, dv, fc) = 23.066$ in

$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12$ in

$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

CONNECTION DESIGN FOR BENT/ABUTMENT TO GIRDER

Bent 3 Connection Design

Note: Also use for Bent 2 connection.

INPUT $Vcolbent := V_{pBent3} = 580$

INPUT $Ngirderperbent := 12$ Ngirderbent = Number of girders per bent

Article 6.5.4.2: Resistance Factors

$\phi_t := 0.8$ Tension for A307

$\phi_s := 0.75$ Shear for A307

$\phi_{bs} := 0.80$ Block Shear

$\phi_{bb} := 0.80$ Bolts Bearing

$\phi_{sc} := 0.85$ Shear Connectors

$\phi_f := 1.00$ Flexure

$\phi_{sangle} := 1.00$ Shear for the Angle

For Type III Girders

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT	$Fub := 58$	ksi	
INPUT	$Dia_b := 1.5$	in	
INPUT	$Ns := 1$		Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT	$Fy := 36$	ksi	Fy = Yield Stress of the Angle
INPUT	$Fu := 58$	ksi	Fu = Ultimate Stress of the Angle
INPUT	$t := 1.00$	in	t = Thickness of Angle
INPUT	$h := 6$	in	h = Height of the Angle
INPUT	$w := 6$	in	w = Width of the Angle
INPUT	$l := 12$	in	l = Length of the Angle
INPUT	$k := 1.5$	in	k = Height of the Bevel
INPUT	$distanchorhole := 4$	in	distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.
INPUT	$diahole := 1.75$	in	diahole = Diameter of bolt hole
INPUT	$BLSHlength := 6$	in	BLSHlength = Block Shear Length
INPUT	$BLSHwidth := 2$	in	BLSHwidth = Block Shear Width
INPUT	$Ubs := 1.0$		Ubs = Shear Lag Factor for Block Shear
INPUT	$a := 2$	in	a = Distance from the center of the bolt to the edge of plate
INPUT	$b := 3.5$	in	b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent}{2Ngirderperbent} = 24.167 \quad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 1.767 \qquad in^2$$

Eq. 6.13.2.12-1 $\quad \phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 36.898 \quad$ kips **CONTROLS MUST USE 1.5" BOLT**

$Shearcheck := ShearCheck(\phi sRn, Vangle) = \text{"OK"}$

Note: If the program returns "FAILURE", either <u>increase diameter of the bolt</u> (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\quad \phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 208.8 \quad$ kips

For Slotted Holes

<u>INPUT</u> $\quad Lc := 2 \quad in \quad$ Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\quad \phi bbRns := Lc \cdot t \cdot Fub = 116 \quad$ kips

$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$

$Bearingscheck := ShearCheck(\phi bbRns, Vangle) = \text{"OK"}$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 6.042 \qquad \text{kips}$$

Eq. 6.13.2.10.2-1 $\phi tTn := \phi_t \cdot 0.76 \cdot A_b \cdot Fub = 62.317$ kips

$$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$$Pu := Vangle$$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2

$$CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) := \left| \begin{array}{l} t \leftarrow 0.76 \cdot A_b \cdot Fub \\ r \leftarrow 0.76 \cdot A_b \cdot Fub \cdot \sqrt{1 - \left(\frac{Pu}{\phi sRn}\right)^2} \\ a \leftarrow t \ \text{ if } \frac{Pu}{\left(\frac{\phi sRn}{\phi_s}\right)} \le 0.33 \\ a \leftarrow r \ \text{ if } \frac{Pu}{\left(\frac{\phi sRn}{\phi_s}\right)} > 0.33 \\ a \end{array} \right.$$

$$Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 58.863 \qquad \text{kips}$$

$$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 47.09 \qquad \text{kips}$$

$$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vangle\right) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$Agv := t \cdot BLSHlength = 6$ in^2

$Anv := t \cdot (BLSHlength - 0.5 \cdot diahole) = 5.125$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 1.125$ in^2

Note this is for if there are one through bolts in the upper leg.

(J4-5) $BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) := \left| \begin{array}{l} b \leftarrow 0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant \\ c \leftarrow 0.6Fy \cdot Agv + Ubs \cdot Fu \cdot Ant \\ a \leftarrow b \ \ \text{if } b \le c \\ a \leftarrow c \ \ \text{if } b > c \\ a \end{array} \right.$

$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 194.85$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 155.88$ kips

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) =$ "OK"

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 4.25$ in^2

(D3-1) $Ae := Ant \cdot Ut = 2.55$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 118.32$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) =$ "OK"

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

Note: This is assuming we need the same size anchor bolt in the top as we do the bottom.

INPUT $\quad Muangle := 72.2 \quad kip \cdot in$

$$Zx := \frac{1 \cdot (t)^2}{4} = 3 \quad in^3$$

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 108 \quad kip \cdot in$$

Had to increase the thickness of the angle to 1.00 in. or can increase the length.

$$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t \cdot w = 6 \quad in^2$$

(G2-1) $$\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 129.6 \quad kips$$

$$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

TYPE BT-72

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $\quad Fub := 58 \quad ksi$

INPUT $\quad Dia_b := 1.5 \quad in$

INPUT $\quad Ns := 1$ $\quad$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT	$Fy := 36$	ksi	Fy = Yield Stress of the Angle
INPUT	$Fu := 58$	ksi	Fu = Ultimate Stress of the Angle
INPUT	$t := 1.00$	in	t = Thickness of Angle
INPUT	$h := 6$	in	h = Height of the Angle
INPUT	$w := 6$	in	w = Width of the Angle
INPUT	$l := 12$	in	l = Length of the Angle
INPUT	$k := 1.50$	in	k = Height of the Bevel
INPUT	$\text{distanchorhole} := 4$	in	distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.
INPUT	$\text{diahole} := 1.75$	in	diahole = Diameter of bolt hole
INPUT	$\text{BLSHlength} := 6$	in	BLSHlength = Block Shear Length
INPUT	$\text{BLSHwidth} := 2$	in	BLSHwicth = Block Shear Width
INPUT	$\text{Ubs} := 1.0$		Ubs = Shear Lag Factor for Block Shear
INPUT	$a := 2$	in	a = Distance from the center of the bolt to the edge of plate
INPUT	$b := 3.5$	in	b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$\text{Vangle} := \frac{\text{Vcolbent}}{2\text{Ngirderperbent}} = 24.167 \quad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\ 2}}{4} = 1.767 \qquad in^2$$

Eq. 6.13.2.12-1 $\quad \phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 36.898 \qquad kips$

$$Shearcheck := ShearCheck(\phi sRn, Vangle) = \text{"OK"}$$

Note: If the program returns "FAILURE", either <u>increase diameter of the bolt</u> (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\quad \phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 208.8 \qquad kips$

For Slotted Holes

<u>INPUT</u> $\quad Lc := 2 \quad in \qquad$ Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\quad \phi bbRns := Lc \cdot t \cdot Fub = 116 \qquad kips$

$$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$$

$$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or **Lc** (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 6.042 \qquad kips$$

Eq. 6.13.2.10.2-1 $\quad \phi tTn := 0.76 \cdot A_b \cdot Fub = 77.896 \qquad kips$

$$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$$Pu := Vangle$$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2 $\quad Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 58.863 \qquad kips$

$$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 47.09 \qquad kips$$

$$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vangle\right) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$$Agv := t \cdot BLSHlength = 6 \qquad in^2$$

$$Anv := t \cdot (BLSHlength - 0.5 \cdot diahole) = 5.125 \qquad in^2$$

$$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 1.125 \qquad in^2$$

(J4-5) $\quad Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 194.85 \qquad kips$

$$\phi bsRn := \phi_{bs} \cdot Rn = 155.88 \qquad kips$$

$$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t\cdot[w - (1\cdot diahole)] = 4.25$ in^2

(D3-1) $Ae := Ant\cdot Ut = 2.55$ in^2

(D2-2) $\phi tPn := \phi_t\cdot Fub\cdot Ae = 118.32$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

INPUT $Muangle := 72.2$ kip·in

Note: This is assuming we need the same size anchor bolt in the top as we do the bottom.

$Zx := \frac{1\cdot(t)^2}{4} = 3$ in^3

$\phi fMn := \phi_f\cdot Fy\cdot Zx = 108$ kip·in

$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$Cv := 1.0$

$Aw := t \cdot w = 6$ in^2

(G2-1) $\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 129.6$ kips

$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Bent 2 Expansion Connection Design

TYPE BT-72

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.5$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT $Fy := 36$ ksi Fy = Yield Stress of the Angle

INPUT $Fu := 58$ ksi Fu = Ultimate Stress of the Angle

INPUT $t := 1.00$ in t = Thickness of Angle

INPUT $h := 6$ in h = Height of the Angle

INPUT $w := 6$ in w = Width of the Angle

INPUT $l := 12$ in l = Length of the Angle

INPUT $k := 1.50$ in k = Height of the Bevel

INPUT $distanchorhole := 4$ in distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT $diahole := 1.75$ in diahole = Diameter of bolt hole

INPUT $SlottedHole := 6$ in SlottedHole = Length of slotted hole

INPUT $BLSHlength := 6$ in BLSHlength = Block Shear Length

INPUT $BLSHwidth := 2$ in BLSHwicth = Block Shear Width

INPUT $Ubs := 1.0$ Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent}{2Ngirderperbent} = 24.167 \quad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 1.767 \quad \text{in}^2$$

Eq. 6.13.2.12-1 $\phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 36.898$ kips

$$Shearcheck := ShearCheck(\phi sRn, Vangle) = \text{"OK"}$$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 208.8$ kips

For Slotted Holes

INPUT $Lc := 2$ in Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\phi bbRns := Lc \cdot t \cdot Fub = 116$ kips

$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$

$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 6.042 \qquad \text{kips}$$

Eq. 6.13.2.10.2-1 $\phi tTn := 0.76 \cdot A_b \cdot Fub = 77.896$ kips

$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$Pu := Vangle$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2 $\quad Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 58.863$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 47.09$ kips

$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vangle\right) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$Agv := t \cdot BLSHlength = 6$ in^2

$Anv := t \cdot \left(BLSHlength - 0.5 \cdot \frac{SlottedHole}{2}\right) = 4.5$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 1.125$ in^2

(J4-5) $\quad Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 194.85$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 155.88$ kips

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$ $\quad$ Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 4.25$ in^2

(D3-1) $\quad Ae := Ant \cdot Ut = 2.55$ in^2

(D2-2) $\quad \phi tPn := \phi_t \cdot Fub \cdot Ae = 118.32$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

Note: This is assuming we need the same size anchor bolt in the top as we do the bottom.

INPUT $Muangle := 72.2 \quad kip \cdot in$

$$Zx := \frac{1 \cdot (t)^2}{4} = 3 \qquad in^3$$

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 108 \qquad kip \cdot in$$

$$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t \cdot w = 6 \qquad in^2$$

(G2-1) $$\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 129.6 \qquad kips$$

$$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Abutment to Girder Connection

INPUT $Vcolbent := 420$ kips

INPUT $Ngirderperbent := 6$ Ngirderbent = Number of girders per bent

For Type III Girders

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.5$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT $Fy := 36$ ksi Fy = Yield Stress of the Angle

INPUT $Fu := 58$ ksi Fu = Ultimate Stress of the Angle

INPUT $t := 1.0$ in t = Thickness of Angle

INPUT $h := 6$ in h = Height of the Angle

INPUT $w := 6$ in w = Width of the Angle

INPUT $l := 20$ in l = Length of the Angle

INPUT $k := 1.5$ in k = Height of the Bevel

INPUT $distanchorhole := 4$ in distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT $diahole := 1.75$ in diahole = Diameter of bolt hole

INPUT $SlottedHole := 6$ in SlottedHole = Length of slotted hole

INPUT $BLSHlength := 15$ in BLSHlength = Block Shear Length

INPUT $BLSHwidth := 2$ in BLSHwicth = Block Shear Width

INPUT $Ubs := 1.0$ Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent}{2Ngirderperbent} = 35 \qquad kips$$

INPUT $\quad n_{bolts} := 2 \qquad n_{bolts}$ = number of bolts per flange

$$Vperbolt := \frac{Vangle}{n_{bolts}} = 17.5 \qquad kips$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 1.767 \qquad in^2$$

Eq. 6.13.2.12-1 $\quad \phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 36.898 \qquad kips$

Note: This is checking to verify that the anchor bolt has enough shear strength.

$$Shearcheck := ShearCheck(\phi sRn, Vperbolt) = \text{"OK"}$$

Note: If the program returns "FAILURE", either <u>increase diameter of the bolt</u> (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\quad \phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 208.8 \qquad kips$

For Slotted Holes

Eq. 6.13.2.9.3 $\quad \phi bbRns := 2.0 \cdot Dia_b \cdot t \cdot Fub = 174 \qquad kips$

$$Bearingcheck := ShearCheck(\phi bbRn, Vperbolt) = \text{"OK"}$$

$$Bearingscheck := ShearCheck(\phi bbRns, Vperbolt) = \text{"OK"}$$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 8.75 \qquad \text{kips}$$

Eq. 6.13.2.10.2-1 $\quad \phi tTn := 0.76 \cdot A_b \cdot Fub = 77.896 \qquad \text{kips}$

$$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$$Pu := Vperbolt$$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2 $\quad Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 68.577 \qquad \text{kips}$

$$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 54.862 \qquad \text{kips}$$

$$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vangle\right) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$Agv := t \cdot BLSHlength = 15 \quad in^2$

$Anv := t \cdot \left(BLSHlength - 1.5 \cdot \frac{SlottedHole}{2}\right) = 10.5 \; in^2$

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 1.125 \quad in^2$

Note this is for if there are two through bolts in the upper leg.

(J4-5) $Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 389.25 \quad kips$

$\phi bsRn := \phi_{bs} \cdot Rn = 311.4 \quad kips$

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 4.25 \quad in^2$

(D3-1) $Ae := Ant \cdot Ut = 2.55 \quad in^2$

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 118.32 \quad kips$

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

INPUT

$Muangle := 104 \quad kip \cdot in$

Note: This is assuming we need the same size anchor bolt in the top as we do the bottom.

$$Zx := \frac{1 \cdot (t)^2}{4} = 5 \qquad in^3$$

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 180 \qquad kip \cdot in$$

Had to increase the thickness of the angle to 1.00 in. or can increase the length.

$$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t \cdot w = 6 \qquad in^2$$

(G2-1) $$\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 129.6 \qquad kips$$

$$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Appendix E: Little Bear Creek Bridge LRFD Specification Design

Designer: Paul Coulston
Project Name: Little Bear Creek Bridge
Job Number:
Date: 8/2/2010

$ORIGIN := 1$

Description of worksheet: This worksheet is a seismic bridge design worksheet for the **AASHTO LRFD Bridge Design Specification**. All preliminary design should already be done for non-seismic loads.

Project Known Information

Location: Franklin County
Zip Code or Coordinates: 35.0069 N 88.2025 W

Superstructure Type: AASTHO I girders
Substructure Type: Circular columns supported on drilled shafts
Abutment Type: Abutment beam supported on drilled shafts

Note: *Input all of the below information.*

$fc := 4000 \quad psi$

$fye := 60000 \quad psi$

$\rho_{conc} := 0.08681 \quad \frac{lb}{in^3}$

$g := 386.4 \quad \frac{in}{s^2}$

Length of Bridge (ft)	$L := 300$	ft
Short Span (ft)	$ShortSpan := 85$	ft
Long Span (ft)	$LongSpan := 130$	ft
Deck Thickness (in)	$t_{deck} := 7$	in
Deck Width (ft)	$DeckWidth := 42.75$	ft
I-Girder X-Sectional Area (in^2)	$IGirderArea := 559.5$	in^2
Bulb Girder X-Sectional Area (in^2)	$BulbGirderArea := 767$	in^2
Guard Rail Area (in^2)	$GuardRailArea := 310$	in^2
Bent Volume (ft^3)	$BentVolume := 40\cdot(7\cdot 5 + 2.4\cdot 2.5) = 1.64\times 10^3$	ft^3
Column Diameter (in)	$Columndia := 54$	in
Drilled Shaft Diameter (in)	$DSdia := 60$	in

Drilled Shaft Abutment Diameter (in) $DSabutdia := 42$ in

Average Column Height (ft) $ColumnHeight := 14.5$ ft

Column Area (in^2) $$Acolumn := \frac{Columndia^2 \cdot \pi}{4} = 2.29 \times 10^3 \quad in^2$$

Drilled Shaft Area (in^2) $$Adrilledshaft := \frac{DSdia^2 \pi}{4} = 2.827 \times 10^3 \quad in^2$$

Drilled Shaft Abutment Area (in^2) $$Adsabut := \frac{DSabutdia^2 \cdot \pi}{4} = 1.385 \times 10^3 \quad in^2$$

Note: These are variables that were easier to input in ft and then convert to inches.

$ShortSpan := ShortSpan \cdot 12 = 1.02 \times 10^3$ in

$LongSpan := LongSpan \cdot 12 = 1.56 \times 10^3$ in

$L := L \cdot 12 = 3.6 \times 10^3$ in

$DeckWidth := DeckWidth \cdot 12 = 513$ in

$BentVolume := BentVolume \cdot 12^3 = 2.834 \times 10^6$ in^3

$ColumnHeight := ColumnHeight \cdot 12 = 174$ in

Steps for Seismic Design

Use Appendix A3 Seismic Design Flowcharts

Description of Difference from Guide Specification
The LRFD Specification is a force based approach to design verses the Guide Specification which is a displacement based approach. The LRFD Specification has an Response Modification Factor (R) that will be used in calculating the loads applied to the structure.

Article 3.10.1: Earthquake Effects - This is just the applicability of the Specifications.

***Article 3.10.2*: Determine Design Response Spectrum**

Note: AASHTO Guide Specifications for LRFD Seismic Bridge Design is accompanied with a program that has the Seismic Hazard maps. This program will calculate several of the variables that are needed for the analysis.

1) *Article 3.10.3.1*:Determine the Site Class.

INPUT Site Class: **D**

2) Enter maps and find PGA, S_s, and S_1. Then enter those values in their respective spot.

$PGA := 0.116\ g$

INPUT $S_s := 0.272\ g$

$S_1 := 0.092\ g$

3) *Article 3.10.3.2:* Site Coefficients. From the PGA, S_s, and S_1 values and site class choose F_{PGA}, F_a, and F_v. Note: straight line interpolation is permitted.

$F_{PGA} := 1.57$

INPUT $F_a := 1.58$

$F_v := 2.4$

$A_s := F_{PGA} \cdot PGA = 0.182\ g$ — A_s : Acceleration Coefficient

$SDS := F_a \cdot S_s = 0.43\ g$ — S_{DS} = Short Period Acceleration Coefficient

$SD1 := F_v \cdot S_1 = 0.221\ g$ — S_{D1} = 1-sec Period Acceleration Coefficient

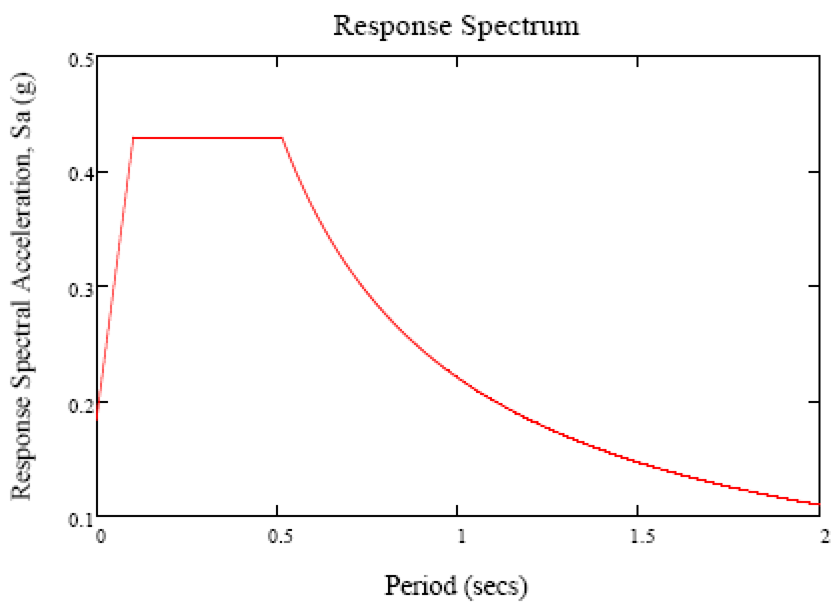
4) Creating a Response Spectrum

Note: The DesignSpectrum Code creates a Response Spectrum for the Bridge. At this time the period of the bridge is unknown; therefore, the Sa value cannot be calculated.

$Tmax := 2\ s$ $Dt := 0.001\ s$

$$
\begin{aligned}
&\mathrm{DesignSpectrum}(SDS, SD1, A_s, Tmax, Dt) := \\
&\quad T_s \leftarrow \frac{SD1}{SDS} \\
&\quad T_o \leftarrow 0.2 \cdot T_s \\
&\quad n_{max} \leftarrow \frac{Tmax}{Dt} \\
&\quad \text{for } i \in 1 .. n_{max} \\
&\quad\quad T_i \leftarrow Dt \cdot i \\
&\quad\quad a_i \leftarrow (SDS - A_s) \cdot \frac{Dt \cdot i}{T_o} + A_s \ \text{ if } Dt \cdot i < T_o \\
&\quad\quad a_i \leftarrow SDS \ \text{ if } Dt \cdot i \ge T_o \wedge Dt \cdot i \le T_s \\
&\quad\quad a_i \leftarrow \frac{SD1}{Dt \cdot i} \ \text{ if } Dt \cdot i > T_s \\
&\quad R \leftarrow \mathrm{augment}(T, a) \\
&\quad R
\end{aligned}
$$

$$BridgeSpectrum := DesignSpectrum(SDS, SD1, A_s, Tmax, Dt)$$



***Article 3.10.6*: Selection of Seismic Performance Zones**

$SD1 = 0.221$ g From Table 3.10.6-1 Choose SPZ

$$SDCprogram(SD1) := \left| \begin{array}{l} \text{for } c \in SD1 \\ \quad \left| \begin{array}{l} c \leftarrow \text{"1"} \text{ if } SD1 \le 0.15 \\ c \leftarrow \text{"2"} \text{ if } SD1 > 0.15 \wedge SD1 \le 0.3 \\ c \leftarrow \text{"3"} \text{ if } SD1 > 0.3 \wedge SD1 \le 0.5 \\ c \leftarrow \text{"4"} \text{ if } SD1 > 0.5 \end{array} \right. \\ Rs \leftarrow c \\ c \end{array} \right.$$

$$SDC := SDCprogram(SD1) = \text{"2"}$$

Article 3.10.5: Bridge Importance Category

Operational Classified: **Other bridges**

Article 3.10.7: Response Modification Factors

For Substructures: Table 3.10.7.1-1

INPUT	Multiple Column Bents	$R_{sub} := 5.0$

For Connections: Table 3.10.7.1-2

INPUT	Superstructure to Abutment	$R_{abutment} := 0.8$
INPUT	Columns to Bent Cap	$R_{columncap} := 1.0$
INPUT	Column to foundation	$R_{foundation} := 1.0$

Article 4.7.4.3: Multispan Bridges

Article 4.7.4.3.1 Selection of Method

Refer to Table 4.7.4.3.1-1 to select the required analysis procedure. This a function of seismic performance zone, regularity, and operational classification. For the worst case in Alabama, we can use either the Uniform Load Elastic Method or Single-Mode Elastic Method.

Note: There are two methods that can be used according to this procedure. The Uniform Load Method is suitable for regular bridges that respond principally in their fundamental mode of vibration. The Single Mode Spectral Method may be a better method if there is a major change in the spans, stiffness of the piers, etc.
The Uniform Load Method is simpler and less time consuming and will give accurate results, and this is the reason it has been chosen in this design.

Article 4.7.4.3.2c **Uniform Load Method**

Step 1: Build a bridge model
Step 2: Apply a uniform load of Po = 1.0 kip/in. in both the longitudinal and transverse direction. Also, the uniform load can be converted into point loads and applied as joint loads in SAP. Calculate the static displacement for both directions. In SAP, tables of the displacements can be exported to EXCEL, and the MAX Function can be used to find the maximum displacement.

Step 3: Calculate the bridge lateral stiffness, K, and total weight, W.

$$p_o := 1.0 \ \frac{kip}{in}$$

INPUT

$$\upsilon_{smaxLong} := 0.647204 \ in$$

$$\upsilon_{smaxTran} := 5.26053 \ in$$

$$K_{Long} := \frac{p_o \cdot L}{\upsilon_{smaxLong}} = 5.562 \times 10^3 \qquad \frac{kip}{in}$$

$$K_{Tran} := \frac{p_o \cdot L}{\upsilon_{smaxTran}} = 684.342 \qquad \frac{kip}{in}$$

<u>INPUT</u>: Multiplying factors

$$W := \frac{\rho_{conc} \cdot \left(\begin{array}{l} L \cdot t_{deck} \cdot DeckWidth + 2 \cdot BentVolume + 4 \cdot Acolumn \cdot ColumnHeight \ldots \\ + 2ShortSpan \cdot 6 \cdot IGirderArea + 6 \cdot LongSpan \cdot BulbGirderArea + 2 \cdot GuardRailArea \cdot L \end{array} \right)}{1000}$$

$$W = 3164.123 \qquad kips$$

Step 4: Calculate the period, T_m.

$$T_{mLong} := 2\pi \cdot \sqrt{\frac{W}{K_{Long} \cdot g}} = 0.241 \qquad s$$

Step 5: Calculate equivalent static earthquake loading p_e.

$$acc(SDS, SD1, T_{mLong}, A_s) := \left| \begin{array}{l} T_s \leftarrow \frac{SD1}{SDS} \\ T_o \leftarrow 0.2 \cdot T_s \\ \text{for } a \in T_{mLong} \\ \quad \left| \begin{array}{l} a \leftarrow (SDS - A_s) \cdot \frac{T_{mLong}}{T_o} + A_s \ \text{ if } T_{mLong} < T_o \\ a \leftarrow SDS \ \text{ if } T_{mLong} \ge T_o \wedge T_{mLong} \le T_s \\ a \leftarrow \frac{SD1}{T_{mLong}} \ \text{ if } T_{mLong} > T_s \end{array} \right. \\ Ra \leftarrow a \\ a \end{array} \right.$$

$$Csm_{Long} := acc(SDS, SD1, T_{mLong}, A_s) = 0.43$$

$$p_{eLong} := \frac{Csm_{Long} \cdot W}{L} = 0.378 \qquad \frac{kip}{in}$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$v_{smaxLong} := \frac{p_{eLong}}{p_o} \cdot \upsilon_{smaxLong} = 0.244 \qquad \text{in}$$

Repeat Steps 4, 5, and 6 for transverse loading.

Step 4: Calculate the period, T_m.

$$T_{mTran} := 2\pi \cdot \sqrt{\frac{W}{K_{Tran} \cdot g}} = 0.687 \qquad \text{s}$$

Step 5: Calculate equivalent static earthquake loading p_e.

$$Csm_{Tran} := acc\left(SDS, SD1, T_{mTran}, A_s\right) = 0.321$$

$$p_{eTran} := \frac{Csm_{Tran} \cdot W}{L} = 0.282 \qquad \frac{\text{kip}}{\text{in}}$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$v_{smaxTran} := \frac{p_{eTran}}{p_o} \cdot \upsilon_{smaxTran} = 1.485 \qquad \text{in}$$

<u>Single-Mode Spectral Method</u>

Article 4.7.4.3.2: Single-Mode Spectral Method

Step 1: Build a bridge model
Step 2: Apply a uniform load of Po = 1.0 in both the longitudinal and transverse direction. Calculate the static displacement for both directions.
Step 3: Calculate factors α, β , and γ.

Note: The Deflection equations come from analysis of the SAP model. The displacement is taken at the joints along the length of the bridge and input into an Excel Worksheet. Then a graph is created of the displacements along the length of the bridge. A best fit line is plotted, and that is the equation that is shown below.

$$v_{stran}(x) := 1\cdot 10^{-7}\cdot x^2 + 0.0017\cdot x + 0.3412 \qquad v_{slong}(x) := -2\cdot 10^{-9}\cdot x^2 + 0.0001\cdot x + 0.2223$$

C4.7.4.3.2b-1 $\alpha_{Tran} := \int_0^L v_{stran}(x)\,dx$ $\qquad \alpha_{Long} := \int_0^L v_{slong}(x)\,dx$

C4.7.4.3.2b-2 $\beta_{Tran} := \int_0^L \frac{W}{L} v_{stran}(x)\,dx$ $\qquad \beta_{Long} := \int_0^L \frac{W}{L}\cdot v_{slong}(x)\,dx$

C4.7.4.3.2b-3 $\gamma_{Tran} := \int_0^L \frac{W}{L}\cdot v_{stran}(x)^2\,dx = 6.102\times 10^4$ $\qquad \gamma_{Long} := \int_0^L \frac{W}{L}\cdot v_{slong}(x)^2\,dx$

α = Displacement along the length

β = Weight per unit length * Displacement

γ = Weight per unit length * Displacement^2

Step 4: Calculate the Period of the Bridge

$$T_{mTran1} := 2\pi\cdot\sqrt{\frac{\gamma_{Tran}}{p_o\cdot g\cdot\alpha_{Tran}}} = 0.672 \qquad s$$

$$T_{mLong1} := 2\pi\cdot\sqrt{\frac{\gamma_{Long}}{p_o\cdot g\cdot\alpha_{Long}}} = 0.194 \qquad s$$

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smLong} := acc\left(SDS, SD1, T_{mLong1}, A_s\right) = 0.43$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$PeLong(x) := \frac{\beta_{Long}\cdot C_{smLong}}{\gamma_{Long}}\cdot\frac{W}{L}\cdot v_{slong}(x)$$

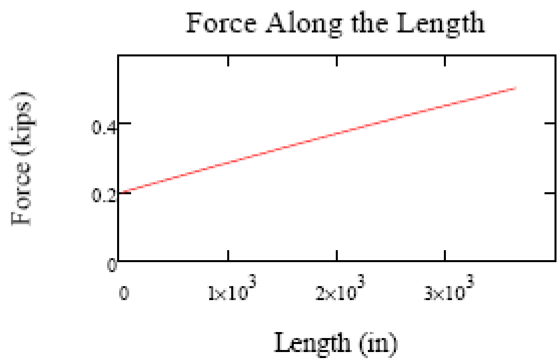
$$PeLong(x) \rightarrow 0.000090517538799082222132\cdot x + -1.8103507759816444426e\text{-}9\cdot x^2 + 0.20122048875035977$$

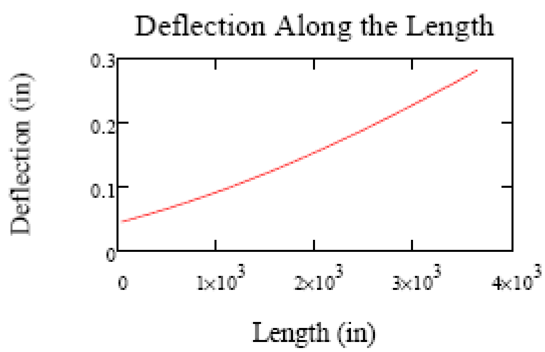
$dW := \frac{L}{100}$

$i := 1..101$

$Pelong_i := PeLong[(i-1)\cdot dW]$ $\qquad$ $\delta long_i := v_{slong}[(i-1)dW]$

$\Delta long_i := Pelong_i \cdot \delta long_i$





Maximum Deflection

$\max(\Delta long) = 0.28$ in

NOTE: Repeat Steps 5 and 6 for Transverse Direction.

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smTran} := acc\left(SDS, SD1, T_{mTran1}, A_s\right) = 0.328$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$PeTran(x) := \frac{\beta_{Tran} \cdot C_{smTran}}{\gamma_{Tran}} \cdot \frac{W}{L} \cdot v_{stran}(x)$$

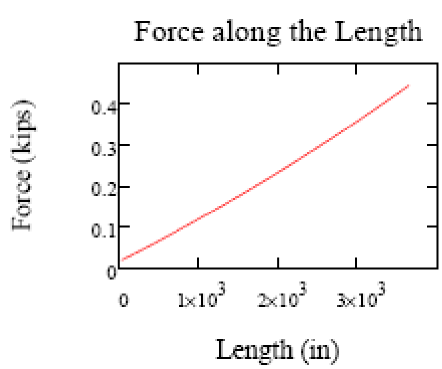
$$PeTran(x) \rightarrow 0.000097554309454804921037 \cdot x + 5.7384887914591130022e\text{-}9 \cdot x^2 + 0.019579723756458493563$$

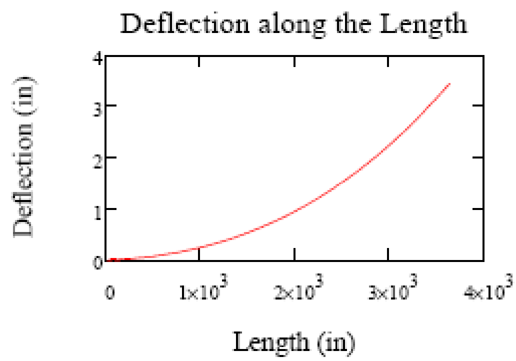
$$dL := \frac{L}{100}$$

$$i := 1 .. 101$$

$$Petran_i := PeTran[(i-1) \cdot dL] \qquad \delta tran_i := v_{stran}[(i-1)dL]$$

$$\Delta tran_i := Petran_i \cdot \delta tran_i$$





Maximum Deflection

$$\max(\Delta tran) = 3.453 \quad in$$

Article 3.10.8: Combination of Seismic Force Effects

$$LoadCase1 := \sqrt{\left(1.0 \cdot p_{eTran}\right)^2 + \left(0.3 \cdot p_{eLong}\right)^2} = 0.304 \quad \frac{kip}{in}$$

$$LoadCase2 := \sqrt{\left(1.0 \cdot p_{eLong}\right)^2 + \left(0.3 \cdot p_{eTran}\right)^2} = 0.387 \quad \frac{kip}{in}$$

Article 3.10.9.3: Determine Design Forces

$$MaxLoadCase(x,y) := \left| \begin{array}{l} a \leftarrow x \ \text{ if } x \ge y \\ a \leftarrow y \ \text{ if } y \ge x \\ a \end{array} \right.$$

$$NominalForce := MaxLoadCase(LoadCase1, LoadCase2) = 0.387 \quad \frac{kip}{in}$$

Note: The Req values are factors that will be used to multiple loads that come out of SAP 2000. The NominalForce variable is truly just a factor also. It is easier to apply these factors to the loading for po than to change the loading in SAP 2000 because if an error is made in calculating pe then the loads will have to re-entered.

Multiple Column Bents $$Req_{substructure} := \frac{NominalForce}{R_{sub}} = 0.077$$

Note: Article 3.10.9.3 specifies that the Drilled shafts be designed for half of the R value. R/2 also must not be taken less than 1.

$$Req_{DrilledShafts} := \frac{NominalForce}{R_{sub} \cdot 0.5} = 0.155$$

Connections

Superstructure to Abutment $\quad Req_{subtoabutcon} := \frac{NominalForce}{R_{abutment}} = 0.484$

Columns to Bent Cap $\quad Req_{coltocapcon} := \frac{NominalForce}{R_{columncap}} = 0.387$

Column to foundation $\quad Req_{coltofoundcon} := \frac{NominalForce}{R_{foundation}} = 0.387$

LOADS FOR DESIGN

NOTE: All the loads calculated in this section are for a single column or drilled shaft.

COLUMN SHEAR PROGRAM

$$\text{Shear}(Vu, Reqsubstructure) := \left| \begin{array}{l} a \leftarrow Vu \cdot Reqsubstructure \\ a \end{array} \right.$$

AXIAL LOAD PROGRAM

$$\text{PDEAD}(Peq, Pd, Rsub, Reqsubstructure) := \left| \begin{array}{l} a \leftarrow \left(\left| Peq \cdot Reqsubstructure - \frac{Pd}{Rsub} \right| \right) \\ a \end{array} \right.$$

Note: The axial load program calculates the minimum axial load on the column. This will needed later in the design process.

BENT 2

Note: Input the maximum shear value between all the columns in the bent from SAP which have not been multiplied by pe/po or divided by the R factor. This will be taken care of in one of the equations below.

INPUT $Vu_{Bent2} := 594$ kips

Note: The Axial load consists of both the Earthquake effect and dead load effect. It should be the minimum axial load the column will encounter during an earthquake.

INPUT $Pueq_{Bent2} := 972.4$ kips

INPUT $Pudead_{Bent2} := 637$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{Bent2} := PDEAD\left(Pueq_{Bent2}, Pudead_{Bent2}, R_{sub}, Req_{substructure}\right) = 52.115$ kips

INPUT $ColumnHeight_{Bent2} := 12.063$ ft

Note: Input the correct variables into the below program to get the shear in the column or drilled shaft.

INPUT

$Vucol_{Bent2} := Shear\left(Vu_{Bent2}, Req_{substructure}\right) = 45.988$ kips

BENT 3

Note: Input the maximum shear value between all the columns in the bent from SAP which have not been multiplied by pe/po or divided by the R factor. This will be taken care in of one of the equations below.

INPUT $Vu_{Bent3} := 1029$ kips

Note: The Axial load consists of both the Earthquake effect and dead load effect. It should be the minimum axial load the column will encounter during an earthquake.

INPUT $Pueq_{Bent3} := 2224$ kips

INPUT $Pudead_{Bent3} := 658$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{Bent3} := PDEAD(Pueq_{Bent3}, Pudead_{Bent3}, R_{sub}, Req_{substructure}) = 40.585$ kips

INPUT $ColumnHeight_{Bent3} := 16.681$ ft

Note: Input the correct variables into the below program to get the shear in the column or drilled shaft.

INPUT

$Vucol_{Bent3} := Shear(Vu_{Bent3}, Req_{substructure}) = 79.667$ kips

DRILLED SHAFT 2

INPUT $Pueq_{DS2} := 1100$ kips

INPUT $Pudead_{DS2} := 637$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{DS2} := PDEAD(Pueq_{DS2}, Pudead_{DS2}, R_{sub}, Req_{DrilledShafts}) = 42.927$ kips

INPUT $ColumnHeight_{DS2} := 4.6$ ft

Note: The drilled shaft is design for an R/2; therefore, the shear in the column can be doubled to equal the design shear for the drilled shaft.

INPUT $Vu_{DS2} := Vucol_{Bent2} \cdot 2 = 91.977$ kips

DRILLED SHAFT 3

INPUT $Pueq_{DS3} := 2224$ kips

INPUT $Pudead_{DS3} := 657$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{DS3} := PDEAD\left(Pueq_{DS3}, Pudead_{DS3}, R_{sub}, Req_{DrilledShafts}\right) = 212.971$ kips

INPUT $ColumnHeight_{DS3} := 3.2$ ft

Note: The drilled shaft is design for an R/2; therefore, the shear in the column can be doubled to equal the design shear for the drilled shaft.

INPUT $Vu_{DS3} := Vucol_{Bent3} \cdot 2 = 159.333$ kips

ABUTMENT DRILLED SHAFTS

Note: Input the loads from SAP which have not been multiplied by pe/po or divided by the R factor. This will be taken care of one of the equations below.

INPUT $Vu_{Abut} := 356$ kips

INPUT $ColumnHeight_{Abut} := 7.639$ ft

Note: Input the correct variables into the below program to get the shear in the column or drilled shaft.

INPUT

$Vu_{DSAbut} := Shear\left(Vu_{Abut}, Req_{subtoabutcon}\right) = 172.263$ kips

Article 4.7.4.4: Minimum Support Length Requirements

Note: May need to add more calculations if column heights are different at the bents.

N (in) = Minimum support length measured normal to the bridge

L (ft) = Length of bridge to adjacent expansion joint or end of the bridge

H (ft) = average height of columns supporting bridge deck for abutments
for columns and piers = column height

S (Degree) = angle of skew

Abutment Support Length Requirement

$Span_{abutment} := \frac{ShortSpan}{12} = 85$ ft $\qquad H_{abutment} := \frac{ColumnHeight}{12} = 14.5$ ft

Note: The $Span_{abutment}$ is divided by number of spans and inches.

$Skew_{abutment} := 0$ Degrees

$Nabutment := 1.5\cdot(8 + 0.02Span_{abutment} + 0.08H_{abutment})\cdot\left(1 + 0.000125Skew_{abutment}^{2}\right) = 16.29$ in

Bent Support Length Requirement

BENT 2

INPUT $Span_{Bent} := \frac{LongSpan}{12} = 130$ ft

Note: The $Span_{abutment}$ is divided by number of spans and inches.

INPUT $H_{Bent} := \frac{202}{12} = 16.83$ ft $\qquad$ *INPUT*: Column Height for this Bent

INPUT $Skew_{Bent} := 0$ Degrees

$N_{Bent} := 1.5\cdot(8 + 0.02Span_{Bent} + 0.08H_{Bent})\cdot\left(1 + 0.000125Skew_{Bent}^{2}\right) = 17.9$ in

BENT 3

INPUT $Span_{Bent} := \frac{LongSpan}{12} = 130$ ft

Note: The $Span_{abutment}$ is divided by number of spans and inches.

INPUT $H_{Bent} := \frac{238.56}{12} = 19.88$ ft $\qquad$ *INPUT*: Column Height for this Bent

INPUT $Skew_{Bent} := 0$ Degrees

$N_{Bent} := 1.5\cdot(8 + 0.02Span_{Bent} + 0.08H_{Bent})\cdot\left(1 + 0.000125Skew_{Bent}^{2}\right) = 18.286$ in

BENT 2 DESIGN

Article 5.10.11.3: Provisions for Seismic Design for Seismic Zone 2

Article 5.10.11.3: Longitudinal Reinforcement

INPUT $A_{longbar} := 1.56$ in^2

INPUT $N_{bars} := 24$

INPUT $Ads := Acolumn$

$$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 37.44 \quad in^2$$

Minimum Longitudinal Reinforcing Check

$$\text{Checkleastlongreinforcing}(Ag, Along) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } Along \geq Ag \cdot 0.01 \\ a \leftarrow \text{"Increase Longitudinal Reinforcing Ratio"} \quad \text{if } Along < 0.01 \cdot Ag \\ a \end{array} \right.$$

$$\text{MinLongRatio} := \text{Checkleastlongreinforcing}\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Maximum Longitudinal Reinforcing Check

$$\text{Checkmaxlongreinforcing}(Ag, Along) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } Along \leq 0.06 \cdot Ag \\ a \leftarrow \text{"Decrease Longitudinal Reinforcing Ratio"} \quad \text{if } Along > 0.06 \cdot Ag \\ a \end{array} \right.$$

$$\text{MaxLongRatio} := \text{Checkmaxlongreinforcing}\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

Article 5.10.11.4.1b: Flexural Resistance

Check biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.9.4.
Flexural Resistance check can be done using some kind of Column Design Program. PCA Column was used for this project. After creating an interaction diagram, verify that all the critical load combinations fall within the diagram.

Article 5.10.11.4.1c: Column Shear and Transverse Reinforcement

Note: This is for the end regions of the column.

Article 5.8.3.3: Nominal Shear Resistance

$$Vu := Vucol_{Bent2} = 45.988 \quad \text{kips}$$

$$Pumin_{Bent2} = 52.115 \quad \text{kips}$$

INPUT	$bv := Columndia$		bv: effective width
INPUT	$\phi_s := 0.9$		
INPUT	$s := 3$	in	s: Spacing of hoops or pitch of spiral (in)
INPUT	$Asp := .31$	in^2	Asp: Area of spiral or hoop reinforcing (in^2)
INPUT	$Dsp := 0.625$	in	Dsp: Diameter of spiral or hoop reinforcing (in)
INPUT	$Cover := 3$	in	Cover: Concrete cover for the Column (in)
INPUT	$Dprime := 48$	in	Dprime: Diameter of spiral or hoop for circular columns (in)
INPUT	$d_{bl} := 1.41$	in	d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 49.67 \quad \text{in}$$

(Equation: C5.8.2.9-2) $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 42.81 \quad \text{in}$$

$$dv := 0.9 \cdot de = 38.529 \quad \text{in}$$

Article 5.10.11.4.1c:

$$\text{VcProgram}(fc, \beta, bv, dv, Ag, Pu) := \left| \begin{array}{l} p \leftarrow Pu \\ v \leftarrow 0.0316 \cdot \beta \cdot \sqrt{\dfrac{fc}{1000}} bv \cdot dv \\ c \leftarrow 0.1 \cdot Ag \cdot \dfrac{fc}{1000} \\ x \leftarrow \dfrac{p \cdot v}{c} \\ a \leftarrow v \ \text{ if } \ p > c \\ a \leftarrow x \ \text{ if } \ p \le c \\ a \end{array} \right.$$

Note: The area of the Column and the Axial Load for this column need to be input into the Vc equation that calls the program above.

$$V_c := \text{VcProgram}\left(fc, \beta, bv, dv, \text{Acolumn}, \text{Pumin}_{\text{Bent2}}\right) = 14.961 \qquad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \dfrac{fye}{1000} dv \cdot \cot(\theta)}{s} = 477.765 \quad \text{kips}$$

Eq. 5.8.3.3-1 $$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 443.453 \qquad \text{kips}$$

$$\text{ShearCheck}\left(\phi Vn, V_u\right) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } \ \phi Vn \ge V_u \\ a \leftarrow \text{"FAILURE"} \quad \text{if } \ \phi Vn < V_u \\ a \end{array} \right.$$

$$\text{Shearcheck} := \text{ShearCheck}\left(\phi V_n, Vu\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

Article 5.10.11.4.1c Length of End Region (Plastic Hinge Region)

End region is assumed to extend from the soffit of girders or cap beams at the top of the columns or from the top of foundations at the bottom of columns.

$$\text{EndRegionProgram}(d,H) := \left| \begin{array}{l} x \leftarrow d \\ y \leftarrow \frac{1}{6}H\cdot 12 \\ z \leftarrow 18 \\ a \leftarrow \max(x,y,z) \\ a \end{array} \right.$$

$$\text{LendgthEndRegion} := \text{EndRegionProgram}\left(\text{Columndia}, \text{ColumnHeight}_{\text{Bent2}}\right) = 54 \qquad \text{in}$$

Article 5.10.11.4.3: Column Connections

Extension into Top and Bottom Connections

Note: This needs to be done whenever the column dimension changes.

$$\text{ExtensionProgram}(d) := \left| \begin{array}{l} z \leftarrow 15 \\ x \leftarrow \frac{1}{2}\cdot d \\ a \leftarrow \max(z,x) \\ a \end{array} \right.$$

$$\text{Extension} := \text{ExtensionProgram}(\text{Columndia}) = 27 \qquad \text{in}$$

Article 5.10.11.4.1e: Spacing of Transverse Reinforcement for Confinement

Transverse Reinforcement for Confinement

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

Shall Not Exceed the Smallest of:

$$\text{Spacingprogram}(\text{Columndia}) := \left| \begin{array}{l} x \leftarrow \frac{1}{4}\cdot \text{Columndia} \\ y \leftarrow 4 \\ a \leftarrow \min(x,y) \\ a \end{array} \right.$$

$$\text{MaximumSpacing} := \text{Spacingprogram}(\text{Columndia}) = 4 \qquad \text{in}$$

$$\text{SpacingCheck(MaximumSpacing, s)} := \left| \begin{array}{l} a \leftarrow s \quad \text{if } s \le \text{MaximumSpacing} \\ a \leftarrow \text{MaximumSpacing} \quad \text{if } s > \text{MaximumSpacing} \\ a \end{array} \right.$$

$$\text{FINALSPACING} := \text{SpacingCheck(MaximumSpacing, s)} = 3 \qquad \text{in}$$

$$\text{scheck} := \text{ShearCheck(MaximumSpacing, s)} = \text{"OK"}$$

Note: If scheck returns "Failure", increase the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.1d: Transverse Reinforcement for Confinement at Plastic Hinges

Required Volumetric Ratio of Spiral or Seismic Hoop Reinforcing

$$\rho_s := \frac{4 \cdot \text{Asp}}{s \cdot \text{Dprime}} = 8.611 \times 10^{-3}$$

$$\text{RatioProgram}\left(fc, fy, \rho_s\right) := \left| \begin{array}{l} z \leftarrow 0.12 \cdot \dfrac{fc}{fy} \\ a \leftarrow \text{"OK"} \quad \text{if } \rho_s \ge z \\ a \leftarrow \text{"Increase Transverse Reinforcing Ratio"} \quad \text{if } \rho_s < z \\ a \end{array} \right.$$

$$\text{Check}\rho_s := \text{RatioProgram}\left(fc, fye, \rho_s\right) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Increase Transverse Reinforcing Ratio", it is recommended to decrease the spacing (s) or increase the area of the transverse reinforcement (Asp) in the inputs.

Note:These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than $6 * d_b$ or **3 in.** at one end and a hook of NOT less than 90 Degrees with an extension **of NOT** less than $6 * d_b$ at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a $6 * d_b$ but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a $6 * d_b$ but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Article 5.10.11.4.1f: Splices

Lap Splices in longitudinal reinforcement shall not be used in plastic hinge region.

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.

5.8.3.3 **Nominal Shear Resistance**

INPUT $Vu_{sub} := Vucol_{Bent2} = 45.988$ kips

INPUT $spaceNOhinge := 10.5$ in

Note: β and θ come from Article 5.8.3.4.1

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 49.67 \quad in$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 42.81 \quad in$$

$$dv := 0.9 \cdot de = 38.529 \quad in$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 262.986 \quad kips$$

Eq. 5.8.3.3-4 $$V_s := \frac{Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 68.252 \quad kips$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 298.115 \quad kips$$

$$ShearCheck\left(\phi Vn, V_u\right) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } \phi Vn \ge V_u \\ a \leftarrow \text{"FAILURE"} \quad \text{if } \phi Vn < V_u \\ a \end{array} \right.$$

$$Shearcheck := ShearCheck\left(\phi V_n, Vu_{sub}\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.597 \quad in^2$$

$$Av := 2 \cdot Asp = 0.62 \quad in^2$$

$$\text{TranCheck}(\text{Avmin}, \text{Av}) := \left| \begin{array}{l} a \leftarrow \text{"Decrease Spacing or Increase Bar Size"} \quad \text{if } \text{Avmin} > \text{Av} \\ a \leftarrow \text{"OK"} \quad \text{if } \text{Avmin} \le \text{Av} \\ a \end{array} \right.$$

$$\text{MinimumTran} := \text{TranCheck}(\text{Avmin}, \text{Av}) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

_5.8.2.7_ Maximum Spacing of Transverse Reinforcement

Eq. 5.8.2.9-1 $\quad \text{vu} := \dfrac{\text{Vu}_{\text{sub}}}{\phi_s \cdot \text{bv} \cdot \text{dv}} = 0.025 \qquad \text{ksi}$

$$\text{spacingProgram}(\text{Vu}, \text{dv}, \text{fc}) := \left| \begin{array}{l} v \leftarrow 0.125 \cdot \dfrac{\text{fc}}{1000} \\ q \leftarrow 0.8 \cdot \text{dv} \\ r \leftarrow 0.4 \cdot \text{dv} \\ z \leftarrow q \quad \text{if } q \le 24 \\ z \leftarrow 24 \quad \text{if } q > 24 \\ t \leftarrow r \quad \text{if } r \le 12 \\ t \leftarrow 12 \quad \text{if } r > 12 \\ a \leftarrow z \quad \text{if } \text{Vu} < v \\ a \leftarrow t \quad \text{if } \text{Vu} \ge v \\ a \end{array} \right.$$

$$\text{MaxSpacing} := \text{spacingProgram}(\text{vu}, \text{dv}, \text{fc}) = 24 \qquad \text{in}$$

$$\text{Spacecheck}(\text{MaxSpacing}, s) := \left| \begin{array}{l} a \leftarrow s \quad \text{if } s \le \text{MaxSpacing} \\ a \leftarrow \text{MaxSpacing} \quad \text{if } s > \text{MaxSpacing} \\ a \end{array} \right.$$

$$\text{MAXSPACING} := \text{Spacecheck}(\text{MaxSpacing}, \text{spaceNOhinge}) = 10.5 \qquad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MAXSPACING}, \text{spaceNOhinge}) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

BENT 3 DESIGN

Article 5.10.11.3: Provisions for Seismic Design for Seismic Zone 2

Article 5.10.11.3: Longitudinal Reinforcement

INPUT $A_{longbar} := 1.56 \quad in^2$

INPUT $N_{bars} := 28$

INPUT $Ads := Acolumn$

$$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 43.68 \quad in^2$$

Minimum Longitudinal Reinforcing Check

$$MinLongRatio := Checkleastlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Maximum Longitudinal Reinforcing Check

$$MaxLongRatio := Checkmaxlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

Article 5.10.11.4.1b: Flexural Resistance

Check biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.9.4.
Flexural Resistance check can be done using some kind of Column Design Program. PCA Column was used for this project. After creating an interaction diagram, verify that all the critical load combinations fall within the diagram.

Article 5.10.11.4.1c: Column Shear and Transverse Reinforcement

Note: This is for the end regions of the column.

Article 5.8.3.3: Nominal Shear Resistance

$Vucol_{Bent3} = 79.667$ kips

$Pumin_{Bent3} = 40.585$ kips

INPUT $bv := Columndia$ bv: effective width

INPUT $\phi_s := 0.9$

INPUT $s := 3$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $Asp := .31$ in^2 Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.625$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 3$ in Cover: Concrete cover for the Column (in)

INPUT $Dprime := 48$ in Dprime: Diameter of spiral or hoop for circular columns (in)

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 49.67 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 42.81 \quad \text{in}$$

$$dv := 0.9 \cdot de = 38.529 \quad \text{in}$$

Article 5.10.11.4.1c: Column Shear and Transverse Reinforcement

INPUT

$$V_c := \mathrm{VcProgram}\left(fc, \beta, bv, dv, Acolumn, Pumin_{Bent3}\right) = 11.651 \quad \text{kips}$$

$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{s} = 477.765 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 440.474 \quad \text{kips}$$

$$\mathrm{Shearcheck} := \mathrm{ShearCheck}\left(\phi V_n, Vu\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

Length of End Region (Plastic Hinge Region)

End region is assumed to extend from the soffit of girders or cap beams at the top of the columns or from the top of foundations at the bottom of columns.

$$\mathrm{LendgthEndRegion} := \mathrm{EndRegionProgram}\left(Columndia, ColumnHeight_{Bent2}\right) = 54 \quad \text{in}$$

Article 5.10.11.4.3: Column Connections

Extension into Top and Bottom Connections

Note: This needs to be done whenever the column dimension changes.

$$\mathrm{Extension} := \mathrm{ExtensionProgram}(Columndia) = 27 \quad \text{in}$$

Article 5.10.11.4.1e: Spacing of Transverse Reinforcement for Confinement

Transverse Reinforcement for Confinement

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

$$\text{MaximumSpacing} := \text{Spacingprogram(Columndia)} = 4 \quad \text{in}$$

$$\text{FINALSPACING} := \text{SpacingCheck(MaximumSpacing, s)} = 3 \quad \text{in}$$

$$\text{scheck} := \text{ShearCheck(MaximumSpacing, s)} = \text{"OK"}$$

Note: If scheck returns "Failure", increase the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.1d: Transverse Reinforcement for Confinement at Plastic Hinges

Required Volumetric Ratio of Spiral or Seismic Hoop Reinforcing

$$\rho_s := \frac{4 \cdot Asp}{s \cdot Dprime} = 8.611 \times 10^{-3}$$

$$\text{Check}\rho_s := \text{RatioProgram}\left(fc, fye, \rho_s\right) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Increase Transverse Reinforcing Ratio", it is recommended to decrease the spacing (s) or increase the area of the transverse reinforcement (Asp) in the inputs.

Note:These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension **of NOT** less than **6*d_b** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Article 5.10.11.4.1f: Splices

Lap Splices in longitudinal reinforcement shall not be used in plastic hinge region.

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.

5.8.3.3 **Nominal Shear Resistance**

INPUT $Vu_{sub} := Vucol_{Bent3} = 79.667$ kips

INPUT $spaceNOhinge := 10.5$ in

Note: β and θ come from Article 5.8.3.4.1

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 49.67 \quad in$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 42.81 \quad in$$

$$dv := 0.9 \cdot de = 38.529 \quad in$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 262.986 \quad kips$$

Eq. 5.8.3.3-4 $$V_s := \frac{Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 68.252 \quad kips$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 298.115 \quad kips$$

$$Shearcheck := ShearCheck\left(\phi V_n, Vu_{sub}\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.597 \quad in^2$$

$$Av := 2 \cdot Asp = 0.62 \quad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $vu := \dfrac{Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 0.043$ ksi

$MaxSpacing := spacingProgram(vu, dv, fc) = 24$ in

$$Spacecheck(MaxSpacing, s) := \left| \begin{array}{l} a \leftarrow s \quad \text{if } s \le MaxSpacing \\ a \leftarrow MaxSpacing \quad \text{if } s > MaxSpacing \\ a \end{array} \right.$$

$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 10.5$ in

$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT DESIGN

DRILLED SHAFT 2

Article 5.13.4.6.2b: Cast-in-place Piles

INPUT $A_{longbar} := 1.56$ in^2

INPUT $N_{bars} := 24$

INPUT $Ads := Adrilledshaft$

$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 37.44$ in^2

Note: Article 5.13.4.6.2b only has a provision for the minimum amount of longitudinal reinforcing. This minimum reinforcing check only applies to the **FIRST 1/3*PILE LENGTH OR 8 FT**.

CHECK: There also needs be a check made if the column Diameter is less than 24.0 in. For these drilled shafts, spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at pitch not exceeding 9.0 in., except that the pitch shall not exceed 4.0 in. within a length below the pile cap reinforcement of not less than 2.0 ft or 1.5 pile diameters, whichever is greater.

Minimum Longitudinal Reinforcing Check (First 1/3*Pile Length or 8 ft)

$$\text{Checkleastlongreinforcing}(Ag, Along) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } Along \geq Ag \cdot 0.005 \\ a \leftarrow \text{"Increase Longitudinal Reinforcing Ratio"} \quad \text{if } Along < 0.005 \cdot Ag \\ a \end{array} \right.$$

$$\text{MinLongRatio} := \text{Checkleastlongreinforcing}\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Article 5.10.11.3: Longitudinal Reinforcement

Minimum Longitudinal Reinforcing Check

$$\text{MinLongRatio} := \text{Checkleastlongreinforcing}\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Maximum Longitudinal Reinforcing Check

$$\text{MaxLongRatio} := \text{Checkmaxlongreinforcing}\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

5.8.3.3 **Nominal Shear Resistance**

INPUT	$Vu_{sub} := Vu_{DS2} = 91.977$	kips	
INPUT	$spaceNOhinge := 12$	in	s: Spacing of hoops or pitch of spiral (in)
INPUT	$bv := DSdia$		bv: effective width
INPUT	$Asp := .44$	in^2	Asp: Area of spiral or hoop reinforcing (in^2)
INPUT	$Dsp := 0.75$	in	Dsp: Diameter of spiral or hoop reinforcing (in)
INPUT	$Cover := 6$	in	Cover: Concrete cover for the Column (in)

INPUT $Dprime := 48$ in Dprime: Diameter of spiral or hoop for circular columns (in)

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 52.545 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 46.726 \quad \text{in}$$

$$dv := 0.9 \cdot de = 42.053 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 318.93 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 185.033 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 453.567 \quad \text{kips}$$

$$Shearcheck := ShearCheck\left(\phi V_n, Vu\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.758 \qquad in^2$$

$$Av := 2 \cdot Asp = 0.88 \qquad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 0.041 \qquad ksi$$

$$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \qquad in$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \qquad in$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of transverse reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT 3

Article 5.13.4.6.2b: Cast-in-place Piles

<u>INPUT</u> $A_{longbar} := 1.56 \qquad in^2$

<u>INPUT</u> $N_{bars} := 32$

<u>INPUT</u> $Ads := Adrilledshaft$

$$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 49.92 \qquad in^2$$

Note: Article 5.13.4.6.2b only has a provision for the minimum amount of longitudinal reinforcing. This minimum reinforcing check only applies to the **FIRST 1/3*PILE LENGTH OR 8 FT**.

CHECK: There also needs be a check made if the column Diameter is less than 24.0 in. For these drilled shafts, spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at pitch not exceeding 9.0 in., except that the pitch shall not exceed 4.0 in. within a length below the pile cap reinforcement of not less than 2.0 ft or 1.5 pile diameters, whichever is greater.

Minimum Longitudinal Reinforcing Check (First 1/3*Pile Length or 8 ft)

$$\mathrm{MinLongRatio} := \mathrm{Checkleastlongreinforcing}\left(\mathrm{Ads}, A_{\mathrm{longreinforcing}}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Article 5.10.11.3: Longitudinal Reinforcement

Minimum Longitudinal Reinforcing Check

$$\mathrm{MinLongRatio} := \mathrm{Checkleastlongreinforcing}\left(\mathrm{Ads}, A_{\mathrm{longreinforcing}}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Maximum Longitudinal Reinforcing Check

$$\mathrm{MaxLongRatio} := \mathrm{Checkmaxlongreinforcing}\left(\mathrm{Ads}, A_{\mathrm{longreinforcing}}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

***5.8.3.3* Nominal Shear Resistance**

INPUT $Vu_{sub} := Vu_{DS3} = 159.333$ kips

INPUT $\mathrm{spaceNOhinge} := 12$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $bv := \mathrm{DSdia}$ bv: effective width

INPUT	$Asp := .44$	in^2	Asp: Area of spiral or hoop reinforcing (in^2)
INPUT	$Dsp := 0.75$	in	Dsp: Diameter of spiral or hoop reinforcing (in)
INPUT	$Cover := 6$	in	Cover: Concrete cover for the Column (in)
INPUT	$Dprime := 48$	in	Dprime: Diameter of spiral or hoop for circular columns (in)
INPUT	$d_{bl} := 1.41$	in	d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 52.545 \quad \text{in}$$

Eq. C5.8.2.9-2

$$de := \frac{bv}{2} + \frac{Dr}{\pi} = 46.726 \quad \text{in}$$

$$dv := 0.9 \cdot de = 42.053 \quad \text{in}$$

Eq. 5.8.3.3-3

$$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 318.93 \quad \text{kips}$$

Eq. 5.8.3.3-4

$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 185.033 \quad \text{kips}$$

$$\phi V_n := (V_c + V_s) \cdot \phi_s = 453.567 \quad \text{kips}$$

$$\phi V_n := (V_c + V_s) \cdot \phi_s = 453.567 \quad \text{kips}$$

$$Shearcheck := ShearCheck(\phi V_n, Vu) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.758$$

$$Av := 2 \cdot Asp = 0.88 \qquad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 0.07 \qquad ksi$$

$$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \qquad in$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \qquad in$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of transverse reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT ABUTMENT

Article 5.13.4.6.2b: Cast-in-place Piles

<u>INPUT</u> $A_{longbar} := 1.56 \qquad in^2$

<u>INPUT</u> $N_{bars} := 12$

<u>INPUT</u> $Ads := Adsabut$

$$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 18.72 \qquad in^2$$

Note: Article 5.13.4.6.2b only has a provision for the minimum amount of longitudinal reinforcing. This minimum reinforcing check only applies to the **FIRST 1/3*PILE LENGTH OR 8 FT**.

CHECK: There also needs be a check made if the column Diameter is less than 24.0 in. For these drilled shafts, spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at pitch not exceeding 9.0 in., except that the pitch shall not exceed 4.0 in. within a length below the pile cap reinforcement of not less than 2.0 ft or 1.5 pile diameters, whichever is greater.

Minimum Longitudinal Reinforcing Check (First 1/3*Pile Length or 8 ft)

$$MinLongRatio := Checkleastlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Article 5.10.11.3: Longitudinal Reinforcement

Minimum Longitudinal Reinforcing Check

$$MinLongRatio := Checkleastlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Maximum Longitudinal Reinforcing Check

$$MaxLongRatio := Checkmaxlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

5.8.3.3 **Nominal Shear Resistance**

INPUT $Vu_{sub} := Vu_{DSAbut} = 172.263$ kips

INPUT $spaceNOhinge := 12$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $bv := DSabutdia$ bv: effective width

INPUT $Asp := .31$ in^2 Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.625$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 6$ in Cover: Concrete cover for the Column (in)

INPUT $Dprime := 30$ in Dprime: Diameter of spiral or hoop for circular columns (in)

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 34.67 \quad \text{in}$$

Eq. C5.8.2.9-2
$$de := \frac{bv}{2} + \frac{Dr}{\pi} = 32.036 \quad \text{in}$$

$$dv := 0.9 \cdot de = 28.832 \quad \text{in}$$

Eq. 5.8.3.3-3
$$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 153.065 \quad \text{kips}$$

Eq. 5.8.3.3-4
$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 89.38 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 218.2 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 218.2 \quad \text{kips}$$

$$Shearcheck := ShearCheck\left(\phi V_n, Vu\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.531$$

$$Av := 2 \cdot Asp = 0.62 \qquad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 0.158 \qquad ksi$$

$$MaxSpacing := spacingProgram(vu, dv, fc) = 23.066 \qquad in$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \qquad in$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of transverse reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

Connection Design for Girder to Bent Cap

INPUT $\mathrm{Vcolbent} := \mathrm{Vucol}_{\mathrm{Bent3}}$

INPUT $\mathrm{Ngirderperbent} := 12$

Article 6.5.4.2: Resistance Factors

$\phi_t := 0.8$	Tension for A307
$\phi_s := 0.75$	Shear for A307
$\phi_{bs} := 0.80$	Block Shear
$\phi_{bb} := 0.80$	Bolts Bearing
$\phi_{sc} := 0.85$	Shear Connectors
$\phi_f := 1.00$	Flexure
$\phi_{sangle} := 1.00$	Shear for the Angle

For Type III Girders

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $\mathrm{Fub} := 58$ ksi

INPUT $\mathrm{Dia}_b := 1.25$ in

INPUT $\mathrm{Ns} := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT	$\mathrm{Fy} := 36$	ksi	Fy = Yield Stress of the Angle
INPUT	$\mathrm{Fu} := 58$	ksi	Fu = Ultimate Stress of the Angle
INPUT	$t := 0.5$	in	t = Thickness of Angle
INPUT	$h := 6$	in	h = Height of the Angle
INPUT	$w := 6$	in	w = Width of the Angle
INPUT	$l := 12$	in	l = Length of the Angle

INPUT $k := 1.00$ in — k = Height of the Bevel

INPUT $\text{distanchorhole} := 4$ in — distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT $\text{diahole} := 1.5$ in — diahole = Diameter of bolt hole

INPUT $\text{BLSHlength} := 6$ in — BLSHlength = Block Shear Length

INPUT $\text{BLSHwidth} := 2$ in — BLSHwicth = Block Shear Width

INPUT $\text{Ubs} := 1.0$ — Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in — a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in — b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$\text{Vangle} := \frac{\text{Vcolbent}\cdot 2}{2\text{Ngirderperbent}} = 6.639 \qquad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi\cdot \text{Dia}_b{}^2}{4} = 1.227 \qquad \text{in}^2$$

Eq. 6.13.2.12-1 $\quad \phi\text{sRn} := \phi_s\cdot 0.48\cdot A_b\cdot \text{Fub}\cdot \text{Ns} = 25.624 \qquad \text{kips}$

Note: This is checking to verify that the anchor bolt has enough shear strength.

$$\text{Shearcheck} := \text{ShearCheck}(\phi\text{sRn}, \text{Vangle}) = \text{"OK"}$$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 87$ kips

For Slotted Holes

INPUT $Lc := 2$ in Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\phi bbRns := Lc \cdot t \cdot Fub = 58$ kips

$$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$$

$$Bearingscheck := ShearCheck(\phi bbRns, Vangle) = \text{"OK"}$$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 1.66 \quad \text{kips}$$

Eq. 6.13.2.10.2-1 $\phi tTn := \phi_t \cdot 0.76 \cdot A_b \cdot Fub = 43.275$ kips

$$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$Pu := Vangle$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2

$$CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) := \left| \begin{array}{l} t \leftarrow 0.76 \cdot A_b \cdot Fub \\ r \leftarrow 0.76 \cdot A_b \cdot Fub \cdot \sqrt{1 - \left(\frac{Pu}{\phi sRn}\right)^2} \\ a \leftarrow t \ \text{ if } \frac{Pu}{\left(\frac{\phi sRn}{\phi_s}\right)} \le 0.33 \\ a \leftarrow r \ \text{ if } \frac{Pu}{\left(\frac{\phi sRn}{\phi_s}\right)} > 0.33 \\ a \end{array} \right.$$

$Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 54.094$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 43.275$ kips

$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vangle\right) =$ "OK"

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

Note this is for if there are one through bolts in the upper leg.

$Agv := t \cdot BLSHlength = 3$ in^2

$Anv := t \cdot (BLSHlength - 0.5 \cdot diahole) = 2.625$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 0.625$ in^2

(J4-5)

$$BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) := \left| \begin{array}{l} b \leftarrow 0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant \\ c \leftarrow 0.6Fy \cdot Agv + Ubs \cdot Fu \cdot Ant \\ a \leftarrow b \ \text{ if } b \le c \\ a \leftarrow c \ \text{ if } b > c \\ a \end{array} \right.$$

$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 101.05$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 80.84$ kips

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = "OK"$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 2.25$ in^2

(D3-1) $Ae := Ant \cdot Ut = 1.35$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 62.64$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) = "OK"$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

INPUT $Muangle := 21.25$ kip·in

$$Zx := \frac{1 \cdot (t)^2}{4} = 0.75 \quad in^3$$

$\phi fMn := \phi_f \cdot Fy \cdot Zx = 27$ kip·in

$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = "OK"$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$Cv := 1.0$

$Aw := t \cdot w = 3$ in^2

(G2-1) $\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 64.8$ kips

ShearAngleCheck := ShearCheck(φsangleVn, Vangle) = "OK"

Note: If program returns "FAILURE", change thickness of angle or width of angle.

TYPE BT-72

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.25$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT $Fy := 36$ ksi Fy = Yield Stress of the Angle

INPUT $Fu := 58$ ksi Fu = Ultimate Stress of the Angle

INPUT $t := 0.5$ in t = Thickness of Angle

INPUT $h := 6$ in h = Height of the Angle

INPUT $w := 6$ in w = Width of the Angle

INPUT $l := 12$ in l = Length of the Angle

INPUT $k := 1.00$ in k = Height of the Bevel

INPUT distanchorhole := 4 in distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT diahole := 1.5 in diahole = Diameter of bolt hole

INPUT BLSHlength := 6 in BLSHlength = Block Shear Length

INPUT BLSHwidth := 2 in BLSHwicth = Block Shear Width

INPUT $Ubs := 1.0$ Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent \cdot 2}{2Ngirderperbent} = 6.639 \quad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\ 2}}{4} = 1.227 \quad \text{in}^2$$

Eq. 6.13.2.12-1 $\phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 25.624$ kips

Note: This is checking to verify that the anchor bolt has enough shear strength.

$$Shearcheck := ShearCheck(\phi sRn, Vangle) = \text{"OK"}$$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 87$ kips

For Slotted Holes

INPUT $Lc := 2$ in Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\phi bbRns := Lc \cdot t \cdot Fub = 58$ kips

$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$

$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 1.66 \quad \text{kips}$$

Eq. 6.13.2.10.2-1 $\phi tTn := 0.76 \cdot A_b \cdot Fub = 54.094$ kips

$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$Pu := Vangle$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2 $Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 54.094$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 43.275$ kips

$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vangle\right) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$Agv := t \cdot BLSHlength = 3$ in^2

$Anv := t \cdot (BLSHlength - 0.5 \cdot diahole) = 2.625$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 0.625$ in^2

(J4-5) $Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 101.05$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 80.84$ kips

BlockShearCheck := ShearCheck(ϕbsRn, Vangle) = "OK"

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 2.25$ in^2

(D3-1) $Ae := Ant \cdot Ut = 1.35$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 62.64$ kips

TensionCheck := ShearCheck(ϕtPn, Vangle) = "OK"

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

INPUT $\quad Muangle := 21.25 \quad kip \cdot in$

$$Zx := \frac{1 \cdot (t)^2}{4} = 0.75 \qquad in^3$$

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 27 \qquad kip \cdot in$$

$$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t \cdot w = 3 \qquad in^2$$

(G2-1) $$\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 64.8 \qquad kips$$

$$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

BENT 2 EXPANSION CONNECTION

TYPE BT-72

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT	$Fub := 58$	ksi	
INPUT	$Dia_b := 1.25$	in	
INPUT	$Ns := 1$		Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT	$Fy := 36$	ksi	Fy = Yield Stress of the Angle
INPUT	$Fu := 58$	ksi	Fu = Ultimate Stress of the Angle
INPUT	$t := 0.5$	in	t = Thickness of Angle
INPUT	$h := 6$	in	h = Height of the Angle
INPUT	$w := 6$	in	w = Width of the Angle
INPUT	$l := 12$	in	l = Length of the Angle
INPUT	$k := 1.00$	in	k = Height of the Bevel
INPUT	$distanchorhole := 4$	in	distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.
INPUT	$diahole := 1.5$	in	diahole = Diameter of bolt hole
INPUT	$SlottedHole := 6$	in	SlottedHole = Length of Slotted Hole
INPUT	$BLSHlength := 6$	in	BLSHlength = Block Shear Length
INPUT	$BLSHwidth := 2$	in	BLSHwicth = Block Shear Width
INPUT	$Ubs := 1.0$		Ubs = Shear Lag Factor for Block Shear
INPUT	$a := 2$	in	a = Distance from the center of the bolt to the edge of plate
INPUT	$b := 3.5$	in	b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent \cdot 2}{2Ngirderperbent} = 6.639 \qquad kips$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 1.227 \qquad in^2$$

Eq. 6.13.2.12-1 $\quad \phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 25.624 \qquad kips$

Note: This is checking to verify that the anchor bolt has enough shear strength.

$$Shearcheck := ShearCheck(\phi sRn, Vangle) = \text{"OK"}$$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\quad \phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 87 \qquad kips$

For Slotted Holes

INPUT $\quad Lc := 2 \quad in \qquad$ Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\quad \phi bbRns := Lc \cdot t \cdot Fub = 58 \qquad kips$

$$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$$

$$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 1.66 \qquad \text{kips}$$

Eq. 6.13.2.10.2-1 $\quad \phi tTn := 0.76 \cdot A_b \cdot Fub = 54.094 \qquad \text{kips}$

$$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$$Pu := Vangle$$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2 $\quad Tn_{combined} := CombinedProgram(Pu, A_b, Fub, \phi sRn, \phi_s) = 54.094 \qquad \text{kips}$

$$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 43.275 \qquad \text{kips}$$

$$Combinedcheck := ShearCheck(\phi tTn_{combined}, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$$Agv := t \cdot BLSHlength = 3 \qquad in^2$$

$$Anv := t \cdot \left(BLSHlength - 0.5 \cdot \frac{SlottedHole}{2}\right) = 2.25 \qquad in^2$$

$$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 0.625 \qquad in^2$$

(J4-5) $\quad Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 101.05 \qquad \text{kips}$

$$\phi bsRn := \phi_{bs} \cdot Rn = 80.84 \qquad \text{kips}$$

$$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 2.25 \quad in^2$

(D3-1) $Ae := Ant \cdot Ut = 1.35 \quad in^2$

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 62.64 \quad kips$

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

INPUT $Muangle := 21.25 \quad kip \cdot in$

$$Zx := \frac{1 \cdot (t)^2}{4} = 0.75 \quad in^3$$

$\phi fMn := \phi_f \cdot Fy \cdot Zx = 27 \quad kip \cdot in$

$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$Cv := 1.0$

$Aw := t \cdot w = 3 \quad in^2$

(G2-1) $\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 64.8 \quad kips$

$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Connection Design for Girder to Abutment

For Type III Girders

INPUT $Vcolbent := Vu_{DSAbut}$

INPUT $Ngirderperbent := 6$

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.5$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT $Fy := 36$ ksi Fy = Yield Stress of the Angle

INPUT $Fu := 58$ ksi Fu = Ultimate Stress of the Angle

INPUT $t := 1$ in t = Thickness of Angle

INPUT $h := 6$ in h = Height of the Angle

INPUT $w := 6$ in w = Width of the Angle

INPUT $l := 20$ in l = Length of the Angle

INPUT $k := 1.5$ in k = Height of the Bevel

INPUT $distanchorhole := 4$ in distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT $diahole := 1.75$ in diahole = Diameter of bolt hole

INPUT $SlottedHole := 6$ in SlottedHole = Length of Slotted Hole

INPUT $BLSHlength := 15$ in BLSHlength = Block Shear Length

INPUT $BLSHwidth := 2$ in BLSHwicth = Block Shear Width

INPUT $Ubs := 1.0$ Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent \cdot 3}{2Ngirderperbent} = 43.066 \quad kips$$

INPUT $n_{bolts} := 2$ n_{bolts} = number of bolts per flange

$$Vperbolt := \frac{Vangle}{n_{bolts}} = 21.533 \quad kips$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\ 2}}{4} = 1.767 \quad in^2$$

Eq. 6.13.2.12-1 $\phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 36.898$ kips

Note: This is checking to verify that the anchor bolt has enough shear strength.

$$Shearcheck := ShearCheck(\phi sRn, Vperbolt) = \text{"OK"}$$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 208.8$ kips

For Slotted Holes

INPUT $Lc := 2$ in Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\phi bbRns := Lc \cdot t \cdot Fub = 116$ kips

$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = "OK"$

$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = "OK"$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 10.766 \quad \text{kips}$$

Eq. 6.13.2.10.2-1 $\phi tTn := 0.76 \cdot A_b \cdot Fub = 77.896$ kips

$Tensioncheck := ShearCheck(\phi tTn, Tu) = "OK"$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$Pu := Vperbolt$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2 $Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 63.256$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 50.605$ kips

$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vangle\right) = "OK"$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$Agv := t \cdot BLSHlength = 15$ in^2

$Anv := t \cdot (BLSHlength - 1.5 \cdot diahole) = 12.375$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 1.125$ in^2

Note: this is for two bolts.

(J4-5) $Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 389.25$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 311.4$ kips

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 4.25$ in^2

(D3-1) $Ae := Ant \cdot Ut = 2.55$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 118.32$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

INPUT $\text{Muangle} := 137.4$ kip·in

$$\text{Zx} := \frac{1\cdot(t)^2}{4} = 5 \qquad \text{in}^3$$

$$\phi\text{fMn} := \phi_f \cdot \text{Fy}\cdot\text{Zx} = 180 \qquad \text{kip}\cdot\text{in}$$

$$\text{BendingAngleCheck} := \text{ShearCheck}(\phi\text{fMn}, \text{Muangle}) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$\text{Cv} := 1.0$$

$$\text{Aw} := t\cdot w = 6 \qquad \text{in}^2$$

(G2-1) $$\phi\text{sangleVn} := \phi_{sangle}\cdot 0.6\cdot\text{Fy}\cdot\text{Aw}\cdot\text{Cv} = 129.6 \qquad \text{kips}$$

$$\text{ShearAngleCheck} := \text{ShearCheck}(\phi\text{sangleVn}, \text{Vangle}) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Designer: PJC
Project Name: LITTLE BEAR CREEK
Job Number:
Date: 11/2/2010

Moment Capacity and Interaction Diagram Data
Column or Drilled Shaft: BENT 2 COLUMN

INPUTS

Column Diameter	54	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	# 11	
Number of Lonitudinal Reinforcing Bars	24	
Transverse Reinforcing Bar Size	# 5	
Spacing of Transverse Reinforcing Bars	12"	in
Column Height	12.063	~~in~~ ft
Cover	3	in

Dead Load Reactions

PuDead ~~(kips)~~ 637 kips
MuDead ~~(kips)~~ 256 kip*ft

Enter into Column Design Software (PCA COLUMN)
Finding Moment Capacity of the column.

Mp = ϕMp/0.9 → 4556 kip*ft

Vpcol = 2*Mp/Heightcolumn → 755.3 kips

Vpbent = 2*Vpcol 1510 kips

Create SAP model of Bent
Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Puoverturning = 867 kips

Pu = PuDead +/- Puoverturning 1504 or 230 kips

Re-enter into Column Design Software (PCA COLUMN)

Mp = ϕMp/0.9 → 4944 kip*ft

Vpcol = 2*Mp/Heightcolumn → 820 kips

Vpbent = 2*Vpcol 1640 kips

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

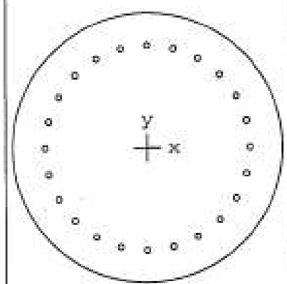
Check = {(Vpbent2 - Vpbent1) / Vpbent2} * 100% 7.93 OK

Check Other Seismic Load Cases

Mupotrans	6444	kip*ft	PCTRAN :	0.288
Putran	1100	kips		
Mupolong	820	kip*ft	PCLONG :	0.378
Pulong	0.301	kips		

Fu = PuDead +/- Putran → (310.2) 947.2 or 326.8 kips
Mu = MuDead + Mupotran → 2073 kip*ft

Pu = PuDead +/- Pulong → (3.7) 637 or 627 kips
Mu = MuDead + Mupolong → 570 kip*ft



60 in diam.

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

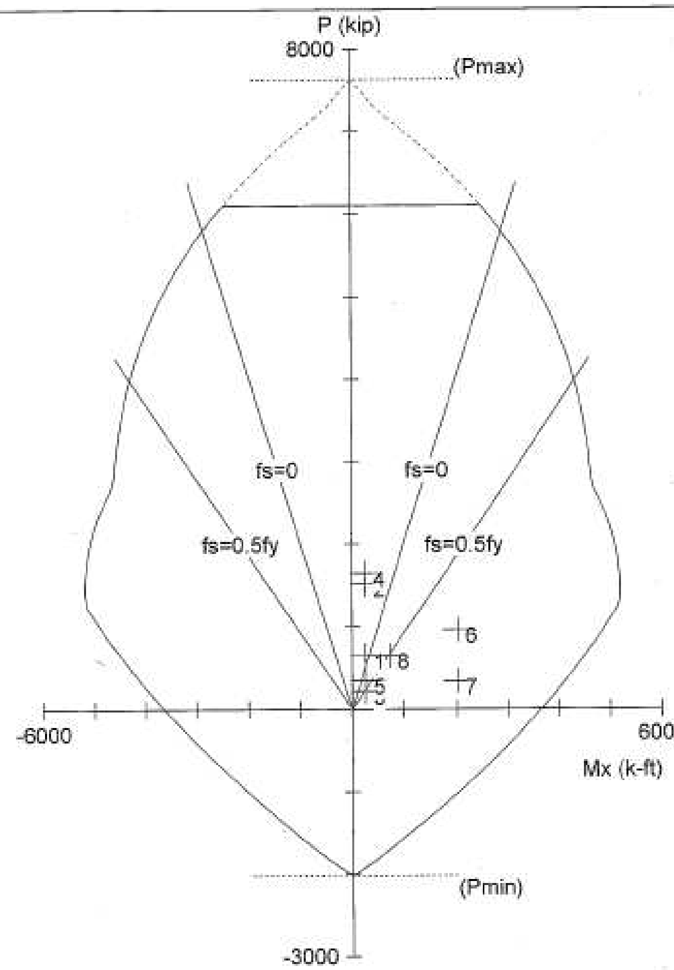
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/02/10

Time: 10:09:52



pcaColumn v3.64. Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59

File: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\Little Bear Cre...\DrilledShaftBent2.col

Project:

Column: Engineer:

f'c = 4 ksi	fy = 60 ksi	Ag = 2827.43 in^2	24 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 37.44 in^2	Rho = 1.32%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 636173 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 636173 in^4
Beta1 = 0.85		Clear spacing = 4.54 in	Clear cover = 6.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

10:09 AM

```
OOOOOOO    OOOOO    OOOOO    OOOOO    OOOOO   OO
OO    OO  OO   OO  OO   OO  OO   OO  OO   OO  OO
OO    OO  OO   OO  OO   OO  OO   OO  OO   OO  OO
OO    OO  OO       OO   OO  OO       OO   OO  OO
OO    OO  OO       OOOOOOO  OO       OO   OO  OO
OOOOOOO   OO   OO  OO   OO  OO   OO  OO   OO  OO
OO        OO   OO  OO   OO  OO   OO  OO   OO  OO
OO         OOOOO   OO   OO   OOOOO    OOOOO   OOOOO  (TM)
```

===
Computer program for the Strength Design of Reinforced Concrete Sections
===

10:09 AM

General Information:
====================
File Name: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\Little Bear Creek\PCA COLUMN\DrilledShaftBent2.col
Project:
Column:
Code: ACI 318-02
Engineer:
Units: English

Run Option: Investigation
Run Axis: X-axis
Slenderness: Not considered
Column Type: Structural

Material Properties:
====================
f'c = 4 ksi
Ec = 3605 ksi
Ultimate strain = 0.003 in/in
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

Section:
========
Circular: Diameter = 60 in

Gross section area, Ag = 2827.43 in^2
Ix = 636173 in^4
Xo = 0 in
Iy = 636173 in^4
Yo = 0 in

Reinforcement:
==============
Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Circular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area, As = 37.44 in^2 at 1.32%
24 #11 Cover = 6 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
==

No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	650.0	260.0	4497.4	17.298
2	1517.0	260.0	5187.0	19.950
3	217.0	260.0	3943.8	15.168
4	1644.0	260.0	5165.0	19.865
5	344.0	260.0	4109.5	15.806
6	960.2	2015.0	4857.4	2.411
7	339.8	2015.0	4104.1	2.037
8	650.0	730.0	4497.4	6.161

*** Program completed as requested! ***

Designer: PTE
Project Name: LITTLE BEAR CREEK
Job Number:
Date: 11/2/2010

Moment Capacity and Interaction Diagram Data
Column or Drilled Shaft: DRILLSHAFT BENT 2

INPUTS

Column Diameter	60	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	#11	
Number of Lonitudinal Reinforcing Bars	24	
Transverse Reinforcing Bar Size	#6	
Spacing of Transverse Reinforcing Bars	12	in
Column Height	13.3	ft
Cover	6	in

Dead Load Reactions

PuDead	650	kips
MuDead	260	kip*ft

Enter into Column Design Software (PCA COLUMN)
Finding Moment Capacity of the column.

Mp = ϕMp/0.9 → 5000 kip*ft

Vpcol = 2*Mp/Heightcolumn → 752 kips

Vpbent = 2*Vpcol 1504 kips

Create SAP model of Bent
Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Puoverturning = 867 kips 994

Pu = PuDead +/- Puoverturning 1517 or 217 kips 1644 or 344

Re-enter into Column Design Software (PCA COLUMN)

Mp = ϕMp/0.9 → 5763 kip*ft 5740

Vpcol = 2*Mp/Heightcolumn → 867 kips 863

Vpbent = 2*Vpcol 1733 kips 1726

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

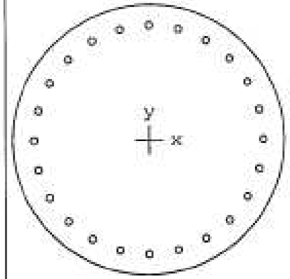
Check = ([Vpbent2 - Vpbent1] / Vpbent2) * 100% 13.2% NO GOOD
0.4% OK

Check Other Seismic Load Cases

Mupotrans	6224	kip*ft	petran =	0.262
Putran	1100	kips	pelong =	0.378
Mupolong	1142	kip*ft		
Pulong	0.291	kips		

Pu = PuDead +/- Putran → = 960.2 or 339.8 kips
Mu = MuDead + Mupotran → 2015 kip*ft

Pu = PuDead +/- Pulong → 650 or 650 kips
Mu = MuDead + Mupolong → 730 kip*ft



54 in diam.

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

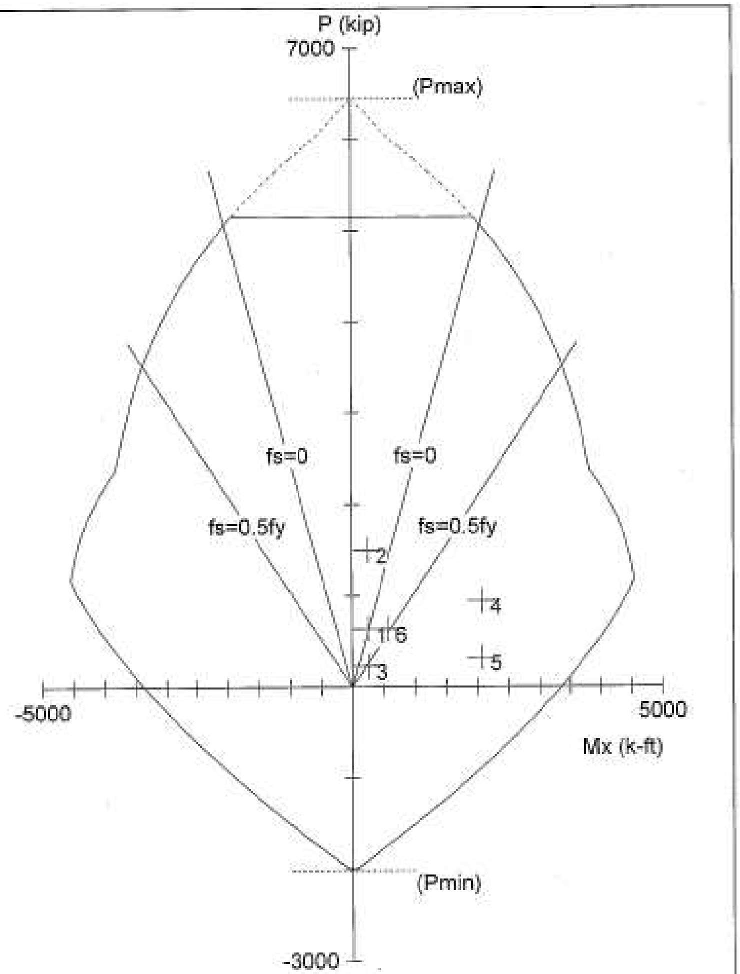
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/02/10

Time: 10:09:05



pcaColumn v3.64. Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59

File: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\Little Bear ...\ColumnDesignBent2.col

Project:

Column:

Engineer:

f'c = 4 ksi	fy = 60 ksi	Ag = 2290.22 in^2	24 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 37.44 in^2	Rho = 1.63%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 417393 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 417393 in^4
Beta1 = 0.85		Clear spacing = 4.54 in	Clear cover = 3.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

10:08 AM

```
OOOOOOO     OOOOO     OOOOO     OOOOO     OOOOO    OO
OO     OO  OO    OO  OO    OO  OO    OO  OO    OO  OO
OO     OO  OO    OO  OO    OO  OO    OO  OO    OO  OO
OO     OO  OO        OO    OO  OO        OO    OO  OO
OO     OO  OO        OOOOOOO   OO        OO    OO  OO
OOOOOOO    OO    OO  OO    OO  OO    OO  OO    OO  OO
OO         OO    OO  OO    OO  OO    OO  OO    OO  OO
OO          OOOOO    OO    OO   OOOOO     OOOOO    OOOOO  (TM)
```

Computer program for the Strength Design of Reinforced Concrete Sections

Licensee stated above acknowledges that Portland Cement Association (PCA) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the pcaColumn(tm) computer program. Furthermore, PCA neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the pcaColumn(tm) program.Although PCA has endeavored to produce pcaColumn(tm) error free, the program is not and can't be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensees. Accordingly, PCA disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the pcaColumn(tm) program.

10:08 AM

General Information:
====================

File Name: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\Little Bear Creek\PCA COLUMN\ColumnDesignBent2.col

Project:
Column:
Code: ACI 318-02

Engineer:
Units: English

Run Option: Investigation
Run Axis: X-axis

Slenderness: Not considered
Column Type: Structural

Material Properties:
====================

f'c = 4 ksi
Ec = 3605 ksi
Ultimate strain = 0.003 in/in
Beta1 = 0.85

fy = 60 ksi
Es = 29000 ksi

Section:
========

Circular: Diameter = 54 in

Gross section area, Ag = 2290.22 in^2
Ix = 417393 in^4
Xo = 0 in

Iy = 417393 in^4
Yo = 0 in

Reinforcement:
==============

Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Circular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area, As = 37.44 in^2 at 1.63%
24 #11 Cover = 3 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
==

No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	637.0	256.0	4098.3	16.009
2	1504.0	256.0	4450.3	17.384
3	230.0	256.0	3637.1	14.208
4	947.2	2073.0	4380.9	2.113
5	326.8	2073.0	3749.9	1.809
6	637.0	570.0	4098.3	7.190

*** Program completed as requested! ***

Designer: PJC
Project Name: LITTLE BEAR CREEK
Job Number:
Date: 11/2/2010

Moment Capacity and Interaction Diagram Data

Column or Drilled Shaft: BENT 3 COLUMN

INPUTS

Column Diameter	54	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	# 11	
Number of Lonitudinal Reinforcing Bars	24	
Transverse Reinforcing Bar Size	# 5	
Spacing of Transverse Reinforcing Bars	12	in
Column Height	16.681	ft
Cover	3	in

Dead Load Reactions

PuDead 657 kips
MuDead 960 kip*ft
MINOR AXIS BENDING
MAJOR AXIS BENDING = 30

Enter into Column Design Software (PCA COLUMN)
Finding Moment Capacity of the column.

Mp = ϕMp/0.9 → 4576 kip*ft

Vpcol = 2*Mp/Heightcolumn → 549 kips

Vpbent = 2*Vpcol 1100 kips

Create SAP model of Bent
Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Puoverturning = 711 kips

Pu = PuDead +/- Puoverturning 1368 or 54 kips

Re-enter into Column Design Software (PCA COLUMN)

Mp = ϕMp/0.9 → 5000 kip*ft

Vpcol = 2*Mp/Heightcolumn → 600 kips

Vpbent = 2*Vpcol 1200 kips

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

Check = ((Vpbent2 - Vpbent1) / Vpbent2) * 100% 8.33 % OK

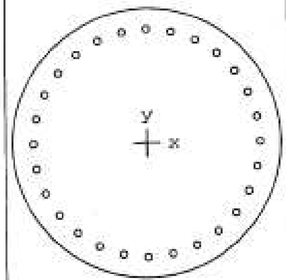
Check Other Seismic Load Cases

Mupotrans	13000	kip*ft	petran =	0.282
Putran	2274	kips	pelong =	0.378
Mupolong	623	kip*ft		
Pulong	4.5	kips		

Pu = PuDead +/- Putran (627) → 1284 or 30 kips
Mu = MuDead + Mupotran → 3696 kip*ft

Pu = PuDead +/- Pulong → 660 or 655 kips
Mu = MuDead + Mupolong (235) → 1195 kip*ft

COMBINATION OF SMALL AXIAL LOAD CAUSES NEED TO INCR. LONG. REINFORCING.
* 28 x #11 *



54 in diam.

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

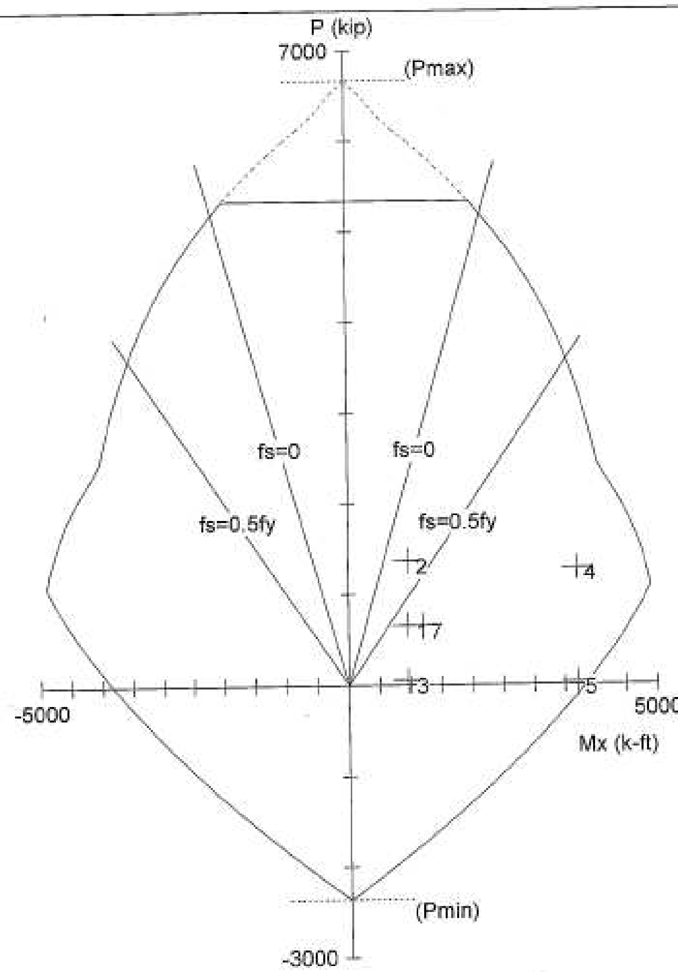
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/02/10

Time: 10:11:52



pcaColumn v3.64. Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59

File: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\Little Bear ...\ColumnDesignBent3.col

Project:

Column:

Engineer:

f'c = 4 ksi	fy = 60 ksi	Ag = 2290.22 in^2	28 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 43.68 in^2	Rho = 1.91%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 417393 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 417393 in^4
Beta1 = 0.85		Clear spacing = 3.69 in	Clear cover = 3.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

10:11 AM

```
0000000    00000     00000     00000     00000    00
00    00  00    00  00    00  00    00  00    00  00
00    00  00    00  00    00  00    00  00    00  00
00    00  00        00    00  00        00    00  00
00    00  00        0000000   00        00    00  00
0000000   00    00  00    00  00    00  00    00  00
00        00    00  00    00  00    00  00    00  00
00         00000    00    00   00000     00000    00000  (TM)
```

Computer program for the Strength Design of Reinforced Concrete Sections

10:11 AM

```
General Information:
====================
    File Name: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\Little Bear Creek\PCA
COLUMN\ColumnDesignBent3.col
    Project:
    Column:                                      Engineer:
    Code:     ACI 318-02                         Units: English

    Run Option: Investigation                    Slenderness: Not considered
    Run Axis:   X-axis                           Column Type: Structural

Material Properties:
====================
    f'c    = 4 ksi                               fy   = 60 ksi
    Ec     = 3605 ksi                            Es   = 29000 ksi
    Ultimate strain = 0.003 in/in
    Beta1 = 0.85

Section:
========
    Circular:    Diameter = 54 in

    Gross section area, Ag =  2290.22 in^2
    Ix =  417393 in^4                            Iy =  417393 in^4
    Xo =  0 in                                   Yo =  0 in

Reinforcement:
==============
    Rebar Database: ASTM A615
    Size Diam (in) Area (in^2)    Size Diam (in) Area (in^2)    Size Diam (in) Area (in^2)
    ---- --------- -----------    ---- --------- -----------    ---- --------- -----------
    #  3     0.38        0.11     #  4     0.50        0.20     #  5     0.63        0.31
    #  6     0.75        0.44     #  7     0.88        0.60     #  8     1.00        0.79
    #  9     1.13        1.00     # 10     1.27        1.27     # 11     1.41        1.56
    # 14     1.69        2.25     # 18     2.26        4.00

    Confinement: Tied; #3 ties with #10 bars,  #4 with larger bars.
    phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.65

    Layout: Circular
    Pattern: All Sides Equal  (Cover to transverse reinforcement)
    Total steel area, As = 43.68 in^2 at 1.91%
    28 #11   Cover = 3 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
=========================================================
                 Pu          Mux         fMnx
    No.          kip         k-ft        k-ft    fMn/Mu
    --- ------------ ------------ ------------ --------
     1         657.0        960.0       4546.5    4.736
     2        1368.0        960.0       4799.4    4.999
     3          54.0        960.0       3902.3    4.065
     4        1284.0       3696.0       4828.1    1.306
     5          30.0       3696.0       3874.5    1.048
     6         660.0       1195.0       4549.2    3.807
     7         655.0       1195.0       4544.6    3.803

    *** Program completed as requested! ***
```

Designer: PTC
Project Name: LITTLE BEAR CREEK
Job Number:
Date: 11/8/2010

Moment Capacity and Interaction Diagram Data

Column or Drilled Shaft: DRILLED SHAFT BENT 3

INPUTS

Column Diameter	60	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	# 11	
Number of Lonitudinal Reinforcing Bars	24	
Transverse Reinforcing Bar Size	# 6	
Spacing of Transverse Reinforcing Bars	12	in
Column Height	12.8	ft
Cover	6	in

Dead Load Reactions

PuDead	666	kips
MuDead	1091	kip*ft

Enter into Column Design Software (PCA COLUMN)
Finding Moment Capacity of the column.

Mp = ϕMp/0.9 → 5019 kip*ft

Vpcol = 2*Mp/Heightcolumn → 784.2 kips

Vpbent = 2*Vpcol 1568 kips

Create SAP model of Bent
Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Puoverturning = 1262 kips

Pu = PuDead +/- Puoverturning 1928 or 596 kips

Re-enter into Column Design Software (PCA COLUMN)

Mp = ϕMp/0.9 → 5680 kip*ft

Vpcol = 2*Mp/Heightcolumn → 887 kips

Vpbent = 2*Vpcol 1775 kips

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

Check = ((Vpbent2 - Vpbent1) / Vpbent2) * 100% 11% OK

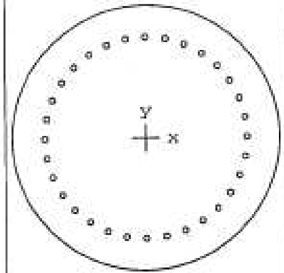
Check Other Seismic Load Cases

Mupotrans	12278	kip*ft	petran =	0.287
Putran	2224	kips	pelong =	0.378
Mupolong	565	kip*ft		
Pulong	4.7	kips		

Pu = PuDead +/- Putran → 1293 or 39 kips
Mu = MuDead + Mupotran → 4553 kip*ft

Pu = PuDead +/- Pulong → 666 or 666 kips
Mu = MuDead + Mupolong → 1304 kip*ft

NOTE: COMBINATION OF SMALL AXIAL LOAD CAUSE NEED TO INCREASE LONG. REINFORCING
* 32 x # 11 *



60 in diam.

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

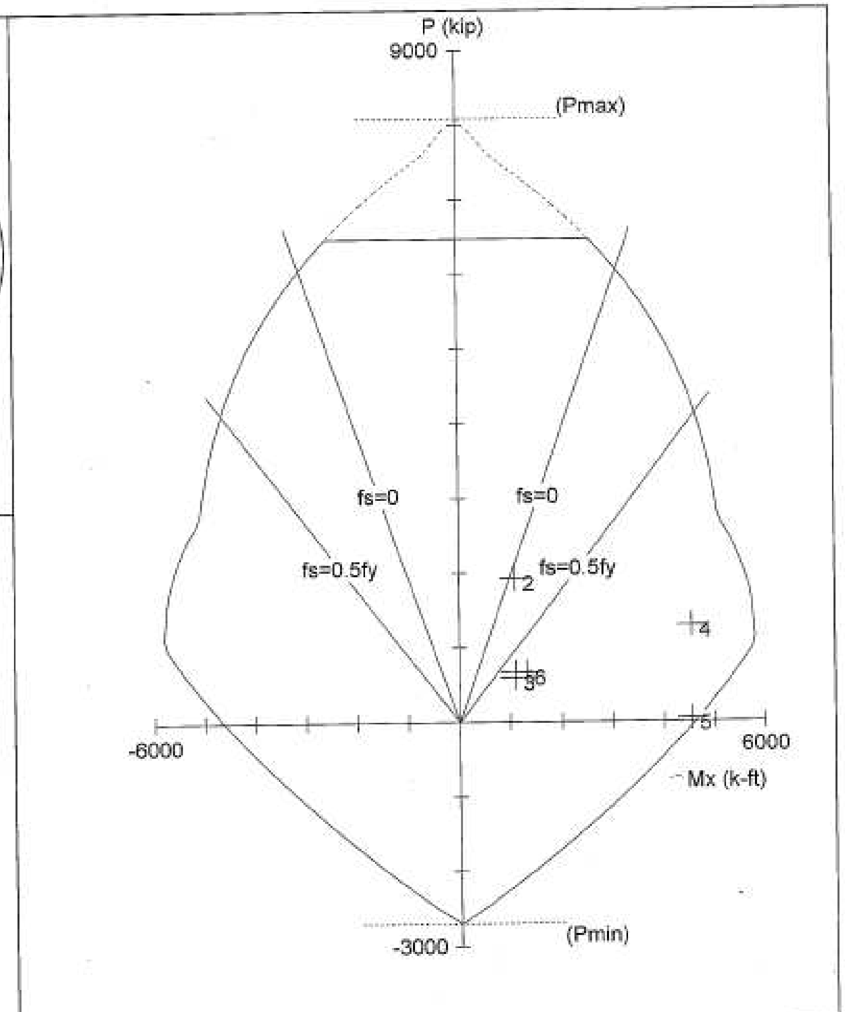
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/02/10

Time: 10:12:59



pcaColumn v3.64. Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59

File: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\Little Bear Cre...\DrilledShaftBent3.col

Project:

Column:		Engineer:	
f'c = 4 ksi	fy = 60 ksi	Ag = 2827.43 in^2	32 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 49.92 in^2	Rho = 1.77%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 636173 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 636173 in^4
Beta1 = 0.85		Clear spacing = 3.06 in	Clear cover = 6.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

```
pcaColumn v3.64 © Portland Cement Association                              Page   1
Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59    11/02/10
C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\Little Bear Creek\PCA COLUMN\

10:12 AM
```

```
OOOOOOO    OOOOO    OOOOO    OOOOO    OOOOO   OO
OO    OO  OO   OO  OO   OO  OO   OO  OO   OO  OO
OO    OO  OO   OO  OO   OO  OO   OO  OO   OO  OO
OO    OO  OO       OO   OO  OO       OO   OO  OO
OO    OO  OO       OOOOOOO  OO       OO   OO  OO
OOOOOOO   OO   OO  OO   OO  OO   OO  OO   OO  OO
OO        OO   OO  OO   OO  OO   OO  OO   OO  OO
OO         OOOOO   OO   OO   OOOOO    OOOOO   OOOOO  (TM)
```

Computer program for the Strength Design of Reinforced Concrete Sections

Licensee stated above acknowledges that Portland Cement Association (PCA) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the pcaColumn(tm) computer program. Furthermore, PCA neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the pcaColumn(tm) program.Although PCA has endeavored to produce pcaColumn(tm) error free, the program is not and can't be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensees. Accordingly, PCA disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the pcaColumn(tm) program.

General Information:

File Name: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\Little Bear C
Project:
Column:
Code: ACI 318-02
Engineer:
Units: English

Run Option: Investigation
Run Axis: X-axis
Slenderness: Not considered
Column Type: Structural

Material Properties:

f'c = 4 ksi
Ec = 3605 ksi
Ultimate strain = 0.003 in/in
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

Section:

Circular: Diameter = 60 in

Gross section area, Ag = 2827.43 in^2
Ix = 636173 in^4
Xo = 0 in
Iy = 636173 in^4
Yo = 0 in

Reinforcement:

Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Circular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area, As = 49.92 in^2 at 1.77%
32 #11 Cover = 6 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)

No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	666.0	1091.0	5423.4	4.971
2	1928.0	1091.0	5638.2	5.168
3	596.0	1091.0	5349.6	4.903
4	1293.0	4553.0	5786.6	1.271
5	39.0	4553.0	4681.0	1.028
6	666.0	1304.0	5423.4	4.159

*** Program completed as requested! ***

Designer: PTC
Project Name: LITTLE BEAR CREEK
Job Number:
Date: 4/2/2010

Moment Capacity and Interaction Diagram Data
Column or Drilled Shaft: ABUTMENT DRILLED SHAFT

INPUTS

Column Diameter	42	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	#11	
Number of Lonitudinal Reinforcing Bars	12	
Transverse Reinforcing Bar Size	#5	
Spacing of Transverse Reinforcing Bars	12	in
Column Height	7.639	ft
Cover	6	in

Dead Load Reactions

PuDead	5.5	kips
MuDead	114.75	kip*ft

Enter Into Column Design Software (PCA COLUMN)
Finding Moment Capacity of the column.

Mp = ϕMp/0.9 → [illegible] kip*ft

Vpcol = 2*Mp/Heightcolumn → [illegible] kips

Vpbent = ~~2~~ 3*Vpcol [illegible] kips

Create SAP model of Bent
Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Puoverturning = [illegible] kips

Pu = PuDead +/- Puoverturning [illegible] or [illegible] kips

Re-enter Into Column Design Software (PCA COLUMN)

Mp = ϕMp/0.9 → [illegible] kip*ft

Vpcol = 2*Mp/Heightcolumn → [illegible] kips

Vpbent = 2*Vpcol [illegible] kips

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

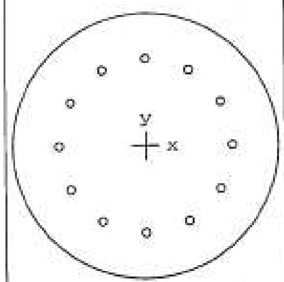
Check = ((Vpbent2 - Vpbent1) / Vpbent2) * 100% 14.

Check Other Seismic Load Cases

Mupotrans	950	kip*ft	petran =	0.282
Putran	0	kips	pelong =	0.378
Mupolong	1909	kip*ft		
Pulong	0	kips		

Pu = PuDead +/- Putran → 5.5 or 5.5 kips
Mu = MuDead + Mupotran → 383 kip*ft

Pu = PuDead +/- Pulong → 5.5 or 5.5 kips
Mu = MuDead + Mupolong → 836 kip*ft



42 in diam.

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

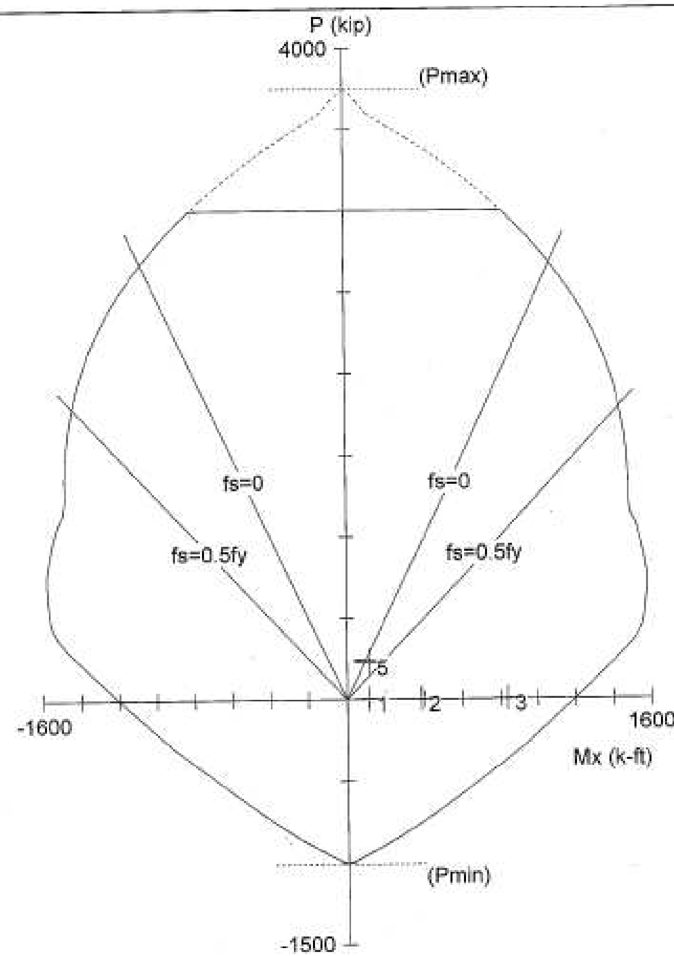
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/02/10

Time: 10:14:02



pcaColumn v3.64. Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59

File: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\Little Bear ...\DrilledShaftAbutment.col

Project:

Column:

Engineer:

f'c = 4 ksi	fy = 60 ksi	Ag = 1385.44 in^2	12 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 18.72 in^2	Rho = 1.35%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 152745 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 152745 in^4
Beta1 = 0.85		Clear spacing = 5.73 in	Clear cover = 6.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

10:13 AM

```
0000000    00000    00000    00000    00000   00
00    00  00   00  00   00  00   00  00   00  00
00    00  00   00  00   00  00   00  00   00  00
00    00  00       00   00  00       00   00  00
00    00  00       0000000  00       00   00  00
0000000   00   00  00   00  00   00  00   00  00
00        00   00  00   00  00   00  00   00  00
00         00000   00   00   00000    00000   00000  (TM)
```

===
Computer program for the Strength Design of Reinforced Concrete Sections
===

10:13 AM

General Information:
====================

File Name: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\Little Bear Creek\PCA COLUMN\DrilledShaftAbutment.col
Project:
Column: Engineer:
Code: ACI 318-02   Units: English

Run Option: Investigation   Slenderness: Not considered
Run Axis: X-axis   Column Type: Structural

Material Properties:
====================

f'c = 4 ksi   fy = 60 ksi
Ec = 3605 ksi   Es = 29000 ksi
Ultimate strain = 0.003 in/in
Beta1 = 0.85

Section:
========

Circular: Diameter = 42 in

Gross section area, Ag = 1385.44 in^2
Ix = 152745 in^4   Iy = 152745 in^4
Xo = 0 in   Yo = 0 in

Reinforcement:
==============

Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Circular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area, As = 18.72 in^2 at 1.35%
12 #11 Cover = 6 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
==

No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	5.5	114.8	1204.5	10.497
2	5.5	383.0	1204.5	3.145
3	5.5	836.0	1204.5	1.441
4	238.5	114.8	1414.2	12.324
5	227.5	114.8	1404.6	12.240

*** Program completed as requested! ***

Designer: Paul Coulston
Project Name: Scarham Creek Bridge
Job Number:
Date: 11/10/2010

$ORIGIN := 1$

Description of worksheet: This worksheet is a seismic bridge design worksheet for the **AASHTO Guide Specifications for LRFD Seismic Bridge Design**. All preliminary design should already be done for service loads.

Project Known Information

Location: Marshall County
Zip Code or Coordinates: 35.0069 N 88.2025 W

Superstructure Type: AASTHO I girders
Substructure Type: Circular columns supported on drilled shafts
Abutment Type: Abutment beam supported on drilled shafts

Note: *Input all of the below information.*

$fc := 4000$ psi

$fye := 60000$ psi

$\rho_{conc} := 0.08681 \ \frac{lb}{in^3}$

$g := 386.4 \ \frac{in}{s^2}$

Length of Bridge (ft)	$L := 520$ ft
Span Length (ft)	$Span := 130$ ft
Deck Thickness (in)	$t_{deck} := 7$ in
Deck Width (ft)	$DeckWidth := 40$ ft
Girder X-Sectional Area (in^2)	$GirderArea := 767$ in^2
Bent Volume (ft^3)	$BentVolume := 7.5 \cdot 5.5 \cdot 40 = 1.65 \times 10^3$ ft^3
Guard Rail Area (in^2)	$GuardRailArea := 310$ in^2
Column 1 Diameter (in)	$Columndia1 := 60$ in
Column 2 Diameter (in)	$Columndia2 := 72$ in
Drill Shaft 1 Diameter (in)	$Drillshaftdia1 := 66$ in
Drill Shaft 2 Diameter (in)	$Drillshaftdia2 := 78$ in
Drill Shaft 3 (Abutment) Diameter (in)	$Drillshaftdia3 := 54$ in

Strut 2 & 4 Depth (in)	$Strut2Depth := 72$ in
Strut 2 & 4 Width (in)	$Strut2Width := 42$ in
Strut 3 Depth (in)	$Strut3Depth := 120$ in
Strut 3 Width (in)	$Strut3Width := 42$ in
Tallest Above Ground Column Height Bent 2 (ft)	$ColumnHeight2 := 34.022$ ft
Talllest Above Ground Column Height Bent 3 (ft)	$ColumnHeight3 := 59.136$ ft
Tallest Above Ground Column Height Bent 4 (ft)	$ColumnHeight4 := 32.156$ ft
Length of Strut 2 & 4 (ft)	$Lstrut2 := 19$ ft
Length of Strut 3 (ft)	$Lstrut3 := 18$ ft

Column 1 Area (in^2) $\quad Acolumn1 := \frac{Columndia1^2 \cdot \pi}{4} = 2.827 \times 10^3$ in^2

Column 2 Area (in^2) $\quad Acolumn2 := \frac{Columndia2^2 \cdot \pi}{4} = 4.072 \times 10^3$ in^2

Bent 2 and 4 Strut Volume (ft^3) $\quad Strut1 := 6 \cdot 3.5 \cdot 19 = 399$ ft^3

Bent 3 Strut Volume (ft^3) $\quad Strut2 := 10 \cdot 3.5 \cdot 18 = 630$ ft^3

Note: These are variables that were easier to input in ft and then convert to inches.

$L := L \cdot 12 = 6.24 \times 10^3$ in

$Span := Span \cdot 12 = 1.56 \times 10^3$ in

$DeckWidth := DeckWidth \cdot 12 = 480$ in

$BentVolume := BentVolume \cdot 12^3 = 2.851 \times 10^6$ in^3

$ColumnHeight2 := ColumnHeight2 \cdot 12 = 408.264$ in

$ColumnHeight3 := ColumnHeight3 \cdot 12 = 709.632$ in

$ColumnHeight4 := ColumnHeight4 \cdot 12 = 385.872$ in

$Strut1 := Strut1 \cdot 12 = 4.788 \times 10^3$ in^3

$Strut2 := Strut2 \cdot 12 = 7.56 \times 10^3$ in^3

Steps for Seismic Design

Article 3.1: The Guide Specification only applies to the design of CONVENTIONAL BRIDGES.

Article 3.2: Bridges are design for the life safety performance objective.

Article 6.2: Requires a subsurface investigation take place.

Article 6.8 and C6.8: Liquefaction Design Requirements - A liquefaction assessment should be made if loose to very loose sands are present to a sufficient extent to impact the bridge stability and A_s is greater than or equal to 0.15.

Article 3.3: The type of Earthquake Resisting System (ERS) should be considered. This is not a requirement as in SDC C and D, but should be considered. A Type 1 ERS has a ductile substructure and essentially elastic superstructure.

Type of Bridge: TYPE 1

***Article 3.4*: Determine Design Response Spectrum**

Note: AASHTO Guide Specifications for LRFD Seismic Bridge Design is accompanied with a program that has the Seismic Hazard maps. This program will calculate several of the variables that are needed for the analysis.

1) *Article 3.4.2.1*:Determine the Site Class. Table 3.4.2.1-1

INPUT Site Class: **D**

2) Enter maps and find PGA, S_s,and S_1. Then enter those values in their respective spot. Also, the the Guide Specification is accompanied with a cd that contains a program that will find these values for the designer.

$PGA := 0.116 \ g$

INPUT $S_s := 0.272 \ g$

$S_1 := 0.092 \ g$

3) *Article 3.4.2.3:* Site Coefficients. From the PGA,S_s, and S_1 values and site class choose F_{PGA}, F_a, and F_v. Note: straight line interpolation is permitted.

$F_{PGA} := 1.57$ Table 3.4.2.3-1

INPUT $F_a := 1.58$ Table 3.4.2.3-1

$F_v := 2.4$ Table 3.4.2.3-2

Eq. 3.4.1-1 $A_s := F_{PGA} \cdot PGA = 0.182 \;\; g$ A_s : Acceleration Coefficient

Eq. 3.4.1-2 $SDS := F_a \cdot S_s = 0.43 \;\; g$ S_{DS} = Short Period Acceleration Coefficient

Eq. 3.4.1-3 $SD1 := F_v \cdot S_1 = 0.221 \;\; g$ S_{D1} = 1-sec Period Acceleration Coefficient

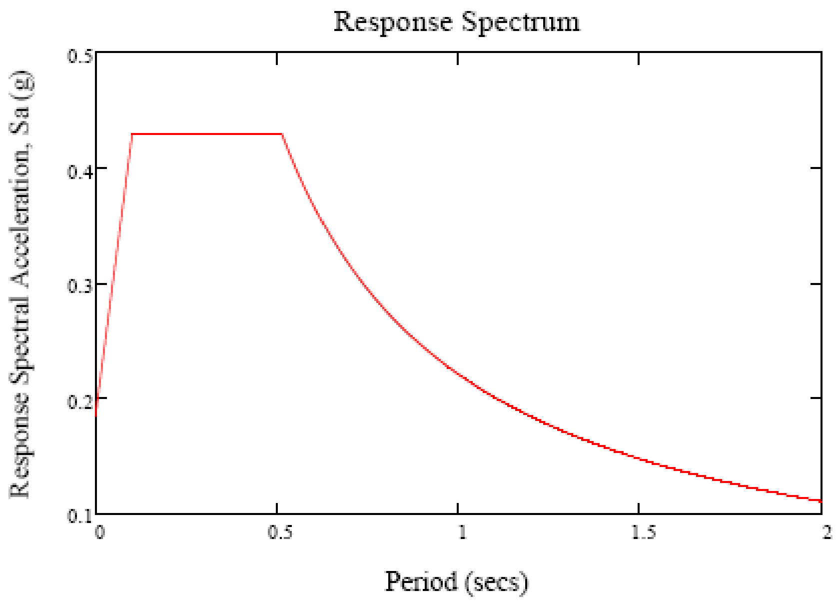
4) Creating a Response Spectrum

Note: The DesignSpectrum Code creates a Response Spectrum for the Bridge.
At this time the period of the bridge is unknown; therefore, the Sa value cannot be calculated.

$Tmax := 2 \;\; s \qquad Dt := 0.001 \;\; s$

$$\mathrm{DesignSpectrum}(SDS, SD1, A_s, Tmax, Dt) := \left| \begin{array}{l} T_s \leftarrow \dfrac{SD1}{SDS} \\ T_o \leftarrow 0.2 \cdot T_s \\ n_{max} \leftarrow \dfrac{Tmax}{Dt} \\ \text{for } i \in 1 .. n_{max} \\ \quad \left| \begin{array}{l} T_i \leftarrow Dt \cdot i \\ a_i \leftarrow (SDS - A_s) \cdot \dfrac{Dt \cdot i}{T_o} + A_s \;\; \text{if } Dt \cdot i < T_o \\ a_i \leftarrow SDS \;\; \text{if } Dt \cdot i \geq T_o \wedge Dt \cdot i \leq T_s \\ a_i \leftarrow \dfrac{SD1}{Dt \cdot i} \;\; \text{if } Dt \cdot i > T_s \end{array} \right. \\ R \leftarrow \mathrm{augment}(T, a) \\ R \end{array} \right.$$

$\mathrm{BridgeSpectrum} := \mathrm{DesignSpectrum}(SDS, SD1, A_s, Tmax, Dt)$



***Article 3.5*: Selection of Seismic Design Category**

SD1 = 0.221 g From Table 3.5-1 Choose SDC

$$\text{SDCprogram(SD1)} := \left| \begin{array}{l} \text{for } c \in \text{SD1} \\ \quad \left| \begin{array}{l} c \leftarrow \text{"A" if SD1} < 0.15 \\ c \leftarrow \text{"B" if SD1} \geq 0.15 \wedge \text{SD1} < 0.3 \\ c \leftarrow \text{"C" if SD1} \geq 0.3 \wedge \text{SD1} < 0.5 \\ c \leftarrow \text{"D" if SD1} \geq 0.5 \end{array} \right. \\ \text{Rs} \leftarrow c \\ c \end{array} \right.$$

SDC := SDCprogram(SD1) = "B"

Displacement Demand Analysis Δ_D

Figure 1.3-2 Demand Analysis Flowchart

***Article 4.2*: Selection of Analysis Procedure**

This is a function of the SDC and the regularity of the bridge.

Procedure 1 = Equivalent Static Method

Article 4.3.3: Displacement Magnification for Short-Period Structures

$u_d := 2$ for SDC B

$$\mathrm{Rdprogram}(T, SDS, SD1, u_d) := \left| \begin{array}{l} Ts \leftarrow \dfrac{SD1}{SDS} \\ Tb \leftarrow 1.25 \cdot Ts \\ x \leftarrow \left(1 - \dfrac{1}{u_d}\right) \cdot \dfrac{Tb}{T} + \dfrac{1}{u_d} \\ y \leftarrow 1.0 \\ a \leftarrow x \ \text{ if } \dfrac{Tb}{T} > 1.0 \\ a \leftarrow y \ \text{ if } \dfrac{Tb}{T} \le 1.0 \\ a \end{array} \right.$$

Note: This Rd value will be calculated when the period of the structure is known. This factor will amplify the displacement demand.

***Article 5.4*: Analytical Procedure 1 (Equivalent Static Analysis)**

Note: There are two methods that can be used according to this procedure. The Uniform Load Method is suitable for regular bridges that respond principally in their fundamental mode of vibration. The Single Mode Spectral Method may be a better method if there is a major change in the spans, stiffness of the piers, etc.
The Uniform Load Method is simpler and less time consuming and will give accurate results, and this is the reason it has been chosen in this design.

<u>Uniform Load Method</u>

Step 1: Build a bridge model
Step 2: Apply a uniform load of Po = 1.0 kip/in. in both the longitudinal and transverse direction. Also, the uniform load can be converted into point loads and applied as joint loads in SAP. Calculate the static displacement for both directions. In SAP, tables of the displacements can be exported to EXCEL, and the MAX Function can be used to find the maximum displacement.

Step 3: Calculate the bridge lateral stiffness, K, and total weight, W.

$$p_o := 1.0 \quad \frac{kip}{in}$$

INPUT

$$\upsilon_{smaxLong} := 0.382075 \quad in$$

$$\upsilon_{smaxTran} := 4.330046 \quad in$$

Eq. C5.4.2-1 $$K_{Long} := \frac{p_o \cdot L}{\upsilon_{smaxLong}} = 1.633 \times 10^4 \quad \frac{kip}{in}$$

Eq. C5.4.2-2 $$K_{Tran} := \frac{p_o \cdot L}{\upsilon_{smaxTran}} = 1.441 \times 10^3 \quad \frac{kip}{in}$$

INPUT: Multiplying factors

$$W := \frac{\rho_{conc} \cdot \begin{bmatrix} L \cdot \left(t_{deck} \cdot DeckWidth + GirderArea \cdot 6 + GuardRailArea\right) + 3 \cdot BentVolume \ldots \\ + 4 \cdot Acolumn1 \cdot (2ColumnHeight2 + 2 \cdot ColumnHeight4) + 2Acolumn2 \cdot ColumnHeight3 \ldots \\ + Strut1 \cdot 2 + Strut2 \end{bmatrix}}{1000}$$

$$W = 7285.919 \quad kips$$

Step 4: Calculate the period, T_m.

Eq. C5.4.2-3 $$T_{mLong} := 2\pi \cdot \sqrt{\frac{W}{K_{Long} \cdot g}} = 0.213 \quad s$$

Step 5: Calculate equivalent static earthquake loading p_e.

$$acc(SDS, SD1, T_{mLong}, A_s) := \left| \begin{array}{l} T_s \leftarrow \dfrac{SD1}{SDS} \\ T_o \leftarrow 0.2 \cdot T_s \\ \text{for } a \in T_{mLong} \\ \quad \left| \begin{array}{l} a \leftarrow (SDS - A_s) \cdot \dfrac{T_{mLong}}{T_o} + A_s \ \text{ if } T_{mLong} < T_o \\ a \leftarrow SDS \ \text{ if } T_{mLong} \ge T_o \wedge T_{mLong} \le T_s \\ a \leftarrow \dfrac{SD1}{T_{mLong}} \ \text{ if } T_{mLong} > T_s \end{array} \right. \\ Ra \leftarrow a \\ a \end{array} \right.$$

$$Sa_{Long} := acc(SDS, SD1, T_{mLong}, A_s) = 0.43$$

Eq. C5.4.2-4 $$p_{eLong} := \frac{Sa_{Long} \cdot W}{L} = 0.502 \quad \frac{kip}{in}$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$Rd_{Long} := Rdprogram(T_{mLong}, SDS, SD1, u_d) = 2.004$$

$$v_{smaxLong} := Rd_{Long} \cdot \frac{p_{eLong}}{p_o} \cdot \upsilon_{smaxLong} = 0.384 \quad in$$

Repeat Steps 4, 5, and 6 for transverse loading.

Step 4: Calculate the period, T_m.

Eq. C5.4.2-3 $$T_{mTran} := 2\pi \cdot \sqrt{\frac{W}{K_{Tran} \cdot g}} = 0.719 \quad s$$

Step 5: Calculate equivalent static earthquake loading p_e.

$$Sa_{Tran} := acc(SDS, SD1, T_{mTran}, A_s) = 0.307$$

Eq. C5.4.2-4 $$p_{eTran} := \frac{Sa_{Tran} \cdot W}{L} = 0.359 \qquad \frac{kip}{in}$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$Rd_{Tran} := Rdprogram(T_{mTran}, SDS, SD1, u_d) = 1$$

$$v_{smaxTran} := Rd_{Tran} \cdot \frac{p_{eTran}}{p_o} \cdot \upsilon_{smaxTran} = 1.553 \qquad in$$

Single-Mode Spectral Method

This procedure is not specifically addressed in the Guide Specifications. The Guide Spec. refers you to the AASHTO LRFD Bridge Design Specifications.

Article 4.7.4.3.2: Single-Mode Spectral Method

Step 1: Build a bridge model
Step 2: Apply a uniform load of Po = 1.0 in both the longitudinal and transverse direction. Calculate the static displacement for both directions.

Step 3: Calculate factors α, β , and γ.

Note: The Deflection equations come from analysis of the SAP model. The displacement is taken at the joints along the length of the bridge and input into an Excel Worksheet. Then a graph is created of the displacements along the length of the bridge. A best fit line is plotted, and that is the equation that is shown below.

INPUT $v_{stran}(x) := -3 \cdot 10^{-7} \cdot x^2 + 0.0016 \cdot x + 1.4093$ $\qquad v_{slong}(x) := -1 \cdot 10^{-8} \cdot x^2 + 0.0001x + 0.1563$

C4.7.4.3.2b-1 $\alpha_{Tran} := \int_0^L v_{stran}(x)\,dx$ $\alpha_{Long} := \int_0^L v_{slong}(x)\,dx$

C4.7.4.3.2b-2 $\beta_{Tran} := \int_0^L \frac{W}{L} v_{stran}(x)\,dx$ $\beta_{Long} := \int_0^L \frac{W}{L} \cdot v_{slong}(x)\,dx$

C4.7.4.3.2b-3 $\gamma_{Tran} := \int_0^L \frac{W}{L} \cdot v_{stran}(x)^2\,dx = 5.308 \times 10^4$ $\gamma_{Long} := \int_0^L \frac{W}{L} \cdot v_{slong}(x)^2\,dx$

α = Displacement along the length

β = Weight per unit length * Displacement

γ = Weight per unit length * Displacement^2

Step 4: Calculate the Period of the Bridge

Eq. 4.7.4.3.2b-4 $T_{mTran1} := 2\pi \cdot \sqrt{\frac{\gamma_{Tran}}{p_o \cdot g \cdot \alpha_{Tran}}} = 0.589$ s

Eq. 4.7.4.3.2b-4 $T_{mLong1} := 2\pi \cdot \sqrt{\frac{\gamma_{Long}}{p_o \cdot g \cdot \alpha_{Long}}} = 0.206$ s

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smLong} := acc\left(SDS, SD1, T_{mLong1}, A_s\right) = 0.43$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

Eq. C4.7.4.3.2b-5 $PeLong(x) := \frac{\beta_{Long} \cdot C_{smLong}}{\gamma_{Long}} \cdot \frac{W}{L} \cdot v_{slong}(x)$

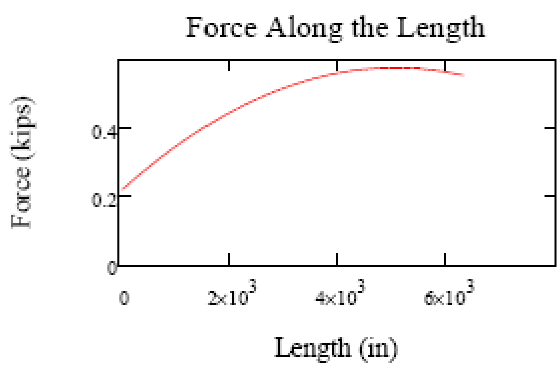
$PeLong(x) \rightarrow 0.00014153071449361569124 \cdot x + -1.4153071449361569124e\text{-}8 \cdot x^2 + 0.221212506753521325$

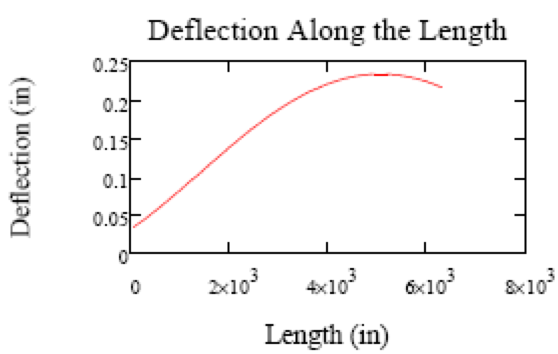
$dW := \frac{L}{100}$

$i := 1 .. 101$

$Pelong_i := PeLong[(i - 1) \cdot dW]$ $\delta long_i := v_{slong}[(i - 1)dW]$

$\Delta long_i := Pelong_i \cdot \delta long_i$





Maximum Deflection

$\max(\Delta long) = 0.234$ in

NOTE: Repeat Steps 5 and 6 for Transverse Direction.

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smTran} := acc\left(SDS, SD1, T_{mTran1}, A_s\right) = 0.375$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

Eq. C4.7.4.3.2b-5 $$PeTran(x) := \frac{\beta_{Tran} \cdot C_{smTran}}{\gamma_{Tran}} \cdot \frac{W}{L} \cdot v_{stran}(x)$$

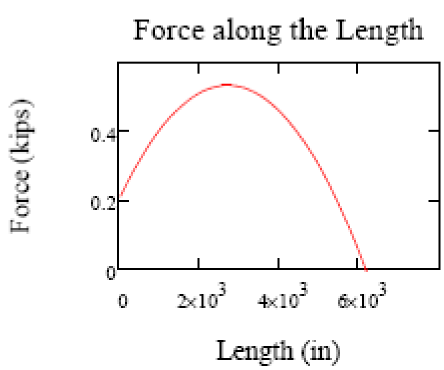
$$PeTran(x) \rightarrow 0.00024113243850551203633 \cdot x + -4.5212332219783506812e\text{-}8 \cdot x^2 + 0.2123924659911363205$$

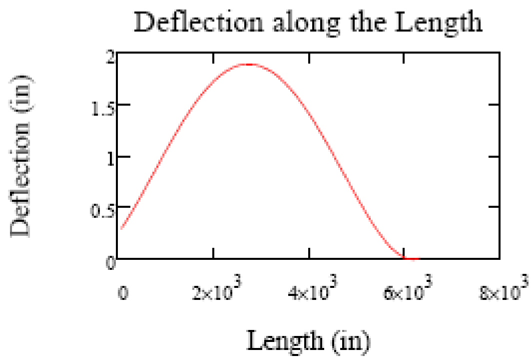
$$dL := \frac{L}{100}$$

$$i := 1..101$$

$$Petran_i := PeTran[(i-1) \cdot dL] \qquad \delta tran_i := v_{stran}[(i-1)dL]$$

$$\Delta tran_i := Petran_i \cdot \delta tran_i$$





Maximum Deflection

$\max(\Delta tran) = 1.891$ in

***Article 5.6*: Effective Section Properties**

Note: Use 0.7*Ig for ductile reinforced concrete members.
Refer to the charts on page 5-20 of the Guide Specification if a more precise value is desired.

***Article 5.2:* Abutment Modeling**

Note: This is taken care of in the SAP model.

***Article 5.3*: Foundations Modeling**

Note: Since in SDC B, Foundation Modeling Methods I can be used.

FMM is dependent on the type of foundation.

For bridges with Pile Bent/Drilled Shaft the depth of fixity can be estimated.
Since details regarding reinforcing are not known, reduce the stiffness of the drilled shafts to **one half** the uncracked section.

Note: Special provisions need to be considered if Liquefaction is present. (Article 6.8)

Article 4.4: Combination of Orthogonal Seismic Displacement Demands

$$\text{LoadCase1} := \sqrt{\left(1 \cdot v_{smaxLong}\right)^2 + \left(0.3 \cdot v_{smaxTran}\right)^2} = 0.604 \quad \text{in}$$

$$\text{LoadCase2} := \sqrt{\left(1 \cdot v_{smaxTran}\right)^2 + \left(0.3 \cdot v_{smaxLong}\right)^2} = 1.557 \quad \text{in}$$

COLUMN DESIGN

Article 4.8: **Displacement Demand/Capacity**

Note: If the column height is different for each bent, a capacity check needs to be made at each bent.

Displacement Demand/Capacity for the Bents $\Delta_D < \Delta_C$

Note: Since the bridge has frame bents, the simplified equations cannot be used; therefore a pushover analysis must be done.

NOTE: IF THE SIMPLIFIED EQUATIONS ABOVE DO NOT WORK, A PUSHOVER ANALYSIS OF THE BRIDGE CAN BE DONE TO VERIFY THE DISPLACEMENT CAPACITY. In SAP 2000, there is an earthquake design program that allows a pushover analysis to be done by setting the SDC to D. Be sure to amplify the demand values by the appropriate Rd value. List the results below to verify that the Displacement Capacity is sufficient. The Demand Displacement must be multiplied by pe/po. The below chart was created in Excel and then brought into Mathcad.

GenDispl	Demand (in)	Capacity (in)	Check
_GD_TR1_DReq1	2.440858	9.7681	OK
_GD_LG1_DReq1	0.54952	2.196964	OK
_GD_TR2_DReq1	6.903604	25.640073	OK
_GD_LG2_DReq1	0.870083	3.574987	OK
_GD_TR3_DReq1	2.870598	11.474908	OK
_GD_LG3_DReq1	0.616989	2.644054	OK

Article 4.12: **Minimum Support Length Requirements**

Note: May need to add more calculations if column heights are different at the bents.

N (in) = Minimum support length measured normal to the bridge

L (ft) = Length of bridge to adjacent expansion joint or end of the bridge

H (ft) = average height of columns supporting bridge deck for abutments
for columns and piers = column height

S (Degree) = angle of skew

Abutment Support Length Requirement

INPUT $\mathrm{Span_{abutment}} := \frac{\mathrm{Span}}{12} = 130$ ft $\quad H_{abutment} := \frac{\mathrm{ColumnHeight2}}{12} = 34.02\text{:}$ ft

Note: The $\mathrm{Span_{abutment}}$ is divided by number of spans and inches.

$\mathrm{Skew_{abutment}} := 0$ Degrees

Eq. 4.12.2-1

$$\mathrm{Nabutment} := 1.5\cdot\left(8 + 0.02\mathrm{Span_{abutment}} + 0.08H_{abutment}\right)\cdot\left(1 + 0.000125\mathrm{Skew_{abutment}}^2\right) = 19.983 \quad \text{in}$$

Bent Support Length Requirement

BENT 2

INPUT $\mathrm{Span_{Bent}} := \frac{\mathrm{Span}}{12} = 130$ ft

Note: The $\mathrm{Span_{abutment}}$ is divided by number of spans and inches.

INPUT $H_{Bent} := \frac{\mathrm{ColumnHeight2}}{12} = 34.022$ ft $\quad$ *INPUT*: Column Height for this Bent

INPUT $Skew_{Bent} := 0$ Degrees

Eq. 4.12.2-1

$$N_{Bent} := 1.5\cdot(8 + 0.02Span_{Bent} + 0.08H_{Bent})\cdot\left(1 + 0.000125Skew_{Bent}^{2}\right) = 19.983 \quad \text{in}$$

BENT 3

INPUT $Span_{Bent} := \frac{Span}{1\cdot 12} = 130$ ft

Note: The $Span_{abutment}$ is divided by number of spans and inches.

INPUT $H_{Bent} := \frac{ColumnHeight3}{12} = 59.136$ ft *INPUT*: Column Height for this Bent

INPUT $Skew_{Bent} := 0$ Degrees

Eq. 4.12.2-1

$$N_{Bent} := 1.5\cdot(8 + 0.02Span_{Bent} + 0.08H_{Bent})\cdot\left(1 + 0.000125Skew_{Bent}^{2}\right) = 22.996 \quad \text{in}$$

BENT 4

INPUT $Span_{Bent} := \frac{Span}{1\cdot 12} = 130$ ft

Note: The $Span_{abutment}$ is divided by number of spans and inches.

INPUT $H_{Bent} := \frac{ColumnHeight4}{12} = 32.156$ ft *INPUT*: Column Height for this Bent

INPUT $Skew_{Bent} := 0$ Degrees

Eq. 4.12.2-1

$$N_{Bent} := 1.5\cdot(8 + 0.02Span_{Bent} + 0.08H_{Bent})\cdot\left(1 + 0.000125Skew_{Bent}^{2}\right) = 19.759 \quad \text{in}$$

***Article 4.14*: Superstructure Shear Keys**

$$V_{ok} := 2 \cdot V_n$$

Note: This does not apply to this bridge.

Figure 1.3-5 SDC B Detailing

Decide what Type of bridge designing.

Structure Type: **Type 1**

***Article 8.3*: Determine Flexure and Shear Demands**

***Article 8.5*: Plastic Moment Capacity**

Note: Article 8.5 refers the designer back to Article 4.11.1-4.

BENT 2 DESIGN

***Article 4.11.1-4*: Steps to find Moment Capacity, Shear Capacity, and Axial Force**

Note: Use some kind of software to find the Moment Capacity of the Column. PCA Column was used to create an Interaction Diagram and to calculate the Moment Capacity. The shear for the bent was found by knowing the Moment.

INPUT $M_p := 75696000 \quad lb \cdot in$

INPUT $Fixity := 300\ in$

Note: Fixity is the point of fixity for the column/drilledshaft.

$$V_p := \frac{2 \cdot M_p}{Fixity \cdot 1000} = 504.64 \quad kips \qquad V_{pBent2} := 2 \cdot V_p = 1.009 \times 10^3 \quad kips$$

Note: If the decision is made to design for **Elastic Forces** then the above variables need to be override. This can be done by simply changing the Vp variable to the elastic force from SAP2000 that has been multiplied by pe/po.

$$V_p := 490 \quad kips \qquad V_{pBent2} := 2 \cdot V_p = 980 \quad kips$$

Note: Pu is the combination of Elastic Axial force from the earthquake and dead load.

INPUT $P_u := 1725000 \ \text{lb}$

Article 8.6: Shear Demand and Capacity for Ductile Concrete Members

Note: It is recommended to use the plastic hinging forces whenever practical.

$V_u := V_p \qquad \phi_s := 0.9$

Article 4.11.6: **Analytical Plastic Hinge Length**

Note: For reinforced concrete columns framing into a footing, an integral bent cap, an oversized shaft, or cased shaft.

INPUT $d_{bl} := 1.41 \ \text{in}$ d_{bl}: Diameter of Longitudinal Bar

Eq. 4.11.6-1

$$\text{PlasticHinge}(\text{Fixity}, \text{fye}, d_{bl}) := \left| \begin{array}{l} lp \leftarrow 0.08 \cdot \text{Fixity} + 0.15 \cdot \frac{\text{fye}}{1000} \cdot d_{bl} \\ m \leftarrow 0.03 \cdot \frac{\text{fye}}{1000} \cdot d_{bl} \\ a \leftarrow lp \ \text{ if } \ lp \geq m \\ a \leftarrow m \ \text{ if } \ lp < m \\ a \end{array} \right.$$

$L_p := \text{PlasticHinge}(\text{Fixity}, \text{fye}, d_{bl}) = 36.69 \ \text{in}$

***Article 4.11.7*:** **Reinforced Concrete Column Plastic Hinge Region**

Note: y is the region of column with a moment demand exceeding 75% of the maximum plastic moment. From the SAP model, find the location at which the moment demand is 0.75*Mp. The 0.75*Mp value should be divided by p_{eTran} to take into account the model loads have not been multiplied by p_{eTran}. The location will also need to be **INPUT** into the PlasticHingeRegion program in inches.

$$Mp75 := 0.75 \cdot M_p = 5.677 \times 10^7 \quad lb \cdot in$$

$$PlasticHingeRegion(L_p, Columndia) := \left| \begin{array}{l} z \leftarrow 1.5 \cdot Columndia \\ x \leftarrow L_p \\ y \leftarrow 0 \\ a \leftarrow \max(z, x, y) \end{array} \right.$$

Note: Input the arguments into the program. Most likely the column diameter is the only variable that has changed.

INPUT $Lpr := PlasticHingeRegion(L_p, Columndia1) = 90 \quad in$

***Article 8.6.2*:** **Concrete Shear Capacity**

$$Ag := Acolumn1$$

Eq. 8.6.2-2 $Ae := 0.8 \cdot Ag = 2.262 \times 10^3 \quad in^2$

$\mu_D := 2$ Specified in Article 8.6.2 of Guide Spec.

INPUT	$s := 6$	in	s: Spacing of hoops or pitch of spiral (in)
INPUT	$Asp := .44$	in^2	Asp: Area of spiral or hoop reinforcing (in^2)
INPUT	$Dsp := 0.75$	in	Dsp: Diameter of spiral or hoop reinforcing (in)
INPUT	$Cover := 3$	in	Cover: Concrete cover for the Column (in)
INPUT	$Dprime := 54$	in	Dprime: Diameter of spiral or hoop for circular columns (in)

Eq. 8.6.2-7 $\rho_s := \frac{4 \cdot Asp}{s \cdot Dprime} = 5.432 \times 10^{-3}$

$fyh := \frac{fye}{1000} = 60$ ksi

Eq. 8.6.2-6 $StressCheck(\rho_s, fyh) :=$ | $fs \leftarrow \rho_s \cdot fyh$
| $a \leftarrow fs \text{ if } fs \le 0.35$

$fs := StressCheck(\rho_s, fyh) = 0.326$

Eq. 8.6.2-5 $\alpha program(fs, \mu_D) :=$ | $\alpha prime \leftarrow \frac{fs}{0.15} + 3.67 - \mu_D$
| $a \leftarrow 0.3 \text{ if } \alpha prime \le 0.3$
| $a \leftarrow \alpha prime \text{ if } \alpha prime > 0.3 \wedge \alpha prime < 3$
| $a \leftarrow 3 \text{ if } \alpha prime \ge 3$
| a

$\alpha Prime := \alpha program(fs, \mu_D) = 3$

If Pu is Compressive

Eq. 8.6.2-3 $vcprogram(\alpha Prime, fc, P_u, Ag) :=$ | $vc \leftarrow 0.032 \cdot \alpha Prime \cdot \left(1 + \frac{P_u}{2Ag \cdot 1000}\right) \cdot \sqrt{\frac{fc}{1000}}$
| $min1 \leftarrow 0.11 \sqrt{\frac{fc}{1000}}$
| $min2 \leftarrow 0.047 \alpha Prime \cdot \sqrt{\frac{fc}{1000}}$
| $minimum \leftarrow \min(min1, min2)$
| $a \leftarrow vc \text{ if } vc \le minimum$
| $a \leftarrow minimum \text{ if } vc > minimum$
| a

Eq. 8.6.2-4

If Pu is **NOT** Compressive VC $= 0$

Note: If Pu is not compressive, will have to manually input 0 for vc. Just input it below the vc:=vcprogram and the variable will assume the new value.

$vc := vcprogram(\alpha Prime, fc, P_u, Ag) = 0.22$ ksi

$Vc := vc \cdot Ae = 497.628$ kips

***Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity**

INPUT $n := 2$ n: number of individual interlocking spiral or hoop core sections

Eq. 8.6.3-1
Eq. 8.6.4-1

$$\text{vsprogram}(n, Asp, fyh, Dprime, s, fc, Ae) := \left| \begin{array}{l} vs \leftarrow \frac{\pi}{2}\cdot\left(\frac{n\,Asp\cdot fyh\cdot Dprime}{s}\right) \\ maxvs \leftarrow 0.25\cdot\sqrt{\frac{fc}{1000}}\cdot Ae \\ a \leftarrow vs \;\; \text{if } vs \le maxvs \\ a \leftarrow maxvs \;\; \text{if } vs > maxvs \\ a \end{array}\right.$$

$$Vs := \text{vsprogram}(n, Asp, fyh, Dprime, s, fc, Ae) = 746.442 \quad \text{kips}$$

Eq. 8.6.1-2 $$\phi Vn := \phi_s\cdot(Vs + Vc) = 1.12\times 10^3 \text{ kips}$$

$$\text{ShearCheck}(\phi Vn, V_u) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \;\; \text{if } \phi Vn \ge V_u \\ a \leftarrow \text{"FAILURE"} \;\; \text{if } \phi Vn < V_u \\ a \end{array}\right.$$

$$\text{Shearcheck} := \text{ShearCheck}(\phi Vn, V_u) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

***Article 8.6.5*: Minimum Shear Reinforcement**

For Circular Columns

Eq. 8.6.5-1 $$\text{mintranprogram}(\rho_s) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \;\; \text{if } \rho_s \ge 0.003 \\ a \leftarrow \text{"Increase Shear Reinforcing Ratio"} \;\; \text{if } \rho_s < 0.003 \\ a \end{array}\right.$$

$$\text{CheckTransverse} := \text{mintranprogram}(\rho_s) = \text{"OK"}$$

Note: If the minimum shear reinforcement program responses "Increase Shear Reinforcing Ratio", it is recommended to decrease the spacing (s) or increase the area of the shear reinforcement (Asp) in the inputs.

Article 8.8: **Longitudinal and Lateral Reinforcement Requirements**

Article 8.8.1: **Maximum Longitudinal Reinforcement**

INPUT $A_{bl} := 1.56 \quad in^2$

INPUT $NumberBars := 24$

$A_{long} := NumberBars \cdot A_{bl} = 37.44 \quad in^2$

Eq. 8.8.1-1
$$\rho program(A_{long}, Ag) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } A_{long} \le 0.04 \cdot Ag \\ a \leftarrow \text{"Section Over Reinforced"} \quad \text{if } A_{long} > 0.04 \cdot Ag \\ a \end{array} \right.$$

$$ReinforcementRaitoCheck := \rho program(A_{long}, Ag) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

Article 8.8.2: **Minimum Longitudinal Reinforcement**

For Columns in SDC B and C:

Eq. 8.8.2-1
$$minAlprogram(A_l, Ag) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } A_{long} \ge 0.007 \cdot Ag \\ a \leftarrow \text{"Increase Longitudinal Reinforcing"} \quad \text{if } A_{long} < 0.007 \cdot Ag \\ a \end{array} \right.$$

$$MinimumA_l := minAlprogram(A_{long}, Ag) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Article 8.8.9: **Requirements for Lateral Reinforcement for SDCs B,C, and D**

These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension of **NOT** less than **6*d_b** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Minimum Size of Lateral Reinforcement

#4 bars for #9 or smaller longitudinal bars
#5 bars for #10 or larger longitudinal bars
#5 bars for bundled longitudinal bars

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

Shall Not Exceed the Smallest of:

$$\text{Spacingprogram}(\text{Columndia}, d_{bl}) := \left| \begin{array}{l} q \leftarrow \left(\frac{1}{5}\right)\text{Columndia} \\ r \leftarrow 6 \cdot d_{bl} \\ t \leftarrow 6 \\ a \leftarrow \min(q, r, t) \\ a \end{array} \right.$$

$$\text{MaximumSpacing} := \text{Spacingprogram}(\text{Columndia1}, d_{bl}) = 6 \quad \text{in}$$

$$\text{SpacingCheck}(\text{MaximumSpacing}, s) := \left| \begin{array}{l} a \leftarrow s \ \text{ if } \ s \le \text{MaximumSpacing} \\ a \leftarrow \text{MaximumSpacing} \ \text{ if } \ s > \text{MaximumSpacing} \\ a \end{array} \right.$$

$$FINALSPACING := SpacingCheck(MaximumSpacing, s) = 6 \quad in$$

$$scheck := ShearCheck(MaximumSpacing, s) = \text{"OK"}$$

Note: If scheck returns "Failure", increase the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.3 (LRFD SPEC.): Column Connections

Note: This needs to be done whenever the column dimension changes. The spacing in the hinge region shall continue into the drilled shaft or cap beam the Extension length.

$$ExtensionProgram(d) := \left| \begin{array}{l} z \leftarrow 15 \\ x \leftarrow \frac{1}{2} \cdot d \\ a \leftarrow \max(z, x) \\ a \end{array} \right.$$

INPUT $Extension := ExtensionProgram(Columndia1) = 30 \quad in$

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.
Refer to the **AASHTO LRFD Bridge Design Specifications**.

5.8.3.3 **Nominal Shear Resistance**

$V_p = 490$ kips

INPUT $spaceNOhinge := 12 \quad in$

INPUT $bv := Columndia1$

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad rad$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 55.545 \quad in$$

Eq. C5.8.2.9-2
$$de := \frac{bv}{2} + \frac{Dr}{\pi} = 47.681 \quad in$$

$$dv := 0.9 \cdot de = 42.912 \quad in$$

Eq. 5.8.3.3-3
$$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 325.448 \quad kips$$

Eq. 5.8.3.3-4
$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 188.815 \quad kips$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 462.837 \quad kips$$

$$ShearCheck\left(\phi Vn, V_u\right) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } \phi Vn \geq V_u \\ a \leftarrow \text{"FAILURE"} \quad \text{if } \phi Vn < V_u \\ a \end{array} \right.$$

$$Shearcheck := ShearCheck\left(\phi Vn, V_p\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1
$$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.758 \quad in^2$$

$$Av := 2 \cdot Asp = 0.88 \quad in^2$$

$$\text{TranCheck}(\text{Avmin}, \text{Av}) := \left| \begin{array}{l} a \leftarrow \text{"Decrease Spacing or Increase Bar Size"} \quad \text{if Avmin} > \text{Av} \\ a \leftarrow \text{"OK"} \quad \text{if Avmin} \le \text{Av} \\ a \end{array} \right.$$

$$\text{MinimumTran} := \text{TranCheck}(\text{Avmin}, \text{Av}) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

***5.8.2.7* Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1

$$\text{vu} := \frac{V_u}{\phi_s \cdot \text{bv} \cdot \text{dv}} = 0.211 \qquad \text{ksi}$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2

$$\text{spacingProgram}(\text{Vu}, \text{dv}, \text{fc}) := \left| \begin{array}{l} v \leftarrow 0.125 \cdot \frac{\text{fc}}{1000} \\ q \leftarrow 0.8 \cdot \text{dv} \\ r \leftarrow 0.4 \cdot \text{dv} \\ z \leftarrow q \quad \text{if } q \le 24 \\ z \leftarrow 24 \quad \text{if } q > 24 \\ t \leftarrow r \quad \text{if } r \le 12 \\ t \leftarrow 12 \quad \text{if } r > 12 \\ a \leftarrow z \quad \text{if Vu} < v \\ a \leftarrow t \quad \text{if Vu} \ge v \\ a \end{array} \right.$$

$$\text{MaxSpacing} := \text{spacingProgram}(\text{vu}, \text{dv}, \text{fc}) = 24 \qquad \text{in}$$

$$\text{Spacecheck}(\text{MaxSpacing}, s) := \left| \begin{array}{l} a \leftarrow s \quad \text{if } s \le \text{MaxSpacing} \\ a \leftarrow \text{MaxSpacing} \quad \text{if } s > \text{MaxSpacing} \\ a \end{array} \right.$$

$$\text{MAXSPACING} := \text{Spacecheck}(\text{MaxSpacing}, \text{spaceNOhinge}) = 12 \qquad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MAXSPACING}, \text{spaceNOhinge}) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

Strut Design for Bent 2

Note: The struts are designed for the linear elastic forces. The loads need to be converted to design loads. This can be done by simply multiplying the SAP load by pe/po.

$V_p := 325$ kips

$M_p := 4280$ kip·ft

$P_u := 226$ kips

Article 4.11.6: **Analytical Plastic Hinge Length**

Note: For reinforced concrete columns framing into a footing, an integral bent cap, an oversized shaft, or cased shaft.

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of Longitudinal Bar

$L_p := \text{PlasticHinge}(\text{Lstrut2}, \text{fye}, d_{bl}) = 14.21$ in

Article 4.11.7: **Reinforced Concrete Column Plastic Hinge Region**

Note: y is the region of column with a moment demand exceeding 75% of the maximum plastic moment. From the SAP model, find the location at which the moment demand is 0.75*Mp. The 0.75*Mp value should be divided by p_{eTran} to take into account the model loads have not been multiplied by p_{eTran}. The location will also need to be **INPUT** into the PlasticHingeRegion program in inches.

$Mp75 := 0.75 \cdot M_p = 3.21 \times 10^3$ lb·in

$$\text{PlasticHingeRegion}(L_p, \text{Columndia}) := \left| \begin{array}{l} z \leftarrow 1.5 \cdot \text{Columndia} \\ x \leftarrow L_p \\ y \leftarrow 0 \\ a \leftarrow \max(z, x, y) \end{array} \right.$$

Note: Input the arguments into the program. Most likely the column diameter is the only variable that has changed.

INPUT $Lpr := \text{PlasticHingeRegion}(L_p, \text{Strut2Depth}) = 108$ in

***Article 8.6.2*: Concrete Shear Capacity**

$Ag := Strut2Depth \cdot Strut2Width$

Eq. 8.6.2-2 $Ae := 0.8 \cdot Ag = 2.419 \times 10^{3}$ in^{2}

$\mu_D := 2$ Specified in Article 6.8.2 Guide Spec.

INPUT $s := 4$ in — s: Spacing of hoops or pitch of spiral (in)

INPUT $Asp := 0.31$ in^{2} — Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.625$ in — Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 2$ in — Cover: Concrete cover for the Column (in)

INPUT $b := Strut2Depth$ in — b: Depth of the Strut (in)

INPUT $d := 70$ in — d: Effective Depth (in)

$Av := 2 \cdot Asp = 0.62$ in^{2}

Eq. 8.6.2-10 $\rho_w := \frac{Av}{s \cdot b} = 2.153 \times 10^{-3}$

$fyh := \frac{fye}{1000} = 60$ ksi

Eq. 8.6.2-9 $StressCheckRect(\rho_w, fyh) := \left| \begin{array}{l} fs \leftarrow 2 \cdot \rho_w \cdot fyh \\ a \leftarrow fs \ \text{ if } \ fs \le 0.35 \end{array} \right.$

$fw := StressCheckRect(\rho_w, fyh) = 0.258$

Eq. 8.6.2-8 $\alpha Prime := \alpha program(fw, \mu_D) = 3$

If Pu is Compressive

Eq. 8.6.2-4

If Pu is **NOT** Compressive $vc = 0$

Note: If Pu is not compressive, will have to manually input 0 for vc. Just input it below the vc:=vcprogram and the variable will assume the new value.

$$vc := vcprogram\left(\alpha Prime, fc, P_u, Ag\right) = 0.192 \quad ksi$$

$$Vc := vc \cdot Ag = 580.63 \qquad kips$$

***Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity**

Eq. 8.6.3-2
Eq. 8.6.4-1

$$vsprogramRect(Av, fyh, d, s, fc, Ae) := \left| \begin{array}{l} vs \leftarrow \dfrac{Av \cdot fyh \cdot d}{s} \\ maxvs \leftarrow 0.25 \cdot \sqrt{\dfrac{fc}{1000}} \cdot Ae \\ a \leftarrow vs \quad \text{if } vs \le maxvs \\ a \leftarrow maxvs \quad \text{if } vs > maxvs \\ a \end{array} \right.$$

$$Vs := vsprogramRect(Av, fyh, d, s, fc, Ae) = 651 \qquad kips$$

Eq. 8.6.1-2

$$\phi Vn := \phi_s \cdot (Vs + Vc) = 1.108 \times 10^3 \, kips$$

$$Shearcheck := ShearCheck\left(\phi Vn, V_u\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

Article 8.6.5: **Minimum Shear Reinforcement**

For Rectangular Shapes

$$\text{mintranprogramRect}(\rho_s) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } \rho_s \geq 0.002 \\ a \leftarrow \text{"Increase Shear Reinforcing Ratio"} \quad \text{if } \rho_s < 0.002 \\ a \end{array} \right.$$

Eq. 8.6.5-2 $\quad \text{CheckTransverse} := \text{mintranprogramRect}(\rho_w) = \text{"OK"}$

Note: If the minimum shear reinforcement program responses "Increase Shear Reinforcing Ratio", it is recommended to decrease the spacing (s) or increase the area of the shear reinforcement (Asp) in the inputs.

Article 8.8: **Longitudinal and Lateral Reinforcement Requirements**

Article 8.8.1: **Maximum Longitudinal Reinforcement**

INPUT $\quad A6_{bl} := 0.31 \quad in^2$

INPUT $\quad \text{Number\_5\_Bars} := 20$

INPUT $\quad A11_{bl} := 1.56 \quad in^2$

INPUT $\quad \text{Number\_11\_Bars} := 16$

$$A_{long} := A6_{bl} \cdot \text{Number\_5\_Bars} + A11_{bl} \cdot \text{Number\_11\_Bars} = 31.16 \quad in^2$$

Eq. 8.8.1-1 $\quad \text{ReinforcementRaitoCheck} := \rho\text{program}(A_{long}, Ag) = \text{"OK"}$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

Article 8.8.2: **Minimum Longitudinal Reinforcement**

For Columns in SDC B and C:

Eq. 8.8.2-1 $\quad \text{MinimumA}_l := \text{minAlprogram}(A_{long}, Ag) = \text{"OK"}$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars) in the inputs.

***Article 8.8.9*: Requirements for Lateral Reinforcement for SDCs B,C, and D**

These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension **of NOT** less than **6*d_b** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed **tie** or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Minimum Size of Lateral Reinforcement

#4 bars for #9 or smaller longitudinal bars
#5 bars for #10 or larger longitudinal bars
#5 bars for bundled longitudinal bars

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

Shall Not Exceed the Smallest of:

$$\text{MaximumSpacing} := \text{Spacingprogram}\left(\text{Columndia2}, d_{bl}\right) = 6 \quad \text{in}$$

$$\text{FINALSPACING} := \text{SpacingCheck}(\text{MaximumSpacing}, s) = 4 \quad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MaximumSpacing}, s) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.
Refer to the **AASHTO LRFD Bridge Design Specifications**.

***5.8.3.3* Nominal Shear Resistance**

$V_p = 325$ kips

INPUT $spaceNOhinge := 12$ in

INPUT $bv := Strut2Width$

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$\beta := 2.0$

$\theta := \frac{\pi}{180} \cdot 45 = 0.785$ rad

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

Eq. 5.8.2.9-2 $de := 69.4$ in — de = ds which is the distance from top of the member to the centriod of the tensile fiber

$dvpreliminary := 66.75$ in — dvpreliminary = distance between compressive and tensile reinforcing

$$dvprogram(de, dv, h) := \left| \begin{array}{l} x \leftarrow 0.9 \cdot de \\ y \leftarrow 0.75 \cdot h \\ z \leftarrow \max(x, y) \\ a \leftarrow dv \ \text{ if } \ dv \geq z \\ a \leftarrow z \ \text{ if } \ dv < z \\ a \end{array} \right.$$

$dv := dvprogram(de, dvpreliminary, Strut2Depth) = 66.75$ in

Eq. 5.8.3.3-3 $V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 354.362$ kips

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp\cdot\frac{fye}{1000}dv\cdot\cot(\theta)}{spaceNOhinge} = 206.925 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right)\cdot\phi_s = 505.159 \quad \text{kips}$$

$$Shearcheck := ShearCheck\left(\phi Vn, V_p\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316\cdot\sqrt{\frac{fc}{1000}}\cdot\frac{bv\cdot spaceNOhinge}{\frac{fye}{1000}} = 0.531 \quad in^2$$

$$Av := 2\cdot Asp = 0.62 \quad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{V_p}{\phi_s\cdot bv\cdot dv} = 0.129 \quad \text{ksi}$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \quad \text{in}$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \quad \text{in}$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

BENT 3 DESIGN

***Article 4.11.1-4*: Steps to find Moment Capacity, Shear Capacity, and Axial Force**

Note: Use some kind of software to find the Moment Capacity of the Column. PCA Column was used to create an Interaction Diagram and to calculate the Moment Capacity. The shear for the bent was found by knowing the Moment.

INPUT $M_p := 132528000 \quad kip \cdot ft$

INPUT $Fixity := 660\,in$

Note: Fixity is the point of fixity for the column/drilledshaft.

$$V_p := \frac{2 \cdot M_p}{Fixity \cdot 1000} = 401.6 \quad kips \qquad V_{pBent3} := 2 \cdot V_p = 803.2 \quad kips$$

Note: If the decision is made to design for **ELASTIC FORCES** then the above variables need to be override. This can be done by simply changing the Vp variable to the elastic force from SAP2000 that has been multiplied by pe/po.

$$V_p := 262 \quad kips \qquad V_{pBent3} := 2 \cdot V_p = 524 \quad kips$$

Note: Pu is the combination of Elastic Axial force from the earthquake and dead load.

INPUT $P_u := 2095000 \quad lb$

Article 8.6: Shear Demand and Capacity for Ductile Concrete Members

Note: It is recommended to use the plastic hinging forces whenever practical.

$$V_u := V_p \qquad \phi_s := 0.9$$

***Article 4.11.6*: Analytical Plastic Hinge Length**

Note: For reinforced concrete columns framing into a footing, an integral bent cap, an oversized shaft, or cased shaft.

INPUT $d_{bl} := 1.41 \quad in$ $\qquad d_{bl}$: Diameter of Longitudinal Bar

$$L_p := PlasticHinge\left(Fixity, fye, d_{bl}\right) = 65.49 \quad in$$

***Article 4.11.7*: Reinforced Concrete Column Plastic Hinge Region**

Note: y is the region of column with a moment demand exceeding 75% of the maximum plastic moment. From the SAP model, find the location at which the moment demand is 0.75*Mp. The 0.75*Mp value should be divided by p_{eTran} to take into account the model loads have not been multiplied by p_{eTran}. The location will also need to be **INPUT** into the PlasticHingeRegion program in inches.

$$Mp75 := 0.75 \cdot M_p = 9.94 \times 10^7 \qquad \text{lb} \cdot \text{in}$$

$$\text{PlasticHingeRegion}\left(L_p, \text{Columndia}\right) := \left| \begin{array}{l} z \leftarrow 1.5 \cdot \text{Columndia} \\ x \leftarrow L_p \\ y \leftarrow 0 \\ a \leftarrow \max(z, x, y) \end{array} \right.$$

Note: Input the arguments into the program. Most likely the column diameter is the only variable that has changed.

INPUT $\quad Lpr := \text{PlasticHingeRegion}\left(L_p, \text{Columndia2}\right) = 108 \qquad \text{in}$

***Article 8.6.2*: Concrete Shear Capacity**

$$Ag := \text{Acolumn2}$$

Eq. 8.6.2-2 $\quad Ae := 0.8 \cdot Ag = 3.257 \times 10^3 \qquad \text{in}^2$

$\mu_D := 2$ Specified in Article 6.8.2 Guide Spec.

INPUT $\quad s := 6 \quad \text{in}$ s: Spacing of hoops or pitch of spiral (in)

INPUT $\quad Asp := 0.44 \quad \text{in}^2$ Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $\quad Dsp := 0.75 \quad \text{in}$ Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $\quad Cover := 3 \quad \text{in}$ Cover: Concrete cover for the Column (in)

INPUT $\quad Dprime := 66 \quad \text{in}$ Dprime: Diameter of spiral or hoop for circular columns (in)

Eq. 8.6.2-7 $\rho_s := \frac{4 \cdot Asp}{s \cdot Dprime} = 4.444 \times 10^{-3}$

$fyh := \frac{fye}{1000} = 60$ ksi

Eq. 8.6.2-6 $fs := StressCheck(\rho_s, fyh) = 0.267$

Eq. 8.6.2-5 $\alpha Prime := \alpha program(fs, \mu_D) = 3$

If Pu is Compressive

Eq. 8.6.2-4

If Pu is **NOT** Compressive $vc = 0$

Note: If Pu is not compressive, will have to manually input 0 for vc. Just input it below the vc:=vcprogram and the variable will assume the new value.

$vc := vcprogram(\alpha Prime, fc, P_u, Ag) = 0.22$ ksi

$Vc := vc \cdot Ag = 895.731$ kips

***Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity**

INPUT $n := 2$ n: number of individual interlocking spiral or hoop core sections

Eq. 8.6.3-1
Eq. 8.6.4-1 $Vs := vsprogram(n, Asp, fyh, Dprime, s, fc, Ae) = 912.319$ kips

Eq. 8.6.1-2 $\phi Vn := \phi_s \cdot (Vs + Vc) = 1.627 \times 10$ kips

$Shearcheck := ShearCheck(\phi Vn, V_u) = \text{"OK"}$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

***Article 8.6.5*: Minimum Shear Reinforcement**

For Circular Columns

Eq. 8.6.5-1 $\quad \mathrm{CheckTransverse} := \mathrm{mintranprogram}(\rho_s) = \text{"OK"}$

Note: If the minimum shear reinforcement program responses "Increase Shear Reinforcing Ratio", it is recommended to <u>decrease the spacing (s)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

***Article 8.8*: Longitudinal and Lateral Reinforcement Requirements**

***Article 8.8.1*: Maximum Longitudinal Reinforcement**

<u>INPUT</u> $\quad A_{bl} := 1.56 \quad in^2$

<u>INPUT</u> $\quad \mathrm{NumberBars} := 32$

$A_{long} := \mathrm{NumberBars} \cdot A_{bl} = 49.92 \quad in^2$

Eq. 8.8.1-1 $\quad \mathrm{ReinforcementRaitoCheck} := \rho\mathrm{program}(A_{long}, Ag) = \text{"OK"}$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either <u>increase the section size (Ag)</u> or <u>decrease the longitudinal reinforcing (Abl and NumberBars)</u> in the inputs.

***Article 8.8.2*: Minimum Longitudinal Reinforcement**

For Columns in SDC B and C:

Eq. 8.8.2-1 $\quad \mathrm{MinimumA}_l := \mathrm{minAlprogram}(A_{long}, Ag) = \text{"OK"}$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars)</u> in the inputs.

***Article 8.8.9*: Requirements for Lateral Reinforcement for SDCs B,C, and D**

These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension **of NOT** less than **6*d_b** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Minimum Size of Lateral Reinforcement

#4 bars for #9 or smaller longitudinal bars
#5 bars for #10 or larger longitudinal bars
#5 bars for bundled longitudinal bars

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

Shall Not Exceed the Smallest of:

$$\text{MaximumSpacing} := \text{Spacingprogram}\left(\text{Columndia2}, d_{bl}\right) = 6 \quad \text{in}$$

$$\text{FINALSPACING} := \text{SpacingCheck}(\text{MaximumSpacing}, s) = 6 \quad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MaximumSpacing}, s) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.3 (LRFD SPEC.): Column Connections

Note: This needs to be done whenever the column dimension changes. The spacing in the hinge region shall continue into the drilled shaft or cap beam the Extension length.

INPUT $\text{Extension} := \text{ExtensionProgram}(\text{Columndia2}) = 36 \quad \text{in}$

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.
Refer to the **AASHTO LRFD Bridge Design Specifications**.

5.8.3.3 **Nominal Shear Resistance**

$V_p = 262$ kips

INPUT $spaceNOhinge := 6$ in

INPUT $bv := Columndia2$

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$\beta := 2.0$

$\theta := \frac{\pi}{180} \cdot 45 = 0.785$ rad

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 67.545 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 57.5 \quad \text{in}$$

$$dv := 0.9 \cdot de = 51.75 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 470.968 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 455.402 \quad \text{kips}$$

$$\phi V_n := (V_c + V_s)\cdot\phi_s = 833.733 \quad \text{kips}$$

$$\text{Shearcheck} := \text{ShearCheck}(\phi Vn, V_p) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$\text{Avmin} := 0.0316\cdot\sqrt{\frac{fc}{1000}}\cdot\frac{bv\cdot\text{spaceNOhinge}}{\frac{fye}{1000}} = 0.455 \quad \text{in}^2$$

$$Av := 2\cdot Asp = 0.88 \quad \text{in}^2$$

$$\text{MinimumTran} := \text{TranCheck}(\text{Avmin}, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{V_u}{\phi_s\cdot bv\cdot dv} = 0.078 \quad \text{ksi}$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $$\text{MaxSpacing} := \text{spacingProgram}(vu, dv, fc) = 24 \quad \text{in}$$

$$\text{MAXSPACING} := \text{Spacecheck}(\text{MaxSpacing}, \text{spaceNOhinge}) = 6 \quad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MAXSPACING}, \text{spaceNOhinge}) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

Strut Design for Bent 3

Note: The struts are designed for the linear elastic forces. The loads need to be converted to design loads. This can be done by simply multiplying the SAP load by pe/po.

$V_p := 545$ kips

$M_p := 6840$ kip·ft

$P_u := 51$ kips

Article 4.11.6: **Analytical Plastic Hinge Length**

Note: For reinforced concrete columns framing into a footing, an integral bent cap, an oversized shaft, or cased shaft.

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of Longitudinal Bar

$L_p := \mathrm{PlasticHinge}(\mathrm{Lstrut3}, \mathrm{fye}, d_{bl}) = 14.13$ in

Article 4.11.7: **Reinforced Concrete Column Plastic Hinge Region**

Note: y is the region of column with a moment demand exceeding 75% of the maximum plastic moment. From the SAP model, find the location at which the moment demand is 0.75*Mp. The 0.75*Mp value should be divided by p_{eTran} to take into account the model loads have not been multiplied by p_{eTran}. The location will also need to be **INPUT** into the PlasticHingeRegion program in inches.

$Mp75 := 0.75 \cdot M_p = 5.13 \times 10^3$ lb·in

$$\mathrm{PlasticHingeRegion}(L_p, \mathrm{Columndia}) := \left| \begin{array}{l} z \leftarrow 1.5 \cdot \mathrm{Columndia} \\ x \leftarrow L_p \\ y \leftarrow 0 \\ a \leftarrow \max(z, x, y) \end{array} \right.$$

Note: Input the arguments into the program. Most likely the column diameter is the only variable that has changed.

INPUT $Lpr := \mathrm{PlasticHingeRegion}(L_p, \mathrm{Strut3Depth}) = 180$ in

Article 8.6.2: **Concrete Shear Capacity**

$Ag := Strut3Depth \cdot Strut3Width$

Eq. 8.6.2-2 $Ae := 0.8 \cdot Ag = 4.032 \times 10^{3}$ in^{2}

$\mu_D := 2$ Specified in Article 6.8.2 Guide Spec.

INPUT $s := 3.5$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $Asp := 0.44$ in^{2} Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.75$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 2$ in Cover: Concrete cover for the Column (in)

INPUT $b := Strut3Depth$ in b: Depth of the Strut (in)

INPUT $d := 116$ in d: Effective Depth (in)

$Av := 2 \cdot Asp = 0.88$ in^{2}

Eq. 8.6.2-10 $\rho_w := \frac{Av}{s \cdot b} = 2.095 \times 10^{-3}$

$fyh := \frac{fye}{1000} = 60$ ksi

Eq. 8.6.2-9 $fw := StressCheckRect(\rho_w, fyh) = 0.251$

Eq. 8.6.2-8 $\alpha Prime := \alpha program(fw, \mu_D) = 3$

If Pu is Compressive

Eq. 8.6.2-4

If Pu is **NOT** Compressive $vc = 0$

Note: If Pu is not compressive, will have to manually input 0 for vc. Just input it below the vc:=vcprogram and the variable will assume the new value.

$vc := vcprogram(\alpha Prime, fc, P_u, Ag) = 0.192$ ksi

$Vc := vc \cdot Ag = 967.685$ kips

***Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity**

Eq. 8.6.3-2
Eq. 8.6.4-1

$$Vs := \text{vsprogramRect}(Av, fyh, d, s, fc, Ae) = 1.75 \times \ \ \text{kips}$$

Eq. 8.6.1-2

$$\phi Vn := \phi_s \cdot (Vs + Vc) = 2.446 \times 10\ \text{kips}$$

$$\text{Shearcheck} := \text{ShearCheck}(\phi Vn, V_u) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

***Article 8.6.5*: Minimum Shear Reinforcement**

Eq. 8.6.5-2 $\quad \text{CheckTransverse} := \text{mintranprogramRect}(\rho_w) = \text{"OK"}$

Note: If the minimum shear reinforcement program responses "Increase Shear Reinforcing Ratio", it is recommended to decrease the spacing (s) or increase the area of the shear reinforcement (Asp) in the inputs.

***Article 8.8*: Longitudinal and Lateral Reinforcement Requirements**

***Article 8.8.1*: Maximum Longitudinal Reinforcement**

INPUT $\quad A5_{bl} := 0.31 \quad in^2$

INPUT $\quad \text{Number\_5\_Bars} := 36$

INPUT $\quad A11_{bl} := 1.56 \quad in^2$

INPUT $\quad \text{Number\_11\_Bars} := 16$

$$A_{long} := A5_{bl} \cdot \text{Number\_5\_Bars} + A11_{bl} \cdot \text{Number\_11\_Bars} = 36.12 \qquad in^2$$

Eq. 8.8.1-1 $\quad \text{ReinforcementRaitoCheck} := \rho\text{program}(A_{long}, Ag) = \text{"OK"}$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

***Article 8.8.2*: Minimum Longitudinal Reinforcement**

For Columns in SDC B and C:

Eq. 8.8.2-1 $\mathrm{MinimumA_l := minAlprogram(A_{long}, Ag) = "OK"}$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars) in the inputs.

***Article 8.8.9*: Requirements for Lateral Reinforcement for SDCs B,C, and D**

These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*$\mathbf{d_b}$** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension **of NOT** less than **6*$\mathbf{d_b}$** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*$\mathbf{d_b}$** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*$\mathbf{d_b}$** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Minimum Size of Lateral Reinforcement

#4 bars for #9 or smaller longitudinal bars
#5 bars for #10 or larger longitudinal bars
#5 bars for bundled longitudinal bars

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

Shall Not Exceed the Smallest of:

$$\mathrm{MaximumSpacing := Spacingprogram(Columndia2, d_{bl}) = 6 \quad in}$$

$$\mathrm{FINALSPACING := SpacingCheck(MaximumSpacing, s) = 3.5 \quad in}$$

$$\mathrm{scheck := ShearCheck(MaximumSpacing, s) = "OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.
Refer to the **AASHTO LRFD Bridge Design Specifications**.

5.8.3.3 **Nominal Shear Resistance**

$V_p = 545$ kips

INPUT $spaceNOhinge := 12$ in

INPUT $bv := Strut3Width$

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$\beta := 2.0$

$\theta := \frac{\pi}{180} \cdot 45 = 0.785$ rad

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

Eq. 5.8.2.9-2 $de := 117$ in de = ds which is the distance from top of the member to the centriod of the tensile fiber

$dvpreliminary := 114$ in dvpreliminary = distance between compressive and tensile reinforcing

$dv := dvprogram(de, dvpreliminary, Strut3Depth) = 114$ in

Eq. 5.8.3.3-3 $V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 605.203$ kips

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 501.6 \qquad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 996.123 \qquad \text{kips}$$

$$Shearcheck := ShearCheck\left(\phi Vn, V_p\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.531 \qquad in^2$$

$$Av := 2 \cdot Asp = 0.88 \qquad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

***5.8.2.7* Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{V_p}{\phi_s \cdot bv \cdot dv} = 0.126 \qquad \text{ksi}$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \qquad \text{in}$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \qquad \text{in}$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

BENT 4 DESIGN

***Article 4.11.1-4*: Steps to find Moment Capacity, Shear Capacity, and Axial Force**

Note: Use some kind of software to find the Moment Capacity of the Column. PCA Column was used to create an Interaction Diagram and to calculate the Moment Capacity. The shear for the bent was found by knowing the Moment.

INPUT $M_p := 75504000 \quad kip \cdot ft$

INPUT $Fixity := 300\,in$

Note: Fixity is the point of fixity for the column/drilledshaft.

$$V_p := \frac{2 \cdot M_p}{Fixity \cdot 1000} = 503.36 \quad kips \qquad V_{pBent4} := 2 \cdot V_p = 1.007 \times 10^{3}\,kips$$

Note: If the decision is made to design for **ELASTIC FORCES** then the above variables need to be override. This can be done by simply changing the Vp variable to the elastic force from SAP2000 that has been multiplied by pe/po.

$$V_p := 525 \quad kips \qquad V_{pBent4} := 2 \cdot V_p = 1.05 \times 10^{3} \quad kips$$

Note: Pu is the combination of Elastic Axial force from the earthquake and dead load.

INPUT $P_u := 991000 \quad lb$

Article 8.6: Shear Demand and Capacity for Ductile Concrete Members

Note: It is recommended to use the plastic hinging forces whenever practical.

$$V_u := V_p \qquad \phi_s := 0.9$$

***Article 4.11.6*: Analytical Plastic Hinge Length**

Note: For reinforced concrete columns framing into a footing, an integral bent cap, an oversized shaft, or cased shaft.

INPUT $d_{bl} := 1.41 \quad in$

d_{bl}: Diameter of Longitudinal Bar

$$L_p := PlasticHinge(Fixity, fye, d_{bl}) = 36.69 \quad in$$

***Article 4.11.7*: Reinforced Concrete Column Plastic Hinge Region**

Note: y is the region of column with a moment demand exceeding 75% of the maximum plastic moment. From the SAP model, find the location at which the moment demand is 0.75*Mp. The 0.75*Mp value should be divided by p_{eTran} to take into account the model loads have not been multiplied by p_{eTran}. The location will also need to be **INPUT** into the PlasticHingeRegion program in inches.

$$Mp75 := 0.75 \cdot M_p = 5.663 \times 10^7 \qquad lb \cdot in$$

$$PlasticHingeRegion(L_p, Columndia) := \left| \begin{array}{l} z \leftarrow 1.5 \cdot Columndia \\ x \leftarrow L_p \\ y \leftarrow 0 \\ a \leftarrow max(z, x, y) \end{array} \right.$$

Note: Input the arguments into the program. Most likely the column diameter is the only variable that has changed.

INPUT $\quad Lpr := PlasticHingeRegion(L_p, Columndia1) = 90 \qquad in$

***Article 8.6.2*: Concrete Shear Capacity**

$$Ag := Acolumn1$$

Eq. 8.6.2-2 $\quad Ae := 0.8 \cdot Ag = 2.262 \times 10^3 \qquad in^2$

$\mu_D := 2$ Specified in Article 6.8.2 Guide Spec.

INPUT $\quad s := 6 \quad in \quad$ s: Spacing of hoops or pitch of spiral (in)

INPUT $\quad Asp := 0.44 \quad in^2 \quad$ Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $\quad Dsp := 0.75 \quad in \quad$ Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $\quad Cover := 3 \quad in \quad$ Cover: Concrete cover for the Column (in)

INPUT $\quad Dprime := 54 \quad in \quad$ Dprime: Diameter of spiral or hoop for circular columns (in)

Eq. 8.6.2-7 $\rho_s := \dfrac{4 \cdot Asp}{s \cdot Dprime} = 5.432 \times 10^{-3}$

$fyh := \dfrac{fye}{1000} = 60$ ksi

Eq. 8.6.2-6 $fs := StressCheck(\rho_s, fyh) = 0.326$

Eq. 8.6.2-5 $\alpha Prime := \alpha program(fs, \mu_D) = 3$

If Pu is Compressive

Eq. 8.6.2-4

If Pu is **NOT** Compressive VC = 0

Note: If Pu is not compressive, will have to manually input 0 for vc. Just input it below the vc:=vcprogram and the variable will assume the new value.

$vc := vcprogram(\alpha Prime, fc, P_u, Ag) = 0.22$ ksi

$Vc := vc \cdot Ag = 622.035$ kips

***Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity**

INPUT $n := 2$ n: number of individual interlocking spiral or hoop core sections

Eq. 8.6.3-1
Eq. 8.6.4-1
$Vs := vsprogram(n, Asp, fyh, Dprime, s, fc, Ae) = 746.442$ kips

Eq. 8.6.1-2 $\phi Vn := \phi_s \cdot (Vs + Vc) = 1.232 \times 10$ kips

$Shearcheck := ShearCheck(\phi Vn, V_u) = "OK"$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

***Article 8.6.5*: Minimum Shear Reinforcement**

For Circular Columns

Eq. 8.6.5-1 $\quad \mathrm{CheckTransverse} := \mathrm{mintranprogram}(\rho_s) = \text{"OK"}$

Note: If the minimum shear reinforcement program responses "Increase Shear Reinforcing Ratio", it is recommended to <u>decrease the spacing (s)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

***Article 8.8*: Longitudinal and Lateral Reinforcement Requirements**

***Article 8.8.1*: Maximum Longitudinal Reinforcement**

<u>INPUT</u> $\quad A_{bl} := 1.56 \quad in^2$

<u>INPUT</u> $\quad \mathrm{NumberBars} := 24$

$A_{long} := \mathrm{NumberBars} \cdot A_{bl} = 37.44 \quad in^2$

Eq. 8.8.1-1 $\quad \mathrm{ReinforcementRaitoCheck} := \rho\mathrm{program}(A_{long}, Ag) = \text{"OK"}$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either <u>increase the section size (Ag)</u> or <u>decrease the longitudinal reinforcing (Abl and NumberBars)</u> in the inputs.

***Article 8.8.2*: Minimum Longitudinal Reinforcement**

For Columns in SDC B and C:

Eq. 8.8.2-1 $\quad \mathrm{MinimumA_l} := \mathrm{minAlprogram}(A_{long}, Ag) = \text{"OK"}$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars)</u> in the inputs.

***Article 8.8.9*: Requirements for Lateral Reinforcement for SDCs B,C, and D**

These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension **of NOT** less than **6*d_b** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Minimum Size of Lateral Reinforcement

#4 bars for #9 or smaller longitudinal bars
#5 bars for #10 or larger longitudinal bars
#5 bars for bundled longitudinal bars

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

Shall Not Exceed the Smallest of:

$$\text{MaximumSpacing} := \text{Spacingprogram}\left(\text{Columndia2}, d_{bl}\right) = 6 \quad \text{in}$$

$$\text{FINALSPACING} := \text{SpacingCheck}(\text{MaximumSpacing}, s) = 6 \quad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MaximumSpacing}, s) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.3 (LRFD SPEC.): Column Connections

Note: This needs to be done whenever the column dimension changes. The spacing in the hinge region shall continue into the drilled shaft or cap beam the Extension length.

INPUT $\text{Extension} := \text{ExtensionProgram}(\text{Columndia1}) = 30 \quad \text{in}$

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.
Refer to the **AASHTO LRFD Bridge Design Specifications**.

5.8.3.3 **Nominal Shear Resistance**

$V_p = 525$ kips

INPUT $\text{spaceNOhinge} := 12$ in

INPUT $\text{bv} := \text{Columndia1}$

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$\beta := 2.0$

$\theta := \frac{\pi}{180} \cdot 45 = 0.785$ rad

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$\text{Dr} := \text{bv} - \text{Cover} - \text{Dsp} - \frac{d_{bl}}{2} = 55.545 \quad \text{in}$$

Eq. C5.8.2.9-2 $$\text{de} := \frac{\text{bv}}{2} + \frac{\text{Dr}}{\pi} = 47.681 \quad \text{in}$$

$$\text{dv} := 0.9 \cdot \text{de} = 42.912 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{\text{fc}}{1000}} \text{bv} \cdot \text{dv} = 325.448 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2\text{Asp} \cdot \frac{\text{fye}}{1000} \text{dv} \cdot \cot(\theta)}{\text{spaceNOhinge}} = 188.815 \quad \text{kips}$$

$$\phi V_n := (V_c + V_s)\cdot\phi_s = 462.837 \qquad \text{kips}$$

$$\text{Shearcheck} := \text{ShearCheck}(\phi Vn, V_p) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1

$$\text{Avmin} := 0.0316\cdot\sqrt{\frac{fc}{1000}}\cdot\frac{bv\cdot\text{spaceNOhinge}}{\frac{fye}{1000}} = 0.758 \qquad \text{in}^2$$

$$\text{Av} := 2\cdot\text{Asp} = 0.88 \qquad \text{in}^2$$

$$\text{MinimumTran} := \text{TranCheck}(\text{Avmin}, \text{Av}) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

***5.8.2.7* Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1

$$vu := \frac{V_u}{\phi_s\cdot bv\cdot dv} = 0.227 \qquad \text{ksi}$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2

$$\text{MaxSpacing} := \text{spacingProgram}(vu, dv, fc) = 24 \qquad \text{in}$$

$$\text{MAXSPACING} := \text{Spacecheck}(\text{MaxSpacing}, \text{spaceNOhinge}) = 12 \qquad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MAXSPACING}, \text{spaceNOhinge}) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

Strut Design for Bent 4

Note: The struts are designed for the linear elastic forces. The loads need to be converted to design loads. This can be done by simply multiplying the SAP load by pe/po.

$V_p := 348$ kips

$M_p := 4498$ kip·ft

$P_u := 226$ kips

***Article 4.11.6*: Analytical Plastic Hinge Length**

Note: For reinforced concrete columns framing into a footing, an integral bent cap, an oversized shaft, or cased shaft.

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of Longitudinal Bar

$L_p := \mathrm{PlasticHinge}(\mathrm{Lstrut2}, \mathrm{fye}, d_{bl}) = 14.21$ in

***Article 4.11.7*: Reinforced Concrete Column Plastic Hinge Region**

Note: y is the region of column with a moment demand exceeding 75% of the maximum plastic moment. From the SAP model, find the location at which the moment demand is 0.75*Mp. The 0.75*Mp value should be divided by p_{eTran} to take into account the model loads have not been multiplied by p_{eTran}. The location will also need to be **INPUT** into the PlasticHingeRegion program in inches.

$Mp75 := 0.75 \cdot M_p = 3.373 \times 10^3$ lb·in

$$\mathrm{PlasticHingeRegion}(L_p, \mathrm{Columndia}) := \left| \begin{array}{l} z \leftarrow 1.5 \cdot \mathrm{Columndia} \\ x \leftarrow L_p \\ y \leftarrow 0 \\ a \leftarrow \max(z, x, y) \end{array} \right.$$

Note: Input the arguments into the program. Most likely the column diameter is the only variable that has changed.

INPUT $Lpr := \mathrm{PlasticHingeRegion}(L_p, \mathrm{Strut2Depth}) = 108$ in

***Article 8.6.2*: Concrete Shear Capacity**

$Ag := Strut2Depth \cdot Strut2Width$

Eq. 8.6.2-2 $Ae := 0.8 \cdot Ag = 2.419 \times 10^{3}$ in^2

$\mu_D := 2$ Specified in Article 6.8.2 Guide Spec.

INPUT $s := 4$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $Asp := 0.31$ in^2 Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.625$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 2$ in Cover: Concrete cover for the Column (in)

INPUT $b := Strut2Depth$ in b: Depth of the Strut (in)

INPUT $d := 70$ in d: Effective Depth (in)

$Av := 2 \cdot Asp = 0.62$ in^2

Eq. 8.6.2-10 $\rho_w := \frac{Av}{s \cdot b} = 2.153 \times 10^{-3}$

$fyh := \frac{fye}{1000} = 60$ ksi

Eq. 8.6.2-9 $fw := StressCheckRect(\rho_w, fyh) = 0.258$

Eq. 8.6.2-8 $\alpha Prime := \alpha program(fw, \mu_D) = 3$

If Pu is Compressive

Eq. 8.6.2-4

If Pu is **NOT** Compressive VC $= 0$

Note: If Pu is not compressive, will have to manually input 0 for vc. Just input it below the vc:=vcprogram and the variable will assume the new value.

$vc := vcprogram(\alpha Prime, fc, P_u, Ag) = 0.192$ ksi

$Vc := vc \cdot Ag = 580.63$ kips

***Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity**

Eq. 8.6.3-2
Eq. 8.6.4-1

$$Vs := \text{vsprogramRect}(Av, fyh, d, s, fc, Ae) = 651 \quad \text{kips}$$

Eq. 8.6.1-2

$$\phi Vn := \phi_s \cdot (Vs + Vc) = 1.108 \times 10^{3} \text{ kips}$$

$$\text{Shearcheck} := \text{ShearCheck}(\phi Vn, V_u) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

***Article 8.6.5*: Minimum Shear Reinforcement**

Eq. 8.6.5-2

$$\text{CheckTransverse} := \text{mintranprogramRect}(\rho_w) = \text{"OK"}$$

Note: If the minimum shear reinforcement program responses "Increase Shear Reinforcing Ratio", it is recommended to decrease the spacing (s) or increase the area of the shear reinforcement (Asp) in the inputs.

***Article 8.8*: Longitudinal and Lateral Reinforcement Requirements**

***Article 8.8.1*: Maximum Longitudinal Reinforcement**

INPUT $A5_{bl} := 0.31 \quad in^2$

INPUT $Number\_5\_Bars := 20$

INPUT $A11_{bl} := 1.56 \quad in^2$

INPUT $Number\_11\_Bars := 16$

$$A_{long} := A5_{bl} \cdot Number\_5\_Bars + A11_{bl} \cdot Number\_11\_Bars = 31.16 \quad in^2$$

Eq. 8.8.1-1

$$\text{ReinforcementRaitoCheck} := \rho\text{program}(A_{long}, Ag) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

***Article 8.8.2*:** **Minimum Longitudinal Reinforcement**

For Columns in SDC B and C:

Eq. 8.8.2-1 $\text{MinimumA}_l := \text{minAlprogram}(A_{long}, Ag) = \text{"OK"}$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars) in the inputs.

***Article 8.8.9*:** **Requirements for Lateral Reinforcement for SDCs B,C, and D**

These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension **of NOT** less than **6*d_b** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Minimum Size of Lateral Reinforcement

#4 bars for #9 or smaller longitudinal bars
#5 bars for #10 or larger longitudinal bars
#5 bars for bundled longitudinal bars

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

Shall Not Exceed the Smallest of:

$$\text{MaximumSpacing} := \text{Spacingprogram}(\text{Strut2Width}, d_{bl}) = 6 \quad \text{in}$$

$$\text{FINALSPACING} := \text{SpacingCheck}(\text{MaximumSpacing}, s) = 4 \quad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MaximumSpacing}, s) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.
Refer to the **AASHTO LRFD Bridge Design Specifications**.

***5.8.3.3* Nominal Shear Resistance**

$V_p = 348$ kips

INPUT $spaceNOhinge := 12$ in

INPUT $bv := Strut2Width$

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$\beta := 2.0$

$\theta := \frac{\pi}{180} \cdot 45 = 0.785$ rad

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

Eq. 5.8.2.9-2 $de := 69.4$ in — de = ds which is the distance from top of the member to the centriod of the tensile fiber

$dvpreliminary := 66.75$ in — dvpreliminary = distance between compressive and tensile reinforcing

$dv := dvprogram(de, dvpreliminary, Strut2Depth) = 66.75$ in

Eq. 5.8.3.3-3 $V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 354.362$ kips

Eq. 5.8.3.3-4 $V_s := \dfrac{2Asp \cdot \dfrac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 206.925$ kips

$\phi V_n := (V_c + V_s) \cdot \phi_s = 505.159$ kips

$Shearcheck := ShearCheck(\phi Vn, V_p) = \text{"OK"}$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $Avmin := 0.0316 \cdot \sqrt{\dfrac{fc}{1000}} \cdot \dfrac{bv \cdot spaceNOhinge}{\dfrac{fye}{1000}} = 0.531$ in^2

$Av := 2 \cdot Asp = 0.62$ in^2

$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

***5.8.2.7* Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $vu := \dfrac{V_p}{\phi_s \cdot bv \cdot dv} = 0.138$ ksi

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $MaxSpacing := spacingProgram(vu, dv, fc) = 24$ in

$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12$ in

$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT DESIGN

Article 6.5: Drilled Shafts

NOTE: The guide specification states that the drilled shafts shall conform to the requirements of columns in SDC B, C, or D as applicable. Also, there are special provisions regarding liquefaction that needs to be investigated if this is a concern for a certain bridge.

Since the hinging will not occur in the drilled shaft, the drilled shaft will be design using the column design from the LRFD Specification.

DRILLED SHAFT 2

Nominal Shear Resistance for members outside Plastic Hinge Region.
Refer to the AASHTO LRFD Bridge Design Specifications.

5.8.3.3 **Nominal Shear Resistance**

INPUT $V_p := 490$ kips

$V_u := V_p$

INPUT $spaceNOhinge := 12$ in

INPUT $Asp := 0.44$ in^2

INPUT $Cover := 6$ in^2

INPUT $bv := Drillshaftdia1$

INPUT $Dsp := 0.75$ in

INPUT $d_{bl} := 1.41$ in

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$\beta := 2.0$

$\theta := \frac{\pi}{180} \cdot 45 = 0.785$ rad

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 58.545 \quad in$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 51.635 \quad in$$

$$dv := 0.9 \cdot de = 46.472 \quad in$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 387.687 \quad kips$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 204.476 \quad kips$$

$$\phi V_n := (V_c + V_s) \cdot \phi_s = 532.947 \quad kips$$

$$Shearcheck := ShearCheck(\phi Vn, V_p) = "OK"$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.834 \quad in^2$$

$$Av := 2 \cdot Asp = 0.88 \quad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = "OK"$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{V_u - \phi_s \cdot V_p}{\phi_s \cdot bv \cdot dv} = 0.018 \quad \text{ksi}$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \quad \text{in}$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \quad \text{in}$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT 3

Nominal Shear Resistance for members outside Plastic Hinge Region.
Refer to the AASHTO LRFD Bridge Design Specifications.

5.8.3.3 **Nominal Shear Resistance**

INPUT $V_p := 262$ kips

$V_u := V_p$

INPUT $spaceNOhinge := 6$ in

INPUT $Asp := 0.44$ in^2

INPUT $Cover := 6$ in^2

INPUT $bv := Drillshaftdia2$

INPUT $Dsp := 0.75$ in

INPUT $d_{bl} := 1.41$ in

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180}\cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 70.545 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 61.455 \quad \text{in}$$

$$dv := 0.9\cdot de = 55.31 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316\cdot\beta\cdot\sqrt{\frac{fc}{1000}}\,bv\cdot dv = 545.309 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp\cdot\frac{fye}{1000}\,dv\cdot\cot(\theta)}{spaceNOhinge} = 486.725 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right)\cdot\phi_s = 928.831 \quad \text{kips}$$

$$Shearcheck := ShearCheck\left(\phi Vn, V_u\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316\cdot\sqrt{\frac{fc}{1000}}\cdot\frac{bv\cdot spaceNOhinge}{\frac{fye}{1000}} = 0.493 \quad \text{in}^2$$

$$Av := 2\cdot Asp = 0.88 \quad \text{in}^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{V_u}{\phi_s \cdot bv \cdot dv} = 0.067 \quad ksi$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \quad in$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 6 \quad in$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT 4

Nominal Shear Resistance for members outside Plastic Hinge Region.
Refer to the AASHTO LRFD Bridge Design Specifications.

5.8.3.3 **Nominal Shear Resistance**

INPUT $V_p := 525$ kips

$V_u := V_p$

INPUT $spaceNOhinge := 12$ in

INPUT $Asp := 0.44$ in^2

INPUT $Cover := 6$ in^2

INPUT $bv := Drillshaftdia1$

INPUT $Dsp := 0.75$ in

INPUT $d_{bl} := 1.41$ in

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 58.545 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 51.635 \quad \text{in}$$

$$dv := 0.9 \cdot de = 46.472 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 387.687 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 204.476 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 532.947 \quad \text{kips}$$

$$Shearcheck := ShearCheck\left(\phi Vn, V_u\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.834 \quad \text{in}^2$$

$$Av := 2 \cdot Asp = 0.88 \quad \text{in}^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $vu := \dfrac{V_u}{\phi_s \cdot bv \cdot dv} = 0.19$ ksi

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $MaxSpacing := spacingProgram(vu, dv, fc) = 24$ in

$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12$ in

$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT ABUTMENT 1

Nominal Shear Resistance for members outside Plastic Hinge Region.
Refer to the AASHTO LRFD Bridge Design Specifications.

5.8.3.3 **Nominal Shear Resistance**

INPUT $V_p := 209$ kips

$V_u := V_p$

INPUT $spaceNOhinge := 10$ in

INPUT $Asp := 0.31$ in^2

INPUT $bv := Drillshaftdia3$

INPUT $Cover := 6$ in

INPUT $Dsp := 0.625$ in

INPUT $d_{bl} := 1.41$ in

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 46.67 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 41.856 \quad \text{in}$$

$$dv := 0.9 \cdot de = 37.67 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 257.12 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 140.132 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 357.527 \quad \text{kips}$$

$$Shearcheck := ShearCheck\left(\phi Vn, V_p\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.569 \quad \text{in}^2$$

$$Av := 2 \cdot Asp = 0.62 \quad \text{in}^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $vu := \frac{V_u}{\phi_s \cdot bv \cdot dv} = 0.114$ ksi

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $MaxSpacing := spacingProgram(vu, dv, fc) = 24$ in

$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 10$ in

$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT ABUTMENT 5

Nominal Shear Resistance for members outside Plastic Hinge Region.
Refer to the AASHTO LRFD Bridge Design Specifications.

5.8.3.3 **Nominal Shear Resistance**

INPUT $V_p := 50$ kips

$V_u := V_p$

INPUT $spaceNOhinge := 10$ in

INPUT $Asp := 0.31$ in^2

INPUT $bv := Drillshaftdia3$

INPUT $Cover := 6$ in

INPUT $Dsp := 0.625$ in

INPUT $d_{bl} := 1.41$ in

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 46.67 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 41.856 \quad \text{in}$$

$$dv := 0.9 \cdot de = 37.67 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 257.12 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 140.132 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 357.527 \quad \text{kips}$$

$$Shearcheck := ShearCheck\left(\phi Vn, V_p\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.569 \quad \text{in}^2$$

$$Av := 2 \cdot Asp = 0.62 \quad \text{in}^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $\quad vu := \dfrac{V_u}{\phi_s \cdot bv \cdot dv} = 0.027 \quad$ ksi

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $\quad MaxSpacing := spacingProgram(vu, dv, fc) = 24 \quad$ in

$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 10 \quad$ in

$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

CONNECTION DESIGN FOR BENT/ABUTMENT TO GIRDER

Bent 2 Connection Design

INPUT $\quad Vcolbent := V_{pBent2} = 980$

INPUT $\quad Ngirderperbent := 12 \quad$ Ngirderbent = Number of girders per bent

Article 6.5.4.2: Resistance Factors

$\phi_t := 0.8$ — Tension for A307

$\phi_s := 0.75$ — Shear for A307

$\phi_{bs} := 0.80$ — Block Shear

$\phi_{bb} := 0.80$ — Bolts Bearing

$\phi_{sc} := 0.85$ — Shear Connectors

$\phi_f := 1.00$ — Flexure

$\phi_{sangle} := 1.00$ — Shear for the Angle

For Type BT-72 Girders

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT	$Fub := 58$	ksi	
INPUT	$Dia_b := 1.5$	in	
INPUT	$Ns := 1$		Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT	$Fy := 36$	ksi	Fy = Yield Stress of the Angle
INPUT	$Fu := 58$	ksi	Fu = Ultimate Stress of the Angle
INPUT	$t := 1.0$	in	t = Thickness of Angle
INPUT	$h := 6$	in	h = Height of the Angle
INPUT	$w := 6$	in	w = Width of the Angle
INPUT	$l := 16$	in	l = Length of the Angle
INPUT	$k := 1.5$	in	k = Height of the Bevel
INPUT	$distanchorhole := 4$	in	distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.
INPUT	$diahole := 1.75$	in	diahole = Diameter of bolt hole
INPUT	$BLSHlength := 11$	in	BLSHlength = Block Shear Length
INPUT	$BLSHwidth := 2$	in	BLSHwidth = Block Shear Width
INPUT	$Ubs := 1.0$		Ubs = Shear Lag Factor for Block Shear
INPUT	$a := 2$	in	a = Distance from the center of the bolt to the edge of plate
INPUT	$b := 3.5$	in	b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent}{2Ngirderperbent} = 40.833 \quad \text{kips}$$

INPUT $n := 2$ n = number of bolts

$$Vperbolt := \frac{Vangle}{n} = 20.417$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 1.767 \quad in^2$$

Eq. 6.13.2.12-1 $\phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 36.898$ kips **CONTROLS MUST USE 1.5" BOLT**

$Shearcheck := ShearCheck(\phi sRn, Vperbolt) = \text{"OK"}$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 208.8$ kips

For Slotted Holes

INPUT $Lc := 2$ in Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\phi bbRns := Lc \cdot t \cdot Fub = 116$ kips

$Bearingcheck := ShearCheck(\phi bbRn, Vperbolt) = \text{"OK"}$

$Bearingscheck := ShearCheck(\phi bbRns, Vperbolt) = \text{"OK"}$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 10.208 \qquad \text{kips}$$

Eq. 6.13.2.10.2-1

$$\phi tTn := \phi_t \cdot 0.76 \cdot A_b \cdot Fub = 62.317 \qquad \text{kips}$$

$$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$$Pu := Vperbolt$$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2

$$CombinedProgram(Pu, A_b, Fub, \phi sRn, \phi_s) := \left| \begin{array}{l} t \leftarrow 0.76 \cdot A_b \cdot Fub \\ r \leftarrow 0.76 \cdot A_b \cdot Fub \cdot \sqrt{1 - \left(\frac{Pu}{\phi sRn}\right)^2} \\ a \leftarrow t \ \text{ if } \frac{Pu}{\left(\frac{\phi sRn}{\phi_s}\right)} \le 0.33 \\ a \leftarrow r \ \text{ if } \frac{Pu}{\left(\frac{\phi sRn}{\phi_s}\right)} > 0.33 \\ a \end{array} \right.$$

$$Tn_{combined} := CombinedProgram(Pu, A_b, Fub, \phi sRn, \phi_s) = 64.884 \qquad \text{kips}$$

$$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 51.908 \qquad \text{kips}$$

$$Combinedcheck := ShearCheck(\phi tTn_{combined}, Vperbolt) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$Agv := t \cdot BLSHlength = 11$ in^2

$Anv := t \cdot (BLSHlength - 1.5 \cdot diahole) = 8.375$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 1.125$ in^2

Note this is for if there are 2 through bolts in the upper leg.

(J4-5) $BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) := \left| \begin{array}{l} b \leftarrow 0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant \\ c \leftarrow 0.6Fy \cdot Agv + Ubs \cdot Fu \cdot Ant \\ a \leftarrow b \ \text{ if } \ b \le c \\ a \leftarrow c \ \text{ if } \ b > c \\ a \end{array} \right.$

$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 302.85$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 242.28$ kips

BlockShearCheck := ShearCheck(ϕbsRn, Vangle) = "OK"

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 4.25$ in^2

(D3-1) $Ae := Ant \cdot Ut = 2.55$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 118.32$ kips

TensionCheck := ShearCheck(ϕtPn, Vangle) = "OK"

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

Note: This is assuming we need the same size anchor bolt in the top as we do the bottom.

INPUT $\quad Muangle := 121 \quad kip \cdot in$

$$Zx := \frac{1 \cdot (t)^2}{4} = 4 \qquad in^3$$

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 144 \qquad kip \cdot in$$

Had to increase the thickness of the angle to 1.00 in. and increase the length 16 in

$$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t \cdot w = 6 \qquad in^2$$

(G2-1) $\quad \phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 129.6 \qquad kips$

$$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

BENT 2 EXPANSION CONNECTION

For Type BT-72 Girders

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT	$Fub := 58$	ksi	
INPUT	$Dia_b := 1.5$	in	
INPUT	$Ns := 1$		Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT	$Fy := 36$	ksi	Fy = Yield Stress of the Angle
INPUT	$Fu := 58$	ksi	Fu = Ultimate Stress of the Angle
INPUT	$t := 1.0$	in	t = Thickness of Angle
INPUT	$h := 6$	in	h = Height of the Angle
INPUT	$w := 6$	in	w = Width of the Angle
INPUT	$l := 20$	in	l = Length of the Angle
INPUT	$k := 1.5$	in	k = Height of the Bevel
INPUT	$distanchorhole := 4$	in	distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.
INPUT	$diahole := 1.75$	in	diahole = Diameter of bolt hole
INPUT	$SlottedHole := 6$	in	SlottedHole = Length of Slotted Hole
INPUT	$BLSHlength := 15$	in	BLSHlength = Block Shear Length
INPUT	$BLSHwidth := 2$	in	BLSHwidth = Block Shear Width
INPUT	$Ubs := 1.0$		Ubs = Shear Lag Factor for Block Shear
INPUT	$a := 2$	in	a = Distance from the center of the bolt to the edge of plate
INPUT	$b := 3.5$	in	b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent}{2Ngirderperbent} = 40.833 \qquad \text{kips}$$

INPUT $\quad n := 2 \qquad$ n = number of bolts

$$Vperbolt := \frac{Vangle}{n} = 20.417$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b{}^2}{4} = 1.767 \qquad in^2$$

Eq. 6.13.2.12-1 $\quad \phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 36.898 \quad$ kips $\quad$ **CONTROLS MUST USE 1.5" BOLT**

$$Shearcheck := ShearCheck(\phi sRn, Vperbolt) = \text{"OK"}$$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\quad \phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 208.8 \quad$ kips

For Slotted Holes

INPUT $\quad Lc := 2 \quad in \qquad$ Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\quad \phi bbRns := Lc \cdot t \cdot Fub = 116 \quad$ kips

$$Bearingcheck := ShearCheck(\phi bbRn, Vperbolt) = \text{"OK"}$$

$$Bearingscheck := ShearCheck(\phi bbRns, Vperbolt) = \text{"OK"}$$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 10.208 \qquad \text{kips}$$

Eq. 6.13.2.10.2-1 $\quad \phi tTn := \phi_t \cdot 0.76 \cdot A_b \cdot Fub = 62.317 \qquad \text{kips}$

$$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$$Pu := Vperbolt$$

$$Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 64.884 \qquad \text{kips}$$

$$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 51.908 \qquad \text{kips}$$

$$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vperbolt\right) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$$Agv := t \cdot BLSHlength = 15 \quad in^2$$

$$Anv := t \cdot \left(BLSHlength - 1.5 \cdot \frac{SlottedHole}{2} \right) = 10.5 \quad in^2$$

Note this is for if there are 2 through bolts in the upper leg.

$$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 1.125 \quad in^2$$

$$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 389.25 \quad kips$$

$$\phi bsRn := \phi_{bs} \cdot Rn = 311.4 \quad kips$$

$$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$$Ut := 0.6$$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$$Ant := t \cdot [w - (1 \cdot diahole)] = 4.25 \quad in^2$$

(D3-1) $$Ae := Ant \cdot Ut = 2.55 \quad in^2$$

(D2-2) $$\phi tPn := \phi_t \cdot Fub \cdot Ae = 118.32 \quad kips$$

$$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

Note: This is assuming we need the same size anchor bolt in the top as we do the bottom.

<u>*INPUT*</u> $Muangle := 121 \quad kip \cdot in$

$$Zx := \frac{1 \cdot (t)^2}{4} = 5 \qquad in^3$$

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 180 \qquad kip \cdot in$$

Had to increase the thickness of the angle to 1.00 in. and increase the length 16 in

$$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t \cdot w = 6 \qquad in^2$$

(G2-1) $\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 129.6 \qquad kips$

$$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Bent 3 Connection Design

INPUT $Vcolbent := V_{pBent3} = 524$

INPUT $Ngirderperbent := 12$ Ngirderbent = Number of girders per bent

Article 6.5.4.2: Resistance Factors

$\phi_t := 0.8$	Tension for A307
$\phi_s := 0.75$	Shear for A307
$\phi_{bs} := 0.80$	Block Shear
$\phi_{bb} := 0.80$	Bolts Bearing
$\phi_{sc} := 0.85$	Shear Connectors
$\phi_f := 1.00$	Flexure
$\phi_{sangle} := 1.00$	Shear for the Angle

For Type BT-72 Girders

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.5$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT	$Fy := 36$	ksi	Fy = Yield Stress of the Angle
INPUT	$Fu := 58$	ksi	Fu = Ultimate Stress of the Angle
INPUT	$t := 1.00$	in	t = Thickness of Angle
INPUT	$h := 6$	in	h = Height of the Angle
INPUT	$w := 6$	in	w = Width of the Angle

INPUT $l := 12$ in — l = Length of the Angle

INPUT $k := 1.5$ in — k = Height of the Bevel

INPUT $distanchorhole := 4$ in — distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT $diahole := 1.75$ in — diahole = Diameter of bolt hole

INPUT $BLSHlength := 6$ in — BLSHlength = Block Shear Length

INPUT $BLSHwidth := 2$ in — BLSHwidth = Block Shear Width

INPUT $Ubs := 1.0$ — Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in — a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in — b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent}{2Ngirderperbent} = 21.833 \quad \text{kips}$$

INPUT $n := 1$ — n = number of bolts

$$Vperbolt := \frac{Vangle}{n} = 21.833$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\ 2}}{4} = 1.767 \quad \text{in}^2$$

Eq. 6.13.2.12-1 $\quad \phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 36.898$ kips **CONTROLS MUST USE 1.5" BOLT**

$Shearcheck := ShearCheck(\phi sRn, Vperbolt) = "OK"$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 208.8$ kips

For Slotted Holes

INPUT $Lc := 2$ in Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\phi bbRns := Lc \cdot t \cdot Fub = 116$ kips

$Bearingcheck := ShearCheck(\phi bbRn, Vperbolt) = \text{"OK"}$

$Bearingscheck := ShearCheck(\phi bbRns, Vperbolt) = \text{"OK"}$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 5.458 \quad \text{kips}$$

Eq. 6.13.2.10.2-1 $\phi tTn := \phi_t \cdot 0.76 \cdot A_b \cdot Fub = 62.317$ kips

$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$Pu := Vperbolt$

$Tn_{combined} := CombinedProgram(Pu, A_b, Fub, \phi sRn, \phi_s) = 62.795$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 50.236$ kips

$Combinedcheck := ShearCheck(\phi tTn_{combined}, Vperbolt) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$Agv := t \cdot BLSHlength = 6$ in^2

$Anv := t \cdot (BLSHlength - 0.5 \cdot diahole) = 5.125$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 1.125$ in^2

Note this is for if there are 1 through bolts in the upper leg.

$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 194.85$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 155.88$ kips

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 4.25$ in^2

(D3-1) $Ae := Ant \cdot Ut = 2.55$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 118.32$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

Note: This is assuming we need the same size anchor bolt in the top as we do the bottom.

INPUT $\quad \text{Muangle} := 65 \quad \text{kip}\cdot\text{in}$

$$Zx := \frac{1\cdot(t)^2}{4} = 3 \qquad in^3$$

Had to increase the thickness of the angle to 1.00 in. or increase the length.

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 108 \qquad \text{kip}\cdot\text{in}$$

$$\text{BendingAngleCheck} := \text{ShearCheck}(\phi fMn, \text{Muangle}) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t\cdot w = 6 \qquad in^2$$

(G2-1) $$\phi\text{sangleVn} := \phi_{sangle}\cdot 0.6\cdot Fy\cdot Aw\cdot Cv = 129.6 \qquad \text{kips}$$

$$\text{ShearAngleCheck} := \text{ShearCheck}(\phi\text{sangleVn}, \text{Vangle}) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Bent 3 Expansion Connection Design

INPUT $Vcolbent := V_{pBent3} = 524$

INPUT $Ngirderperbent := 12$ Ngirderbent = Number of girders per bent

Article 6.5.4.2: Resistance Factors

$\phi_t := 0.8$ Tension for A307

$\phi_s := 0.75$ Shear for A307

$\phi_{bs} := 0.80$ Block Shear

$\phi_{bb} := 0.80$ Bolts Bearing

$\phi_{sc} := 0.85$ Shear Connectors

$\phi_f := 1.00$ Flexure

$\phi_{sangle} := 1.00$ Shear for the Angle

For Type BT-72 Girders

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.5$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT $Fy := 36$ ksi Fy = Yield Stress of the Angle

INPUT $Fu := 58$ ksi Fu = Ultimate Stress of the Angle

INPUT $t := 1.00$ in t = Thickness of Angle

INPUT $h := 6$ in h = Height of the Angle

INPUT $w := 6$ in w = Width of the Angle

INPUT $l := 12$ in l = Length of the Angle

INPUT $k := 1.5$ in k = Height of the Bevel

INPUT	$distanchorhole := 4$	in	distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.
INPUT	$diahole := 1.75$	in	diahole = Diameter of bolt hole
INPUT	$SlottedHole := 6$	in	SlottedHole = Length of Slotted Hole
INPUT	$BLSHlength := 6$	in	BLSHlength = Block Shear Length
INPUT	$BLSHwidth := 2$	in	BLSHwidth = Block Shear Width
INPUT	$Ubs := 1.0$		Ubs = Shear Lag Factor for Block Shear
INPUT	$a := 2$	in	a = Distance from the center of the bolt to the edge of plate
INPUT	$b := 3.5$	in	b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent}{2Ngirderperbent} = 21.833 \qquad \text{kips}$$

INPUT $n := 1$ n = number of bolts

$$Vperbolt := \frac{Vangle}{n} = 21.833$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 1.767 \qquad \text{in}^2$$

Eq. 6.13.2.12-1 $\phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 36.898$ kips **CONTROLS MUST USE 1.5" BOLT**

$Shearcheck := ShearCheck(\phi sRn, Vperbolt) = \text{"OK"}$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 208.8$ kips

For Slotted Holes

INPUT $Lc := 2$ in Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\phi bbRns := Lc \cdot t \cdot Fub = 116$ kips

$Bearingcheck := ShearCheck(\phi bbRn, Vperbolt) = \text{"OK"}$

$Bearingscheck := ShearCheck(\phi bbRns, Vperbolt) = \text{"OK"}$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 5.458 \quad \text{kips}$$

Eq. 6.13.2.10.2-1 $\phi tTn := \phi_t \cdot 0.76 \cdot A_b \cdot Fub = 62.317$ kips

$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$Pu := Vperbolt$

$Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 62.795$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 50.236$ kips

$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vperbolt\right) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$Agv := t \cdot BLSHlength = 6$ in^2

$Anv := t \cdot \left(BLSHlength - 0.5 \cdot \frac{SlottedHole}{2}\right) = 4.5$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 1.125$ in^2

Note this is for if there are 1 through bolts in the upper leg.

$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 194.85$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 155.88$ kips

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 4.25$ in^2

(D3-1) $Ae := Ant \cdot Ut = 2.55$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 118.32$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

Note: This is assuming we need the same size anchor bolt in the top as we do the bottom.

INPUT $\quad Muangle := 65 \quad kip \cdot in$

$$Zx := \frac{1 \cdot (t)^2}{4} = 3 \qquad in^3$$

Had to increase the thickness of the angle to 1.00 in. or increase the length.

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 108 \qquad kip \cdot in$$

$$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t \cdot w = 6 \qquad in^2$$

(G2-1) $$\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 129.6 \qquad kips$$

$$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Bent 4 Connection Design

INPUT $Vcolbent := V_{pBent4}$

INPUT $Ngirderperbent := 12$ Ngirderbent = Number of girders per bent

Article 6.5.4.2: Resistance Factors

$\phi_t := 0.8$ Tension for A307

$\phi_s := 0.75$ Shear for A307

$\phi_{bs} := 0.80$ Block Shear

$\phi_{bb} := 0.80$ Bolts Bearing

$\phi_{sc} := 0.85$ Shear Connectors

$\phi_f := 1.00$ Flexure

$\phi_{sangle} := 1.00$ Shear for the Angle

For Type BT-72 Girders

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.5$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT $Fy := 36$ ksi Fy = Yield Stress of the Angle

INPUT $Fu := 58$ ksi Fu = Ultimate Stress of the Angle

INPUT $t := 1.0$ in t = Thickness of Angle

INPUT $h := 6$ in h = Height of the Angle

INPUT $w := 6$ in w = Width of the Angle

INPUT $l := 16$ in — l = Length of the Angle

INPUT $k := 1.5$ in — k = Height of the Bevel

INPUT $distanchorhole := 4$ in — distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT $diahole := 1.75$ in — diahole = Diameter of bolt hole

INPUT $BLSHlength := 11$ in — BLSHlength = Block Shear Length

INPUT $BLSHwidth := 2$ in — BLSHwidth = Block Shear Width

INPUT $Ubs := 1.0$ — Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in — a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in — b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent}{2Ngirderperbent} = 43.75 \qquad \text{kips}$$

INPUT $n := 2$ — n = number of bolts

$$Vperbolt := \frac{Vangle}{n} = 21.875$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\ 2}}{4} = 1.767 \qquad in^2$$

Eq. 6.13.2.12-1 $\phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 36.898$ kips **CONTROLS MUST USE 1.5" BOLT**

$Shearcheck := ShearCheck(\phi sRn, Vperbolt) = \text{"OK"}$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 208.8$ kips

For Slotted Holes

INPUT $Lc := 2$ in Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\phi bbRns := Lc \cdot t \cdot Fub = 116$ kips

$Bearingcheck := ShearCheck(\phi bbRn, Vperbolt) = \text{"OK"}$

$Bearingscheck := ShearCheck(\phi bbRns, Vperbolt) = \text{"OK"}$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 10.938 \quad \text{kips}$$

Eq. 6.13.2.10.2-1 $\phi tTn := \phi_t \cdot 0.76 \cdot A_b \cdot Fub = 62.317$ kips

$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$Pu := Vperbolt$

$Tn_{combined} := CombinedProgram(Pu, A_b, Fub, \phi sRn, \phi_s) = 62.73$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 50.184$ kips

$Combinedcheck := ShearCheck(\phi tTn_{combined}, Vperbolt) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$Agv := t \cdot BLSHlength = 11$ in^2

$Anv := t \cdot (BLSHlength - 1.5 \cdot diahole) = 8.375$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 1.125$ in^2

Note this is for if there are 2 through bolts in the upper leg.

$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 302.85$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 242.28$ kips

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 4.25$ in^2

(D3-1) $Ae := Ant \cdot Ut = 2.55$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 118.32$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

Note: This is assuming we need the same size anchor bolt in the top as we do the bottom.

INPUT $\quad Muangle := 125 \quad kip \cdot in$

$$Zx := \frac{1 \cdot (t)^2}{4} = 4 \qquad in^3$$

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 144 \qquad kip \cdot in$$

Had to increase the thickness of the angle to 1.00 in. or increase the length 16 in.

$$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t \cdot w = 6 \qquad in^2$$

(G2-1) $$\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 129.6 \qquad kips$$

$$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Abutment 1 to Girder Connection

INPUT $Vcolbent := 627$ kips

INPUT $Ngirderperbent := 6$ Ngirderbent = Number of girders per bent

For Type BT-72 Girders

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.5$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT $Fy := 36$ ksi Fy = Yield Stress of the Angle

INPUT $Fu := 58$ ksi Fu = Ultimate Stress of the Angle

INPUT $t := 1.0$ in t = Thickness of Angle

INPUT $h := 6$ in h = Height of the Angle

INPUT $w := 6$ in w = Width of the Angle

INPUT $l := 20$ in l = Length of the Angle

INPUT $k := 1.5$ in k = Height of the Bevel

INPUT $distanchorhole := 4$ in distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT $diahole := 1.75$ in diahole = Diameter of bolt hole

INPUT $SlottedHole := 6$ in SlottedHole = Length of Slotted Hole

INPUT $BLSHlength := 15$ in BLSHlength = Block Shear Length

INPUT $BLSHwidth := 2$ in BLSHwicth = Block Shear Width

INPUT $Ubs := 1.0$ Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent}{2Ngirderperbent} = 52.25 \qquad kips$$

INPUT $n := 2$ n_{bolts} = number of bolts per flange

$$Vperbolt := \frac{Vangle}{n} = 26.125 \qquad kips$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 1.767 \qquad in^2$$

Eq. 6.13.2.12-1 $\phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 36.898$ kips

Note: This is checking to verify that the anchor bolt has enough shear strength.

$$Shearcheck := ShearCheck(\phi sRn, Vperbolt) = \text{"OK"}$$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 208.8$ kips

For Slotted Holes

Eq. 6.13.2.9.3 $\phi bbRns := 2.0 \cdot Dia_b \cdot t \cdot Fub = 174$ kips

$$Bearingcheck := ShearCheck(\phi bbRn, Vperbolt) = \text{"OK"}$$

$$Bearingscheck := ShearCheck(\phi bbRns, Vperbolt) = \text{"OK"}$$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 13.063 \qquad \text{kips}$$

Eq. 6.13.2.10.2-1

$$\phi tTn := 0.76 \cdot A_b \cdot Fub = 77.896 \qquad \text{kips}$$

$$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$$Pu := Vperbolt$$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2

$$Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 55.008 \qquad \text{kips}$$

$$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 44.007 \qquad \text{kips}$$

$$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vperbolt\right) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$$Agv := t \cdot BLSHlength = 15 \qquad in^2$$

$$Anv := t \cdot \left(BLSHlength - 1.5 \cdot \frac{SlottedHole}{2}\right) = 10.5 \qquad in^2$$

$$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 1.125 \qquad in^2$$

Note this is for if there are 2 through bolts in the upper leg.

(J4-5) $$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 389.25 \qquad kips$$

$$\phi bsRn := \phi_{bs} \cdot Rn = 311.4 \qquad kips$$

$$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$$Ut := 0.6$$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$$Ant := t \cdot [w - (1 \cdot diahole)] = 4.25 \qquad in^2$$

(D3-1) $$Ae := Ant \cdot Ut = 2.55 \qquad in^2$$

(D2-2) $$\phi tPn := \phi_t \cdot Fub \cdot Ae = 118.32 \qquad kips$$

$$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

Note: This is assuming we need the same size anchor bolt in the top as we do the bottom.

INPUT $Muangle := 155$ kip·in

$Zx := \frac{1 \cdot (t)^2}{4} = 5$ in^3

$\phi fMn := \phi_f \cdot Fy \cdot Zx = 180$ kip·in

Had to increase the thickness of the angle to 1.00 in. and increase the length 18 in.

$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = $ "OK"

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$Cv := 1.0$

$Aw := t \cdot w = 6$ in^2

(G2-1) $\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 129.6$ kips

$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = $ "OK"

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Abutment 5 to Girder Connection

INPUT $Vcolbent := 150$ kips

INPUT $Ngirderperbent := 6$ Ngirderbent = Number of girders per bent

For Type BT-72 Girders

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.25$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT $Fy := 36$ ksi Fy = Yield Stress of the Angle

INPUT $Fu := 58$ ksi Fu = Ultimate Stress of the Angle

INPUT $t := 0.75$ in t = Thickness of Angle

INPUT $h := 6$ in h = Height of the Angle

INPUT $w := 6$ in w = Width of the Angle

INPUT $l := 12$ in l = Length of the Angle

INPUT $k := 1.5$ in k = Height of the Bevel

INPUT $distanchorhole := 4$ in distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT $diahole := 1.5$ in diahole = Diameter of bolt hole

INPUT $SlottedHole := 6$ in SlottedHole = Length of Slotted Hole

INPUT $BLSHlength := 6$ in BLSHlength = Block Shear Length

INPUT $BLSHwidth := 2$ in BLSHwicth = Block Shear Width

INPUT $Ubs := 1.0$ Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent}{2Ngirderperbent} = 12.5 \quad \text{kips}$$

INPUT $n := 1$ n_{bolts} = number of bolts per flange

$$Vperbolt := \frac{Vangle}{n} = 12.5 \quad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 1.227 \quad in^2$$

Eq. 6.13.2.12-1 $\phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 25.624$ kips

Note: This is checking to verify that the anchor bolt has enough shear strength.

$$Shearcheck := ShearCheck(\phi sRn, Vperbolt) = \text{"OK"}$$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 130.5$ kips

For Slotted Holes

Eq. 6.13.2.9.3 $\phi bbRns := 2.0 \cdot Dia_b \cdot t \cdot Fub = 108.75$ kips

$$Bearingcheck := ShearCheck(\phi bbRn, Vperbolt) = \text{"OK"}$$

$$Bearingscheck := ShearCheck(\phi bbRns, Vperbolt) = \text{"OK"}$$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 3.125 \qquad \text{kips}$$

Eq. 6.13.2.10.2-1 $\qquad \phi tTn := 0.76 \cdot A_b \cdot Fub = 54.094 \qquad \text{kips}$

$$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$$Pu := Vperbolt$$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2 $\qquad Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 47.221 \qquad \text{kips}$

$$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 37.777 \qquad \text{kips}$$

$$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vangle\right) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$Agv := t \cdot BLSHlength = 4.5 \quad in^2$

$Anv := t \cdot \left(BLSHlength - 0.5 \cdot \frac{SlottedHole}{2}\right) = 3.375 \quad in^2$

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 0.938 \quad in^2$

Note this is for if there are one through bolts in the upper leg.

(J4-5) $Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 151.575 \quad kips$

$\phi bsRn := \phi_{bs} \cdot Rn = 121.26 \quad kips$

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 3.375 \quad in^2$

(D3-1) $Ae := Ant \cdot Ut = 2.025 \quad in^2$

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 93.96 \quad kips$

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

Note: This is assuming we need the same size anchor bolt in the top as we do the bottom.

INPUT $\quad Muangle := 44 \quad kip \cdot in$

$$Zx := \frac{1 \cdot (t)^2}{4} = 1.688 \qquad in^3$$

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 60.75 \qquad kip \cdot in$$

Had to increase the thickness of the angle to 0.75 in. or increase the length.

$$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t \cdot w = 4.5 \qquad in^2$$

(G2-1) $$\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 97.2 \qquad kips$$

$$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Appendix H: Scarham Creek Bridge LRFD Specification Design

Designer: Paul Coulston
Project Name: Little Bear Creek Bridge
Job Number:
Date: 8/2/2010

ORIGIN := 1

Description of worksheet: This worksheet is a seismic bridge design worksheet for the **AASHTO LRFD Bridge Design Specification**. All preliminary design should already be done for non-seismic loads.

Project Known Information

Location: Marshall County
Zip Code or Coordinates: 35.0069 N 88.2025 W

Superstructure Type: AASTHO I girders
Substructure Type: Circular columns supported on drilled shafts
Abutment Type: Abutment beam supported on drilled shafts

Note: *Input all of the below information.*

$fc := 4000$ psi

$fye := 60000$ psi

$\rho_{conc} := 0.08681 \ \frac{lb}{in^3}$

$g := 386.4 \ \frac{in}{s^2}$

Length of Bridge (ft)	$L := 520$ ft
Span Length (ft)	$Span := 130$ ft
Deck Thickness (in)	$t_{deck} := 7$ in
Deck Width (ft)	$DeckWidth := 40$ ft
Girder X-Sectional Area (in^2)	$GirderArea := 767$ in^2
Bent Volume (ft^3)	$BentVolume := 7.5 \cdot 5.5 \cdot 40 = 1.65 \times 10^3$ ft^3
Guard Rail Area (in^2)	$GuardRailArea := 310$ in^2
Column 1 Diameter (in)	$Columndia1 := 60$ in
Column 2 Diameter (in)	$Columndia2 := 72$ in
Drill Shaft 1 Diameter (in)	$Drillshaftdia1 := 66$ in
Drill Shaft 2 Diameter (in)	$Drillshaftdia2 := 78$ in
Drill Shaft 3 (Abutment) Diameter (in)	$Drillshaftdia3 := 54$ in

Strut 2 & 4 Depth (in) $Strut2Depth := 72$ in

Strut 2 & 4 Width (in) $Strut2Width := 42$ in

Strut 3 Depth (in) $Strut3Depth := 120$ in

Strut 3 Width (in) $Strut3Width := 42$ in

Tallest Above Ground Column Height Bent 2 (ft) $ColumnHeight2 := 34.022$ ft

Tallest Above Ground Column Height Bent 3 (ft) $ColumnHeight3 := 59.136$ ft

Tallest Above Ground Column Height Bent 4 (ft) $ColumnHeight4 := 32.156$ ft

Length of Strut 2 & 4 (ft) $Lstrut2 := 19$ ft

Length of Strut 3 (ft) $Lstrut3 := 18$ ft

Column 1 Area (in^2) $Acolumn1 := \frac{Columndia1^2 \cdot \pi}{4} = 2.827 \times 10^3$ in^2

Column 2 Area (in^2) $Acolumn2 := \frac{Columndia2^2 \cdot \pi}{4} = 4.072 \times 10^3$ in^2

Drilled Shaft 1 Area (in^2) $Ads1 := \frac{Drillshaftdia1^2 \cdot \pi}{4} = 3.421 \times 10^3$ in^2

Drilled Shaft 2 Area (in^2) $Ads2 := \frac{Drillshaftdia2^2 \cdot \pi}{4} = 4.778 \times 10^3$ in^2

Drilled Shaft 3 Area (in^2) $Ads3 := \frac{Drillshaftdia3^2 \cdot \pi}{4} = 2.29 \times 10^3$ in^2

Bent 2 and 4 Strut Volume (ft^3) $Strut1 := 6 \cdot 3.5 \cdot 19 = 399$ ft^3

Bent 3 Strut Volume (ft^3) $Strut2 := 10 \cdot 3.5 \cdot 18 = 630$ ft^3

Note: These are variables that were easier to input in ft and then convert to inches.

$L := L \cdot 12 = 6.24 \times 10^3$ in

$Span := Span \cdot 12 = 1.56 \times 10^3$ in

$DeckWidth := DeckWidth \cdot 12 = 480$ in

$BentVolume := BentVolume \cdot 12^3 = 2.851 \times 10^6$ in^3

$ColumnHeight2 := ColumnHeight2 \cdot 12 = 408.264$ in

$ColumnHeight3 := ColumnHeight3 \cdot 12 = 709.632$ in

$ColumnHeight4 := ColumnHeight4 \cdot 12 = 385.872$ in

$Strut1 := Strut1 \cdot 12 = 4.788 \times 10^3$ in^3

$Strut2 := Strut2 \cdot 12 = 7.56 \times 10^3$ in^3

Steps for Seismic Design

Use Appendix A3 Seismic Design Flowcharts

Description of Difference from Guide Specification
The LRFD Specification is a force based approach to design verses the Guide Specification which is a displacement based approach. The LRFD Specification has an Response Modification Factor (R) that will be used in calculating the loads applied to the structure.

Article 3.10.1: Earthquake Effects - This is just the applicability of the Specifications.

***Article 3.10.2*: Determine Design Response Spectrum**

Note: AASHTO Guide Specifications for LRFD Seismic Bridge Design is accompanied with a program that has the Seismic Hazard maps. This program will calculate several of the variables that are needed for the analysis.

1) *Article 3.10.3.1*:Determine the Site Class.

INPUT Site Class: **D**

2) Enter maps and find PGA, S_s,and S_1. Then enter those values in their respective spot.

$PGA := 0.116$ g

INPUT $S_s := 0.272$ g

$S_1 := 0.092$ g

3) *Article 3.10.3.2:* Site Coefficients. From the PGA,S_s, and S_1 values and site class choose F_{PGA}, F_a, and F_v. Note: straight line interpolation is permitted.

$F_{PGA} := 1.57$

INPUT $F_a := 1.58$

$F_v := 2.4$

$A_s := F_{PGA} \cdot PGA = 0.182$ g A_s : Acceleration Coefficient

$SDS := F_a \cdot S_s = 0.43 \quad g$ $\qquad$ S_{DS} = Short Period Acceleration Coefficient

$SD1 := F_v \cdot S_1 = 0.221 \quad g$ $\qquad$ S_{D1} = 1-sec Period Acceleration Coefficient

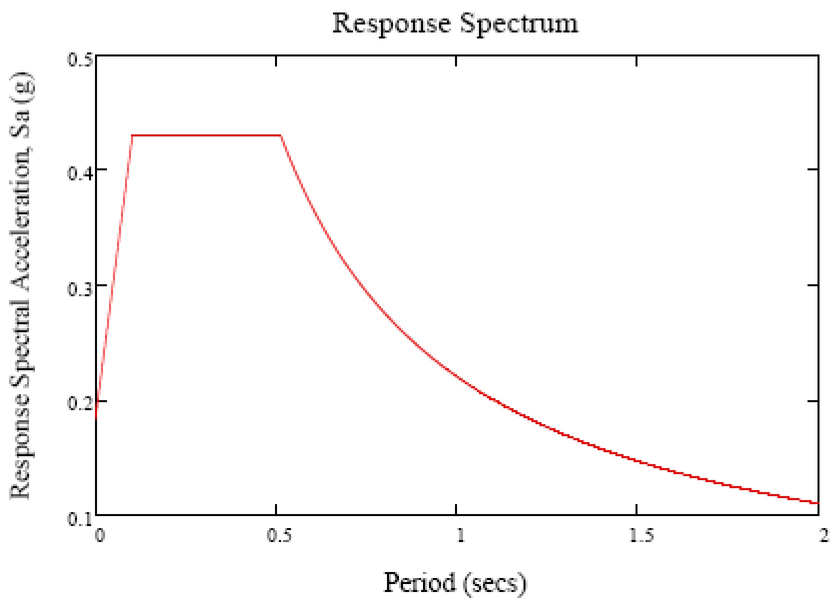
4) Creating a Response Spectrum

Note: The DesignSpectrum Code creates a Response Spectrum for the Bridge. At this time the period of the bridge is unknown; therefore, the Sa value cannot be calculated.

$Tmax := 2 \ s \qquad Dt := 0.001 \ s$

$$
\text{DesignSpectrum}(SDS, SD1, A_s, Tmax, Dt) := \left|
\begin{array}{l}
T_s \leftarrow \dfrac{SD1}{SDS} \\
T_o \leftarrow 0.2 \cdot T_s \\
n_{max} \leftarrow \dfrac{Tmax}{Dt} \\
\text{for } i \in 1 .. n_{max} \\
\quad \left| \begin{array}{l}
T_i \leftarrow Dt \cdot i \\
a_i \leftarrow (SDS - A_s) \cdot \dfrac{Dt \cdot i}{T_o} + A_s \ \text{ if } Dt \cdot i < T_o \\
a_i \leftarrow SDS \ \text{ if } Dt \cdot i \ge T_o \wedge Dt \cdot i \le T_s \\
a_i \leftarrow \dfrac{SD1}{Dt \cdot i} \ \text{ if } Dt \cdot i > T_s
\end{array} \right. \\
R \leftarrow \text{augment}(T, a) \\
R
\end{array} \right.
$$

$\text{BridgeSpectrum} := \text{DesignSpectrum}(SDS, SD1, A_s, Tmax, Dt)$



***Article 3.10.6*: Selection of Seismic Performance Zones**

$SD1 = 0.221 \quad g$ From Table 3.10.6-1 Choose SPZ

$$
SDCprogram(SD1) := \left| \begin{array}{l} \text{for } c \in SD1 \\ \quad \left| \begin{array}{l} c \leftarrow "1" \quad \text{if } SD1 \le 0.15 \\ c \leftarrow "2" \quad \text{if } SD1 > 0.15 \wedge SD1 \le 0.3 \\ c \leftarrow "3" \quad \text{if } SD1 > 0.3 \wedge SD1 \le 0.5 \\ c \leftarrow "4" \quad \text{if } SD1 > 0.5 \end{array} \right. \\ Rs \leftarrow c \\ c \end{array} \right.
$$

$SDC := SDCprogram(SD1) = "2"$

Article 3.10.5: Bridge Importance Category

Operational Classified: **Other bridges**

Article 3.10.7: Response Modification Factors

For Substructures: Table 3.10.7.1-1

INPUT	Multiple Column Bents	$R_{sub} := 5.0$

For Connections: Table 3.10.7.1-2

INPUT	Superstructure to Abutment	$R_{abutment} := 0.8$
INPUT	Columns to Bent Cap	$R_{columncap} := 1.0$
INPUT	Column to foundation	$R_{foundation} := 1.0$

Article 4.7.4.3: Multispan Bridges

Article 4.7.4.3.1 Selection of Method

Refer to Table 4.7.4.3.1-1 to select the required analysis procedure. This a function of seismic performance zone, regularity, and operational classification. For the worst case in Alabama, we can use either the Uniform Load Elastic Method or Single-Mode Elastic Method.

Note: There are two methods that can be used according to this procedure. The Uniform Load Method is suitable for regular bridges that respond principally in their fundamental mode of vibration. The Single Mode Spectral Method may be a better method if there is a major change in the spans, stiffness of the piers, etc.
The Uniform Load Method is simpler and less time consuming and will give accurate results, and this is the reason it has been chosen in this design.

Article 4.7.4.3.2c **Uniform Load Method**

Step 1: Build a bridge model
Step 2: Apply a uniform load of Po = 1.0 kip/in. in both the longitudinal and transverse direction. Also, the uniform load can be converted into point loads and applied as joint loads in SAP. Calculate the static displacement for both directions. In SAP, tables of the displacements can be exported to EXCEL, and the MAX Function can be used to find the maximum displacement.

Step 3: Calculate the bridge lateral stiffness, K, and total weight, W.

$$p_o := 1.0 \quad \frac{kip}{in}$$

$$\upsilon_{smaxLong} := 0.382075 \quad \text{in}$$

INPUT

$$\upsilon_{smaxTran} := 4.330046 \quad \text{in}$$

$$K_{Long} := \frac{p_o \cdot L}{\upsilon_{smaxLong}} = 1.633 \times 10^{4} \qquad \frac{\text{kip}}{\text{in}}$$

$$K_{Tran} := \frac{p_o \cdot L}{\upsilon_{smaxTran}} = 1.441 \times 10^{3} \qquad \frac{\text{kip}}{\text{in}}$$

INPUT: Multiplying factors

$$W := \frac{\rho_{conc} \cdot \begin{bmatrix} L \cdot \left(t_{deck} \cdot DeckWidth + GirderArea \cdot 6 + GuardRailArea\right) + 3 \cdot BentVolume \; ... \\ + 4 \cdot Acolumn1 \cdot (2ColumnHeight2 + 2 \cdot ColumnHeight4) + 2Acolumn2 \cdot ColumnHeight3 \; ... \\ + Strut1 \cdot 2 + Strut2 \end{bmatrix}}{1000}$$

$$W = 7285.919 \qquad \text{kips}$$

Step 4: Calculate the period, T_m.

$$T_{mLong} := 2\pi \cdot \sqrt{\frac{W}{K_{Long} \cdot g}} = 0.213 \qquad s$$

Step 5: Calculate equivalent static earthquake loading p_e.

$$acc\left(SDS, SD1, T_{mLong}, A_s\right) := \left| \begin{array}{l} T_s \leftarrow \dfrac{SD1}{SDS} \\ T_o \leftarrow 0.2 \cdot T_s \\ \text{for } a \in T_{mLong} \\ \quad \left| \begin{array}{l} a \leftarrow \left(SDS - A_s\right) \cdot \dfrac{T_{mLong}}{T_o} + A_s \;\; \text{if } T_{mLong} < T_o \\ a \leftarrow SDS \;\; \text{if } T_{mLong} \ge T_o \wedge T_{mLong} \le T_s \\ a \leftarrow \dfrac{SD1}{T_{mLong}} \;\; \text{if } T_{mLong} > T_s \end{array} \right. \\ Ra \leftarrow a \\ a \end{array} \right.$$

$$Csm_{Long} := acc(SDS, SD1, T_{mLong}, A_s) = 0.43$$

$$p_{eLong} := \frac{Csm_{Long} \cdot W}{L} = 0.502 \qquad \frac{kip}{in}$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$v_{smaxLong} := \frac{p_{eLong}}{p_o} \cdot \upsilon_{smaxLong} = 0.192 \qquad in$$

Repeat Steps 4, 5, and 6 for transverse loading.

Step 4: Calculate the period, T_m.

$$T_{mTran} := 2\pi \cdot \sqrt{\frac{W}{K_{Tran} \cdot g}} = 0.719 \qquad s$$

Step 5: Calculate equivalent static earthquake loading p_e.

$$Csm_{Tran} := acc(SDS, SD1, T_{mTran}, A_s) = 0.307$$

$$p_{eTran} := \frac{Csm_{Tran} \cdot W}{L} = 0.359 \qquad \frac{kip}{in}$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$v_{smaxTran} := \frac{p_{eTran}}{p_o} \cdot \upsilon_{smaxTran} = 1.553 \qquad in$$

Single-Mode Spectral Method

Article 4.7.4.3.2: Single-Mode Spectral Method

Step 1: Build a bridge model
Step 2: Apply a uniform load of Po = 1.0 in both the longitudinal and transverse direction. Calculate the static displacement for both directions.

Step 3: Calculate factors α, β , and γ.

Note: The Deflection equations come from analysis of the SAP model. The displacement is taken at the joints along the length of the bridge and input into an Excel Worksheet. Then a graph is created of the displacements along the length of the bridge. A best fit line is plotted, and that is the equation that is shown below.

INPUT $v_{stran}(x) := -3\cdot 10^{-7}\cdot x^2 + 0.0016\cdot x + 1.4093$ $v_{slong}(x) := -1\cdot 10^{-8}\cdot x^2 + 0.0001x + 0.1563$

C4.7.4.3.2b-1 $\alpha_{Tran} := \int_0^L v_{stran}(x)\,dx$ $\alpha_{Long} := \int_0^L v_{slong}(x)\,dx$

C4.7.4.3.2b-2 $\beta_{Tran} := \int_0^L \frac{W}{L} v_{stran}(x)\,dx$ $\beta_{Long} := \int_0^L \frac{W}{L}\cdot v_{slong}(x)\,dx$

C4.7.4.3.2b-3 $\gamma_{Tran} := \int_0^L \frac{W}{L}\cdot v_{stran}(x)^2\,dx = 5.308\times 10^4$ $\gamma_{Long} := \int_0^L \frac{W}{L}\cdot v_{slong}(x)^2\,dx$

α = Displacement along the length

β = Weight per unit length * Displacement

γ = Weight per unit length * Displacement^2

Step 4: Calculate the Period of the Bridge

$$T_{mTran1} := 2\pi\cdot\sqrt{\frac{\gamma_{Tran}}{p_o\cdot g\cdot \alpha_{Tran}}} = 0.589 \quad s$$

$$T_{mLong1} := 2\pi\cdot\sqrt{\frac{\gamma_{Long}}{p_o\cdot g\cdot \alpha_{Long}}} = 0.206 \quad s$$

Step 5: Calculate the equivalent Static Earthquake Loading

$C_{smLong} := acc(SDS, SD1, T_{mLong1}, A_s) = 0.43$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$PeLong(x) := \frac{\beta_{Long} \cdot C_{smLong}}{\gamma_{Long}} \cdot \frac{W}{L} \cdot v_{slong}(x)$$

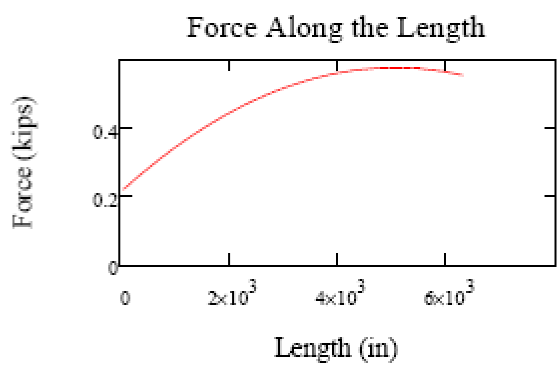
$PeLong(x) \rightarrow 0.00014153071449361569124 \cdot x + -1.4153071449361569124e\text{-}8 \cdot x^2 + 0.221212506753521325$

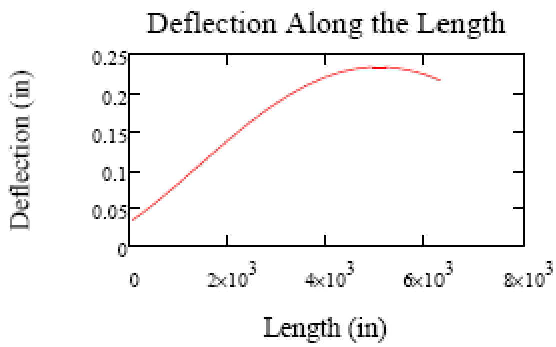
$$dW := \frac{L}{100}$$

$i := 1 .. 101$

$Pelong_i := PeLong[(i-1) \cdot dW]$ $\qquad$ $\delta long_i := v_{slong}[(i-1)dW]$

$\Delta long_i := Pelong_i \cdot \delta long_i$





Maximum Deflection

$$\max(\Delta long) = 0.234 \quad in$$

NOTE: Repeat Steps 5 and 6 for Transverse Direction.

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smTran} := acc(SDS, SD1, T_{mTran1}, A_s) = 0.375$$

Step 6: Calculate the displacements and member forces for use in design by applying p_e to the model or by scaling the results by p_e/p_o.

$$PeTran(x) := \frac{\beta_{Tran} \cdot C_{smTran}}{\gamma_{Tran}} \cdot \frac{W}{L} \cdot v_{stran}(x)$$

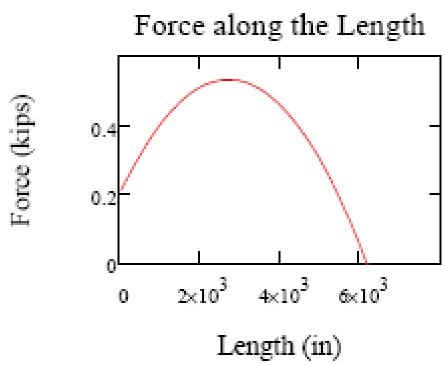
$$PeTran(x) \rightarrow 0.00024113243850551203633 \cdot x + -4.5212332219783506812e\text{-}8 \cdot x^2 + 0.21239246599113632 05$$

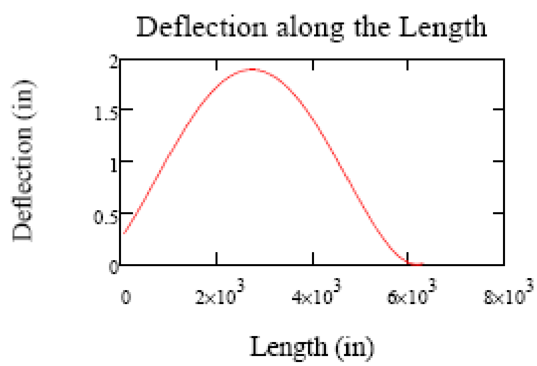
$$dL := \frac{L}{100}$$

$$i := 1..101$$

$$Petran_i := PeTran[(i-1) \cdot dL] \qquad \delta tran_i := v_{stran}[(i-1)dL]$$

$$\Delta tran_i := Petran_i \cdot \delta tran_i$$





Maximum Deflection

$\max(\Delta\text{tran}) = 1.891$ in

Article 3.10.8: Combination of Seismic Force Effects

$$\text{LoadCase1} := \sqrt{\left(1.0 \cdot p_{eTran}\right)^2 + \left(0.3 \cdot p_{eLong}\right)^2} = 0.389 \quad \frac{\text{kip}}{\text{in}}$$

$$\text{LoadCase2} := \sqrt{\left(1.0 \cdot p_{eLong}\right)^2 + \left(0.3 \cdot p_{eTran}\right)^2} = 0.513 \quad \frac{\text{kip}}{\text{in}}$$

Article 3.10.9.3: Determine Design Forces

$$\text{MaxLoadCase}(x,y) := \left| \begin{array}{l} a \leftarrow x \;\text{ if }\; x \ge y \\ a \leftarrow y \;\text{ if }\; y \ge x \\ a \end{array} \right.$$

$$\text{NominalForce} := \text{MaxLoadCase}(\text{LoadCase1}, \text{LoadCase2}) = 0.513 \qquad \frac{\text{kip}}{\text{in}}$$

Note: The Req values are factors that will be used to multiple loads that come out of SAP 2000. The NominalForce variable is truly just a factor also. It is easier to apply these factors to the loading for po than to change the loading in SAP 2000 because if an error is made in calculating pe then the loads will have to re-entered.

Multiple Column Bents $$\text{Req}_{\text{substructure}} := \frac{\text{NominalForce}}{R_{\text{sub}}} = 0.103$$

Note: Article 3.10.9.3 specifies that the Drilled shafts be designed for half of the R value. R/2 also must not be taken less than 1.

$$\text{Req}_{\text{DrilledShafts}} := \frac{\text{NominalForce}}{R_{\text{sub}} \cdot 0.5} = 0.205$$

Connections

Superstructure to Abutment $$\text{Req}_{\text{subtoabutcon}} := \frac{\text{NominalForce}}{R_{\text{abutment}}} = 0.642$$

Columns to Bent Cap $$\text{Req}_{\text{coltocapcon}} := \frac{\text{NominalForce}}{R_{\text{columncap}}} = 0.513$$

Column to foundation $$\text{Req}_{\text{coltofoundcon}} := \frac{\text{NominalForce}}{R_{\text{foundation}}} = 0.513$$

LOADS FOR DESIGN

NOTE: All the loads calculated in this section are for a single column or drilled shaft.

COLUMN SHEAR PROGRAM

$$\mathrm{Shear}(Vu, Reqsubstructure) := \left| \begin{array}{l} a \leftarrow Vu \cdot Reqsubstructure \\ a \end{array} \right.$$

AXIAL LOAD PROGRAM

$$\mathrm{PDEAD}(Peq, Pd, Rsub, Reqsubstructure) := \left| \begin{array}{l} a \leftarrow \left(\left| Peq \cdot Reqsubstructure - \frac{Pd}{Rsub} \right| \right) \\ a \end{array} \right.$$

Note: The axial load program calculates the minimum axial load on the column. This will needed later in the design process.

BENT 2

Note: Input the maximum shear value between all the columns in the bent from SAP which have not been multiplied by pe/po or divided by the R factor. This will be taken care of in one of the equations below.

INPUT $Vu_{Bent2} := 894$ kips

Note: The Axial load consists of both the Earthquake effect and dead load effect. It should be the minimum axial load the column will encounter during an earthquake.

INPUT $Pueq_{Bent2} := 2431$ kips

INPUT $Pudead_{Bent2} := 853$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{Bent2} := \mathrm{PDEAD}\left(Pueq_{Bent2}, Pudead_{Bent2}, R_{sub}, Req_{substructure}\right) = 78.92$ kips

INPUT $ColumnHeight_{Bent2} := \frac{ColumnHeight2}{12} = 34.022$ ft

Note: Input the correct variables into the below program to get the shear in the column or drilled shaft.

INPUT

$Vucol_{Bent2} := \mathrm{Shear}\left(Vu_{Bent2}, Req_{substructure}\right) = 91.761$ kips

BENT 3

Note: Input the maximum shear value between all the columns in the bent from SAP which have not been multiplied by pe/po or divided by the R factor. This will be taken care in of one of the equations below.

INPUT $Vu_{Bent3} := 623$ kips

Note: The Axial load consists of both the Earthquake effect and dead load effect. It should be the minimum axial load the column will encounter during an earthquake.

INPUT $Pueq_{Bent3} := 2895$ kips

INPUT $Pudead_{Bent3} := 1055$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{Bent3} := PDEAD\left(Pueq_{Bent3}, Pudead_{Bent3}, R_{sub}, Req_{substructure}\right) = 86.145$ kips

INPUT $ColumnHeight_{Bent3} := \frac{ColumnHeight3}{12} = 59.136$ ft

Note: Input the correct variables into the below program to get the shear in the column or drilled shaft.

INPUT

$Vucol_{Bent3} := Shear\left(Vu_{Bent3}, Req_{substructure}\right) = 63.945$ kips

BENT 4

Note: Input the maximum shear value between all the columns in the bent from SAP which have not been multiplied by pe/po or divided by the R factor. This will be taken care in of one of the equations below.

INPUT $Vu_{Bent4} := 948$ kips

Note: The Axial load consists of both the Earthquake effect and dead load effect. It should be the minimum axial load the column will encounter during an earthquake.

INPUT $Pueq_{Bent4} := 2760$ kips

INPUT $Pudead_{Bent4} := 872$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{Bent4} := PDEAD(Pueq_{Bent4}, Pudead_{Bent4}, R_{sub}, Req_{substructure}) = 108.888$ kips

INPUT $ColumnHeight_{Bent4} := \frac{ColumnHeight4}{12} = 32.156$ ft

Note: Input the correct variables into the below program to get the shear in the column or drilled shaft.

INPUT

$Vucol_{Bent4} := Shear(Vu_{Bent4}, Req_{substructure}) = 97.303$ kips

DRILLED SHAFT 2

INPUT $Pueq_{DS2} := 2431$ kips

INPUT $Pudead_{DS2} := 872$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{DS2} := PDEAD(Pueq_{DS2}, Pudead_{DS2}, R_{sub}, Req_{DrilledShafts}) = 324.639$ kips

INPUT $ColumnHeight_{DS2} := 14$ ft

Note: The drilled shaft is design for an R/2; therefore, the shear in the column can be doubled to equal the design shear for the drilled shaft.

INPUT $Vu_{DS2} := Vucol_{Bent2} \cdot 2 = 183.522$ kips

DRILLED SHAFT 3

INPUT $Pueq_{DS3} := 2895$ kips

INPUT $Pudead_{DS3} := 1075$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{DS3} := PDEAD(Pueq_{DS3}, Pudead_{DS3}, R_{sub}, Req_{DrilledShafts}) = 379.29$ kips

INPUT $ColumnHeight_{DS3} := 17$ ft

Note: The drilled shaft is design for an R/2; therefore, the shear in the column can be doubled to equal the design shear for the drilled shaft.

INPUT $Vu_{DS3} := Vucol_{Bent3} \cdot 2 = 127.89$ kips

DRILLED SHAFT 4

INPUT $Pueq_{DS4} := 2760$ kips

INPUT $Pudead_{DS4} := 884$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{DS4} := PDEAD(Pueq_{DS4}, Pudead_{DS4}, R_{sub}, Req_{DrilledShafts}) = 389.777$ kips

INPUT $ColumnHeight_{DS4} := 16$ ft

Note: The drilled shaft is design for an R/2; therefore, the shear in the column can be doubled to equal the design shear for the drilled shaft.

INPUT $Vu_{DS4} := Vucol_{Bent4} \cdot 2 = 194.607$ kips

ABUTMENT 1 DRILLED SHAFT

Note: Input the loads from SAP which have not been multiplied by pe/po or divided by the R factor. This will be taken care of one of the equations below.

INPUT $Vu_{Abut1} := 417$ kips

INPUT $ColumnHeight_{Abut1} := 12.5$ ft

Note: Input the correct variables into the below program to get the shear in the column or drilled shaft.

INPUT

$Vu_{DSAbut1} := \text{Shear}\left(Vu_{Abut1}, Req_{subtoabutcon}\right) = 267.507$ kips

ABUTMENT 5 DRILLED SHAFT

Note: Input the loads from SAP which have not been multiplied by pe/po or divided by the R factor. This will be taken care of one of the equations below.

INPUT $Vu_{Abut5} := 274$ kips

INPUT $ColumnHeight_{Abut2} := 27$ ft

Note: Input the correct variables into the below program to get the shear in the column or drilled shaft.

INPUT

$Vu_{DSAbut5} := \text{Shear}\left(Vu_{Abut5}, Req_{subtoabutcon}\right) = 175.772$ kips

BENT 2 STRUT

Note: Input the maximum shear value between all the columns in the bent from SAP which have not been multiplied by pe/po or divided by the R factor. This will be taken care in of one of the equations below.

INPUT $Vu_{Strut2} := 901$ kips

Note: The Axial load consists of both the Earthquake effect and dead load effect. It should be the minimum axial load the column will encounter during an earthquake.

INPUT $Pueq_{Strut2} := 598$ kips

INPUT $Pudead_{Strut2} := 11$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{Strut2} := \text{PDEAD}\left(Pueq_{Strut2}, Pudead_{Strut2}, R_{sub}, Req_{substructure}\right) = 59.179$

Note: Input the correct variables into the below program to get the shear in the column or drilled shaft.

INPUT

$Vucol_{Strut2} := Shear(Vu_{Strut2}, Req_{substructure}) = 92.479$ kips

BENT 3 STRUT

Note: Input the maximum shear value between all the columns in the bent from SAP which have not been multiplied by pe/po or divided by the R factor. This will be taken care in of one of the equations below.

INPUT $Vu_{Strut3} := 1516$ kips

Note: The Axial load consists of both the Earthquake effect and dead load effect. It should be the minimum axial load the column will encounter during an earthquake.

INPUT $Pueq_{Strut3} := 125$ kips

INPUT $Pudead_{Strut3} := 6$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{Strut3} := PDEAD(Pueq_{Strut3}, Pudead_{Strut2}, R_{sub}, Req_{substructure}) = 10.63$

Note: Input the correct variables into the below program to get the shear in the column or drilled shaft.

INPUT

$Vucol_{Strut3} := Shear(Vu_{Strut3}, Req_{substructure}) = 155.603$ kips

BENT 4 STRUT

Note: Input the maximum shear value between all the columns in the bent from SAP which have not been multiplied by pe/po or divided by the R factor. This will be taken care in of one of the equations below.

INPUT $Vu_{Strut4} := 967$ kips

Note: The Axial load consists of both the Earthquake effect and dead load effect. It should be the minimum axial load the column will encounter during an earthquake.

INPUT $Pueq_{Strut4} := 605$ kips

INPUT $Pudead_{Strut4} := 9$ kips

Note: Input the correct variables into the below program to get the axial load in the column or drilled shaft.

INPUT $Pumin_{Strut4} := PDEAD\left(Pueq_{Strut4}, Pudead_{Strut4}, R_{sub}, Req_{substructure}\right) = 60.298$

Note: Input the correct variables into the below program to get the shear in the column or drilled shaft.

INPUT

$Vucol_{Strut4} := Shear\left(Vu_{Strut4}, Req_{substructure}\right) = 99.254$ kips

Article 4.7.4.4: Minimum Support Length Requirements

Note: May need to add more calculations if column heights are different at the bents.

N (in) = Minimum support length measured normal to the bridge

L (ft) = Length of bridge to adjacent expansion joint or end of the bridge

H (ft) = average height of columns supporting bridge deck for abutments
for columns and piers = column height

S (Degree) = angle of skew

Abutment Support Length Requirement

$Span_{abutment} := \frac{Span}{12} = 130$ ft $H_{abutment} := \frac{ColumnHeight2}{12} = 34.022$ ft

Note: The $Span_{abutment}$ is divided by number of spans and inches.

$Skew_{abutment} := 0$ Degrees

$Nabutment := 1.5\cdot\left(8 + 0.02Span_{abutment} + 0.08H_{abutment}\right)\cdot\left(1 + 0.000125Skew_{abutment}{}^{2}\right) = 19.983$ in

Bent Support Length Requirement

BENT 2

INPUT $Span_{Bent} := \frac{Span}{12} = 130$ ft

Note: The $Span_{abutment}$ is divided by number of spans and inches.

INPUT $H_{Bent} := \frac{ColumnHeight2}{12} = 34.022$ ft *INPUT*: Column Height for this Bent

INPUT $Skew_{Bent} := 0$ Degrees

$$N_{Bent} := 1.5 \cdot (8 + 0.02 Span_{Bent} + 0.08 H_{Bent}) \cdot \left(1 + 0.000125 Skew_{Bent}^{2}\right) = 19.98 \text{ in}$$

BENT 3

INPUT $Span_{Bent} := \frac{Span}{12} = 130$ ft

Note: The $Span_{abutment}$ is divided by number of spans and inches.

INPUT $H_{Bent} := \frac{ColumnHeight3}{12} = 59.136$ ft *INPUT*: Column Height for this Bent

INPUT $Skew_{Bent} := 0$ Degrees

$$N_{Bent} := 1.5 \cdot (8 + 0.02 Span_{Bent} + 0.08 H_{Bent}) \cdot \left(1 + 0.000125 Skew_{Bent}^{2}\right) = 22.996 \text{ in}$$

BENT 4

INPUT $Span_{Bent} := \frac{Span}{12} = 130$ ft

Note: The $Span_{abutment}$ is divided by number of spans and inches.

INPUT $H_{Bent} := \frac{ColumnHeight4}{12} = 32.156$ ft *INPUT*: Column Height for this Bent

INPUT $Skew_{Bent} := 0$ Degrees

$$N_{Bent} := 1.5 \cdot (8 + 0.02 Span_{Bent} + 0.08 H_{Bent}) \cdot \left(1 + 0.000125 Skew_{Bent}^{2}\right) = 19.759 \text{ in}$$

BENT 2 DESIGN

Article 5.10.11.3: Provisions for Seismic Design for Seismic Zone 2

Article 5.10.11.3: Longitudinal Reinforcement

INPUT $A_{longbar} := 1.56$ in^2

INPUT $N_{bars} := 24$

INPUT $Ads := Acolumn1$

$$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 37.44 \quad in^2$$

Minimum Longitudinal Reinforcing Check

$$\text{Checkleastlongreinforcing}(Ag, Along) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } Along \geq Ag \cdot 0.01 \\ a \leftarrow \text{"Increase Longitudinal Reinforcing Ratio"} \quad \text{if } Along < 0.01 \cdot Ag \\ a \end{array} \right.$$

$$MinLongRatio := \text{Checkleastlongreinforcing}\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars</u> in the inputs.

Maximum Longitudinal Reinforcing Check

$$\text{Checkmaxlongreinforcing}(Ag, Along) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } Along \leq 0.06 \cdot Ag \\ a \leftarrow \text{"Decrease Longitudinal Reinforcing Ratio"} \quad \text{if } Along > 0.06 \cdot Ag \\ a \end{array} \right.$$

$$MaxLongRatio := \text{Checkmaxlongreinforcing}\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either <u>increase the section size (Ag)</u> or <u>decrease the longitudinal reinforcing (Abl and NumberBars)</u> in the inputs.

Article 5.10.11.4.1b: Flexural Resistance

Check biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.9.4.
Flexural Resistance check can be done using some kind of Column Design Program. PCA Column was used for this project. After creating an interaction diagram, verify that all the critical load combinations fall within the diagram.

Article 5.10.11.4.1c: Column Shear and Transverse Reinforcement

Note: This is for the end regions of the column.

Article 5.8.3.3: Nominal Shear Resistance

$$Vu := Vucol_{Bent2} = 91.761 \quad \text{kips}$$

$$Pumin_{Bent2} = 78.92 \quad \text{kips}$$

INPUT $bv := Columndia1$ bv: effective width

INPUT $\phi_s := 0.9$

INPUT $s := 4$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $Asp := .44$ in^2 Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.725$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 3$ in Cover: Concrete cover for the Column (in)

INPUT $Dprime := 54$ in Dprime: Diameter of spiral or hoop for circular columns (in)

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 55.57 \quad \text{in}$$

(Equation: C5.8.2.9-2) $de := \frac{bv}{2} + \frac{Dr}{\pi} = 47.688$ in

$$dv := 0.9 \cdot de = 42.92 \quad \text{in}$$

Article 5.10.11.4.1c:

$$\mathrm{VcProgram}(fc, \beta, bv, dv, Ag, Pu) := \left| \begin{array}{l} p \leftarrow Pu \\ v \leftarrow 0.0316 \cdot \beta \cdot \sqrt{\dfrac{fc}{1000}} bv \cdot dv \\ c \leftarrow 0.1 \cdot Ag \cdot \dfrac{fc}{1000} \\ x \leftarrow \dfrac{p \cdot v}{c} \\ a \leftarrow v \ \text{ if } \ p > c \\ a \leftarrow x \ \text{ if } \ p \le c \\ a \end{array} \right.$$

Note: The area of the Column and the Axial Load for this column need to be input into the Vc equation that calls the program above.

$$V_c := \mathrm{VcProgram}\left(fc, \beta, bv, dv, Ads, Pumin_{Bent2}\right) = 22.714 \qquad \text{kips}$$

Eq. 5.8.3.3-4

$$V_s := \frac{2Asp \cdot \dfrac{fye}{1000} dv \cdot \cot(\theta)}{s} = 566.539 \qquad \text{kips}$$

Eq. 5.8.3.3-1

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 530.328 \qquad \text{kips}$$

$$\mathrm{ShearCheck}\left(\phi Vn, V_u\right) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } \ \phi Vn \ge V_u \\ a \leftarrow \text{"FAILURE"} \quad \text{if } \ \phi Vn < V_u \\ a \end{array} \right.$$

$$\mathrm{Shearcheck} := \mathrm{ShearCheck}\left(\phi V_n, Vu\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

Article 5.10.11.4.1c Length of End Region (Plastic Hinge Region)

End region is assumed to extend from the soffit of girders or cap beams at the top of the columns or from the top of foundations at the bottom of columns.

$$\mathrm{EndRegionProgram}(d,H) := \left| \begin{array}{l} x \leftarrow d \\ y \leftarrow \frac{1}{6}H \cdot 12 \\ z \leftarrow 18 \\ a \leftarrow \max(x,y,z) \\ a \end{array} \right.$$

$$\mathrm{LendgthEndRegion} := \mathrm{EndRegionProgram}\left(\mathrm{Columndia1}, \mathrm{ColumnHeight}_{\mathrm{Bent2}}\right) = 68.044 \quad \mathrm{in}$$

Article 5.10.11.4.3: Column Connections

Extension into Top and Bottom Connections

Note: This needs to be done whenever the column dimension changes.

$$\mathrm{ExtensionProgram}(d) := \left| \begin{array}{l} z \leftarrow 15 \\ x \leftarrow \frac{1}{2} \cdot d \\ a \leftarrow \max(z,x) \\ a \end{array} \right.$$

$$\mathrm{Extension} := \mathrm{ExtensionProgram}(bv) = 30 \qquad \mathrm{in}$$

Article 5.10.11.4.1e: Spacing of Transverse Reinforcement for Confinement

Transverse Reinforcement for Confinement

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

Shall Not Exceed the Smallest of:

$$\mathrm{Spacingprogram}(\mathrm{Columndia}) := \left| \begin{array}{l} x \leftarrow \frac{1}{4} \cdot \mathrm{Columndia} \\ y \leftarrow 4 \\ a \leftarrow \min(x,y) \\ a \end{array} \right.$$

$$\mathrm{MaximumSpacing} := \mathrm{Spacingprogram}(bv) = 4 \qquad \mathrm{in}$$

$$\text{SpacingCheck(MaximumSpacing, s)} := \left| \begin{array}{l} a \leftarrow s \quad \text{if } s \le \text{MaximumSpacing} \\ a \leftarrow \text{MaximumSpacing} \quad \text{if } s > \text{MaximumSpacing} \\ a \end{array} \right.$$

$$\text{FINALSPACING} := \text{SpacingCheck(MaximumSpacing, s)} = 4 \qquad \text{in}$$

$$\text{scheck} := \text{ShearCheck(MaximumSpacing, s)} = \text{"OK"}$$

Note: If scheck returns "Failure", increase the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.1d: Transverse Reinforcement for Confinement at Plastic Hinges

Required Volumetric Ratio of Spiral or Seismic Hoop Reinforcing

$$\rho_s := \frac{4 \cdot \text{Asp}}{s \cdot \text{Dprime}} = 8.148 \times 10^{-3}$$

$$\text{RatioProgram}(fc, fy, \rho_s) := \left| \begin{array}{l} z \leftarrow 0.12 \cdot \frac{fc}{fy} \\ a \leftarrow \text{"OK"} \quad \text{if } \rho_s \ge z \\ a \leftarrow \text{"Increase Transverse Reinforcing Ratio"} \quad \text{if } \rho_s < z \\ a \end{array} \right.$$

$$\text{Check}\rho_s := \text{RatioProgram}(fc, fye, \rho_s) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Increase Transverse Reinforcing Ratio", it is recommended to decrease the spacing (s) or increase the area of the transverse reinforcement (Asp) in the inputs.

Note:These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension of **NOT** less than **6*d_b** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Article 5.10.11.4.1f: Splices

Lap Splices in longitudinal reinforcement shall not be used in plastic hinge region.

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.

5.8.3.3 **Nominal Shear Resistance**

INPUT $Vu_{sub} := Vucol_{Bent2} = 91.761$ kips

INPUT $spaceNOhinge := 12$ in

Note: β and θ come from Article 5.8.3.4.1

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 55.57 \quad in$$

Eq. C5.8.2.9-2

$$de := \frac{bv}{2} + \frac{Dr}{\pi} = 47.688 \quad in$$

$$dv := 0.9 \cdot de = 42.92 \quad in$$

Eq. 5.8.3.3-3

$$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 325.502 \quad kips$$

Eq. 5.8.3.3-4

$$V_s := \frac{Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 94.423 \quad kips$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 377.933 \quad kips$$

$$ShearCheck\left(\phi Vn, V_u\right) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } \phi Vn \ge V_u \\ a \leftarrow \text{"FAILURE"} \quad \text{if } \phi Vn < V_u \\ a \end{array} \right.$$

$$Shearcheck := ShearCheck\left(\phi V_n, Vu_{sub}\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1

$$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.758 \quad in^2$$

$$Av := 2 \cdot Asp = 0.88 \quad in^2$$

$$\text{TranCheck}(\text{Avmin}, \text{Av}) := \left| \begin{array}{l} a \leftarrow \text{"Decrease Spacing or Increase Bar Size"} \quad \text{if } \text{Avmin} > \text{Av} \\ a \leftarrow \text{"OK"} \quad \text{if } \text{Avmin} \le \text{Av} \\ a \end{array} \right.$$

$$\text{MinimumTran} := \text{TranCheck}(\text{Avmin}, \text{Av}) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

***5.8.2.7* Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $\quad vu := \dfrac{Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 0.04$ ksi

$$\text{spacingProgram}(\text{Vu}, \text{dv}, \text{fc}) := \left| \begin{array}{l} v \leftarrow 0.125 \cdot \dfrac{fc}{1000} \\ q \leftarrow 0.8 \cdot dv \\ r \leftarrow 0.4 \cdot dv \\ z \leftarrow q \quad \text{if } q \le 24 \\ z \leftarrow 24 \quad \text{if } q > 24 \\ t \leftarrow r \quad \text{if } r \le 12 \\ t \leftarrow 12 \quad \text{if } r > 12 \\ a \leftarrow z \quad \text{if } Vu < v \\ a \leftarrow t \quad \text{if } Vu \ge v \\ a \end{array} \right.$$

$\text{MaxSpacing} := \text{spacingProgram}(vu, dv, fc) = 24$ in

$$\text{Spacecheck}(\text{MaxSpacing}, s) := \left| \begin{array}{l} a \leftarrow s \quad \text{if } s \le \text{MaxSpacing} \\ a \leftarrow \text{MaxSpacing} \quad \text{if } s > \text{MaxSpacing} \\ a \end{array} \right.$$

$\text{MAXSPACING} := \text{Spacecheck}(\text{MaxSpacing}, \text{spaceNOhinge}) = 12$ in

$$\text{scheck} := \text{ShearCheck}(\text{MAXSPACING}, \text{spaceNOhinge}) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

STRUT 2 DESIGN

Article 5.10.11.3: Provisions for Seismic Design for Seismic Zone 2

Article 5.10.11.3: Longitudinal Reinforcement

INPUT $A5_{bl} := 0.31 \quad in^2$

INPUT $Number\_5\_Bars := 20$

INPUT $A11_{bl} := 1.56 \quad in^2$

INPUT $Number\_11\_Bars := 16$

$$A_{long} := A5_{bl} \cdot Number\_5\_Bars + A11_{bl} \cdot Number\_11\_Bars = 31.16 \quad in^2$$

$$Ag := Strut2Depth \cdot Strut2Width = 3.024 \times 10^3 \quad in^2$$

Minimum Longitudinal Reinforcing Check

$$Checkleastlongreinforcing(Ag, Along) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } Along \ge Ag \cdot 0.01 \\ a \leftarrow \text{"Increase Longitudinal Reinforcing Ratio"} \quad \text{if } Along < 0.01 \cdot Ag \\ a \end{array} \right.$$

$$MinLongRatio := Checkleastlongreinforcing\left(Ag, A_{long}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars</u> in the inputs.

Maximum Longitudinal Reinforcing Check

$$Checkmaxlongreinforcing(Ag, Along) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } Along \le 0.06 \cdot Ag \\ a \leftarrow \text{"Decrease Longitudinal Reinforcing Ratio"} \quad \text{if } Along > 0.06 \cdot Ag \\ a \end{array} \right.$$

$$MaxLongRatio := Checkmaxlongreinforcing\left(Ag, A_{long}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either <u>increase the section size (Ag)</u> or <u>decrease the longitudinal reinforcing (Abl and NumberBars)</u> in the inputs.

Article 5.10.11.4.1b: Flexural Resistance

Check biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.9.4.
Flexural Resistance check can be done using some kind of Column Design Program. PCA Column was used for this project. After creating an interaction diagram, verify that all the critical load combinations fall within the diagram.

Article 5.10.11.4.1c: Column Shear and Transverse Reinforcement

Note: This is for the end regions of the column.

Article 5.8.3.3: Nominal Shear Resistance

$Vu := Vucol_{Strut2} = 92.479$ kips

$Pumin_{Strut2} = 59.179$ kips

INPUT	$bv := Strut2Width$		bv: effective width
INPUT	$\phi_s := 0.9$		
INPUT	$s := 3.5$	in	s: Spacing of hoops or pitch of spiral (in)
INPUT	$Asp := 0.44$	in^2	Asp: Area of spiral or hoop reinforcing (in^2)
INPUT	$Dsp := 0.75$	in	Dsp: Diameter of spiral or hoop reinforcing (in)
INPUT	$Cover := 2$	in	Cover: Concrete cover for the Column (in)
INPUT	$Dprime := 54$	in	Dprime: Diameter of spiral or hoop for circular columns (in)
INPUT	$d_{bl} := 1.41$	in	d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$\beta := 2.0$

$\theta := \frac{\pi}{180} \cdot 45 = 0.785$ rad

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

Eq. 5.8.2.9-2 $\quad de := 69.4 \quad \text{in}$ $\quad$ de = ds which is the distance from top of the member to the centriod of the tensile fiber

$dvpreliminary := 66.75 \quad \text{in}$ $\quad$ dvpreliminary = distance between compressive and tensile reinforcing

$$dvprogram(de, dv, h) := \left| \begin{array}{l} x \leftarrow 0.9 \cdot de \\ y \leftarrow 0.75 \cdot h \\ z \leftarrow \max(x, y) \\ a \leftarrow dv \quad \text{if } dv \geq z \\ a \leftarrow z \quad \text{if } dv < z \\ a \end{array} \right.$$

$$dv := dvprogram(de, dvpreliminary, Strut2Depth) = 66.75 \qquad \text{in}$$

Article 5.10.11.4.1c:

$$VcProgram(fc, \beta, bv, dv, Ag, Pu) := \left| \begin{array}{l} p \leftarrow Pu \\ v \leftarrow 0.0316 \cdot \beta \cdot \sqrt{\dfrac{fc}{1000}} bv \cdot dv \\ c \leftarrow 0.1 \cdot Ag \cdot \dfrac{fc}{1000} \\ x \leftarrow \dfrac{p \cdot v}{c} \\ a \leftarrow v \quad \text{if } p > c \\ a \leftarrow x \quad \text{if } p \leq c \\ a \end{array} \right.$$

Note: The area of the Column and the Axial Load for this column need to be input into the Vc equation that calls the program above.

$$V_c := VcProgram\left(fc, \beta, bv, dv, Ag, Pumin_{Strut2}\right) = 17.337 \qquad \text{kips}$$

Eq. 5.8.3.3-4

$$V_s := \frac{2Asp \cdot \dfrac{fye}{1000} dv \cdot \cot(\theta)}{s} = 1.007 \times 10^{-} \text{kips}$$

Eq. 5.8.3.3-1 $\phi V_n := \left(V_c + V_s\right)\cdot\phi_s = 921.878$ kips

$\text{Shearcheck} := \text{ShearCheck}\left(\phi V_n, Vu\right) = \text{"OK"}$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

Article 5.10.11.4.1c Length of End Region (Plastic Hinge Region)

End region is assumed to extend from the soffit of girders or cap beams at the top of the columns or from the top of foundations at the bottom of columns.

$$\text{EndRegionProgram}(d, H) := \left| \begin{array}{l} x \leftarrow d \\ y \leftarrow \frac{1}{6} H \cdot 12 \\ z \leftarrow 18 \\ a \leftarrow \max(x, y, z) \\ a \end{array} \right.$$

$\text{LendgthEndRegion} := \text{EndRegionProgram}(\text{Strut2Depth}, \text{Lstrut2}) = 72$ in

Article 5.10.11.4.1e: Spacing of Transverse Reinforcement for Confinement

Transverse Reinforcement for Confinement

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

Shall Not Exceed the Smallest of:

$$\text{Spacingprogram(Columndia)} := \left| \begin{array}{l} x \leftarrow \frac{1}{4} \cdot \text{Columndia} \\ y \leftarrow 4 \\ a \leftarrow \min(x, y) \\ a \end{array} \right.$$

$$\text{MaximumSpacing} := \text{Spacingprogram(Strut2Width)} = 4 \quad \text{in}$$

$$\text{SpacingCheck(MaximumSpacing}, s) := \left| \begin{array}{l} a \leftarrow s \ \text{ if } \ s \le \text{MaximumSpacing} \\ a \leftarrow \text{MaximumSpacing} \ \text{ if } \ s > \text{MaximumSpacing} \\ a \end{array} \right.$$

$$\text{FINALSPACING} := \text{SpacingCheck(MaximumSpacing}, s) = 3.5 \quad \text{in}$$

$$\text{scheck} := \text{ShearCheck(MaximumSpacing}, s) = \text{"OK"}$$

Note: If scheck returns "Failure", increase the spacing of <u>shear reinforcing spacing (s)</u>. The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.1d: Transverse Reinforcement for Confinement at Plastic Hinges

Required Volumetric Ratio Seismic Hoop Reinforcing

INPUT $hc := 68 \quad \text{in}$ hc = core dimension of strut

INPUT $Ac := hc \cdot (\text{Strut2Width} - \text{Cover} \cdot 2) = 2.584 \times 10^3 \quad \text{in}^2$

$Av := 2 \cdot Asp = 0.88 \quad \text{in}^2$

$$\text{RectangularProgram}(fc, fy, s, Ag, Ac, Av, hc) := \left| \begin{array}{l} z \leftarrow 0.3 \cdot s \cdot hc \cdot \frac{fc}{fy}\left(\frac{Ag}{Ac} - 1\right) \\ a \leftarrow \text{"OK"} \quad \text{if } Av \geq z \\ a \leftarrow \text{"Increase Transverse Reinforcing Ratio"} \quad \text{if } Av < z \\ a \end{array} \right.$$

$$\text{Check}\rho_s := \text{RectangularProgram}(fc, fye, s, Ag, Ac, Av, hc) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Increase Transverse Reinforcing Ratio", it is recommended to decrease the spacing (s) or increase the area of the transverse reinforcement (Asp) in the inputs.

Note:These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension **of NOT** less than **6*d_b** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Article 5.10.11.4.1f: Splices

Lap Splices in longitudinal reinforcement shall not be used in plastic hinge region.

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.

***5.8.3.3* Nominal Shear Resistance**

INPUT $Vu_{sub} := Vucol_{Strut2} = 92.479$ kips

INPUT $spaceNOhinge := 12$ in

INPUT $bv := Strut2Width$

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$\beta := 2.0$

$\theta := \frac{\pi}{180} \cdot 45 = 0.785$ rad

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

Eq. 5.8.2.9-2 $de := 69.4$ in de = ds which is the distance from top of the member to the centriod of the tensile fiber

$dvpreliminary := 66.75$ in dvpreliminary = distance between compressive and tensile reinforcing

$$dvprogram(de, dv, h) := \left| \begin{array}{l} x \leftarrow 0.9 \cdot de \\ y \leftarrow 0.75 \cdot h \\ z \leftarrow \max(x, y) \\ a \leftarrow dv \ \text{ if } \ dv \geq z \\ a \leftarrow z \ \text{ if } \ dv < z \\ a \end{array} \right.$$

$dv := dvprogram(de, dvpreliminary, Strut2Depth) = 66.75$ in

Eq. 5.8.3.3-3 $V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 354.362$ kips

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp\cdot\frac{fye}{1000}dv\cdot\cot(\theta)}{spaceNOhinge} = 293.7 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right)\cdot\phi_s = 583.256 \quad \text{kips}$$

$$\text{Shearcheck} := \text{ShearCheck}\left(\phi V_n, Vu_{sub}\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316\cdot\sqrt{\frac{fc}{1000}}\cdot\frac{bv\cdot spaceNOhinge}{\frac{fye}{1000}} = 0.531 \quad in^2$$

$$Av := 2\cdot Asp = 0.88 \quad in^2$$

$$\text{MinimumTran} := \text{TranCheck}(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{Vu_{sub}}{\phi_s\cdot bv\cdot dv} = 0.037 \quad \text{ksi}$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $$\text{MaxSpacing} := \text{spacingProgram}(vu, dv, fc) = 24 \quad \text{in}$$

$$\text{MAXSPACING} := \text{Spacecheck}(\text{MaxSpacing}, spaceNOhinge) = 12 \quad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MAXSPACING}, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

BENT 3 DESIGN

Article 5.10.11.3: Provisions for Seismic Design for Seismic Zone 2

Article 5.10.11.3: Longitudinal Reinforcement

INPUT $A_{longbar} := 1.56$ in^2

INPUT $N_{bars} := 32$

INPUT $Ads := Acolumn2$

$$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 49.92 \quad in^2$$

Minimum Longitudinal Reinforcing Check

$$MinLongRatio := Checkleastlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Maximum Longitudinal Reinforcing Check

$$MaxLongRatio := Checkmaxlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

Article 5.10.11.4.1b: Flexural Resistance

Check biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.9.4.
Flexural Resistance check can be done using some kind of Column Design Program. PCA Column was used for this project. After creating an interaction diagram, verify that all the critical load combinations fall within the diagram.

Article 5.10.11.4.1c: Column Shear and Transverse Reinforcement

Note: This is for the end regions of the column.

Article 5.8.3.3: Nominal Shear Resistance

$Vucol_{Bent3} = 63.945$ kips

$Pumin_{Bent3} = 86.145$ kips

INPUT $bv := Columndia2$ bv: effective width

INPUT $\phi_s := 0.9$

INPUT $s := 3$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $Asp := .44$ in^2 Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.725$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 3$ in Cover: Concrete cover for the Column (in)

INPUT $Dprime := 66$ in Dprime: Diameter of spiral or hoop for circular columns (in)

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 67.57 \quad \text{in}$$

Eq. C5.8.2.9-2

$$de := \frac{bv}{2} + \frac{Dr}{\pi} = 57.508 \quad \text{in}$$

$$dv := 0.9 \cdot de = 51.757 \quad \text{in}$$

Article 5.10.11.4.1c: Column Shear and Transverse Reinforcement

INPUT $V_c := \mathrm{VcProgram}(fc, \beta, bv, dv, Ads, Pumin_{Bent3}) = 24.915$ kips

$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{s} = 910.93 \quad \text{kips}$$

$$\phi V_n := (V_c + V_s) \cdot \phi_s = 842.261 \quad \text{kips}$$

$$\mathrm{Shearcheck} := \mathrm{ShearCheck}(\phi V_n, Vu) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

Length of End Region (Plastic Hinge Region)

End region is assumed to extend from the soffit of girders or cap beams at the top of the columns or from the top of foundations at the bottom of columns.

$$\mathrm{LendgthEndRegion} := \mathrm{EndRegionProgram}(bv, ColumnHeight_{Bent3}) = 118.272 \quad \text{in}$$

Article 5.10.11.4.3: Column Connections

Extension into Top and Bottom Connections

Note: This needs to be done whenever the column dimension changes.

$$\mathrm{Extension} := \mathrm{ExtensionProgram}(bv) = 36 \quad \text{in}$$

Article 5.10.11.4.1e: Spacing of Transverse Reinforcement for Confinement

Transverse Reinforcement for Confinement

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

$$\text{MaximumSpacing} := \text{Spacingprogram(bv)} = 4 \qquad \text{in}$$

$$\text{FINALSPACING} := \text{SpacingCheck(MaximumSpacing, s)} = 3 \qquad \text{in}$$

$$\text{scheck} := \text{ShearCheck(MaximumSpacing, s)} = \text{"OK"}$$

Note: If scheck returns "Failure", increase the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.1d: Transverse Reinforcement for Confinement at Plastic Hinges

Required Volumetric Ratio of Spiral or Seismic Hoop Reinforcing

$$\rho_s := \frac{4 \cdot \text{Asp}}{s \cdot \text{Dprime}} = 8.889 \times 10^{-3}$$

$$\text{Check}\rho_s := \text{RatioProgram}\left(\text{fc}, \text{fye}, \rho_s\right) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Increase Transverse Reinforcing Ratio", it is recommended to decrease the spacing (s) or increase the area of the transverse reinforcement (Asp) in the inputs.

Note:These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension **of NOT** less than **6*d_b** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a 6*d_b but **NOT** less than **3 in.** extension that engages the longitudinal reinforcement.

Article 5.10.11.4.1f: Splices

Lap Splices in longitudinal reinforcement shall not be used in plastic hinge region.

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.

5.8.3.3 **Nominal Shear Resistance**

<u>*INPUT*</u> $Vu_{sub} := Vucol_{Bent3} = 63.945$ kips

<u>*INPUT*</u> $spaceNOhinge := 6$ in

Note: β and θ come from Article 5.8.3.4.1

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 67.57 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 57.508 \quad \text{in}$$

$$dv := 0.9 \cdot de = 51.757 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 471.034 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{Asp \cdot \frac{fye}{1000} \cdot dv \cdot cot(\theta)}{spaceNOhinge} = 227.732 \quad \text{kips}$$

$$\phi V_n := (V_c + V_s) \cdot \phi_s = 628.889 \quad \text{kips}$$

$$Shearcheck := ShearCheck(\phi V_n, Vu_{sub}) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.455 \quad \text{in}^2$$

$$Av := 2 \cdot Asp = 0.88 \quad \text{in}^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

***5.8.2.7* Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 0.019 \quad \text{ksi}$$

$$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \quad \text{in}$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 6 \quad \text{in}$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

STRUT 3 DESIGN

Article 5.10.11.3: Provisions for Seismic Design for Seismic Zone 2

Article 5.10.11.3: Longitudinal Reinforcement

INPUT $A8_{bl} := 0.79 \quad in^2$

INPUT $Number\_8\_Bars := 36$

INPUT $A11_{bl} := 1.56 \quad in^2$

INPUT $Number\_11\_Bars := 16$

$$A_{long} := A8_{bl} \cdot Number\_8\_Bars + A11_{bl} \cdot Number\_11\_Bars = 53.4 \quad in^2$$

$$Ag := Strut3Depth \cdot Strut3Width = 5.04 \times 10^3 \quad in^2$$

Minimum Longitudinal Reinforcing Check

$$MinLongRatio := Checkleastlongreinforcing\left(Ag, A_{long}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Maximum Longitudinal Reinforcing Check

$$MaxLongRatio := Checkmaxlongreinforcing\left(Ag, A_{long}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

Article 5.10.11.4.1b: Flexural Resistance

Check biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.9.4.
Flexural Resistance check can be done using some kind of Column Design Program. PCA Column was used for this project. After creating an interaction diagram, verify that all the critical load combinations fall within the diagram.

Article 5.10.11.4.1c: Column Shear and Transverse Reinforcement

Note: This is for the end regions of the column.

Article 5.8.3.3: Nominal Shear Resistance

$Vu := Vucol_{Strut3} = 155.603$ kips

$Pumin_{Strut3} = 10.63$ kips

INPUT $bv := Strut3Width$ bv: effective width

INPUT $\phi_s := 0.9$

INPUT $s := 3.5$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $Asp := 0.60$ in^2 Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.875$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 2$ in Cover: Concrete cover for the Column (in)

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$\beta := 2.0$

$\theta := \frac{\pi}{180} \cdot 45 = 0.785$ rad

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

Eq. 5.8.2.9-2 $de := 117$ in de = ds which is the distance from top of the member to the centriod of the tensile fiber

$dvpreliminary := 114$ in dvpreliminary = distance between compressive and tensile reinforcing

$dv := dvprogram(de, dvpreliminary, Strut2Depth) = 114$ in

Article 5.10.11.4.1c:

Note: The area of the Column and the Axial Load for this column need to be input into the Vc equation that calls the program above.

$$V_c := \mathrm{VcProgram}\left(fc, \beta, bv, dv, Ag, Pumin_{Strut3}\right) = 3.191 \qquad \text{kips}$$

Eq. 5.8.3.3-4

$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{s} = 2.345 \times 10^{-} \text{kips}$$

Eq. 5.8.3.3-1

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 2.114 \times 10^3 \text{ kips}$$

$$\mathrm{Shearcheck} := \mathrm{ShearCheck}\left(\phi V_n, Vu\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

Article 5.10.11.4.1c Length of End Region (Plastic Hinge Region)

End region is assumed to extend from the soffit of girders or cap beams at the top of the columns or from the top of foundations at the bottom of columns.

$$\mathrm{LendgthEndRegion} := \mathrm{EndRegionProgram}(\mathrm{Strut3Depth}, \mathrm{Lstrut3}) = 120 \qquad \text{in}$$

Article 5.10.11.4.1e: Spacing of Transverse Reinforcement for Confinement

Transverse Reinforcement for Confinement

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

$$\text{MaximumSpacing} := \text{Spacingprogram}(\text{Strut3Width}) = 4 \quad \text{in}$$

$$\text{FINALSPACING} := \text{SpacingCheck}(\text{MaximumSpacing}, s) = 3.5 \quad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MaximumSpacing}, s) = \text{"OK"}$$

Note: If scheck returns "Failure", increase the spacing of <u>shear reinforcing spacing (s)</u>. The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.1d: Transverse Reinforcement for Confinement at Plastic Hinges

Required Volumetric Ratio Seismic Hoop Reinforcing

<u>INPUT</u> $hc := 116 \quad \text{in}$ hc = core dimension of strut

<u>INPUT</u> $Ac := hc \cdot (\text{Strut3Width} - \text{Cover} \cdot 2) = 4.408 \times 10^{3} \quad \text{in}^2$

$Av := 2 \cdot Asp = 1.2 \quad \text{in}^2$

$$\text{Check}\rho_s := \text{RectangularProgram}(fc, fye, s, Ag, Ac, Av, hc) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Increase Transverse Reinforcing Ratio", it is recommended to <u>decrease the spacing (s)</u> or <u>increase the area of the transverse reinforcement (Asp)</u> in the inputs.

Note:These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension **of NOT** less than **6*d_b** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Article 5.10.11.4.1f: Splices

Lap Splices in longitudinal reinforcement shall not be used in plastic hinge region.

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.

***5.8.3.3* Nominal Shear Resistance**

<u>*INPUT*</u> $Vu_{sub} := Vucol_{Strut3} = 155.603$ kips

<u>*INPUT*</u> $spaceNOhinge := 12$ in

<u>*INPUT*</u> $bv := Strut3Width$

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$\beta := 2.0$

$\theta := \frac{\pi}{180} \cdot 45 = 0.785$ rad

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

Eq. 5.8.2.9-2 $de := 117$ in de = ds which is the distance from top of the member to the centriod of the tensile fiber

$dvpreliminary := 114$ in dvpreliminary = distance between compressive and tensile reinforcing

$$dv := dvprogram(de, dvpreliminary, Strut2Depth) = 114 \quad in$$

Eq. 5.8.3.3-3 $$V_c := 0.0316\cdot\beta\cdot\sqrt{\frac{fc}{1000}}bv\cdot dv = 605.203 \quad kips$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp\cdot\frac{fye}{1000}dv\cdot\cot(\theta)}{spaceNOhinge} = 684 \quad kips$$

$$\phi V_n := \left(V_c + V_s\right)\cdot\phi_s = 1.16\times10^3 \quad kips$$

$$Shearcheck := ShearCheck\left(\phi V_n, Vu_{sub}\right) = "OK"$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316\cdot\sqrt{\frac{fc}{1000}}\cdot\frac{bv\cdot spaceNOhinge}{\frac{fye}{1000}} = 0.531 \quad in^2$$

$$Av := 2\cdot Asp = 1.2 \quad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = "OK"$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

***5.8.2.7* Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 0.036 \quad \text{ksi}$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \quad \text{in}$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \quad \text{in}$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

BENT 4 DESIGN

Article 5.10.11.3: Provisions for Seismic Design for Seismic Zone 2

Article 5.10.11.3: Longitudinal Reinforcement

<u>*INPUT*</u> $A_{longbar} := 1.56 \quad in^2$

<u>*INPUT*</u> $N_{bars} := 24$

<u>*INPUT*</u> $Ads := Acolumn1$

$$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 37.44 \quad in^2$$

Minimum Longitudinal Reinforcing Check

$$MinLongRatio := Checkleastlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars</u> in the inputs.

Maximum Longitudinal Reinforcing Check

$$MaxLongRatio := Checkmaxlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either <u>increase the section size (Ag)</u> or <u>decrease the longitudinal reinforcing (Abl and NumberBars)</u> in the inputs.

Article 5.10.11.4.1b: Flexural Resistance

Check biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.9.4.
Flexural Resistance check can be done using some kind of Column Design Program. PCA Column was used for this project. After creating an interaction diagram, verify that all the critical load combinations fall within the diagram.

Article 5.10.11.4.1c: Column Shear and Transverse Reinforcement

Note: This is for the end regions of the column.

Article 5.8.3.3: Nominal Shear Resistance

$Vucol_{Bent4} = 97.303$ kips

$Pumin_{Bent4} = 108.888$ kips

INPUT $bv := Columndia1$ bv: effective width

INPUT $\phi_s := 0.9$

INPUT $s := 4$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $Asp := .44$ in^2 Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.725$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 3$ in Cover: Concrete cover for the Column (in)

INPUT $Dprime := 54$ in Dprime: Diameter of spiral or hoop for circular columns (in)

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 55.57 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 47.688 \quad \text{in}$$

$$dv := 0.9 \cdot de = 42.92 \quad \text{in}$$

Article 5.10.11.4.1c: Column Shear and Transverse Reinforcement

INPUT $V_c := \text{VcProgram}\left(fc, \beta, bv, dv, Ads, Pumin_{Bent3}\right) = 24.793$ kips

$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{s} = 566.539 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 532.199 \quad \text{kips}$$

$$\text{Shearcheck} := \text{ShearCheck}\left(\phi V_n, Vu\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

Length of End Region (Plastic Hinge Region)

End region is assumed to extend from the soffit of girders or cap beams at the top of the columns or from the top of foundations at the bottom of columns.

$$\text{LendgthEndRegion} := \text{EndRegionProgram}\left(bv, \text{ColumnHeight}_{Bent4}\right) = 64.312 \quad \text{in}$$

Article 5.10.11.4.3: Column Connections

Extension into Top and Bottom Connections

Note: This needs to be done whenever the column dimension changes.

$$\text{Extension} := \text{ExtensionProgram}(bv) = 30 \quad \text{in}$$

Article 5.10.11.4.1e: Spacing of Transverse Reinforcement for Confinement

Transverse Reinforcement for Confinement

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

$$\text{MaximumSpacing} := \text{Spacingprogram}(bv) = 4 \quad \text{in}$$

$$\text{FINALSPACING} := \text{SpacingCheck}(\text{MaximumSpacing}, s) = 4 \quad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MaximumSpacing}, s) = \text{"OK"}$$

Note: If scheck returns "Failure", increase the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.1d: Transverse Reinforcement for Confinement at Plastic Hinges

Required Volumetric Ratio of Spiral or Seismic Hoop Reinforcing

$$\rho_s := \frac{4 \cdot Asp}{s \cdot Dprime} = 8.148 \times 10^{-3}$$

$$\text{Check}\rho_s := \text{RatioProgram}\left(fc, fye, \rho_s\right) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Increase Transverse Reinforcing Ratio", it is recommended to decrease the spacing (s) or increase the area of the transverse reinforcement (Asp) in the inputs.

Note:These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **$6*d_b$** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension of **NOT** less than **$6*d_b$** at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a $6*d_b$ but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a $6*d_b$ but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Article 5.10.11.4.1f: Splices

Lap Splices in longitudinal reinforcement shall not be used in plastic hinge region.

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.

5.8.3.3 **Nominal Shear Resistance**

INPUT $Vu_{sub} := Vucol_{Bent4} = 97.303$ kips

INPUT $spaceNOhinge := 12$ in

Note: β and θ come from Article 5.8.3.4.1

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 55.57 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 47.688 \quad \text{in}$$

$$dv := 0.9 \cdot de = 42.92 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 325.502 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 94.423 \quad \text{kips}$$

$$\phi V_n := (V_c + V_s) \cdot \phi_s = 377.933 \quad \text{kips}$$

$$Shearcheck := ShearCheck(\phi V_n, Vu_{sub}) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.758 \quad \text{in}^2$$

$$Av := 2 \cdot Asp = 0.88 \quad \text{in}^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

***5.8.2.7* Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{Vu_{sub} - \phi_s \cdot Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 4.198 \times 10^{-3} \quad \text{ksi}$$

$$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \quad \text{in}$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \quad \text{in}$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

STRUT 4 DESIGN

Article 5.10.11.3: Provisions for Seismic Design for Seismic Zone 2

Article 5.10.11.3: Longitudinal Reinforcement

INPUT $A5_{bl} := 0.31 \quad in^2$

INPUT $Number\_5\_Bars := 20$

INPUT $A11_{bl} := 1.56 \quad in^2$

INPUT $Number\_11\_Bars := 16$

$$A_{long} := A5_{bl} \cdot Number\_5\_Bars + A11_{bl} \cdot Number\_11\_Bars = 31.16 \quad in^2$$

$$Ag := Strut2Depth \cdot Strut2Width = 3.024 \times 10^3 \quad in^2$$

Minimum Longitudinal Reinforcing Check

$$MinLongRatio := Checkleastlongreinforcing(Ag, A_{long}) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Maximum Longitudinal Reinforcing Check

$$MaxLongRatio := Checkmaxlongreinforcing(Ag, A_{long}) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

Article 5.10.11.4.1b: Flexural Resistance

Check biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.9.4.
Flexural Resistance check can be done using some kind of Column Design Program. PCA Column was used for this project. After creating an interaction diagram, verify that all the critical load combinations fall within the diagram.

Article 5.10.11.4.1c: Column Shear and Transverse Reinforcement

Note: This is for the end regions of the column.

Article 5.8.3.3: Nominal Shear Resistance

$Vu := Vucol_{Strut4} = 99.254$ kips

$Pumin_{Strut4} = 60.298$ kips

INPUT	$bv := Strut2Width$		bv: effective width
INPUT	$\phi_s := 0.9$		
INPUT	$s := 3.5$	in	s: Spacing of hoops or pitch of spiral (in)
INPUT	$Asp := 0.44$	in^2	Asp: Area of spiral or hoop reinforcing (in^2)
INPUT	$Dsp := 0.75$	in	Dsp: Diameter of spiral or hoop reinforcing (in)
INPUT	$Cover := 2$	in	Cover: Concrete cover for the Column (in)
INPUT	$Dprime := 54$	in	Dprime: Diameter of spiral or hoop for circular columns (in)
INPUT	$d_{bl} := 1.41$	in	d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

Eq. 5.8.2.9-2 $de := 69.4$ in de = ds which is the distance from top of the member to the centriod of the tensile fiber

$dvpreliminary := 66.75$ in dvpreliminary = distance between compressive and tensile reinforcing

$$dv := dvprogram(de, dvpreliminary, Strut2Depth) = 66.75 \quad \text{in}$$

Article 5.10.11.4.1c:

Note: The area of the Column and the Axial Load for this column need to be input into the Vc equation that calls the program above.

$$V_c := \mathrm{VcProgram}\left(fc, \beta, bv, dv, Ag, Pumin_{Strut4}\right) = 17.665 \qquad \text{kips}$$

Eq. 5.8.3.3-4

$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{s} = 1.007 \times 10 \ \text{kips}$$

Eq. 5.8.3.3-1

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 922.173 \qquad \text{kips}$$

$$\mathrm{Shearcheck} := \mathrm{ShearCheck}\left(\phi V_n, Vu\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

Article 5.10.11.4.1c Length of End Region (Plastic Hinge Region)

End region is assumed to extend from the soffit of girders or cap beams at the top of the columns or from the top of foundations at the bottom of columns.

$$\mathrm{LendgthEndRegion} := \mathrm{EndRegionProgram}(\mathrm{Strut2Depth}, \mathrm{Lstrut2}) = 72 \qquad \text{in}$$

Article 5.10.11.4.1e: Spacing of Transverse Reinforcement for Confinement

Transverse Reinforcement for Confinement

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

$$MaximumSpacing := Spacingprogram(Strut2Width) = 4 \quad in$$

$$FINALSPACING := SpacingCheck(MaximumSpacing, s) = 3.5 \quad in$$

$$scheck := ShearCheck(MaximumSpacing, s) = \text{"OK"}$$

Note: If scheck returns "Failure", increase the spacing of shear reinforcing spacing (s). The spacing value may be FINALSPACING, but verify this works for all other checks.

Article 5.10.11.4.1d: Transverse Reinforcement for Confinement at Plastic Hinges

Required Volumetric Ratio Seismic Hoop Reinforcing

INPUT $hc := 68 \quad in$ hc = core dimension of strut

INPUT $Ac := hc \cdot (Strut2Width - Cover \cdot 2) = 2.584 \times 10^{3} \quad in^{2}$

$Av := 2 \cdot Asp = 0.88 \quad in^{2}$

$$Check\rho_{s} := RectangularProgram(fc, fye, s, Ag, Ac, Av, hc) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Increase Transverse Reinforcing Ratio", it is recommended to decrease the spacing (s) or increase the area of the transverse reinforcement (Asp) in the inputs.

Note:These Requirements need to be checked and satisfied.

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension **NOT** less than **6*d_b** or **3 in**. at one end and a hook of NOT less than 90 Degrees with an extension of **NOT** less than 6*d_b at the other end.
2) The hooks must engage peripheral longitudinal bars.
3) The 90 Degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Hoop Requirements

1) Bar shall be a closed tie or continuously wound tie.
2) A closed tie may be made up of several reinforcing elements with 135 Degree hooks having a **6*d_b** but **NOT** less than **3 in**. extension at each end.
3) A continuously wound tie shall have at each end a 135 Degree hook with a **6*d_b** but **NOT** less than **3 in**. extension that engages the longitudinal reinforcement.

Article 5.10.11.4.1f: Splices

Lap Splices in longitudinal reinforcement shall not be used in plastic hinge region.

Nominal Shear Resistance for members **OUTSIDE** Plastic Hinge Region.

***5.8.3.3* Nominal Shear Resistance**

INPUT $Vu_{sub} := Vucol_{Strut4} = 99.254$ kips

INPUT $spaceNOhinge := 12$ in

INPUT $bv := Strut2Width$

$\phi_s = 0.9$

Note: β and θ come from **Article 5.8.3.4.1**

$\beta := 2.0$

$\theta := \frac{\pi}{180} \cdot 45 = 0.785$ rad

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

Eq. 5.8.2.9-2 $de := 69.4$ in — de = ds which is the distance from top of the member to the centriod of the tensile fiber

$dvpreliminary := 66.75$ in — dvpreliminary = distance between compressive and tensile reinforcing

$$dv := \text{dvprogram}(de, \text{dvpreliminary}, \text{Strut2Depth}) = 66.75 \qquad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 354.362 \qquad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{\text{spaceNOhinge}} = 293.7 \qquad \text{kips}$$

$$\phi V_n := (V_c + V_s) \cdot \phi_s = 583.256 \qquad \text{kips}$$

$$\text{Shearcheck} := \text{ShearCheck}(\phi V_n, Vu_{sub}) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$\text{Avmin} := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot \text{spaceNOhinge}}{\frac{fye}{1000}} = 0.531 \qquad \text{in}^2$$

$$Av := 2 \cdot Asp = 0.88 \qquad \text{in}^2$$

$$\text{MinimumTran} := \text{TranCheck}(\text{Avmin}, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 Maximum Spacing of Transverse Reinforcement

Eq. 5.8.2.9-1 $$vu := \frac{Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 0.039 \quad \text{ksi}$$

Eq. 5.8.2.7-1
Eq. 5.8.2.7-2 $$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \quad \text{in}$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \quad \text{in}$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of shear reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT DESIGN

DRILLED SHAFT 2

Article 5.13.4.6.2b: Cast-in-place Piles

INPUT $A_{longbar} := 1.56 \quad in^2$

INPUT $N_{bars} := 24$

INPUT $Ads := Ads1$

$$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 37.44 \quad in^2$$

Note: Article 5.13.4.6.2b only has a provision for the minimum amount of longitudinal reinforcing. This minimum reinforcing check only applies to the **FIRST 1/3*PILE LENGTH OR 8 FT**.

CHECK: There also needs be a check made if the column Diameter is less than 24.0 in. For these drilled shafts, spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at pitch not exceeding 9.0 in., except that the pitch shall not exceed 4.0 in. within a length below the pile cap reinforcement of not less than 2.0 ft or 1.5 pile diameters, whichever is greater.

Minimum Longitudinal Reinforcing Check (First 1/3*Pile Length or 8 ft)

$$\text{Checkleastlongreinforcing}(Ag, Along) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } Along \geq Ag \cdot 0.005 \\ a \leftarrow \text{"Increase Longitudinal Reinforcing Ratio"} \quad \text{if } Along < 0.005 \cdot Ag \\ a \end{array} \right.$$

$$\text{MinLongRatio} := \text{Checkleastlongreinforcing}\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars</u> in the inputs.

Article 5.10.11.3: Longitudinal Reinforcement

Minimum Longitudinal Reinforcing Check

$$\text{MinLongRatio} := \text{Checkleastlongreinforcing}(\text{Ads}, A_{\text{longreinforcing}}) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Maximum Longitudinal Reinforcing Check

$$\text{MaxLongRatio} := \text{Checkmaxlongreinforcing}(\text{Ads}, A_{\text{longreinforcing}}) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

***5.8.3.3* Nominal Shear Resistance**

INPUT $Vu_{sub} := Vu_{DS2} = 183.522$ kips

INPUT $\text{spaceNOhinge} := 12$ in — s: Spacing of hoops or pitch of spiral (in)

INPUT $bv := \text{Drillshaftdia1}$ — bv: effective width

INPUT $Asp := .44$ in^2 — Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.75$ in — Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 6$ in — Cover: Concrete cover for the Column (in)

INPUT $Dprime := 54$ in — Dprime: Diameter of spiral or hoop for circular columns (in)

INPUT $d_{bl} := 1.41$ in — d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 58.545 \quad in$$

Eq. C5.8.2.9-2
$$de := \frac{bv}{2} + \frac{Dr}{\pi} = 51.635 \quad in$$

$$dv := 0.9 \cdot de = 46.472 \quad in$$

Eq. 5.8.3.3-3
$$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 387.687 \quad kips$$

Eq. 5.8.3.3-4
$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 204.476 \quad kips$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 532.947 \quad kips$$

$$Shearcheck := ShearCheck\left(\phi V_n, Vu\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1
$$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.834 \quad in^2$$

$$Av := 2 \cdot Asp = 0.88 \quad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{Vu_{sub} - \phi_s \cdot Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 6.648 \times 10^{-3} \quad \text{ksi}$$

$$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \quad \text{in}$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \quad \text{in}$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of transverse reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT 3

Article 5.13.4.6.2b: Cast-in-place Piles

INPUT $A_{longbar} := 1.56 \quad in^2$

INPUT $N_{bars} := 32$

INPUT $Ads := Ads2$

$$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 49.92 \quad in^2$$

Note: Article 5.13.4.6.2b only has a provision for the minimum amount of longitudinal reinforcing. This minimum reinforcing check only applies to the **FIRST 1/3*PILE LENGTH OR 8 FT**.

CHECK: There also needs be a check made if the column Diameter is less than 24.0 in. For these drilled shafts, spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at pitch not exceeding 9.0 in., except that the pitch shall not exceed 4.0 in. within a length below the pile cap reinforcement of not less than 2.0 ft or 1.5 pile diameters, whichever is greater.

Minimum Longitudinal Reinforcing Check (First 1/3*Pile Length or 8 ft)

$$MinLongRatio := Checkleastlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Article 5.10.11.3: Longitudinal Reinforcement

Minimum Longitudinal Reinforcing Check

$$MinLongRatio := Checkleastlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Maximum Longitudinal Reinforcing Check

$$MaxLongRatio := Checkmaxlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

5.8.3.3 **Nominal Shear Resistance**

INPUT	$Vu_{sub} := Vu_{DS3} = 127.89$	kips	
INPUT	$spaceNOhinge := 12$	in	s: Spacing of hoops or pitch of spiral (in)
INPUT	$bv := Drillshaftdia2$		bv: effective width
INPUT	$Asp := .44$	in^2	Asp: Area of spiral or hoop reinforcing (in^2)
INPUT	$Dsp := 0.75$	in	Dsp: Diameter of spiral or hoop reinforcing (in)
INPUT	$Cover := 6$	in	Cover: Concrete cover for the Column (in)
INPUT	$Dprime := 66$	in	Dprime: Diameter of spiral or hoop for circular columns (in)

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 70.545 \quad \text{in}$$

Eq. C5.8.2.9-2 $$de := \frac{bv}{2} + \frac{Dr}{\pi} = 61.455 \quad \text{in}$$

$$dv := 0.9 \cdot de = 55.31 \quad \text{in}$$

Eq. 5.8.3.3-3 $$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 545.309 \quad \text{kips}$$

Eq. 5.8.3.3-4 $$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 243.362 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 709.804 \quad \text{kips}$$

$$Shearcheck := ShearCheck\left(\phi V_n, Vu\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.986$$

$$Av := 2 \cdot Asp = 0.88 \qquad in^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"Decrease Spacing or Increase Bar Size"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to <u>decrease the spacing (spaceNOhinge)</u> or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{Vu_{sub} - \phi_s \cdot Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 3.294 \times 10^{-3} \qquad ksi$$

$$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \qquad in$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 12 \qquad in$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of transverse reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT 4

Article 5.13.4.6.2b: Cast-in-place Piles

<u>INPUT</u> $A_{longbar} := 1.56 \qquad in^2$

<u>INPUT</u> $N_{bars} := 24$

<u>INPUT</u> $Ads := Ads1$

$$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 37.44 \qquad in^2$$

Note: Article 5.13.4.6.2b only has a provision for the minimum amount of longitudinal reinforcing. This minimum reinforcing check only applies to the **FIRST 1/3*PILE LENGTH OR 8 FT**.

CHECK: There also needs be a check made if the column Diameter is less than 24.0 in. For these drilled shafts, spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at pitch not exceeding 9.0 in., except that the pitch shall not exceed 4.0 in. within a length below the pile cap reinforcement of not less than 2.0 ft or 1.5 pile diameters, whichever is greater.

Minimum Longitudinal Reinforcing Check (First 1/3*Pile Length or 8 ft)

$$\mathrm{MinLongRatio} := \mathrm{Checkleastlongreinforcing}\left(\mathrm{Ads}, A_{\mathrm{longreinforcing}}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Article 5.10.11.3: Longitudinal Reinforcement

Minimum Longitudinal Reinforcing Check

$$\mathrm{MinLongRatio} := \mathrm{Checkleastlongreinforcing}\left(\mathrm{Ads}, A_{\mathrm{longreinforcing}}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Maximum Longitudinal Reinforcing Check

$$\mathrm{MaxLongRatio} := \mathrm{Checkmaxlongreinforcing}\left(\mathrm{Ads}, A_{\mathrm{longreinforcing}}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

5.8.3.3 Nominal Shear Resistance

<u>*INPUT*</u> $Vu_{sub} := Vu_{DS4} = 194.607$ kips

<u>*INPUT*</u> $spaceNOhinge := 12$ in — s: Spacing of hoops or pitch of spiral (in)

<u>*INPUT*</u> $bv := Drillshaftdia1$ — bv: effective width

<u>*INPUT*</u> $Asp := .44$ in^2 — Asp: Area of spiral or hoop reinforcing (in^2)

<u>*INPUT*</u> $Dsp := 0.75$ in — Dsp: Diameter of spiral or hoop reinforcing (in)

<u>*INPUT*</u> $Cover := 6$ in — Cover: Concrete cover for the Column (in)

<u>*INPUT*</u> $Dprime := 54$ in — Dprime: Diameter of spiral or hoop for circular columns (in)

<u>*INPUT*</u> $d_{bl} := 1.41$ in — d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 58.545 \quad \text{in}$$

Eq. C5.8.2.9-2
$$de := \frac{bv}{2} + \frac{Dr}{\pi} = 51.635 \quad \text{in}$$

$$dv := 0.9 \cdot de = 46.472 \quad \text{in}$$

Eq. 5.8.3.3-3
$$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 387.687 \quad \text{kips}$$

Eq. 5.8.3.3-4
$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 204.476 \quad \text{kips}$$

$$\phi V_n := (V_c + V_s)\cdot\phi_s = 532.947 \qquad \text{kips}$$

$$\text{Shearcheck} := \text{ShearCheck}(\phi V_n, Vu) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $$\text{Avmin} := 0.0316\cdot\sqrt{\frac{fc}{1000}}\cdot\frac{bv\cdot\text{spaceNOhinge}}{\frac{fye}{1000}} = 0.834$$

$$Av := 2\cdot Asp = 0.88 \qquad in^2$$

$$\text{MinimumTran} := \text{TranCheck}(\text{Avmin}, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

***5.8.2.7* Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{Vu_{sub} - \phi_s\cdot Vu_{sub}}{\phi_s\cdot bv\cdot dv} = 7.05\times 10^{-3} \qquad \text{ksi}$$

$$\text{MaxSpacing} := \text{spacingProgram}(vu, dv, fc) = 24 \qquad \text{in}$$

$$\text{MAXSPACING} := \text{Spacecheck}(\text{MaxSpacing}, \text{spaceNOhinge}) = 12 \qquad \text{in}$$

$$\text{scheck} := \text{ShearCheck}(\text{MAXSPACING}, \text{spaceNOhinge}) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of transverse reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT ABUTMENT 1

Article 5.13.4.6.2b: Cast-in-place Piles

INPUT $A_{longbar} := 1.56 \quad in^2$

INPUT $N_{bars} := 24$

INPUT $Ads := Ads3$

$$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 37.44 \quad in^2$$

Note: Article 5.13.4.6.2b only has a provision for the minimum amount of longitudinal reinforcing. This minimum reinforcing check only applies to the **FIRST 1/3*PILE LENGTH OR 8 FT**.

CHECK: There also needs be a check made if the column Diameter is less than 24.0 in. For these drilled shafts, spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at pitch not exceeding 9.0 in., except that the pitch shall not exceed 4.0 in. within a length below the pile cap reinforcement of not less than 2.0 ft or 1.5 pile diameters, whichever is greater.

Minimum Longitudinal Reinforcing Check (First 1/3*Pile Length or 8 ft)

$$MinLongRatio := Checkleastlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars</u> in the inputs.

Article 5.10.11.3: Longitudinal Reinforcement

Minimum Longitudinal Reinforcing Check

$$MinLongRatio := Checkleastlongreinforcing\left(Ads, A_{longreinforcing}\right) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars</u> in the inputs.

Maximum Longitudinal Reinforcing Check

$$\text{MaxLongRatio} := \text{Checkmaxlongreinforcing}\left(\text{Ads}, A_{\text{longreinforcing}}\right) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either <u>increase the section size (Ag)</u> or <u>decrease the longitudinal reinforcing (Abl and NumberBars)</u> in the inputs.

5.8.3.3 **Nominal Shear Resistance**

INPUT $Vu_{sub} := Vu_{DSAbut1} = 267.507$ kips

INPUT $\text{spaceNOhinge} := 10$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $bv := \text{Drillshaftdia3}$ bv: effective width

INPUT $Asp := .31$ in^2 Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.625$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 6$ in Cover: Concrete cover for the Column (in)

INPUT $Dprime := 42$ in Dprime: Diameter of spiral or hoop for circular columns (in)

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180}\cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 46.67 \quad \text{in}$$

Eq. C5.8.2.9-2

$$de := \frac{bv}{2} + \frac{Dr}{\pi} = 41.856 \quad \text{in}$$

$$dv := 0.9\cdot de = 37.67 \quad \text{in}$$

Eq. 5.8.3.3-3 $V_c := 0.0316\cdot\beta\cdot\sqrt{\frac{fc}{1000}}bv\cdot dv = 257.12$ kips

Eq. 5.8.3.3-4 $V_s := \frac{2Asp\cdot\frac{fye}{1000}dv\cdot\cot(\theta)}{spaceNOhinge} = 140.132$ kips

$\phi V_n := (V_c + V_s)\cdot\phi_s = 357.527$ kips

$\phi V_n := (V_c + V_s)\cdot\phi_s = 357.527$ kips

$Shearcheck := ShearCheck(\phi V_n, Vu) = "OK"$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1 $Avmin := 0.0316\cdot\sqrt{\frac{fc}{1000}}\cdot\frac{bv\cdot spaceNOhinge}{\frac{fye}{1000}} = 0.569$

$Av := 2\cdot Asp = 0.62$ in^2

$MinimumTran := TranCheck(Avmin, Av) = "OK"$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $vu := \frac{Vu_{sub} - \phi_s\cdot Vu_{sub}}{\phi_s\cdot bv\cdot dv} = 0.015$ ksi

$MaxSpacing := spacingProgram(vu, dv, fc) = 24$ in

$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 10$ in

$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$

Note: If scheck returns "Failure", change the spacing of transverse reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

DRILLED SHAFT ABUTMENT 5

Article 5.13.4.6.2b: Cast-in-place Piles

INPUT $A_{longbar} := 1.56$ in^2

INPUT $N_{bars} := 24$

INPUT $Ads := Ads3$

$A_{longreinforcing} := A_{longbar} \cdot N_{bars} = 37.44$ in^2

Note: Article 5.13.4.6.2b only has a provision for the minimum amount of longitudinal reinforcing. This minimum reinforcing check only applies to the **FIRST 1/3*PILE LENGTH OR 8 FT**.

CHECK: There also needs be a check made if the column Diameter is less than 24.0 in. For these drilled shafts, spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at pitch not exceeding 9.0 in., except that the pitch shall not exceed 4.0 in. within a length below the pile cap reinforcement of not less than 2.0 ft or 1.5 pile diameters, whichever is greater.

Minimum Longitudinal Reinforcing Check (First 1/3*Pile Length or 8 ft)

$MinLongRatio := Checkleastlongreinforcing(Ads, A_{longreinforcing}) = \text{"OK"}$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either <u>decrease the section size (Ag)</u> or <u>increase the longitudinal reinforcing (Abl and NumberBars</u> in the inputs.

Article 5.10.11.3: Longitudinal Reinforcement

Minimum Longitudinal Reinforcing Check

$$MinLongRatio := Checkleastlongreinforcing(Ads, A_{longreinforcing}) = \text{"OK"}$$

Note: If the Minimum Longitudinal Reinforcing program returns "Increase Longitudinal Reinforcing", either decrease the section size (Ag) or increase the longitudinal reinforcing (Abl and NumberBars in the inputs.

Maximum Longitudinal Reinforcing Check

$$MaxLongRatio := Checkmaxlongreinforcing(Ads, A_{longreinforcing}) = \text{"OK"}$$

Note: If the Maximum Longitudinal Reinforcing program returns "Section Over Reinforced", either increase the section size (Ag) or decrease the longitudinal reinforcing (Abl and NumberBars) in the inputs.

5.8.3.3 **Nominal Shear Resistance**

INPUT $Vu_{sub} := Vu_{DSAbut5} = 175.772$ kips

INPUT $spaceNOhinge := 10$ in s: Spacing of hoops or pitch of spiral (in)

INPUT $bv := Drillshaftdia3$ bv: effective width

INPUT $Asp := .31$ in^2 Asp: Area of spiral or hoop reinforcing (in^2)

INPUT $Dsp := 0.625$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT $Cover := 6$ in Cover: Concrete cover for the Column (in)

INPUT $Dprime := 42$ in Dprime: Diameter of spiral or hoop for circular columns (in)

INPUT $d_{bl} := 1.41$ in d_{bl}: Diameter of the longitudinal bar

Note: β and θ come from **Article 5.8.3.4.1**

$$\beta := 2.0$$

$$\theta := \frac{\pi}{180} \cdot 45 = 0.785 \quad \text{rad}$$

Note: Dr, de, and dv equations come from **Article 5.8.2.9**.

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 46.67 \quad \text{in}$$

Eq. C5.8.2.9-2

$$de := \frac{bv}{2} + \frac{Dr}{\pi} = 41.856 \quad \text{in}$$

$$dv := 0.9 \cdot de = 37.67 \quad \text{in}$$

Eq. 5.8.3.3-3

$$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 257.12 \quad \text{kips}$$

Eq. 5.8.3.3-4

$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{spaceNOhinge} = 140.132 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 357.527 \quad \text{kips}$$

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 357.527 \quad \text{kips}$$

$$Shearcheck := ShearCheck\left(\phi V_n, Vu\right) = \text{"OK"}$$

Note: If ShearCheck returns "Failure", either decrease the spacing (s) of the shear reinforcing (Asp), increase the area of shear reinforcing, or increase the section size (Acolumn). These variables can be changed in the inputs.

5.8.2.5 Minimum Transverse Reinforcement

Eq. 5.8.2.5-1

$$Avmin := 0.0316 \cdot \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot spaceNOhinge}{\frac{fye}{1000}} = 0.569$$

$$Av := 2 \cdot Asp = 0.62 \quad \text{in}^2$$

$$MinimumTran := TranCheck(Avmin, Av) = \text{"OK"}$$

Note: If the minimum transverse reinforcement program responses "Decrease Spacing or Increase Bar Size", it is recommended to decrease the spacing (spaceNOhinge) or increase the area of the shear reinforcement (Asp) in the inputs.

5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

Eq. 5.8.2.9-1 $$vu := \frac{Vu_{sub} - \phi_s \cdot Vu_{sub}}{\phi_s \cdot bv \cdot dv} = 9.601 \times 10^{-3} \qquad \text{ksi}$$

$$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \qquad \text{in}$$

$$MAXSPACING := Spacecheck(MaxSpacing, spaceNOhinge) = 10 \qquad \text{in}$$

$$scheck := ShearCheck(MAXSPACING, spaceNOhinge) = \text{"OK"}$$

Note: If scheck returns "Failure", change the spacing of transverse reinforcing spacing (spaceNOhinge). The spacing value may be MAXSPACING, but verify this works for all other checks.

Connection Design for Girder to Bent Cap

INPUT $Vcolbent := Vucol_{Bent4}$

Note: Since Bent 4 has the highest shear out of the 3 bents, use bent 4 shear force.

INPUT $Ngirderperbent := 12$

Article 6.5.4.2: Resistance Factors

$\phi_t := 0.8$	Tension for A307
$\phi_s := 0.75$	Shear for A307
$\phi_{bs} := 0.80$	Block Shear
$\phi_{bb} := 0.80$	Bolts Bearing
$\phi_{sc} := 0.85$	Shear Connectors
$\phi_f := 1.00$	Flexure
$\phi_{sangle} := 1.00$	Shear for the Angle

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT	$Fub := 58$	ksi	
INPUT	$Dia_b := 1.25$	in	
INPUT	$Ns := 1$		Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT	$Fy := 36$	ksi	Fy = Yield Stress of the Angle
INPUT	$Fu := 58$	ksi	Fu = Ultimate Stress of the Angle
INPUT	$t := 0.625$	in	t = Thickness of Angle
INPUT	$h := 6$	in	h = Height of the Angle
INPUT	$w := 6$	in	w = Width of the Angle
INPUT	$l := 12$	in	l = Length of the Angle
INPUT	$k := 1.125$	in	k = Height of the Bevel
INPUT	$distanchorhole := 4$	in	distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.
INPUT	$diahole := 1.5$	in	diahole = Diameter of bolt hole
INPUT	$BLSHlength := 6$	in	BLSHlength = Block Shear Length
INPUT	$BLSHwidth := 2$	in	BLSHwicth = Block Shear Width
INPUT	$Ubs := 1.0$		Ubs = Shear Lag Factor for Block Shear
INPUT	$a := 2$	in	a = Distance from the center of the bolt to the edge of plate
INPUT	$b := 3.5$	in	b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent \cdot 2}{2Ngirderperbent} = 8.109 \quad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 1.227 \qquad in^2$$

Eq. 6.13.2.12-1 $\quad \phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 25.624 \qquad kips$

Note: This is checking to verify that the anchor bolt has enough shear strength.

$$Shearcheck := ShearCheck(\phi sRn, Vangle) = \text{"OK"}$$

Note: If the program returns "FAILURE", either <u>increase diameter of the bolt</u> (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\quad \phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 108.75 \qquad kips$

For Slotted Holes

<u>INPUT</u> $\quad Lc := 2 \quad in \qquad$ Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\quad \phi bbRns := Lc \cdot t \cdot Fub = 72.5 \qquad kips$

$$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$$

$$Bearingscheck := ShearCheck(\phi bbRns, Vangle) = \text{"OK"}$$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 2.027 \qquad kips$$

Eq. 6.13.2.10.2-1

$$\phi tTn := \phi_t \cdot 0.76 \cdot A_b \cdot Fub = 43.275 \qquad kips$$

$$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$$Pu := Vangle$$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2

$$CombinedProgram(Pu, A_b, Fub, \phi sRn, \phi_s) := \left| \begin{array}{l} t \leftarrow 0.76 \cdot A_b \cdot Fub \\ r \leftarrow 0.76 \cdot A_b \cdot Fub \cdot \sqrt{1 - \left(\frac{Pu}{\phi sRn}\right)^2} \\ a \leftarrow t \ \text{ if } \frac{Pu}{\left(\frac{\phi sRn}{\phi_s}\right)} \le 0.33 \\ a \leftarrow r \ \text{ if } \frac{Pu}{\left(\frac{\phi sRn}{\phi_s}\right)} > 0.33 \\ a \end{array} \right.$$

$$Tn_{combined} := CombinedProgram(Pu, A_b, Fub, \phi sRn, \phi_s) = 54.094 \qquad kips$$

$$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 43.275 \qquad kips$$

$$Combinedcheck := ShearCheck(\phi tTn_{combined}, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

Note this is for if there are one through bolts in the upper leg.

$Agv := t \cdot BLSHlength = 3.75 \quad in^2$

$Anv := t \cdot (BLSHlength - 0.5 \cdot diahole) = 3.281 \quad in^2$

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 0.781 \quad in^2$

(J4-5) $BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) := \left| \begin{array}{l} b \leftarrow 0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant \\ c \leftarrow 0.6Fy \cdot Agv + Ubs \cdot Fu \cdot Ant \\ a \leftarrow b \ \text{ if } b \le c \\ a \leftarrow c \ \text{ if } b > c \\ a \end{array} \right.$

$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 126.312 \quad kips$

$\phi bsRn := \phi_{bs} \cdot Rn = 101.05 \quad kips$

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 2.813 \quad in^2$

(D3-1) $Ae := Ant \cdot Ut = 1.688 \quad in^2$

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 78.3 \quad kips$

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

INPUT $\quad Muangle := 26.5 \quad kip \cdot in$

$$Zx := \frac{1 \cdot (t)^2}{4} = 1.172 \qquad in^3$$

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 42.188 \qquad kip \cdot in$$

$$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t \cdot w = 3.75 \qquad in^2$$

(G2-1) $$\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 81 \qquad kips$$

$$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Expansion Connection Design for Girder to Bent Cap

INPUT $Vcolbent := Vucol_{Bent4}$

Note: Since Bent 4 has the highest shear out of the 3 bents, use bent 4 shear force.

INPUT $Ngirderperbent := 12$

Article 6.5.4.2: Resistance Factors

$\phi_t := 0.8$	Tension for A307
$\phi_s := 0.75$	Shear for A307
$\phi_{bs} := 0.80$	Block Shear
$\phi_{bb} := 0.80$	Bolts Bearing
$\phi_{sc} := 0.85$	Shear Connectors
$\phi_f := 1.00$	Flexure
$\phi_{sangle} := 1.00$	Shear for the Angle

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.25$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT	$Fy := 36$	ksi	Fy = Yield Stress of the Angle
INPUT	$Fu := 58$	ksi	Fu = Ultimate Stress of the Angle
INPUT	$t := 0.625$	in	t = Thickness of Angle
INPUT	$h := 6$	in	h = Height of the Angle
INPUT	$w := 6$	in	w = Width of the Angle
INPUT	$l := 12$	in	l = Length of the Angle
INPUT	$k := 1.125$	in	k = Height of the Bevel

INPUT distanchorhole := 4 in — distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT diahole := 1.5 in — diahole = Diameter of bolt hole

INPUT SlottedHole := 6 in — SlottedHole = Length of Slotted Hole

INPUT BLSHlength := 6 in — BLSHlength = Block Shear Length

INPUT BLSHwidth := 2 in — BLSHwicth = Block Shear Width

INPUT Ubs := 1.0 — Ubs = Shear Lag Factor for Block Shear

INPUT a := 2 in — a = Distance from the center of the bolt to the edge of plate

INPUT b := 3.5 in — b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$\text{Vangle} := \frac{\text{Vcolbent}\cdot 2}{2\text{Ngirderperbent}} = 8.109 \qquad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi\cdot \text{Dia}_b^{\ 2}}{4} = 1.227 \qquad \text{in}^2$$

Eq. 6.13.2.12-1 $\phi\text{sRn} := \phi_s\cdot 0.48\cdot A_b\cdot \text{Fub}\cdot \text{Ns} = 25.624$ kips

Note: This is checking to verify that the anchor bolt has enough shear strength.

$$\text{Shearcheck} := \text{ShearCheck}(\phi\text{sRn}, \text{Vangle}) = \text{"OK"}$$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 108.75$ kips

For Slotted Holes

INPUT $Lc := 2$ in Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\phi bbRns := Lc \cdot t \cdot Fub = 72.5$ kips

$Bearingcheck := ShearCheck(\phi bbRn, Vangle) = \text{"OK"}$

$Bearingscheck := ShearCheck(\phi bbRns, Vangle) = \text{"OK"}$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 2.027 \quad \text{kips}$$

Eq. 6.13.2.10.2-1 $\phi tTn := \phi_t \cdot 0.76 \cdot A_b \cdot Fub = 43.275$ kips

$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$Pu := Vangle$

$Tn_{combined} := CombinedProgram(Pu, A_b, Fub, \phi sRn, \phi_s) = 54.094$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 43.275$ kips

$Combinedcheck := ShearCheck(\phi tTn_{combined}, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

Note this is for if there are one through bolts in the upper leg.

$Agv := t \cdot BLSHlength = 3.75$ in^2

$Anv := t \cdot (BLSHlength - 0.5 \cdot diahole) = 3.281$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 0.781$ in^2

(J4-5) $BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) := \left| \begin{array}{l} b \leftarrow 0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant \\ c \leftarrow 0.6Fy \cdot Agv + Ubs \cdot Fu \cdot Ant \\ a \leftarrow b \text{ if } b \le c \\ a \leftarrow c \text{ if } b > c \\ a \end{array} \right.$

$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 126.312$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 101.05$ kips

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 2.813$ in^2

(D3-1) $Ae := Ant \cdot Ut = 1.688$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 78.3$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

INPUT $Muangle := 26.5$ kip·in

$$Zx := \frac{1 \cdot (t)^2}{4} = 1.172 \quad in^3$$

$\phi fMn := \phi_f \cdot Fy \cdot Zx = 42.188$ kip·in

$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$Cv := 1.0$

$Aw := t \cdot w = 3.75$ in^2

(G2-1) $\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 81$ kips

$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = "OK"$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Connection Design for Girder to Abutment 1

INPUT $Vcolbent := Vu_{DSAbut1}$

INPUT $Ngirderperbent := 6$

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.5$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT $Fy := 36$ ksi Fy = Yield Stress of the Angle

INPUT $Fu := 58$ ksi Fu = Ultimate Stress of the Angle

INPUT $t := 1$ in t = Thickness of Angle

INPUT $h := 6$ in h = Height of the Angle

INPUT $w := 6$ in w = Width of the Angle

INPUT $l := 26$ in l = Length of the Angle

INPUT $k := 1.5$ in k = Height of the Bevel

INPUT $distanchorhole := 4$ in distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT $diahole := 1.75$ in — diahole = Diameter of bolt hole

INPUT $SlottedHole := 6$ in — SlottedHole = Length of Slotted Hole

INPUT $BLSHlength := 21$ in — BLSHlength = Block Shear Length

INPUT $BLSHwidth := 2$ in — BLSHwicth = Block Shear Width

INPUT $Ubs := 1.0$ — Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in — a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in — b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$Vangle := \frac{Vcolbent \cdot 3}{2Ngirderperbent} = 66.877 \quad \text{kips}$$

INPUT $n_{bolts} := 3$ — n_{bolts} = number of bolts per flange

$$Vperbolt := \frac{Vangle}{n_{bolts}} = 22.292 \quad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 1.767 \quad \text{in}^2$$

Eq. 6.13.2.12-1 $$\phi sRn := \phi_s \cdot 0.48 \cdot A_b \cdot Fub \cdot Ns = 36.898 \quad \text{kips}$$

Note: This is checking to verify that the anchor bolt has enough shear strength.

$$Shearcheck := ShearCheck(\phi sRn, Vperbolt) = \text{"OK"}$$

Note: If the program returns "FAILURE", either increase diameter of the bolt (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\phi bbRn := 2.4 \cdot Dia_b \cdot t \cdot Fub = 208.8$ kips

For Slotted Holes

INPUT $Lc := 2$ in Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\phi bbRns := Lc \cdot t \cdot Fub = 116$ kips

$Bearingcheck := ShearCheck(\phi bbRn, Vperbolt) = \text{"OK"}$

$Bearingcheck := ShearCheck(\phi bbRn, Vperbolt) = \text{"OK"}$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 16.719 \quad \text{kips}$$

Eq. 6.13.2.10.2-1 $\phi tTn := 0.76 \cdot A_b \cdot Fub = 77.896$ kips

$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$Pu := Vperbolt$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2 $Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 62.072$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 49.658$ kips

$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vperbolt\right) = \text{"OK"}$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$Agv := t \cdot BLSHlength = 21$ in^2

$Anv := t \cdot \left(BLSHlength - 2.5 \cdot \frac{SlottedHole}{2}\right) = 13.5$ in^2

Note: this is for three bolts.

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 1.125$ in^2

(J4-5) $Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 518.85$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 415.08$ kips

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 4.25$ in^2

(D3-1) $Ae := Ant \cdot Ut =$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 118.32$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

INPUT $\quad Muangle := 200. \quad kip \cdot in$

$$Zx := \frac{1 \cdot (t)^2}{4} = 6.5 \qquad in^3$$

Note Need to ADD stiffners in order for this to work.

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 234 \qquad kip \cdot in$$

$$BendingAngleCheck := ShearCheck(\phi fMn, Muangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t \cdot w = 6 \qquad in^2$$

(G2-1) $$\phi sangleVn := \phi_{sangle} \cdot 0.6 \cdot Fy \cdot Aw \cdot Cv = 129.6 \qquad kips$$

$$ShearAngleCheck := ShearCheck(\phi sangleVn, Vangle) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Connection Design for Girder to Abutment 5

INPUT $Vcolbent := Vu_{DSAbut5}$

INPUT $Ngirderperbent := 6$

Note: Select the grade of bolt being used. It is assumed that a ASTM A307 Grade C bolt is used.

INPUT $Fub := 58$ ksi

INPUT $Dia_b := 1.5$ in

INPUT $Ns := 1$ Ns = Number of Shear Planes per Bolt

Angle Properties

INPUT $Fy := 36$ ksi Fy = Yield Stress of the Angle

INPUT $Fu := 58$ ksi Fu = Ultimate Stress of the Angle

INPUT $t := 0.875$ in t = Thickness of Angle

INPUT $h := 6$ in h = Height of the Angle

INPUT $w := 6$ in w = Width of the Angle

INPUT $l := 20$ in l = Length of the Angle

INPUT $k := 1.375$ in k = Height of the Bevel

INPUT $distanchorhole := 4$ in distanchorhole = Distance from the vertical leg to the center of the hole. This is the location of the holes.

INPUT $diahole := 1.75$ in diahole = Diameter of bolt hole

INPUT $SlottedHole := 6$ in SlottedHole = Length of Slotted Hole

INPUT $BLSHlength := 15$ in BLSHlength = Block Shear Length

INPUT $BLSHwidth := 2$ in BLSHwicth = Block Shear Width

INPUT $Ubs := 1.0$ Ubs = Shear Lag Factor for Block Shear

INPUT $a := 2$ in a = Distance from the center of the bolt to the edge of plate

INPUT $b := 3.5$ in b = distance from center of bolt to toe of fillet of connected part

Shear Force per Angle:

$$\text{Vangle} := \frac{\text{Vcolbent}\cdot 3}{2\text{Ngirderperbent}} = 43.943 \qquad \text{kips}$$

INPUT $\quad n_{bolts} := 2 \qquad n_{bolts}$ = number of bolts per flange

$$\text{Vperbolt} := \frac{\text{Vangle}}{n_{bolts}} = 21.972 \qquad \text{kips}$$

Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot \text{Dia}_b^{\,2}}{4} = 1.767 \qquad \text{in}^2$$

Eq. 6.13.2.12-1 $\quad \phi\text{sRn} := \phi_s \cdot 0.48 \cdot A_b \cdot \text{Fub} \cdot \text{Ns} = 36.898 \qquad \text{kips}$

Note: This is checking to verify that the anchor bolt has enough shear strength.

$$\text{Shearcheck} := \text{ShearCheck}(\phi\text{sRn}, \text{Vperbolt}) = \text{"OK"}$$

Note: If the program returns "FAILURE", either <u>increase diameter of the bolt</u> (Diab), change grade of bolt, increase number of bolts, etc.

Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

Eq. 6.13.2.9-1 $\quad \phi\text{bbRn} := 2.4 \cdot \text{Dia}_b \cdot t \cdot \text{Fub} = 182.7 \qquad \text{kips}$

For Slotted Holes

INPUT $\quad \text{Lc} := 2 \quad \text{in} \qquad$ Lc = Clear dist. between the hole and the end of the member

Eq. 6.13.2.9-4 $\quad \phi\text{bbRns} := \text{Lc} \cdot t \cdot \text{Fub} = 101.5 \qquad \text{kips}$

$$\text{Bearingcheck} := \text{ShearCheck}(\phi\text{bbRn}, \text{Vperbolt}) = \text{"OK"}$$

$$\text{Bearingcheck} := \text{ShearCheck}(\phi\text{bbRn}, \text{Vperbolt}) = \text{"OK"}$$

Note: If the program returns "FAILURE", either change the diameter of the bolt, thickness of the angle, or Lc (hole location).

Article 6.13.2.10: Tensile Resistance

Note: This a calculation of the Tension force on the anchor bolt due to the shear. A moment is taken about the through bolt in the vertical leg of the angle. The line of action for the shear force is assumed to enter the angle at 1" below the through bolt; therefore, the moment due to shear is Vangle* 1". The distance to the anchor bolt in the cap beam is 4", and that is how the Tu equation was derived.

$$Tu := \frac{Vangle \cdot 1}{distanchorhole} = 10.986 \qquad \text{kips}$$

Eq. 6.13.2.10.2-1

$$\phi tTn := 0.76 \cdot A_b \cdot Fub = 77.896 \qquad \text{kips}$$

$$Tensioncheck := ShearCheck(\phi tTn, Tu) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

Article 6.13.2.11: Combined Tension and Shear

$$Pu := Vperbolt$$

Eq. 6.13.2.11-1
Eq. 6.13.2.11-2

$$Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 62.58 \qquad \text{kips}$$

$$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 50.064 \qquad \text{kips}$$

$$Combinedcheck := ShearCheck\left(\phi tTn_{combined}, Vperbolt\right) = \text{"OK"}$$

Note: If program returns "FAILURE", change area of the bolt or grade of bolt.

AISC J4 Block Shear

$Agv := t \cdot BLSHlength = 13.125$ in^2

$Anv := t \cdot \left(BLSHlength - 1.5 \cdot \frac{SlottedHole}{2} \right) = 9.188$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 \cdot diahole) = 0.984$ in^2

Note: this is for 2 bolts.

(J4-5) $Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 340.594$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 272.475$ kips

$BlockShearCheck := ShearCheck(\phi bsRn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change diameter of the bolt, number of bolts, thickness of angle, length of angle, etc.

AISC D2: Tension Member

$Ut := 0.6$

Ut = Shear Lag factor for single Angles. Refer to Table D3.1 in AISC Manual

$Ant := t \cdot [w - (1 \cdot diahole)] = 3.719$ in^2

(D3-1) $Ae := Ant \cdot Ut = 2.231$ in^2

(D2-2) $\phi tPn := \phi_t \cdot Fub \cdot Ae = 103.53$ kips

$TensionCheck := ShearCheck(\phi tPn, Vangle) = \text{"OK"}$

Note: If program returns "FAILURE", change thickness of angle, length of angle, width of angle, etc.

AISC CH. F: Bending of Angle

Note: A SAP Model of the angle was created. The shear of the angle was applied at 4 in. (which is at the bolt location). Then the moment was found just above the k height for this angle.
The Critical section is a k distance away on the horizontal leg. This location has the greatest moment.

INPUT $\text{Muangle} := 68.75 \quad \text{kip}\cdot\text{in}$

$$Zx := \frac{1\cdot(t)^2}{4} = 3.828 \qquad \text{in}^3$$

$$\phi fMn := \phi_f \cdot Fy \cdot Zx = 137.813 \qquad \text{kip}\cdot\text{in}$$

$$\text{BendingAngleCheck} := \text{ShearCheck}(\phi fMn, \text{Muangle}) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or length of angle

AISC G: Shear Check

$$Cv := 1.0$$

$$Aw := t\cdot w = 5.25 \qquad \text{in}^2$$

(G2-1) $$\phi\text{sangleVn} := \phi_{sangle}\cdot 0.6\cdot Fy\cdot Aw\cdot Cv = 113.4 \qquad \text{kips}$$

$$\text{ShearAngleCheck} := \text{ShearCheck}(\phi\text{sangleVn}, \text{Vangle}) = \text{"OK"}$$

Note: If program returns "FAILURE", change thickness of angle or width of angle.

Appendix I: Interaction Diagrams for Scarham Creek Bridge

Designer: PJC
Project Name: SCARHAM
Job Number:
Date: 11-10-2010

Moment Capacity and Interaction Diagram Data
Column or Drilled Shaft: BENT 2 COLUMN

INPUTS

Column Diameter	60	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	# 11	
Number of Lonitudinal Reinforcing Bars	24	
Transverse Reinforcing Bar Size	# 6	
Spacing of Transverse Reinforcing Bars	12	in
Column Height	25	ft
Cover	3	in

Dead Load Reactions

PuDead 853 kips
MuDead 185 kip*ft

Enter into Column Design Software (PCA COLUMN)
Finding Moment Capacity of the column.

Mp = ϕMp/0.9 → 5595 kip*ft

Vpcol = 2*Mp/Heightcolumn → 448 kips

Vpbent = 2*Vpcol 895 kips

Create SAP model of Bent
Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Puoverturning = 1165 kips 1315 k

Pu = PuDead +/- Puoverturning 2018 or 312 kips 2168 k or 462 kips

Re-enter into Column Design Software (PCA COLUMN)

Mp = ϕMp/0.9 → 6308 kip*ft 6226

Vpcol = 2*Mp/Heightcolumn → 505 kips 478

Vpbent = 2*Vpcol 1010 kips 996

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

Check = ((Vpbent2 - Vpbent1) / Vpbent2) * 100% 11.38 1.38

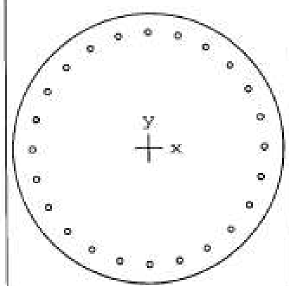
Check Other Seismic Load Cases

Mupotrans 3130 kip*ft
Putran 872 kips
Mupolong 247 kip*ft
Pulong 0.6 kips

petran = 0.359
pelong = 0.502

Pu = PuDead +/- Putran → 1725 or 19 kips
Mu = MuDead + Mupotran → 3315 kip*ft

Pu = PuDead +/- Pulong → 853 or 853 kips
Mu = MuDead + Mupolong → 432 kip*ft



60 in diam.

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

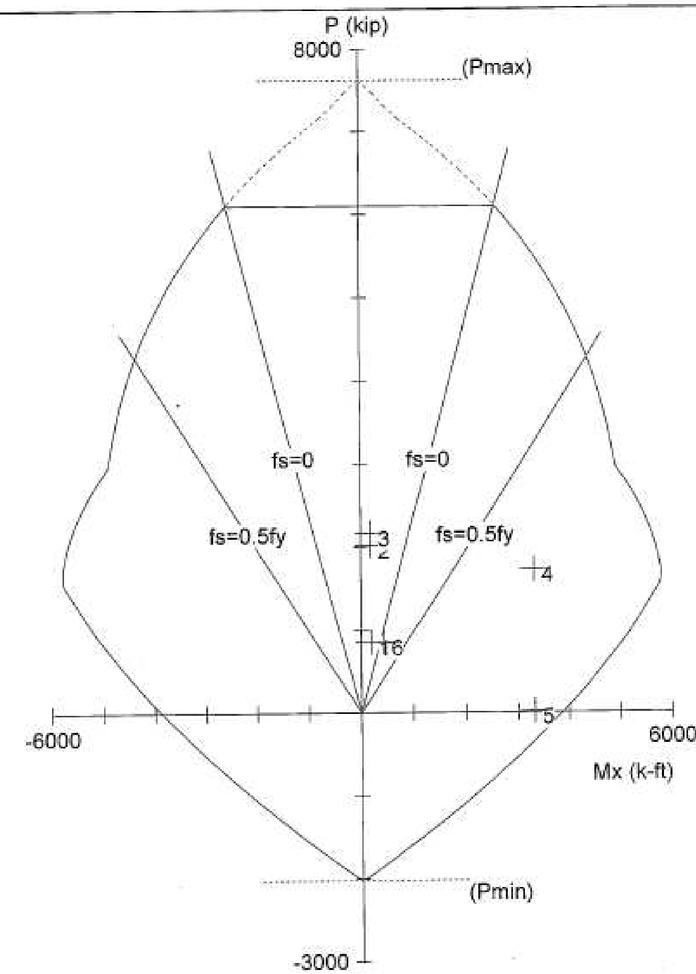
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/10/10

Time: 14:05:40



pcaColumn v3.64. Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59

File: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\SCARHAM CR...\BENT2 COLUMN.col

Project: Scarham Creek

Column: Bent 2		Engineer: PJC	
f'c = 4 ksi	fy = 60 ksi	Ag = 2827.43 in^2	24 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 37.44 in^2	Rho = 1.32%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 636173 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 636173 in^4
Beta1 = 0.85		Clear spacing = 5.32 in	Clear cover = 3.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

02:05 PM

```
0000000    00000    00000    00000    00000   00
00     00 00   00 00   00 00   00 00   00 00
00     00 00   00 00   00 00   00 00   00 00
00     00 00      00   00 00      00   00 00
00     00 00      0000000 00      00   00 00
0000000   00   00 00   00 00   00 00   00 00
00        00   00 00   00 00   00 00   00 00
00         00000  00   00  00000   00000  00000  (TM)
```

Computer program for the Strength Design of Reinforced Concrete Sections

02:05 PM

General Information:
====================
File Name: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\SCARHAM CREEK\PCA COLUMN\BENT2 COLUMN.col
Project: Scarham Creek
Column: Bent 2 Engineer: PJC
Code: ACI 318-02 Units: English

Run Option: Investigation Slenderness: Not considered
Run Axis: X-axis Column Type: Structural

Material Properties:
====================
f'c = 4 ksi fy = 60 ksi
Ec = 3605 ksi Es = 29000 ksi
Ultimate strain = 0.003 in/in
Beta1 = 0.85

Section:
========
Circular: Diameter = 60 in

Gross section area, Ag = 2827.43 in^2
Ix = 636173 in^4 Iy = 636173 in^4
Xo = 0 in Yo = 0 in

Reinforcement:
==============
Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Circular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area, As = 37.44 in^2 at 1.32%
24 #11 Cover = 3 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
==

No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	853.0	185.0	5035.5	27.219
2	2018.0	185.0	5677.2	30.688
3	2168.0	185.0	5604.4	30.294
4	1725.0	3315.0	5782.9	1.744
5	19.0	3315.0	3895.5	1.175
6	853.0	432.0	5035.5	11.656

*** Program completed as requested! ***

Designer: P J C
Project Name: GRAHAM CREEK
Job Number:
Date: 11 / 10 / 2010

Moment Capacity and Interaction Diagram Data

Column or Drilled Shaft: BENT 2 DS

INPUTS

Column Diameter	66	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	# 11	
Number of Lonitudinal Reinforcing Bars	24	
Transverse Reinforcing Bar Size	# 6	
Spacing of Transverse Reinforcing Bars	12	in
Column Height	X	ft
Cover	6	in

Dead Load Reactions

PuDead	872	kips
MuDead	185	kip*ft

Enter into Column Design Software (PCA COLUMN)
Finding Moment Capacity of the column.

Mp = ϕMp/0.9 → ______ kip*ft

Vpcol = 2*Mp/Heightcolumn → ______ kips

Vpbent = 2*Vpcol ______ kips

Create SAP model of Bent
Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Puoverturning = ______ kips

Pu = PuDead +/- Puoverturning ______ or ______ kips

Re-enter into Column Design Software (PCA COLUMN)

Mp = ϕMp/0.9 → ______ kip*ft

Vpcol = 2*Mp/Heightcolumn → ______ kips

Vpbent = 2*Vpcol ______ kips

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

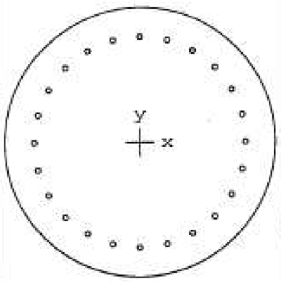
Check = (Vpbent2 - Vpbent1) / Vpbent2) * 100% ______

Check Other Seismic Load Cases

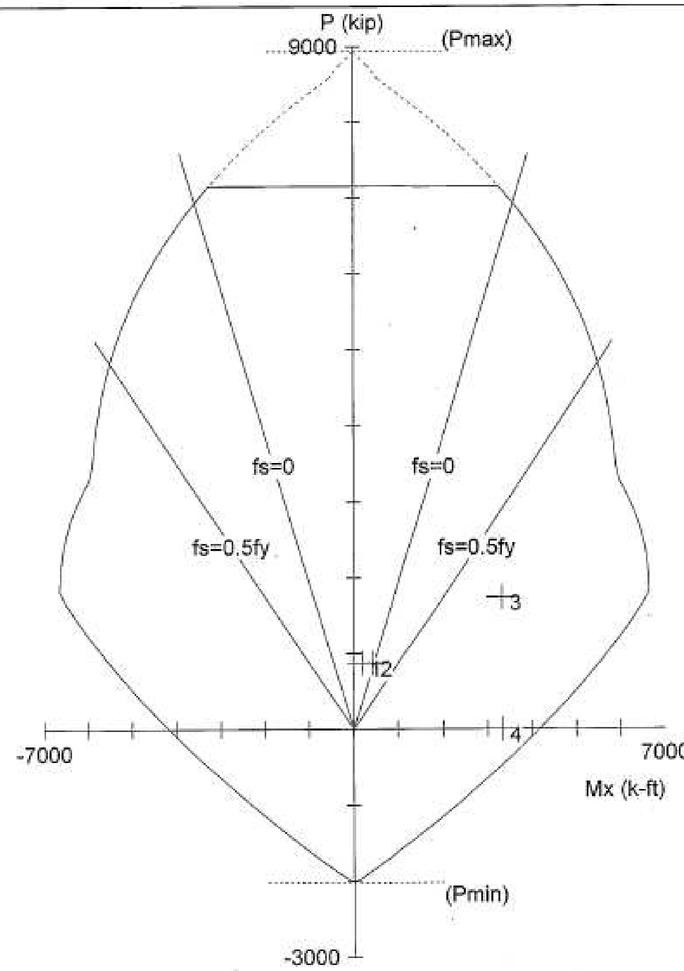
Mupotrans	3130	kip*ft	petran =	0.359
Putran	872	kips	pelong =	0.502
Mupolong	247	kip*ft		
Pulong	0.6	kips		

Pu = PuDead +/- Putran → 1744 or 0 kips
Mu = MuDead + Mupotran → 3315 kip*ft

Pu = PuDead +/- Pulong → 872 or 872 kips
Mu = MuDead + Mupolong → 432 kip*ft



66 in diam.



Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/10/10

Time: 14:10:06

pcaColumn v3.64. Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59

File: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\SCARHAM CREEK\...\BENT2 DS.col

Project: SCARHAM CREEK

Column: BENT 2 DS		Engineer: PJC	
f'c = 4 ksi	fy = 60 ksi	Ag = 3421.19 in^2	24 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 37.44 in^2	Rho = 1.09%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 931420 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 931420 in^4
Beta1 = 0.85		Clear spacing = 5.32 in	Clear cover = 6.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

02:09 PM

General Information:
====================
File Name: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\SCARHAM CREEK\PCA COLUMN\BENT2 DS.col
Project: SCARHAM CREEK
Column: BENT 2 DS Engineer: PJC
Code: ACI 318-02 Units: English

Run Option: Investigation Slenderness: Not considered
Run Axis: X-axis Column Type: Structural

Material Properties:
====================
f'c = 4 ksi fy = 60 ksi
Ec = 3605 ksi Es = 29000 ksi
Ultimate strain = 0.003 in/in
Beta1 = 0.85

Section:
========
Circular: Diameter = 66 in

Gross section area, Ag = 3421.19 in^2
Ix = 931420 in^4 Iy = 931420 in^4
Xo = 0 in Yo = 0 in

Reinforcement:
==============
Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Circular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area, As = 37.44 in^2 at 1.09%
24 #11 Cover = 6 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
==

No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	872.0	185.0	5512.9	29.799
2	872.0	432.0	5512.9	12.761
3	1744.0	3315.0	6602.5	1.992
4	0.0	3315.0	4160.4	1.255

*** Program completed as requested! ***

Designer: PJC
Project Name: SCABHAR CREEK
Job Number:
Date: 11 / 10 / 2010

Moment Capacity and Interaction Diagram Data
Column or Drilled Shaft: BENT 3 COLUMN

INPUTS

Column Diameter	72	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	# 11	
Number of Lonitudinal Reinforcing Bars	32	
Transverse Reinforcing Bar Size	# 6	
Spacing of Transverse Reinforcing Bars	12	in
Column Height	55	ft
Cover	3	in

Dead Load Reactions

PuDead 1055 kips
MuDead 150 kip*ft

Enter into Column Design Software (PCA COLUMN)
Finding Moment Capacity of the column.

Mp = ϕMp/0.9 → 9050 kip*ft

Vpcol = 2*Mp/Heightcolumn → 330 kips

Vpbent = 2*Vpcol 660 kips

Create SAP model of Bent
Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Puoverturning = 1504 kips 1832

Pu = PuDead +/- Puoverturning 2560 or 449 kips 2887 or 777

Re-enter into Column Design Software (PCA COLUMN)

Mp = ϕMp/0.9 → 11044 kip*ft 10872

Vpcol = 2*Mp/Heightcolumn → 402 kips 396

Vpbent = 2*Vpcol 804 kips 790

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

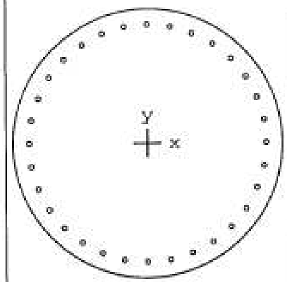
Check = ((Vpbent2 - Vpbent1) / Vpbent2) * 100% 18 % 1.7 %

Check Other Seismic Load Cases

Mupotrans 3967 kip*ft petran = 0.359
Putran 1040 kips pelong = 0.502
Mupolong 143 kip*ft
Pulong 2.3 kips

Pu = PuDead +/- Putran → 2095 or 15 kips
Mu = MuDead + Mupotran → 4117 kip*ft

Pu = PuDead +/- Pulong → 1055 or 1055 kips
Mu = MuDead + Mupolong → 293 kip*ft



72 in diam.

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

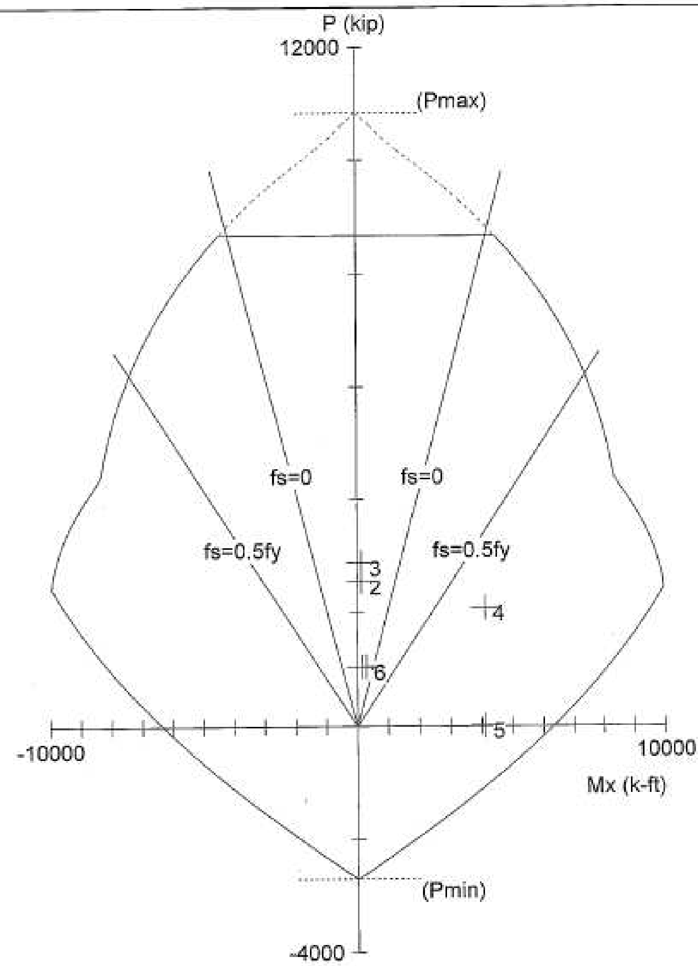
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/10/10

Time: 14:11:07



pcaColumn v3.64. Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59

File: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\SCARHAM CR...\BENT3 COLUMN.col

Project: SCARHAM CREEK

Column: BENT3 COLUMN		Engineer: PJC	
f'c = 4 ksi	fy = 60 ksi	Ag = 4071.5 in^2	32 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 49.92 in^2	Rho = 1.23%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 1.31917e+006 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 1.31917e+006 in^4
Beta1 = 0.85		Clear spacing = 4.82 in	Clear cover = 3.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

02:10 PM

```
   General Information:
   ====================
      File Name: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\SCARHAM
CREEK\PCA COLUMN\BENT3 COLUMN.col
      Project:  SCARHAM CREEK
      Column:   BENT3 COLUMN                   Engineer: PJC
      Code:     ACI 318-02                     Units: English

      Run Option: Investigation                Slenderness: Not considered
      Run Axis:   X-axis                       Column Type: Structural

   Material Properties:
   ====================
      f'c   = 4 ksi                            fy   = 60 ksi
      Ec    = 3605 ksi                         Es   = 29000 ksi
      Ultimate strain = 0.003 in/in
      Beta1 = 0.85

   Section:
   ========
      Circular:    Diameter = 72 in

      Gross section area, Ag =  4071.5 in^2
      Ix =  1.31917e+006 in^4                  Iy =  1.31917e+006 in^4
      Xo =  0 in                               Yo =  0 in

   Reinforcement:
   ==============
      Rebar Database: ASTM A615
      Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)
      ---- --------- -----------   ---- --------- -----------   ---- --------- -----------
      #  3      0.38        0.11   #  4      0.50        0.20   #  5      0.63        0.31
      #  6      0.75        0.44   #  7      0.88        0.60   #  8      1.00        0.79
      #  9      1.13        1.00   # 10      1.27        1.27   # 11      1.41        1.56
      # 14      1.69        2.25   # 18      2.26        4.00

      Confinement: Tied; #3 ties with #10 bars,  #4 with larger bars.
      phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.65

      Layout: Circular
      Pattern: All Sides Equal  (Cover to transverse reinforcement)
      Total steel area, As = 49.92 in^2 at 1.23%
      32 #11    Cover = 3 in

   Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
   =========================================================
                   Pu          Mux          fMnx
      No.          kip         k-ft         k-ft    fMn/Mu
      --- ------------ ------------ ------------ --------
        1       1055.0        150.0       8145.0   54.300
        2       2560.0        150.0       9940.0   66.267
        3       2887.0        150.0       9785.4   65.236
        4       2095.0       4117.0       9557.4    2.321
        5         15.0       4117.0       6369.0    1.547
        6       1055.0        293.0       8145.0   27.799

      *** Program completed as requested! ***
```

Designer: PJC
Project Name: SCHEHAN CREEK
Job Number:
Date: 11/10/2010

Moment Capacity and Interaction Diagram Data

Column or Drilled Shaft: BENT 3 DS

INPUTS

Column Diameter	78	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	# 11	
Number of Lonitudinal Reinforcing Bars	32	
Transverse Reinforcing Bar Size	# 6	
Spacing of Transverse Reinforcing Bars	12	in
Column Height	X	ft
Cover	6	in

Dead Load Reactions

PuDead	1075	kips
MuDead	150	kip*ft

Enter into Column Design Software (PCA COLUMN)
Finding Moment Capacity of the column.

Mp = ϕMp/0.9 → ________ kip*ft

Vpcol = 2*Mp/Heightcolumn → ________ kips

Vpbent = 2*Vpcol ________ kips

Create SAP model of Bent
Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Puoverturning = ________ kips

Pu = PuDead +/- Puoverturning ________ or ________ kips

Re-enter into Column Design Software (PCA COLUMN)

Mp = ϕMp/0.9 → ________ kip*ft

Vpcol = 2*Mp/Heightcolumn → ________ kips

Vpbent = 2*Vpcol ________ kips

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

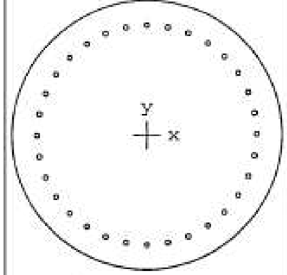
Check = ((Vpbent2 - Vpbent1) / Vpbent2) * 100% ________

Check Other Seismic Load Cases

Mupotrans	3967	kip*ft	petran =	0.359
Putran	1040	kips	pelong =	0.502
Mupolong	143	kip*ft		
Pulong	2.3	kips		

Pu = PuDead +/- Putran → 2115 or 35 kips
Mu = MuDead + Mupotran → 4117 kip*ft

Pu = PuDead +/- Pulong → 1075 or 1075 kips
Mu = MuDead + Mupolong → 293 kip*ft



78 in diam.

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

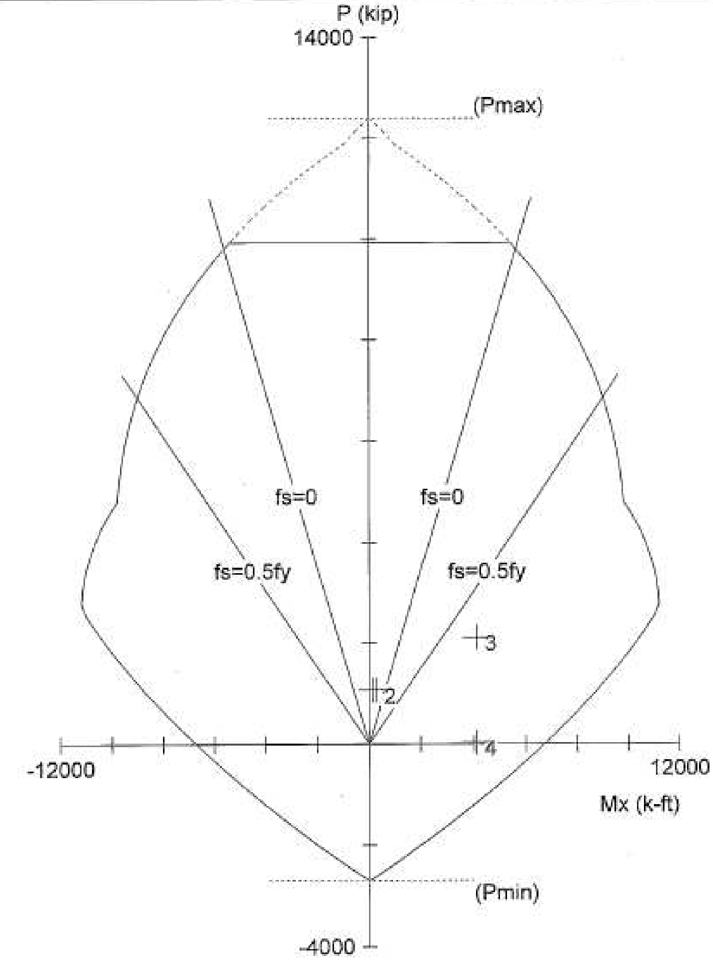
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/10/10

Time: 14:11:42



pcaColumn v3.64. Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59

File: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\SCARHAM CREEK\...\BENT3 DS.col

Project: SCARHAM CREEK

Column: BENT3 DS		Engineer: PJC	
f'c = 4 ksi	fy = 60 ksi	Ag = 4778.36 in^2	32 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 49.92 in^2	Rho = 1.04%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 1.81697e+006 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 1.81697e+006 in^4
Beta1 = 0.85		Clear spacing = 4.82 in	Clear cover = 6.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

02:11 PM

```
General Information:
====================
   File Name: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\SCARHAM
CREEK\PCA COLUMN\BENT3 DS.col
   Project:  SCARHAM CREEK
   Column:   BENT3 DS                       Engineer: PJC
   Code:     ACI 318-02                     Units: English

   Run Option: Investigation                Slenderness: Not considered
   Run Axis:   X-axis                       Column Type: Structural

Material Properties:
====================
   f'c   = 4 ksi                            fy   = 60 ksi
   Ec    = 3605 ksi                         Es   = 29000 ksi
   Ultimate strain = 0.003 in/in
   Beta1 = 0.85

Section:
========
   Circular:    Diameter = 78 in

   Gross section area, Ag =  4778.36 in^2
   Ix =  1.81697e+006 in^4                  Iy =  1.81697e+006 in^4
   Xo =  0 in                               Yo =  0 in

Reinforcement:
==============
   Rebar Database: ASTM A615
   Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)
   ---- --------- -----------   ---- --------- -----------   ---- --------- -----------
   #  3     0.38        0.11    #  4     0.50        0.20    #  5     0.63        0.31
   #  6     0.75        0.44    #  7     0.88        0.60    #  8     1.00        0.79
   #  9     1.13        1.00    # 10     1.27        1.27    # 11     1.41        1.56
   # 14     1.69        2.25    # 18     2.26        4.00

   Confinement: Tied; #3 ties with #10 bars,  #4 with larger bars.
   phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.65

   Layout: Circular
   Pattern: All Sides Equal  (Cover to transverse reinforcement)
   Total steel area, As = 49.92 in^2 at 1.04%
   32 #11   Cover = 6 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
=========================================================
                  Pu          Mux         fMnx
   No.           kip         k-ft         k-ft   fMn/Mu
   --- ------------ ------------ ------------ --------
     1       1075.0        150.0       8779.0   58.527
     2       1075.0        293.0       8779.0   29.963
     3       2115.0       4117.0      10431.5    2.534
     4         35.0       4117.0       6793.5    1.650

   *** Program completed as requested! ***
```

Designer: PJC

Project Name: SCRE HAM CREEK

Job Number:

Date: 11/10/2010

Moment Capacity and Interaction Diagram Data

Column or Drilled Shaft: BENT 4 COLUMN

INPUTS

Column Diameter	60	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	# 11	
Number of Lonitudinal Reinforcing Bars	24	
Transverse Reinforcing Bar Size	# 6	
Spacing of Transverse Reinforcing Bars	12	in
Column Height	25	ft
Cover	3	in

Dead Load Reactions

PuDead	860	kips
MuDead	155	kip*ft

Enter into Column Design Software (PCA COLUMN)

Finding Moment Capacity of the column.

Mp = ϕMp/0.9 → 5604 kip*ft

Vpcol = 2*Mp/Heightcolumn → 448 kips

Vpbent = 2*Vpcol 896 kips

Create SAP model of Bent

Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Puoverturning = 1191 kips 1337

Pu = PuDead +/- Puoverturning 2051 or 331 kips 2197 or 477

Re-enter into Column Design Software (PCA COLUMN)

Mp = ϕMp/0.9 → 6292 kip*ft 6207

Vpcol = 2*Mp/Heightcolumn → 503 kips 497

Vpbent = 2*Vpcol 1006 kips 994

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

Check = ((Vpbent2 - Vpbent1) / Vpbent2) * 100% 10.9% 1.2%

Check Other Seismic Load Cases

Mupotrans	3752	kip*ft	petran =	0.359
Putran	991	kips	pelong =	0.502
Mupolong	289	kip*ft		
Pulong	2.5	kips		

Pu = PuDead +/- Putran → 1851 or 151 kips

Mu = MuDead + Mupotran → 3907 kip*ft

Pu = PuDead +/- Pulong → 860 or 860 kips

Mu = MuDead + Mupolong → 444 kip*ft

02:12 PM

General Information:
====================

File Name: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\SCARHAM CREEK\PCA COLUMN\BENT4 COLUMN.col
Project: SCARHAM CREEK
Column: BENT4 COLUMN
Code: ACI 318-02
Engineer: PJC
Units: English

Run Option: Investigation
Run Axis: X-axis
Slenderness: Not considered
Column Type: Structural

Material Properties:
====================

f'c = 4 ksi
Ec = 3605 ksi
Ultimate strain = 0.003 in/in
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

Section:
========

Circular: Diameter = 60 in

Gross section area, Ag = 2827.43 in^2
Ix = 636173 in^4
Iy = 636173 in^4
Xo = 0 in
Yo = 0 in

Reinforcement:
==============

Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

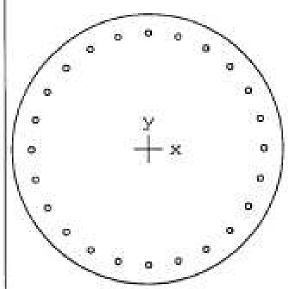
Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Circular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area, As = 37.44 in^2 at 1.32%
24 #11 Cover = 3 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
===

No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	860.0	155.0	5044.1	32.543
2	2051.0	155.0	5663.1	36.536
3	2197.0	155.0	5586.6	36.042
4	1851.0	3907.0	5741.4	1.470
5	131.0	3907.0	4069.6	1.042
6	860.0	444.0	5044.1	11.361

*** Program completed as requested! ***



60 in diam.

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

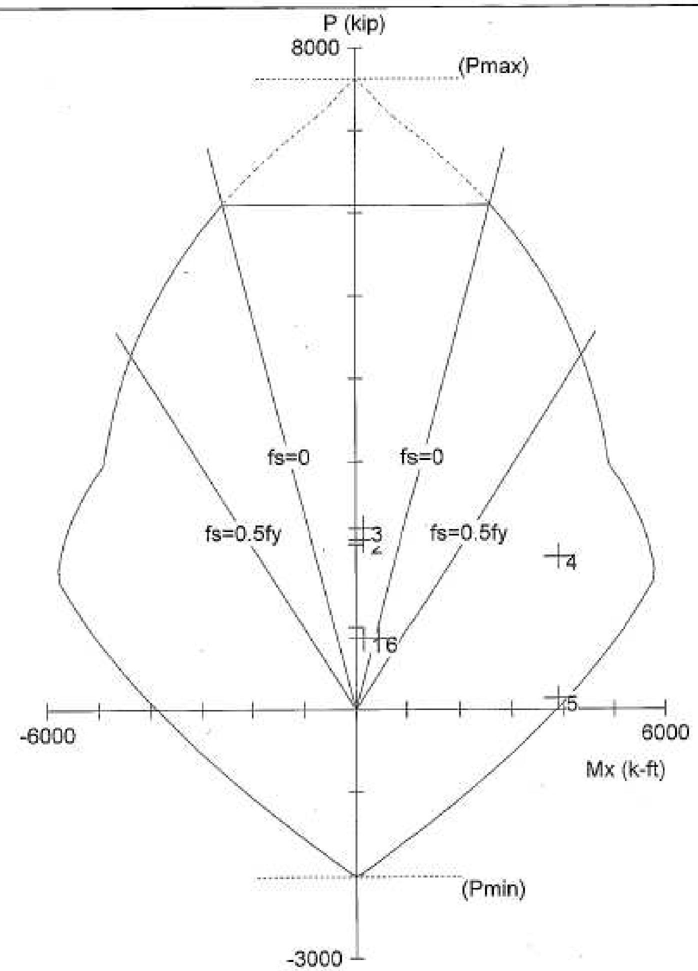
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/10/10

Time: 14:12:39



pcaColumn v3.64. Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59

File: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\SCARHAM CR...\BENT4 COLUMN.col

Project: SCARHAM CREEK

Column: BENT4 COLUMN		Engineer: PJC	
f'c = 4 ksi	fy = 60 ksi	Ag = 2827.43 in^2	24 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 37.44 in^2	Rho = 1.32%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 636173 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 636173 in^4
Beta1 = 0.85		Clear spacing = 5.32 in	Clear cover = 3.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

Designer: PJC

Project Name: SCREHAM CREEK

Job Number:

Date: 11 / 10 / 2010

Moment Capacity and Interaction Diagram Data

Column or Drilled Shaft: BENT 4 TO 5

INPUTS

Column Diameter	60	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	# 11	
Number of Lonitudinal Reinforcing Bars	24	
Transverse Reinforcing Bar Size	# 6	
Spacing of Transverse Reinforcing Bars	12	in
Column Height	K	ft
Cover	6	in

Dead Load Reactions

PuDead 884 kips

MuDead 155 kip*ft

Enter into Column Design Software (PCA COLUMN)

Finding Moment Capacity of the column.

Mp = ϕMp/0.9 → ________ kip*ft

Vpcol = 2*Mp/Heightcolumn → ________ kips

Vpbent = 2*Vpcol ________ kips

Create SAP model of Bent

Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Puoverturning = ________ kips

Pu = PuDead +/- Puoverturning ________ or ________ kips

Re-enter into Column Design Software (PCA COLUMN)

Mp = ϕMp/0.9 → ________ kip*ft

Vpcol = 2*Mp/Heightcolumn → ________ kips

Vpbent = 2*Vpcol ________ kips

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

Check = [(Vpbent2 - Vpbent1) / Vpbent2] * 100% ________

Check Other Seismic Load Cases

Mupotrans 3752 kip*ft   petran = 0.359

Putran 991 kips   pelong = 0.502

Mupolong 289 kip*ft

Pulong 2.5 kips

Pu = PuDead +/- Putran → 1875 or 107 kips

Mu = MuDead + Mupotran → 2907 kip*ft

Pu = PuDead +/- Pulong → 884 or 884 kips

Mu = MuDead + Mupolong → 444 kip*ft

02:12 PM

General Information:
====================
File Name: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\SCARHAM CREEK\PCA COLUMN\BENT4 DS.col
Project: SCARHAM CREEK
Column: BENT4 DS   Engineer: PJC
Code: ACI 318-02   Units: English

Run Option: Investigation   Slenderness: Not considered
Run Axis: X-axis   Column Type: Structural

Material Properties:
====================
f'c = 4 ksi   fy = 60 ksi
Ec = 3605 ksi   Es = 29000 ksi
Ultimate strain = 0.003 in/in
Beta1 = 0.85

Section:
========
Circular: Diameter = 66 in

Gross section area, Ag = 3421.19 in^2
Ix = 931420 in^4   Iy = 931420 in^4
Xo = 0 in   Yo = 0 in

Reinforcement:
==============
Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Circular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area, As = 37.44 in^2 at 1.09%
24 #11 Cover = 6 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
===

No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	884.0	155.0	5529.8	35.676
2	1875.0	3907.0	6649.2	1.702
3	107.0	3907.0	4341.4	1.111
4	884.0	444.0	5529.8	12.455

*** Program completed as requested! ***

Designer: PJC
Project Name: SCARHAM CREEK
Job Number:
Date: 11/10/2010

Moment Capacity and Interaction Diagram Data
Column or Drilled Shaft: ABUTMENT 1 DS

INPUTS

Column Diameter	~~42~~ 54	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	# 11	
Number of Lonitudinal Reinforcing Bars	~~18~~ 24	
Transverse Reinforcing Bar Size	# 5	
Spacing of Transverse Reinforcing Bars	12	in
Column Height		ft
Cover	6	in

Dead Load Reactions

PuDead	305	kips
MuDead	2352	kip*ft

Enter into Column Design Software (PCA COLUMN)
Finding Moment Capacity of the column.

Mp = ϕMp/0.9 → ______ kip*ft

Vpcol = 2*Mp/Heightcolumn → ______ kips

Vpbent = 2*Vpcol ______ kips

Create SAP model of Bent
Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Puoverturning = ______ kips

Pu = PuDead +/- Puoverturning ______ or ______ kips

Re-enter into Column Design Software (PCA COLUMN)

Mp = ϕMp/0.9 → ______ kip*ft

Vpcol = 2*Mp/Heightcolumn → ______ kips

Vpbent = 2*Vpcol ______ kips

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

Check = ((Vpbent2 - Vpbent1) / Vpbent2) * 100% ______

Check Other Seismic Load Cases

MAJOR ✓ 374 = MINOR

Mupotrans	0.	kip*ft	petran =	0.359
Putran	3	kips	pelong =	0.502
Mupolong	864	kip*ft		
Pulong	0.7	kips		

Pu = PuDead +/- Putran → 305 or 305 kips
Mu = MuDead + Mupotran → 2352 or 374 kip*ft

Pu = PuDead +/- Pulong → 305 or 305 kips
Mu = MuDead + Mupolong → 3216 kip*ft

02:13 PM

General Information:

File Name: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\SCARHAM CREEK\PCA COLUMN\ABUTMENT1.col

Project: SCARHAM CREEK
Column: ABUTMENT1
Code: ACI 318-02

Engineer: PJC
Units: English

Run Option: Investigation
Run Axis: X-axis

Slenderness: Not considered
Column Type: Structural

Material Properties:

f'c = 4 ksi
Ec = 3605 ksi
Ultimate strain = 0.003 in/in
Beta1 = 0.85

fy = 60 ksi
Es = 29000 ksi

Section:

Circular: Diameter = 54 in

Gross section area, Ag = 2290.22 in^2
Ix = 417393 in^4
Xo = 0 in

Iy = 417393 in^4
Yo = 0 in

Reinforcement:

Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

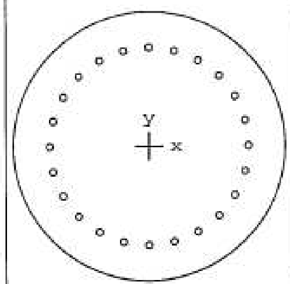
Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Circular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area, As = 37.44 in^2 at 1.63%
24 #11 Cover = 6 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)

No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	305.0	374.0	3452.1	9.230
2	305.0	2352.0	3452.1	1.468
3	305.0	3216.0	3452.1	1.073

*** Program completed as requested! ***



54 in diam.

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

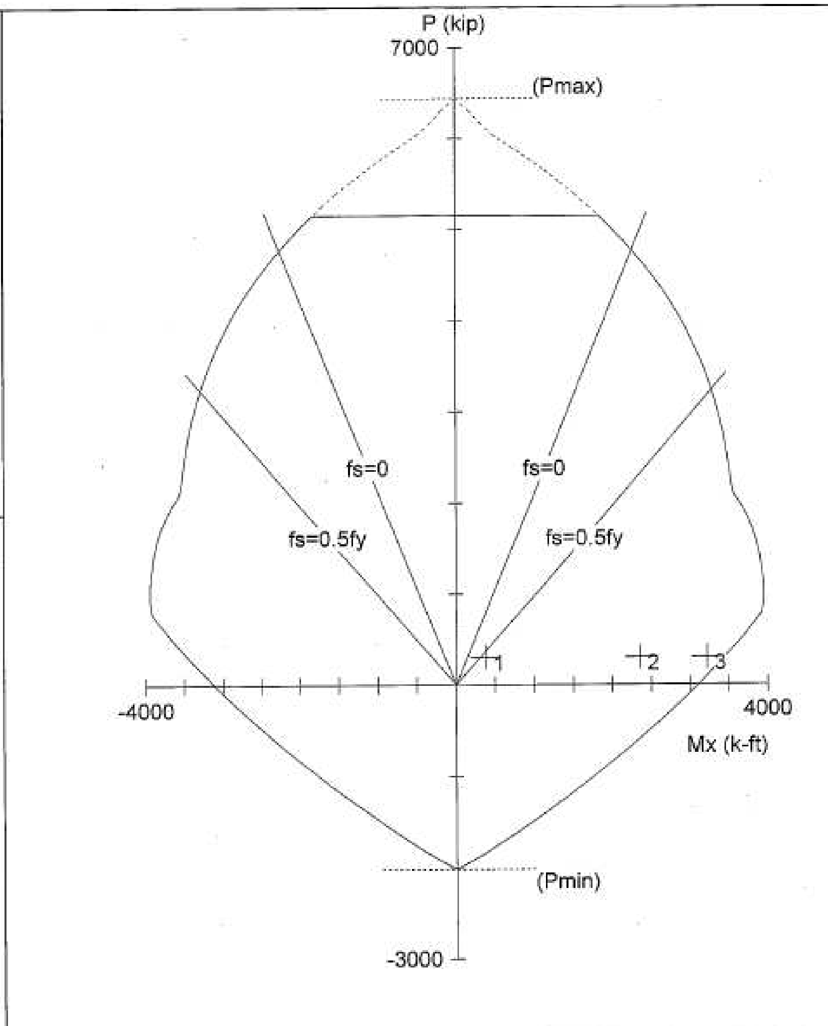
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/10/10

Time: 14:14:12



pcaColumn v3.64. Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59

File: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\SCARHAM CREE...\ABUTMENT1.col

Project: SCARHAM CREEK

Column: ABUTMENT1 Engineer: PJC

f'c = 4 ksi	fy = 60 ksi	Ag = 2290.22 in^2	24 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 37.44 in^2	Rho = 1.63%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 417393 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 417393 in^4
Beta1 = 0.85		Clear spacing = 3.76 in	Clear cover = 6.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

Designer: PSC
Project Name: SCHEHUM CREEK
Job Number:
Date: 11/10/2010

Moment Capacity and Interaction Diagram Data

Column or Drilled Shaft: ABUTMENT 5 DS

INPUTS

Column Diameter	~~42~~ 54	in
f'c	4	ksi
fy	60	ksi
Longitudinal Reinforcing Bar Size	# 11	
Number of Lonitudinal Reinforcing Bars	~~16~~ 24	
Transverse Reinforcing Bar Size	# 5	
Spacing of Transverse Reinforcing Bars	12	in
Column Height		ft
Cover	6	in

Dead Load Reactions

PuDead	325	kips
MuDead	2255	kip*ft

Enter into Column Design Software (PCA COLUMN)
Finding Moment Capacity of the column.

Mp = ϕMp/0.9 → ______ kip*ft

Vpcol = 2*Mp/Heightcolumn → ______ kips

Vpbent = 2*Vpcol ______ kips

Create SAP model of Bent
Apply shear (Vpbent) at the center of mass of the substructure

Axial Force due to Vpbent → Puoverturning = ______ kips

Pu = PuDead +/- Puoverturning ______ or ______ kips

Re-enter into Column Design Software (PCA COLUMN)

Mp = ϕMp/0.9 → ______ kip*ft

Vpcol = 2*Mp/Heightcolumn → ______ kips

Vpbent = 2*Vpcol ______ kips

Verify that Vpbent is within 10% of first Vpbent. If not, repeat the above process.

Check = ((Vpbent2 - Vpbent1) / Vpbent2) * 100% ______

Check Other Seismic Load Cases

MAJOR MINOR = 1021

Mupotrans	0	kip*ft	petran =	0.351
Putran	3.59	kips	pelong =	0.502
Mupolong	689	kip*ft		
Pulong	1.5	kips		

Pu = PuDead +/- Putran → 325 or 325 kips
Mu = MuDead + Mupotran → 1021 / 1084 2255 kip*ft

Pu = PuDead +/- Pulong → 325 or 325 kips
Mu = MuDead + Mupolong → 2939 kip*ft

02:14 PM

General Information:
====================
File Name: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\SCARHAM CREEK\PCA COLUMN\ABUTMENT2.col
Project: SCARHAM CREEK
Column: ABUTMENT2 Engineer: PJC
Code: ACI 318-02 Units: English

Run Option: Investigation Slenderness: Not considered
Run Axis: X-axis Column Type: Structural

Material Properties:
====================
f'c = 4 ksi fy = 60 ksi
Ec = 3605 ksi Es = 29000 ksi
Ultimate strain = 0.003 in/in
Beta1 = 0.85

Section:
========
Circular: Diameter = 54 in

Gross section area, Ag = 2290.22 in^2
Ix = 417393 in^4 Iy = 417393 in^4
Xo = 0 in Yo = 0 in

Reinforcement:
==============
Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

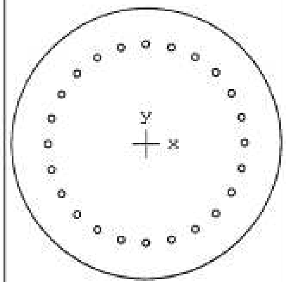
Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Circular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area, As = 37.44 in^2 at 1.63%
24 #11 Cover = 6 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
===

No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	325.0	2255.0	3474.0	1.541
2	325.0	2939.0	3474.0	1.182
3	325.0	1021.0	3474.0	3.403

*** Program completed as requested! ***



54 in diam.

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

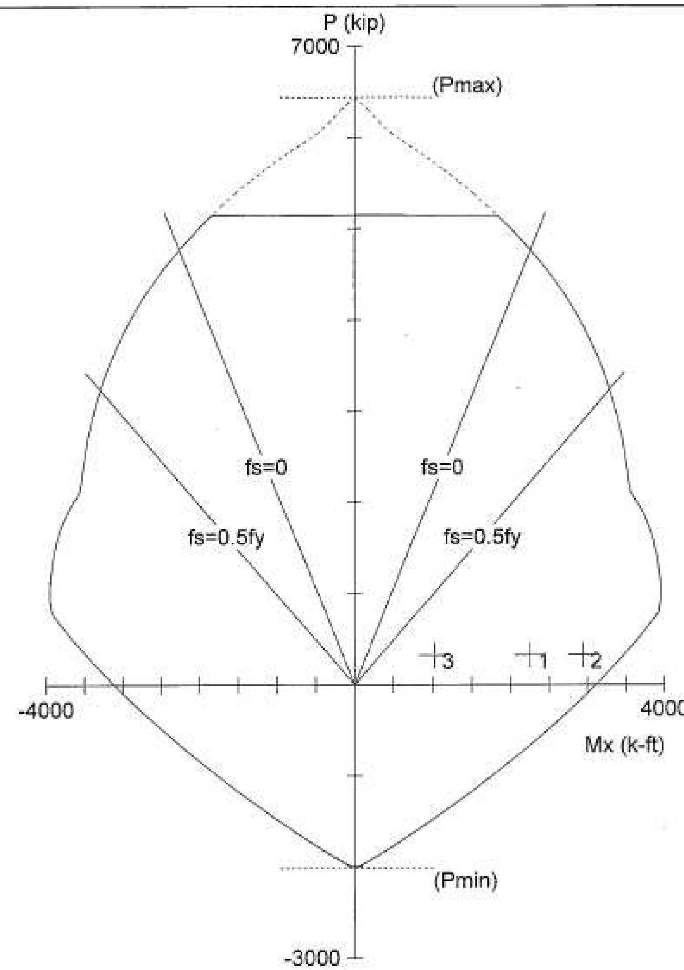
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/10/10

Time: 14:14:44



pcaColumn v3.64. Licensed to: Auburn University. License ID: 53161-1011561-4-20C30-27D59

File: C:\Documents and Settings\coulspj\Desktop\Seismic Design Research\SCARHAM CREE...\ABUTMENT2.col

Project: SCARHAM CREEK

Column: ABUTMENT2 Engineer: PJC

f'c = 4 ksi	fy = 60 ksi	Ag = 2290.22 in^2	24 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 37.44 in^2	Rho = 1.63%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 417393 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 417393 in^4
Beta1 = 0.85		Clear spacing = 3.76 in	Clear cover = 6.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		