Update of Bridge Design Standards in Alabama for AASHTO LRFD Seismic Design Requirements

by

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Abstract

The Alabama Department of Transportation (ALDOT) has been required to update their bridge design specifications from the Standard Specifications for Highway Bridges to the LRFD Bridge Design Specifications. This transition has resulted in changes to the seismic design standards of bridges in the state. These changes, as well as their resulting effects on the design of bridges, have been researched and are discussed in this thesis. One of the goals was to determine if standard drawings and details for bridges in Seismic Design Categories A and B, which are low to moderate seismic regions, could be generated. Multiple bridges, provided by ALDOT, were re-designed so that they satisfied the requirements of the LRFD Specifications. These new design details were used to create standard drawings for bridges in SDC A and B. The superstructure-to-substructure connection was also investigated to determine if it was adequate to resist the expected horizontal design forces. It was determined to be inadequate, but instead of proposing a new connection design, the original connection was recommended along with supplying an extended seat width in the longitudinal direction. A new equation for determining the minimum seat width was recommended, and this new design philosophy was incorporated into the re-design of the bridges.

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List of Symbols

A_v = area of shear reinforcement

$A_{v,min}$ = minimum area of shear reinforcement

b_v = effective web width

B = width of superstructure

B_o = column diameter or width measured parallel to direction under consideration

C_{sm} = dimensionless elastic seismic response coefficient

d_{bl} = nominal diameter of longitudinal column reinforcing bars

d_v = effective web depth

f_c' = compressive strength of concrete

f_y = specified minimum yield strength of steel

f_{ye} = expected yield strength of longitudinal column reinforcing steel bars

F_v = site coefficient for 1.0-sec period spectral acceleration

g = acceleration due to gravity

H = average column height

H_o = average clear height of column

K = effective lateral bridge stiffness

L = distance between joints (Eq. 3.1, Eq. 4.2, Eq. 4.3)

L = column length from point of maximum moment to point of moment contraflexure (Eq. 3.7)

L = total length of the structure (Eq. 3.8, Eq. 3.10, Eq. 3.11, Eq. 3.12, Eq. 3.13)

L_p = plastic hinge length

N = minimum seat length

p_e = equivalent static earthquake loading

$p_e(x)$ = intensity of the equivalent static earthquake loading

p_o = uniform lateral load applied over the length of the structure

s = spacing of transverse reinforcement

S = skew angle of bridge

Sa = design response spectral acceleration coefficient

S_1 = 1.0-sec period spectral acceleration

S_{D1} = design earthquake response spectral acceleration coefficient at 1.0-sec period

T_m = period of the bridge

$v_{s,max}$ = maximum lateral displacement due to uniform load p_o

$v_s(x)$ = deformation corresponding to p_o

V_c = nominal shear resistance of the concrete

V_s = nominal shear resistance of the steel

w(x) = nominal, unfactored dead load of the bridge superstructure and substructure

W = total weight of structure

α = skew angle of bridge (Eq. 2.1)

α = generalized flexibility (Eq. 3.11)

β = factor indicating ability of diagonally cracked concrete to transmit tension and shear (3.2)

β = generalized participation (Eq. 3.12)

γ = generalized mass

θ = angle of inclination of diagonal compressive stresses

Δ_C = displacement capacity taken along the local principal axis corresponding to Δ_D of the ductile member

Δ_D = global seismic displacement demand

$\Delta_{D,LONG}$ = seismic displacement demand in the longitudinal direction

$\Delta_{D,\ TRAN}$ = seismic displacement demand in the transverse direction

Δ_{EQ} = expected displacement demand

Λ = factor for column end restraint condition

Chapter 1: Introduction

1.1 Problem Statement

The Alabama Department of Transportation (ALDOT) currently designs precast prestressed concrete bridges in the state of Alabama using the latest edition (17^{th}) of the American Association of State Highway and Transportation (AASHTO) Standard Specification (Standard Specifications for Highway Bridges, 2002). This specification, which was originally based on allowable stress design (ASD) theory and since updated to include Load and Resistance Factor (LRFD) principles, has not been updated since 2002. Recently, ALDOT has been required to update their bridge design specifications to the AASHTO LRFD Design Specifications (LRFD Bridge Design Specifications, 2009). This specification is based on LRFD principles and is updated every few years by AASHTO. Some of the major changes in the new specification have been in the area of seismic design, which prompted ALDOT to update their seismic design criteria. A previous study by Coulston and Marshall (2011) concluded that the AASHTO Guide Specifications for LRFD Seismic Bridge Design (Guide Specifications for LRFD Seismic Bridge Design, 2009) is an acceptable alternative to the seismic design criteria in the LRFD Specification. This project deals specifically with updating the seismic design criteria for ALDOT in low seismic regions (SDC A and B) as well as addressing the superstructure-to-substructure horizontal strength connection.

1.2 Problem Overview

During an earthquake, inertial forces are generated by the bridge in response to the ground accelerations. The larger the ground accelerations, the larger the inertial forces in the bridge. During the design process for low seismic regions, such as Seismic Design Category (SDC) B, these expected forces are typically applied as static lateral loads on the bridge. The bridge must maintain a complete load path from the point of load application to the foundation, with each element being able to resist the loads acting on the bridge. Since bridge design is focused on preventing collapse and ensuring that bridges remain open to at least emergency vehicles after a design earthquake, the desirable behavior for a bridge experiencing extreme loading conditions is for the substructure of the bridge to receive damage without loss of span. This allows the superstructure of the bridge, the roadway deck and girders, to be passable. Therefore, the superstructure of the bridge is designed to remain elastic during a seismic event, while the substructure of the bridge is designed to dissipate energy through inelastic response. This is accomplished by designing for plastic hinging to occur in the columns and/or foundations, which allows the substructure to dissipate energy through cracking of the concrete and yielding of steel. Plastic hinges form when reinforcement in one cross section yields, without failure, and allows the element to redistribute moments from additional loads to cross sections that have not yielded (Wight & MacGregor, 2009). In order for these plastic hinges to occur in the columns, the columns must be designed as ductile elements. Ductility is defined as the ability of the structure to absorb and dissipate energy without significant strength loss. Research following the 1994 Northridge and 1995 Kobe earthquakes showed the importance of having ductile substructures to prevent failure of a bridge. If the substructure is not ductile, it will not be able to dissipate all the energy from the earthquake and the entire bridge will be at risk of collapse. Specific reinforcement detailing is required to allow for plastic hinging to occur in bridge columns. Both the Standard Specification and the Guide Specification address the importance of detailing for ductility in SDC B. However, the Standard Specification results in most of the state being classified as SDC A, for which no minimum detailing is required. This occurs because the seismic hazard maps used in the Standard Specification were last updated in 1988 and are based on a return period of 475 years. The research that has been incorporated into new seismic hazard maps is included in the Guide Specifications, which uses maps from 2007. They are based on a design earthquake of 1000 years that has been determined by seismological research. These maps result in the classification of many more bridges in the state as SDC B. Therefore, the Standard Specification does not require bridges in the state to be designed as having ductile substructures, while the Guide Specification does.

1.3 Project Deliverables

This thesis reports on two separate objectives that are related to the changes in the bridge design specifications. The first is a recommendation for a new superstructure to substructure connection. It was assumed that the current connection would not allow for a complete load path during an earthquake. One of the first steps was studying already established connections used by other state DOTs. These different options were analyzed based on safety, constructability and economy. Once a final recommendation concerning the connection was approved by the ALDOT Bridge Bureau, it was included in the new bridge designs.

The second objective was a refinement of design standards for those bridges classified as SDC B and the development of standard drawings and design sheets for bridges in SDC A. Some design standards had been developed in a previous project by Coulston and Marshall

(2011); these were refined by using two additional case studies to show the differences between the two specifications. Computer aided design sheets were created for each of the bridges studied in SDC B, and each of the bridges studied in SDC A. Also, two additional bridge models were created for the two additional bridges studied.

1.4 Project Outline

This thesis is organized into five chapters and multiple appendices. The first chapter is an introduction to the problem and description of the thesis. The second chapter is a literature review, including a discussion on the differences between the two design specifications. The third chapter describes an analysis of the current superstructure-to-substructure connection and recommends a new design. The fourth chapter reviews the seismic design process for bridges in SDC A and B and provides detailed procedures used for the design of each bridge. The fifth chapter concludes the thesis and presents the final design recommendations. The appendices contain the design sheets for each of the bridges studied, moment-axial load interaction diagrams for bridges (where appropriate) and the connection design calculations.

Chapter 2: Literature Review

2.1 Introduction

The Standard Specification and the LRFD Specification express different design philosophies, which control the design procedure of the bridge. Research is constantly completed in the area of seismic design that results in a better understanding of bridge behavior during an earthquake and, consequently, better design procedures to mitigate poor behavior. The Standard Specification was first compiled in 1921 using allowable stress design (ASD) standards. ASD uses elastic analysis to determine the stresses in an element. It requires that these calculated stresses be less than the allowable stress the material can withstand divided by a factor of safety. Only one factor of safety is used, incorporating uncertainties in both the load and material resistance. However, this factor of safety does not recognize that some loads are more variable than others. In the 1970s, load factor design (LFD) was introduced to the Standard Specifications. It requires the nominal strength to be greater than the factored load demand and uses two factors of safety, one for the load and one for strength reduction, which allow more efficient structures to be designed. The load factors are calibrated for specific loads because LFD recognizes that some loads are more variable than others. In 1994, the first edition of the LRFD Specifications, on which the AASHTO Guide Specifications for LRFD Seismic Bridge Design are based, was published. It uses load and resistance factor (LRFD) design and both elastic and plastic analysis to determine the nominal strength. It also requires the factored nominal strength to be greater than the factored load. LRFD is an extension of LFD, but uses various load and resistance factors that are specifically analyzed for each limit state to account for variability in both resistance and load while achieving a uniform level of safety (Caltrans, 2011). In 2000, the Federal Highway Administration decided to stop updating the Standard Specifications and only maintain the LRFD Specifications. In 2007, states were required to adopt the new LRFD Specification for all bridge design. Any new research in the area of bridge seismic design, such as return periods for design earthquakes, has been addressed in the LRFD Specifications but not in the Standard Specifications. Therefore, the differences in the seismic design of the two specifications is due mainly to continuing research, which has been included in the LRFD Specifications, but not in the Standard Specifications.

2.2 Specification Comparison

One part of this thesis was to develop standard details for bridges in SDC A and B. In order to accomplish this task, the design specifications needed to be compared. This section will discuss the design procedures of each specification and examine the differences between them. As mentioned earlier, the LRFD Specifications are required to be used for bridge design since the Standard Specifications can no longer be used. But another alternative to the LRFD Specifications, in the area of seismic bridge design, is the Guide Specifications. These specifications use a displacement-based design, while the LRFD Specifications use a force-based design. A displacement-based design requires a bridge to meet certain displacement criteria, determined by estimating the inelastic displacement of the bridge using a model that represents the first mode of vibration. The forces are determined from this displacement demand. For example, in Figure 2.1 the actual force at an expected displacement of 6 inches would be about 300 kips, while the elastic force would be 600 kips.

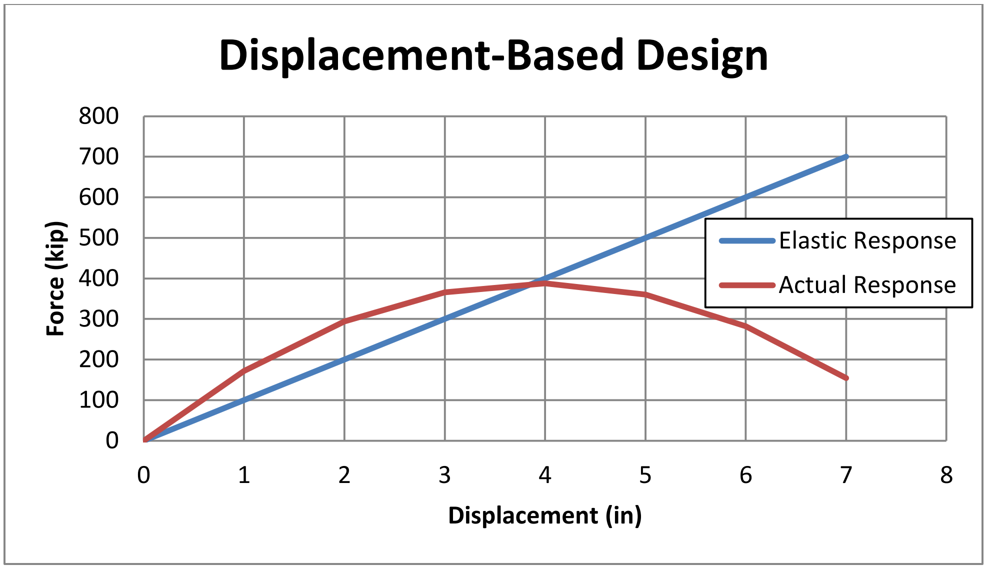

Figure 2.1: Displacement Based Design

A force based design determines the design loads by dividing the elastic force by a response modification factor. The bridge is designed for this lower force, but still expected to achieve the same lateral displacement from the elastic force. For example, in Figure 2.2, the elastic force is 800 kips, but the design force is 200 kips. Both are expected to reach the ultimate displacement of 8 inches, but the elasto-plastic response allows for smaller design forces. In order to achieve this displacement, the structure must be designed to be ductile so that it can dissipate the additional energy expected from the inelastic response.

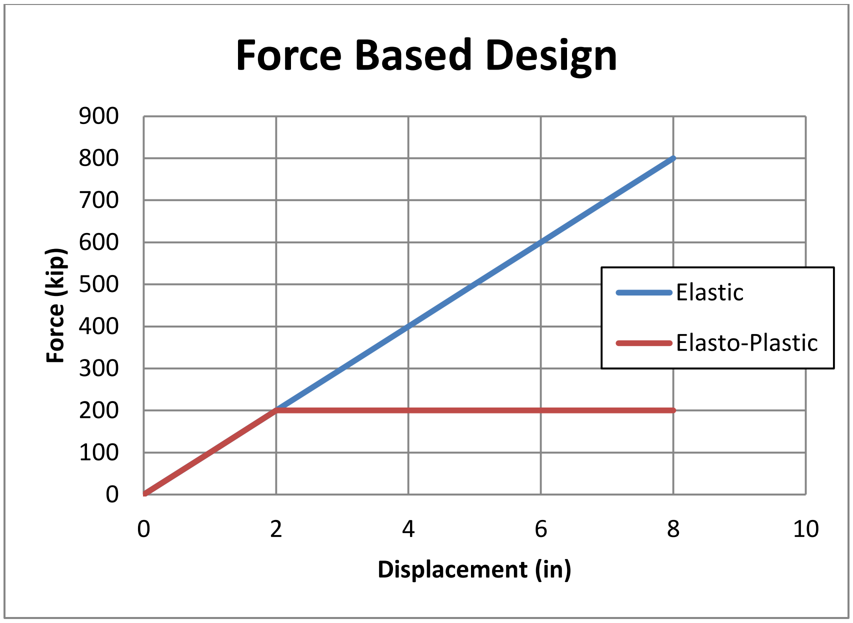

Figure 2.2: Force-Based Design

While most of the current design codes feature a force-based design, recent research has suggested that a displacement-based design better estimates the true response of a bridge. This is one of the reasons why the Guide Specifications are recommended for design instead of the LRFD Specifications. These two design specifications are compared later in this chapter. In order to understand how changes in research have influenced bridge design, the Standard Specifications are discussed first.

2.2.1 Standard Specifications

Like the LRFD Specifications, the Standard Specifications are a force based design. They are applicable only for conventional bridges, meaning those of steel or concrete girder construction with spans less than 500 feet. Bridge sites are classified as one of four Seismic Performance Categories (SPC) based on the acceleration coefficient at the site and importance classification of the bridge. The importance classification comes from the bridge being classified as either “Essential” or “Other.” Bridges classified as “Essential” must remain functional during and after a design earthquake, and "Other" encompasses all other bridges. The acceleration coefficient is determined from the seismic hazard maps, which were last updated in 1988. These maps are based on an estimated return period of 475 years with the soil assumed to be rock. Once the bridge SPC has been classified, the response coefficient is determined based on the acceleration coefficient, soil profile type and bridge period. The soil profiles are based on the type of soil present at the bridge site or by a shear wave velocity test or "other appropriate means of classification" (AASHTO, 2002). Applying these procedures to bridge sites in Alabama results in most bridges in the state being classified as SPC A.

For SPC A, no structural analysis is required to determine the design forces. The horizontal design forces are determined to be 20% of the tributary weight resisted by the substructure. The only other requirement is for the minimum seat width to be provided.

For SPC B, the design forces are determined from an elastic structural analysis and are divided by a response modification factor. The minimum seat width is also required to be provided. One additional requirement in this SPC is minimum detailing requirements in the top and bottom of a column. These minimum details are intended to provide a limited measure of ductility to the column.

2.2.2 LRFD Specifications

The LRFD Specification uses a force based design and is applicable to bridges with conventional construction only. Bridges are classified as one of four Seismic Design Categories (SDC) that are roughly equivalent to the SPC in the Standard Specification. The SDC of a bridge is based on the soil site class and 1.0-second spectral response acceleration coefficient. The soil site classes are divided into six categories, determined using the shear wave velocity, undrained shear strength, or average blow count of the soil. Whereas in the Standard Specifications the soil profile affects the forces after the SPC was determined, in the LRFD Specifications the soil profile is used to determine the SDC and not the design forces. One key difference is the seismic hazard maps used in the LRFD Specifications. Three maps are used to determine the peak ground acceleration, 0.2-second spectral response acceleration, and 1.0-second spectral response acceleration. These maps were updated in 2007 and based on an estimated return period of 1000 years. This results in the ground accelerations in the LRFD Specifications being much larger than those in the Standard Specifications. Also, bridges are classified into three categories: “Critical,” “Essential,” and “Other.” Both “Critical” and “Essential” bridges must remain open after a design earthquake, but “Essential” bridges are designed for earthquakes with 1000-year return period, and “Critical” bridges for earthquakes with 2500-year return period. The LRFD Specifications result in many more bridges in the state of Alabama classified as “Essential” or “Other” to be SDC B. So the biggest difference between the two specifications is the change in the seismic design classification of a bridge, which has a significant effect on its design.

For SDC A, only the horizontal connection forces and minimum seat width are designed. The horizontal connection force is either 15% or 25% of the vertical reaction due to the tributary load depending on the acceleration coefficient at the site. For sites with an acceleration coefficient of less than 0.05g, the connection force is 15% of the vertical reaction, otherwise it is 25%. The Standard Specifications do not allow for a reduction at sites with smaller expected accelerations. The minimum seat width in this design category is calculated using the same equation as the Standard Specification, but is also allowed to be reduced by 25% if the expected acceleration is less than 0.05g.

For SDC B, a structural analysis is required to determine the elastic forces. These elastic forces are then divided by a response modification factor to determine the seismic forces. In this SDC, the minimum seat width is still calculated with the same equation, but the supplied seat width is required to be 150% of the minimum seat width equation to accommodate the full capacity of the plastic hinging mechanism. The major difference in this category compared to the Standard Specifications is the more extensive detailing requirements. These requirements are the same as those required for SDC C and D, with the exception of a larger maximum longitudinal reinforcement ratio limit. These details include designing a plastic hinge zone at the top and bottom of the column that adheres to specific transverse reinforcement spacing requirements, maximum and minimum longitudinal reinforcement ratio limits, and splicing requirements. These design requirements are the result of research in earthquake engineering that has been incorporated into the LRFD Specifications, but not the Standard Specifications.

2.2.3 Guide Specifications

The differences between the Guide Specifications and Standard Specifications are the same as those between the Standard Specifications and LRFD Specifications. For this reason, this section will focus on the differences between the Guide and LRFD Specifications. The Guide Specifications are not applicable for use of "Critical" or "Essential" bridges. They are only for conventional bridges, which fall into the "Other" category in the LRFD Specifications. The largest difference is that the Guide Specifications use a displacement based design, meaning the bridge must satisfy displacement demands at each of the bents and abutments. This makes sure the bridge is capable of transmitting the maximum force effects developed by the plastic hinges into the foundation. The calculation of the horizontal design forces will be discussed next.

The calculation of the horizontal design force in SDC A is the same as in the LRFD Specifications, where it is equal to either 15% or 25% of the vertical reaction. The one difference in this design category is the requirement of bridges to satisfy the minimum detailing requirements of SDC B if they are within 0.05g of the SDC B classification.

A structural analysis is still required for SDC B, but the design forces are not divided by a response modification factor. Once the bridge is determined to have satisfied the displacement demand, the design forces that result from the displacement analysis are used unless the plastic forces are greater. The minimum detailing requirements are similar, with two exceptions. The maximum spacing of the transverse reinforcement in the plastic hinge zone is 6 inches, whereas in the LRFD Specification it is 4 inches. And there is no requirement of an extension of the plastic hinge zone into the bent cap or foundation in the Guide Specifications. The largest difference is the determination of the design forces. The previous study by Coulston and Marshall (2011) determined the Guide Specifications to be an acceptable and more economical alternate for seismic bridge design. These specifications will be used to design the remainder of the bridges in this thesis, except where they specifically require the LRFD Specifications.

2.3 State DOT Connections

Another part of this thesis is the investigation of the superstructure-to-substructure connection. This connection is an important link in the load path. The current connection used by ALDOT was assumed to be inadequate because of its inability to resist the expected loads. In order to find a better connection between the superstructure and substructure, connections currently in use by other state departments of transportations (DOTs) were researched and surveyed for their potential use. States that are in similar or higher seismic hazards as Alabama were contacted. These states include the following: Alaska, Oregon, Georgia, Illinois, North Carolina, South Carolina, Missouri, and Tennessee. The connections will be shown and discussed in Chapter 3.

2.4 Bridge Locations

Seven different bridges were chosen to be re-designed using the Guide Specifications in order to create new bridge standards. These bridge locations can be seen below in Figure 2.3, and are listed in Table 2.1. These bridges were supplied by ALDOT and were chosen because they are representative of many different bridges throughout the state. The two bridges in the southern part of the state, Stave Creek and County Road 39, are in low seismic hazard zones but assumed to be in poor soil conditions. The three bridges in the northern part of the state, Little Bear Creek, Scarham Creek, and Norfolk Southern Railroad, are in the highest seismic zones of the state but are assumed to be over rock. The three remaining bridges are a combination of the two. Having bridges in different locations allowed the standards to be applicable for bridges not just in high seismic zones, but throughout the state.

Table 2.1: Bridge Locations

Number	Bridge	Location
1	Little Bear Creek	Russellville
2	Scarham Creek	Albertville
3	Norfolk Southern RR	Gadsden
4	Oseligee Creek	Lanett
5	Bent Creek Road	Auburn
6	Stave Creek	Jackson
7	County Road 39	Mobile

Figure 2.3: Bridge Locations

2.5 Summary

This chapter has reviewed the reasons why new design standards are necessary for bridges in Alabama. Changes in the seismic hazard maps and research in earthquake engineering have been included in the LRFD and Guide Specifications, but not in the Standard Specifications. These changes have resulted in the bridges in Alabama being classified in higher seismic design categories, which requires different design procedures and has significant impacts on the design requirements for a bridge. The old standards are not based on the new design requirements, and therefore must be updated. The horizontal design forces have also changed, and the superstructure to substructure connection needs to be updated to ensure it can resist these new forces and maintain the load path.

Chapter 3: Superstructure-to-Substructure Connection

3.1 Introduction

The first part of this thesis is the investigation of the superstructure-to-substructure connection. One of the most important aspects of bridge engineering is ensuring a complete load path exists. If there is any element of the bridge that is unable to provide a complete load path, the bridge will not behave as designed and may suffer unexpected failure. The superstructure should be able to resist all of the forces and transfer them to the ductile substructure. Thus, the connection between the superstructure and substructure is very important to ensuring the ductility of the bridge. It must be able to resist the loads in each orthogonal direction and transfer them to the substructure. ALDOT had expressed concern about the current connection and wanted to find another option that is simple to construct, cost effective, and structurally safe. So the first step was to analyze the current ALDOT connection and determine if it was adequate to transfer the loads. Once the problem areas of the connection were identified, other connections from state DOTs were studied to determine if they could be used to design a new connection that addressed the design issues as well as be constructible and economical. This chapter will detail the steps that were taken to design the new connection. All design checks and calculations for this chapter can be found in Appendix A.

3.2 Connection Study

The first step was to review the current connection used by ALDOT, seen below in Figure 3.1. The precast beam rests on the bearing pad and is connected to the bent cap by two steel angles. A 3-inch cap screw with a diameter of 0.875 inches is attached to the side of the beam and an anchor bolt is attached to the bent cap (Alabama DOT, 2012). The two directions of movement are transverse and longitudinal. In the transverse direction, the angles are expected to transfer the loads into the anchor bolts, and in the longitudinal direction, the cap screws would transfer the loads into the anchor bolts. However, after discussion with the Bridge Bureau, it was determined that the cap screw inserts were not adequate to resist the longitudinal forces and a new design in the longitudinal direction was necessary. With this in mind, the other connections from other state DOTs were studied. The clip angles that resist loads in the transverse direction were assumed to be adequate, but this assumption is discussed in Chapter 4.

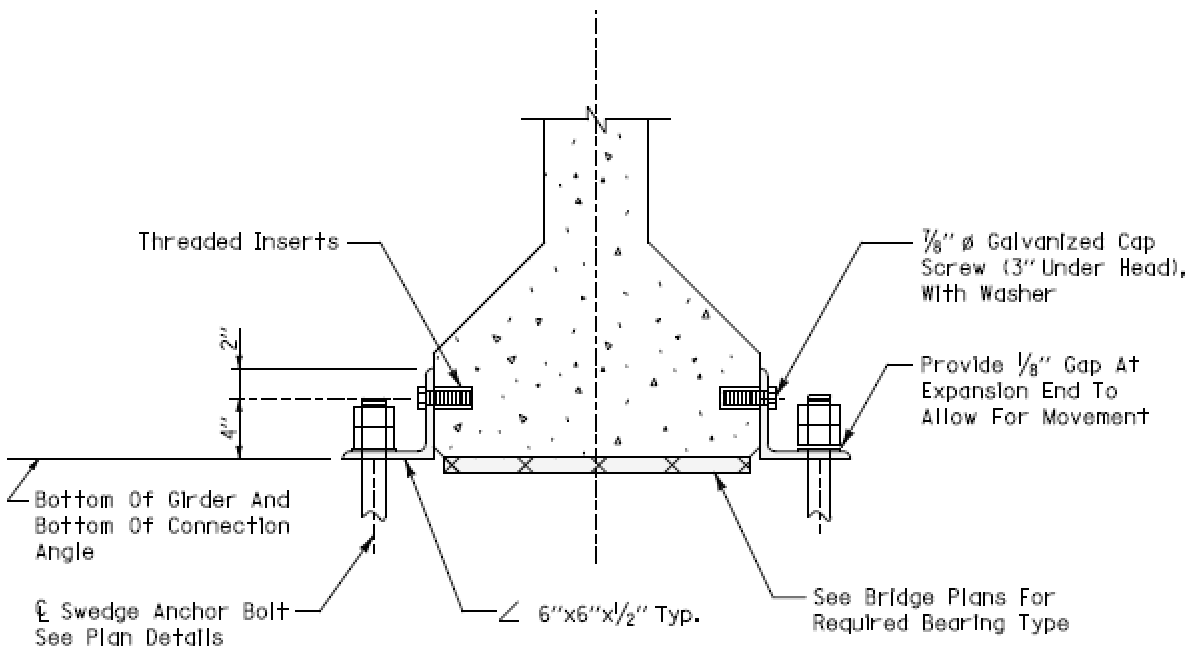

Figure 3.1: Alabama DOT Connection

3.2.1 Modified ALDOT Connection

In a previous study conducted at Auburn University, a modified connection was proposed. This connection is seen in Figure 3.2. By placing a bolt through the bottom of the girder, the longitudinal restraint of the connection was achieved by increasing the bearing area of the concrete which would allow the forces to be transferred into the anchor bolts. The rest of the connection stayed the same, so this design allowed the connection to transfer the forces into the bent cap. However, this bolt interferes with the prestressing strands in the concrete girder. Since these strands in the girder could not be moved without sacrificing strength and ductility, it was determined that the modified connection would not be acceptable (Coulston & Marshall, 2011).

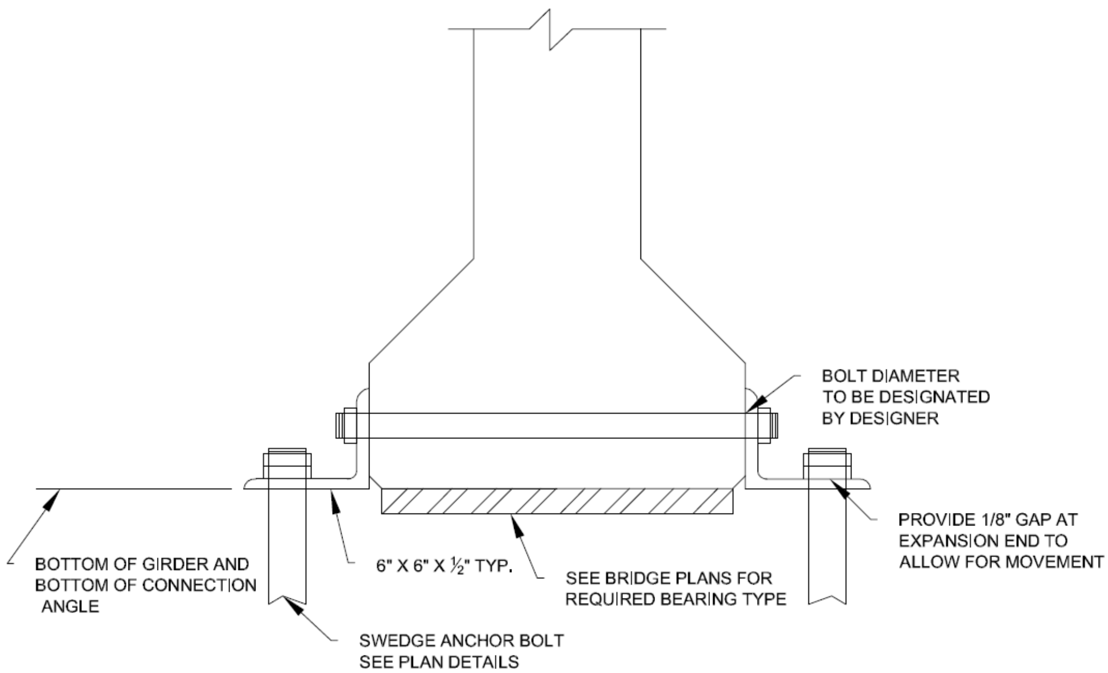

Figure 3.2: Previously Modified ALDOT Connection

3.2.2 Alaska DOT

The next connection to be studied was the Alaska DOT connection, seen in Figure 3.3. It is only used at the abutments. Over the pier bents, the beams rest on a bearing pad and longitudinal bars are used to transfer the horizontal forces into the diaphragms and end abutment walls. The concrete diaphragms and abutment walls transfer the horizontal load to the foundation. Modular bridge joint systems are used at the expansion joints to allow for thermal movement of the bridge. The connection at the abutment has shear studs cast into the bottom of the precast beam. These studs are cast on an anchor plate that is bolted to a steel sliding plate that rests on top of the elastomeric bearing pad (Alaska DOT, 2008). This connection was able to resist the forces in both directions; however, it was designed to be used only at the abutments and not the pier cap seats. ALDOT wanted to use the same connection at the bent and abutment, so this design was not investigated further.

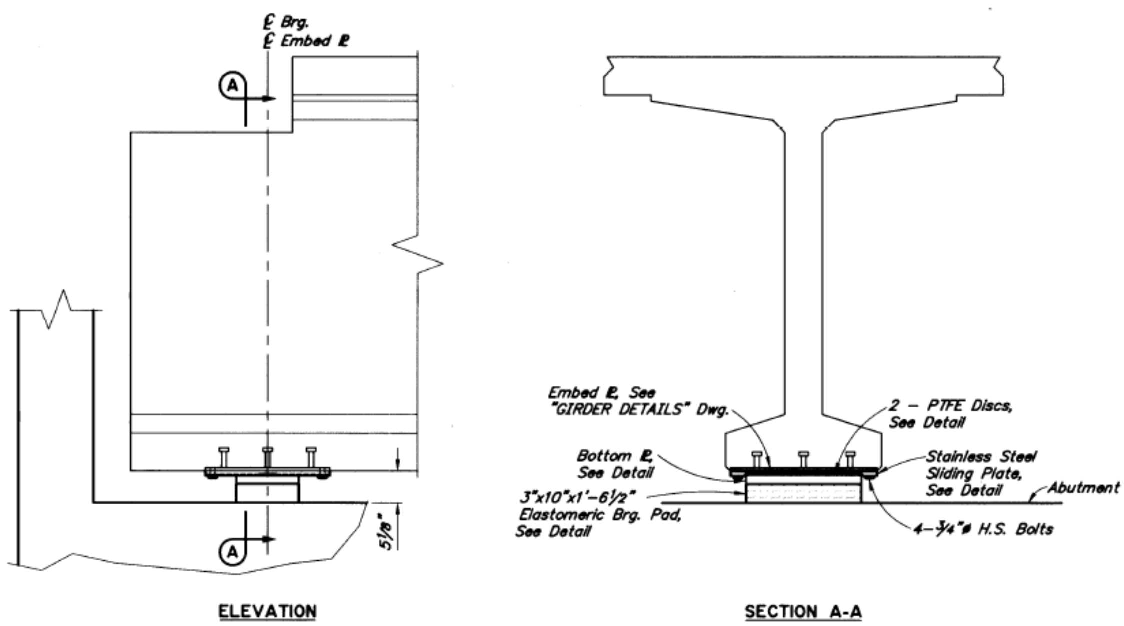

Figure 3.3: Alaska DOT Connection

3.2.3 Oregon DOT

Figure 3.4 and Figure 3.5 show the connection used by Oregon DOT, which was studied next. This connection uses a sole plate with anchor studs cast with the concrete girder to transfer the forces from the girder to the seat. The sole plate rests on a bearing that transfers the forces into the bent cap. The bearing could be of any type (pot, disc, radial, etc.) based on the intended use, such as allowing rotational motion or translational motion. However, it only provides minimal connecting force between the girder and bent. Restrainer rods and cables, which engage beyond a certain deflection, were to be added between the superstructure and substructure if large forces are expected. Transverse shear lugs can also be used to resist lateral forces (Oregon DOT, 2012). This connection did not have a clearly defined load path without adding restrainer rods or cables. From a constructability viewpoint, this connection required more work and would not be something with which ALDOT's contractors would be familiar. For this reason, this connection was not used in the new design.

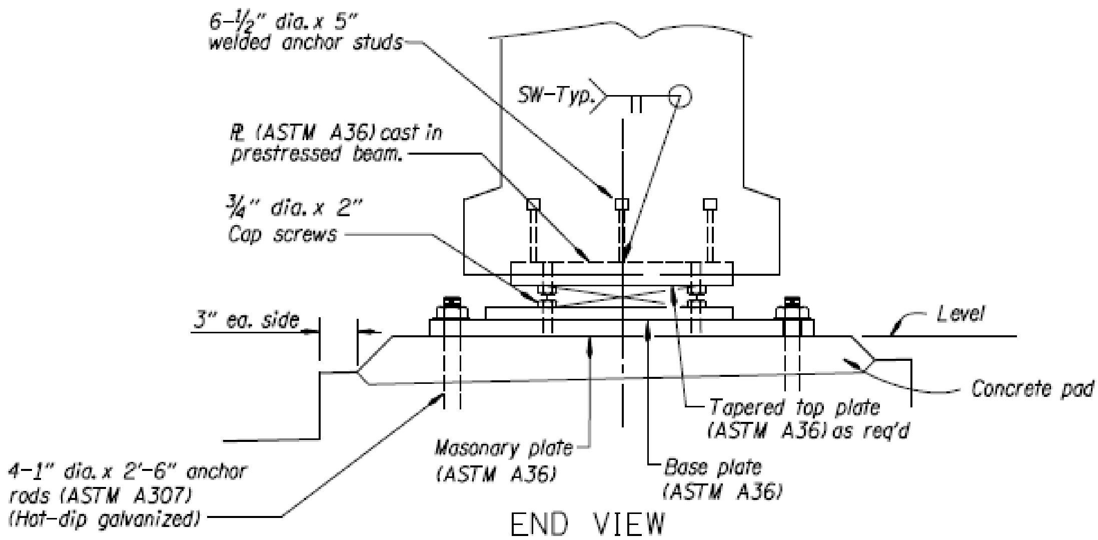

Figure 3.4: Oregon DOT Connection (End View)

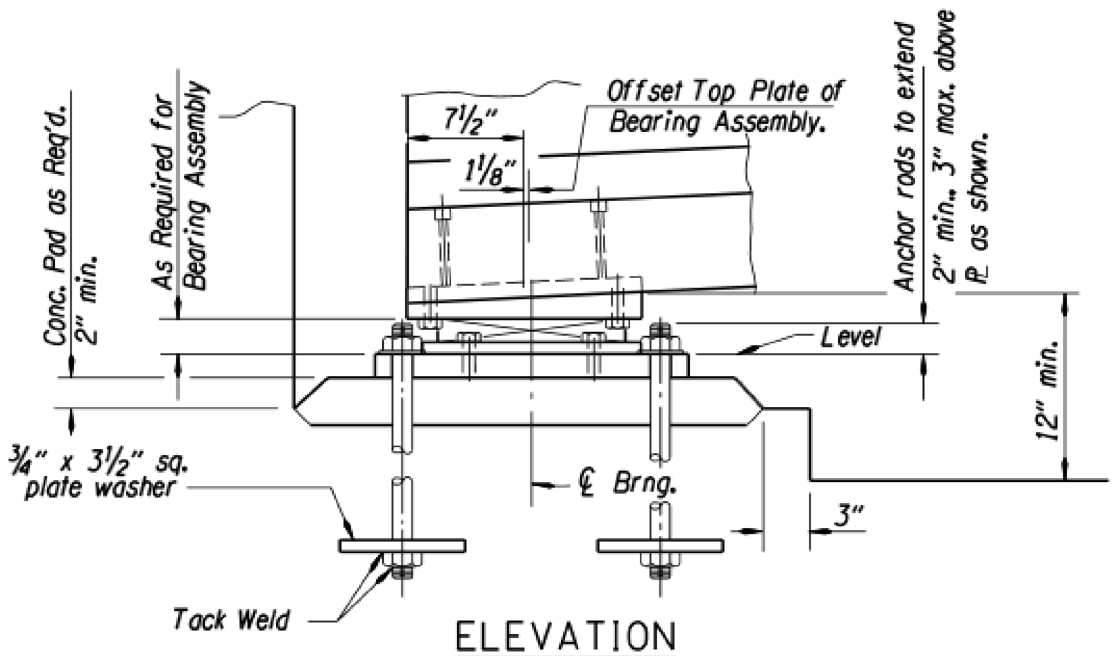

Figure 3.5: Oregon DOT Connection (Elevation View)

3.2.4 Georgia DOT

The next connection studied was the Georgia DOT connection, seen in Figure 3.6. It is a very simple connection from a design standpoint. A steel dowel is embedded at least 10 inches into the bent cap and extended through the bearing pad, at least 5 inches into the beam. At expansion joints, the slot in the beam is typically 6 inches long to allow for day to day thermal movement and construction tolerance. During a seismic event, the dowel will engage the beam and provide restraint in both directions. A 1.5 inch Grade 50 dowel is typically used based on the calculated shear force. While a 1.25 inch steel dowel rod is specified in the figure, Georgia DOT is soon planning to begin using a 1.5 inch rod. Figure 3.7 shows the detail for the end of the precast concrete girder. The reinforcement in the girder is arranged such that the dowel has enough space to anchor through the bottom without affecting the prestressing strands (Georgia DOT, 2012). This connection was simple from a design standpoint, but not very easy to construct. The girders would have to be placed exactly on top of the dowel rod, which was something ALDOT did not feel their contractors would be able to do. For this reason, this connection was not studied further.

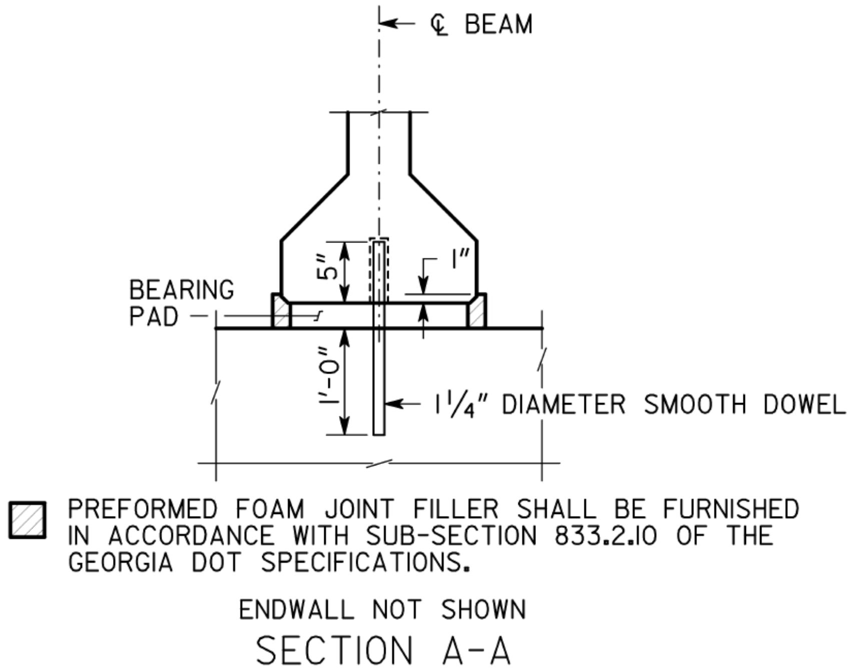

Figure 3.6: Georgia DOT Connection

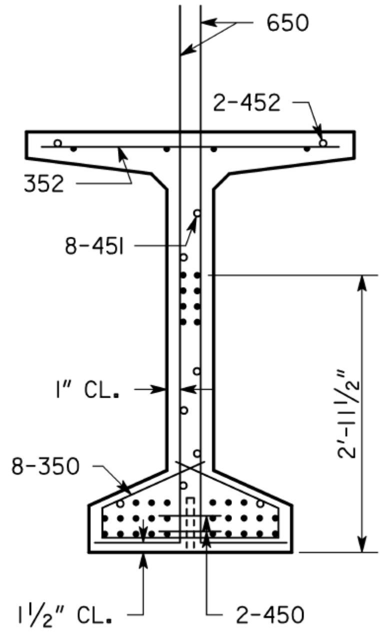

Figure 3.7: Georgia DOT Connection (End Beam)

3.2.5 Illinois DOT

The next connection studied is used by the Illinois DOT. This connection was designed after Illinois conducted research into its earthquake resisting system (ERS). Their new ERS utilizes three tiers to prevent span loss. The first tier is the connection between the superstructure and substructure, seen in Figure 3.8. This connection is designed to provide resistance in the transverse direction. In the longitudinal direction, no restraint is provided, as is evident in the figure. This will result in the connection slipping during a design earthquake, which will dissipate energy. However, the seat width must be large enough to allow the superstructure to "ride out" the remainder of the earthquake since it will not be restrained in the longitudinal direction. The second tier of the ERS is to provide additional seat length. This seat length, calculated using Equation 3.1, is larger than the seat length as calculated in the LRFD Specifications. The third tier includes plastic hinging of columns and foundation elements. The connection has a steel plate cast with the bottom of the concrete girder. The bearing pad assembly is then connected to the girder by pintles. The assembly resists transverse movement by side retainers connected to the bent cap by two anchor bolts (Tobias, Anderson, Hodel, Kramer, Wahab, & Chaput, 2008). This connection has the same problem of the current ALDOT connection, which is a lack of restraint in the longitudinal direction. For this reason, it was not studied further. Other aspects of the Illinois ERS, however, were studied and will be discussed later in this chapter.

$$N = \left(0.10 + 0.0017L + 0.007H + 0.05\sqrt{H}\sqrt{1 + (2\frac{B}{L})^2}\right) * \left(\frac{1+1.25F_v S_1}{\cos(\alpha)}\right) \qquad \text{Equation 3.1}$$

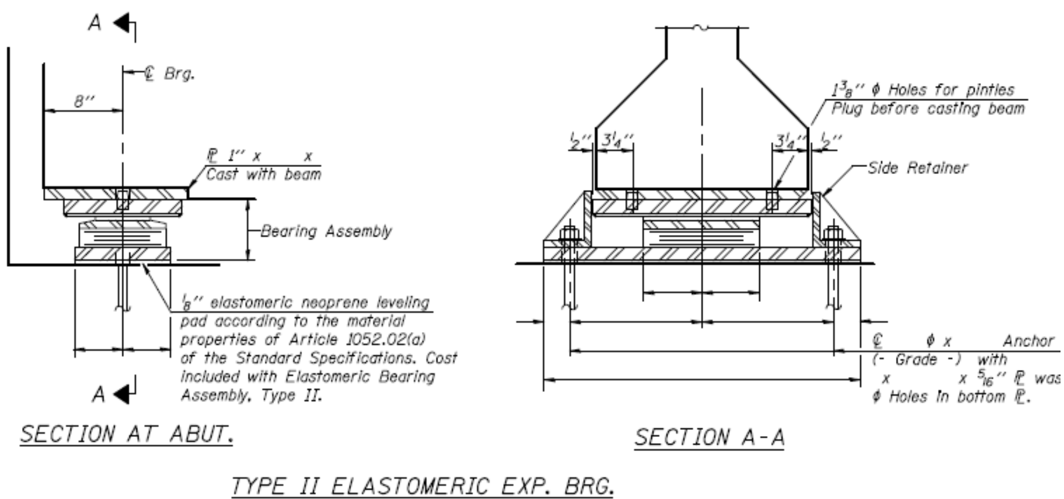

Figure 3.8: Illinois Connection

3.2.6 North Carolina DOT

The next connection that was studied can be seen in Figure 3.9 and is the North Carolina DOT connection. A steel plate is cast on the bottom of the concrete girder. This plate is welded to the sole plate in accordance with detail "A" (Figure 3.10). The sole plate is placed on top of an elastomeric bearing pad, and the entire assembly is connected to the bent cap through two anchor bolts. The sole plate and bearing pad can be slotted to accommodate an expansion joint.

The weld and the anchor bolts are designed to resist the horizontal forces in both the transverse and longitudinal directions (North Carolina DOT, 2012). This connection is very similar to the next two connections to be studied, the South Carolina DOT connection and Missouri DOT connection. Because of their similarity, the other two connections will be discussed before a specific analysis is performed.

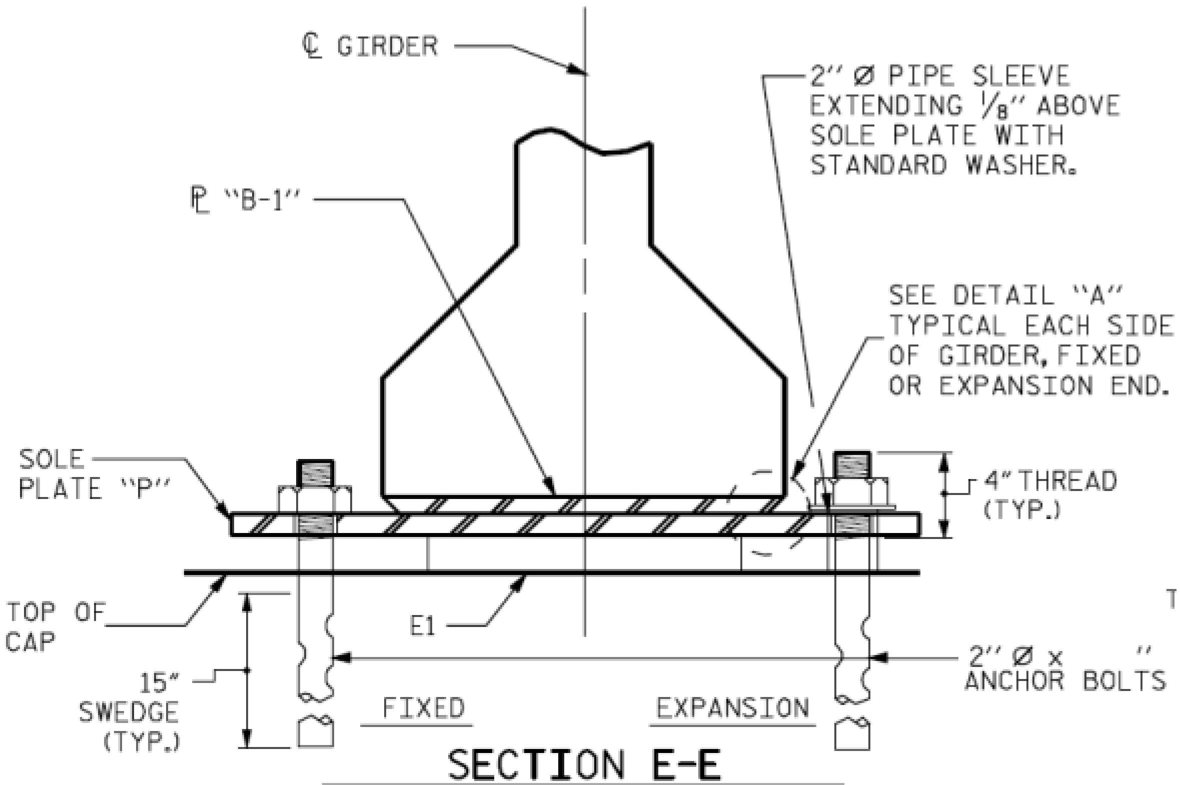

Figure 3.9: North Carolina DOT Connection

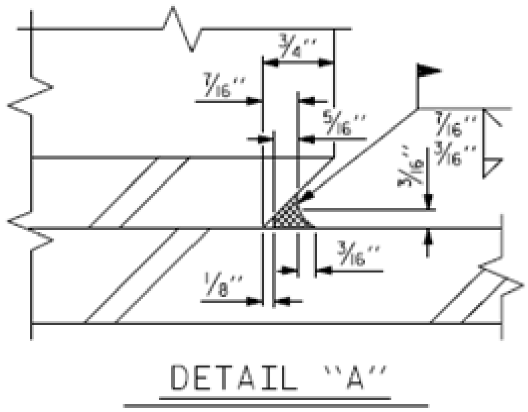

Figure 3.10: North Carolina DOT Connection Detail "A"

3.2.7 South Carolina DOT

The South Carolina DOT connection, seen in Figure 3.11, is used for both expansion and non-expansion bearings. A sole plate is cast with the precast beam and welded to the bearing plate. Two anchor bolts connect the entire assembly to the bent cap. The welds and anchor bolts are designed for the horizontal forces in each direction. For expansion bearings, the bearing plate is slotted to allow for movement (South Carolina DOT, South Carolina Bridge Design Manual, 2006). This connection is very similar to the North Carolina connection because it uses an embedded sole plate welded to a bearing plate that transfers the forces into the bent cap. The Missouri connection will be discussed next, and then the results of an analysis will be presented.

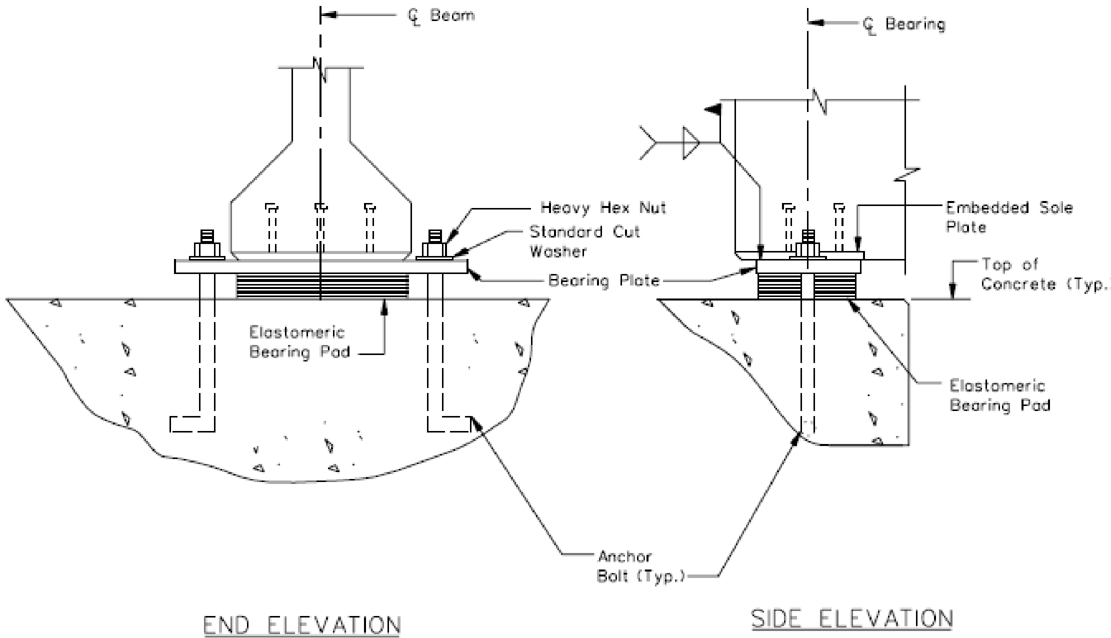

Figure 3.11: South Carolina DOT Connection

3.2.8 Missouri DOT

The Missouri DOT Connection is detailed in Figure 3.12. An anchor plate is cast at the bottom of the girder and welded to a steel plate on top of the elastomeric bearing. This steel plate is bolted to the bent cap with two anchor bolts, which transfer the loads to the bent cap. The anchor bolts are placed above the bearing pad to reduce the deformations in the pad. The weld and anchor bolts provide the resistance for the longitudinal and transverse horizontal forces (Missouri DOT, Bridge Standard Drawings - Bearings, 2009). As mentioned earlier, this connection is very similar to the North Carolina and South Carolina connections. The weld resists the forces in both directions and allows the anchor bolts to transfer the forces into the bent cap. ALDOT has a welded connection design in its standard drawings, seen in Figure 3.13, so it was assumed that the contractors would be familiar with it and be able to construct it. The weld could be designed to resist the appropriate horizontal design force for a specific bridge and also would eliminate the need for cap screws, which were assumed not to transfer any load. For these reasons, a welded design was chosen to be used as the basis for a new connection design. This design will be discussed below.

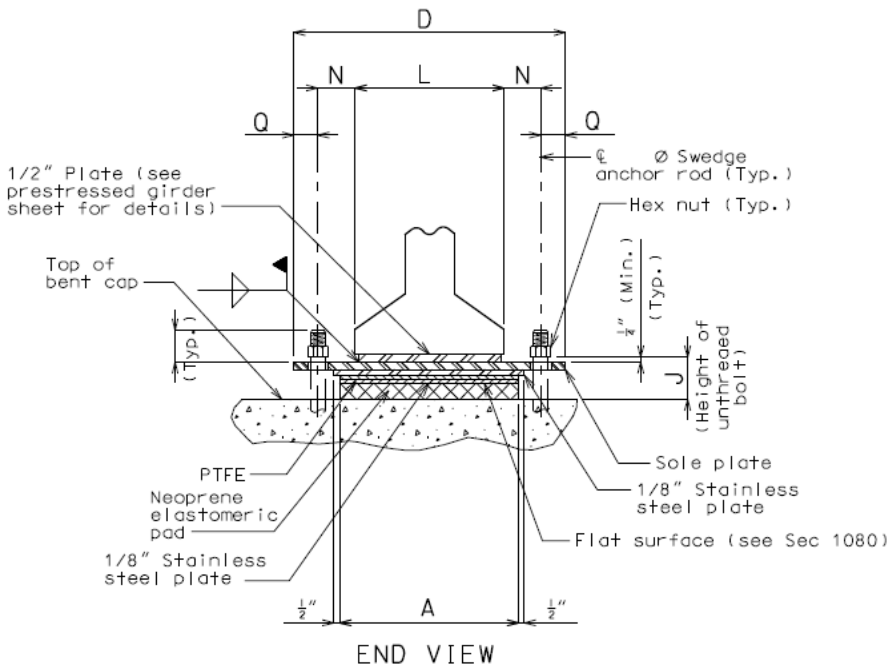

Figure 3.12: Missouri DOT Connection

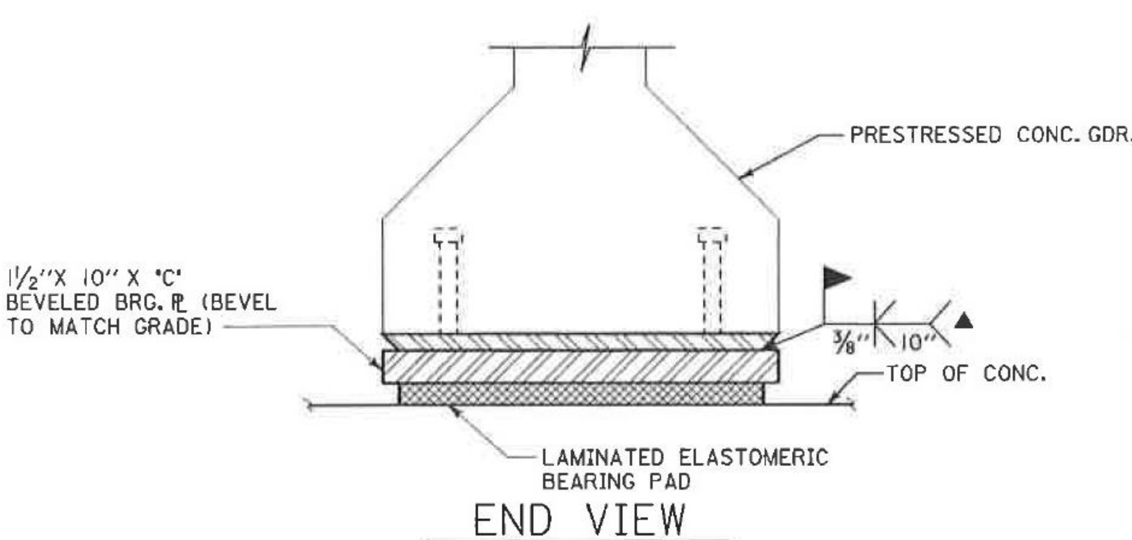

Figure 3.13: ALDOT Welded Connection

3.2.9 Connection Recommendation

The welded plate connection used by ALDOT was used as the backbone for the new design connection. The new connection can be seen in Figure 3.14 and has a sole plate that is cast with the bottom of the concrete girder. Shear studs protrude from the plate into the girder to transfer the forces from the girder. The sole plate would be welded to another plate that rested on the bearing and anchor bolts would connect the assembly with the bent cap. The weld would provide sufficient restraint in both directions, and the anchor bolts would transfer the loads into the bent cap. Another option would be to have the anchor bolts cast with the sole plate, eliminating the need for a weld. However, after discussion with ALDOT, it was decided to keep the current connection because there was concern about the ability of their contractors to be able to transition to a new connection design. Instead, it was decided to allow the connection to move in the longitudinal direction. In order to prevent span loss, either the displacements would be decreased by using longitudinal restrainers, or the seat width would be increased. Both options will be addressed below.

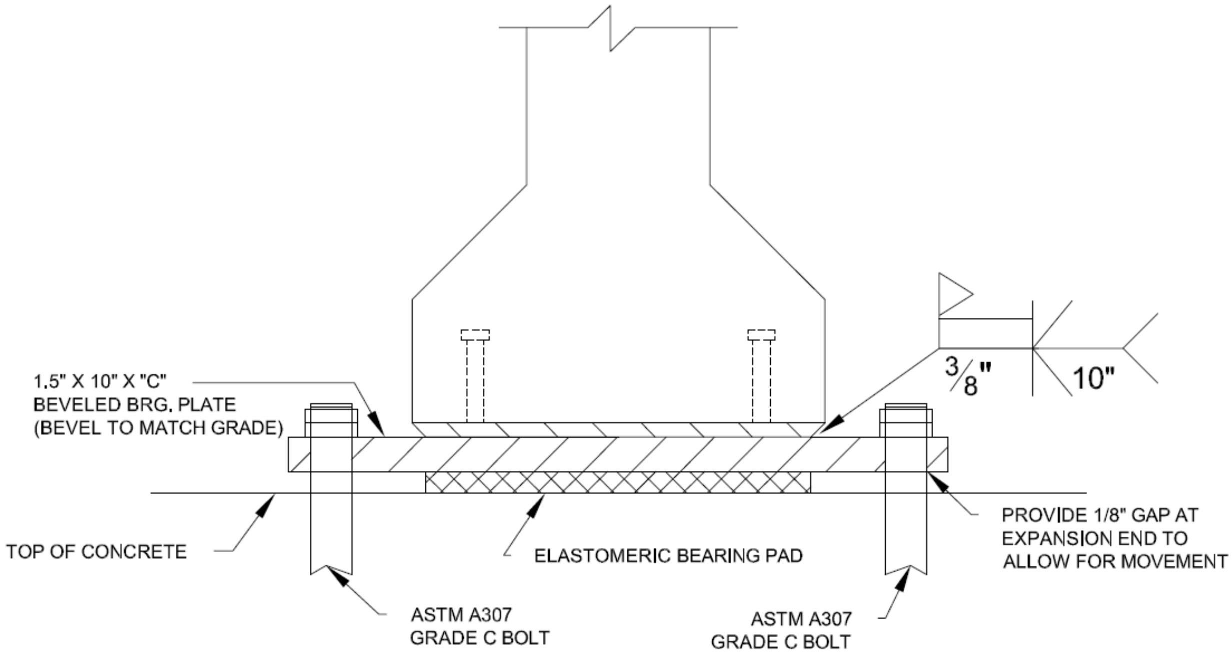

Figure 3.14: Proposed Weld Connection

3.3 Longitudinal Restrainers

The first option for longitudinal design was to limit the displacement of the girders to prevent seating loss. Longitudinal restrainers are placed between girders at expansion joints that prevent the girders from moving in the longitudinal direction and transfer the forces generated by preventing the movement into the bridge deck and girders. Once again, state DOTs were surveyed in order to see what types of longitudinal restrainers existed. The following state DOT details were surveyed because their seismic hazard is equivalent to the most severe hazards expected in Alabama: Tennessee, South Carolina, and Missouri. ALDOT recommended another detail that was designed by Volkert and Associates as a hurricane tie-down in the southern parts of the state. This detail was also studied and will be discussed below along with the other details.

3.3.1 Volkert Heavy Chain Detail

The Volkert detail can be seen in Figure 3.15 and Figure 3.16. It was created by Volkert and Associates, Inc. for ALDOT as a hurricane tie-down connection to keep the girders from becoming unseated during the event of a flood caused by a hurricane. It was thought that these same heavy chain details could be used to limit the longitudinal displacement of the bridge, but their primary design purpose is a vertical restraint against uplift. This detail was not studied further because it would not provide longitudinal restraint (ALDOT, 1971).

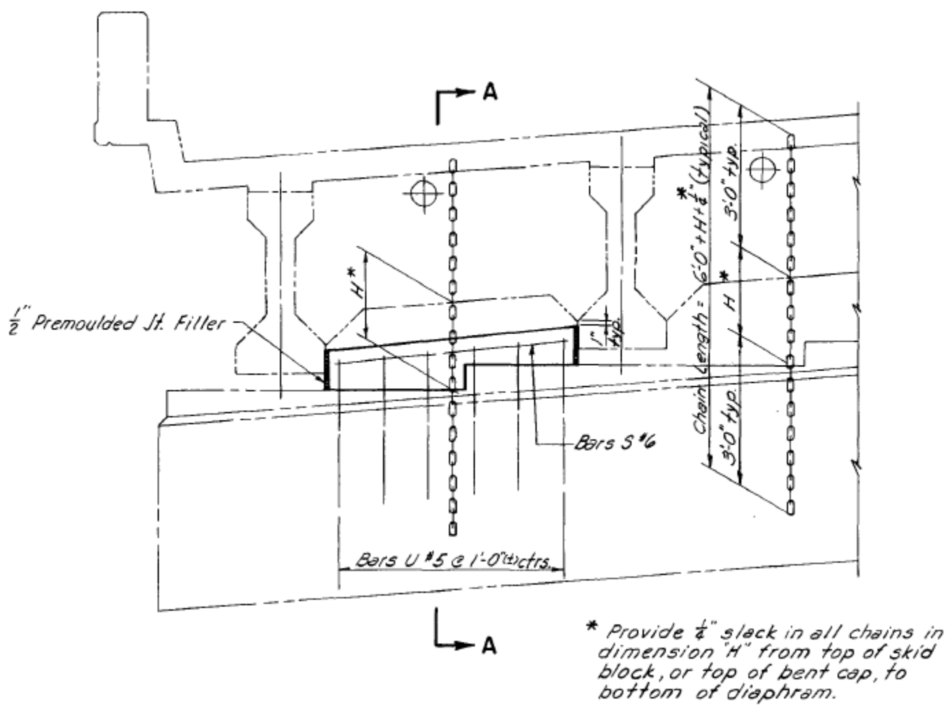

Figure 3.15: Volkert Connection (Elevation)

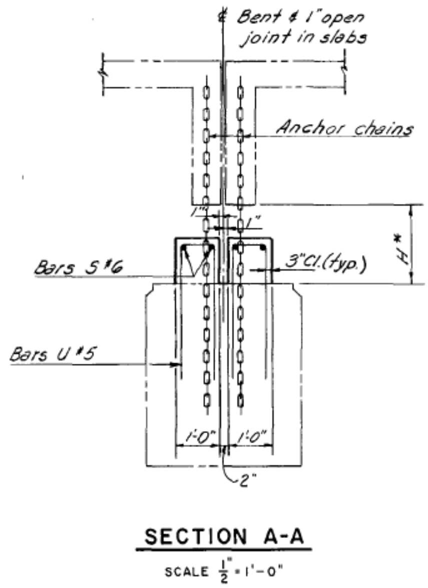

Figure 3.16: Volkert Connection (Section A-A)

3.3.2 Tennessee DOT Longitudinal Restrainer

Figure 3.17 and Figure 3.18 show the longitudinal restrainer detail used by Tennessee. The diaphragm is integral with the bent cap so both lateral and longitudinal horizontal forces are transferred directly into the bent cap through anchor bolts. There is no bearing pad connection and no expansion joints are present in the bridges because expansion is performed by rotation of the substructure. This option provides resistance to the horizontal forces along the entire length of the bent cap instead of just at the bearing pad connection, but requires a deeper diaphragm capable of connecting to the bent cap (Tennessee DOT, 2010). Because ALDOT wanted to use a bearing pad connection, this detail was not studied further.

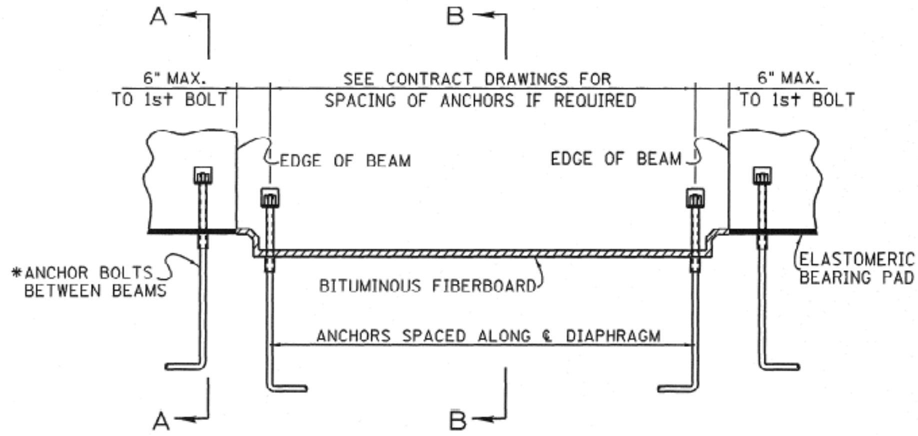

Figure 3.17: Tennessee DOT Longitudinal Connection

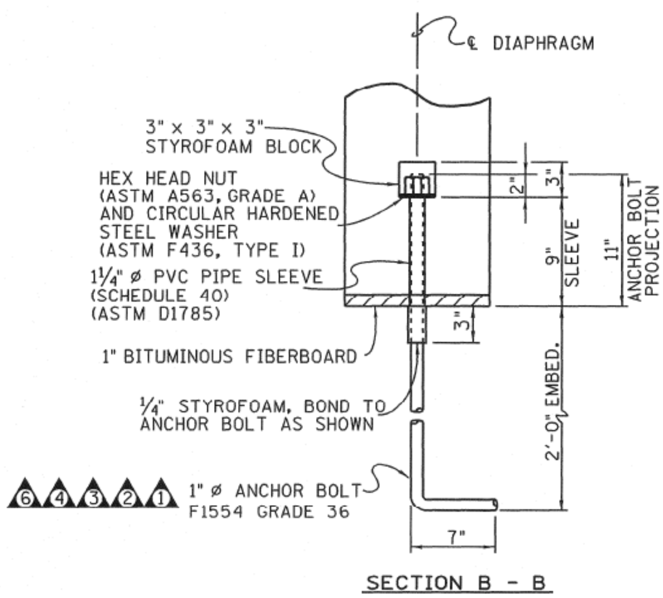

Figure 3.18: Tennessee DOT Longitudinal Connection (Section B-B)

3.3.3 South Carolina DOT Longitudinal Restrainer

The South Carolina DOT uses a cable restrainer system as its longitudinal restrainer. As seen in Figure 3.19 and Figure 3.20, the cables run between the girders and extend a distance specified by the engineer. Sufficient slack in the cable is provided to allow for thermal expansion before the system engages. When the restrainer unit engages, it transfers the forces into the deck, which transfers the forces into the girders and, through the connection shown in Figure 3.11, into the bent cap. The cable restrainer unit attached at either end of the cable is seen in Figure 3.21 (South Carolina DOT, Seismic Restrainer Details, 2005). This connection was considered, but the use of cables required more analysis and design than the use of a bar restrainer system, like the one used by Missouri DOT discussed next. Also, if this connection were used by ALDOT, the current bearing pad connection would be unable to transfer the longitudinal forces into the bent cap because the current connection does not have any restraint in the longitudinal direction. While the displacements would be limited, the load path would not be complete and the forces would not be able to be transferred into the substructure. So even with this restrainer detail, a change to the connection would be necessary to prevent the girders from becoming unseated.

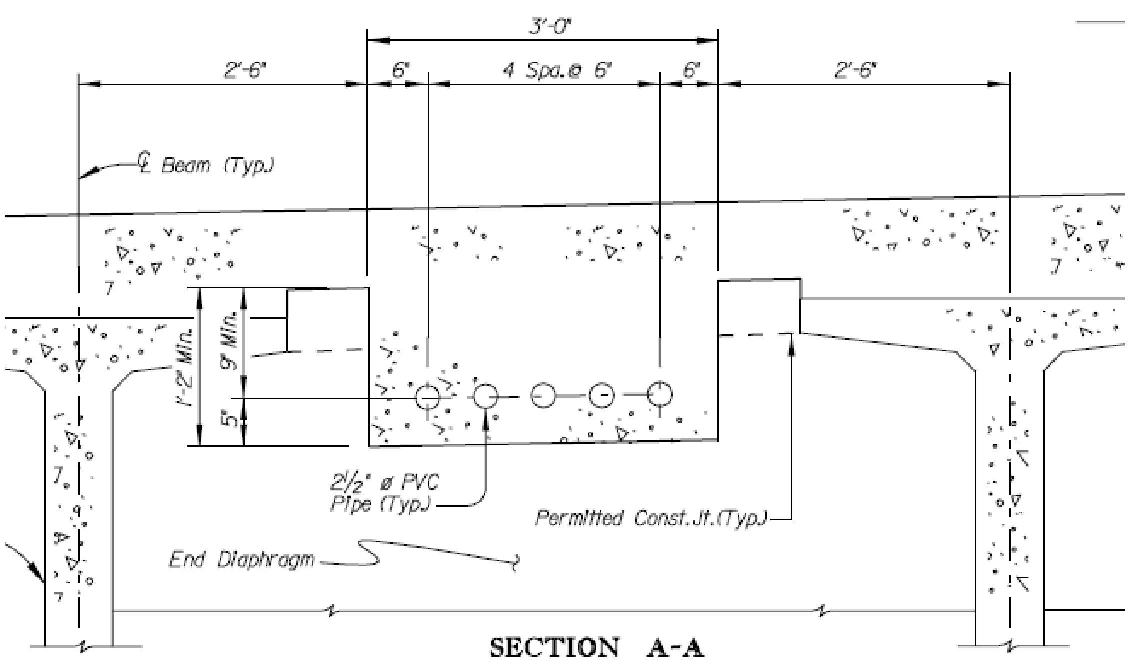

Figure 3.19: South Carolina DOT Longitudinal Restrainer (End View)

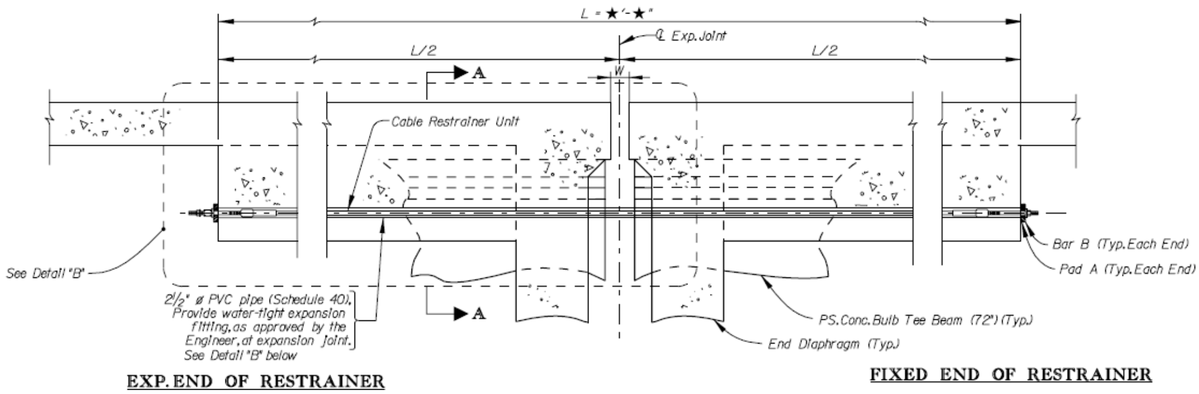

Figure 3.20: South Carolina DOT Longitudinal Restrainer (Elevation View)

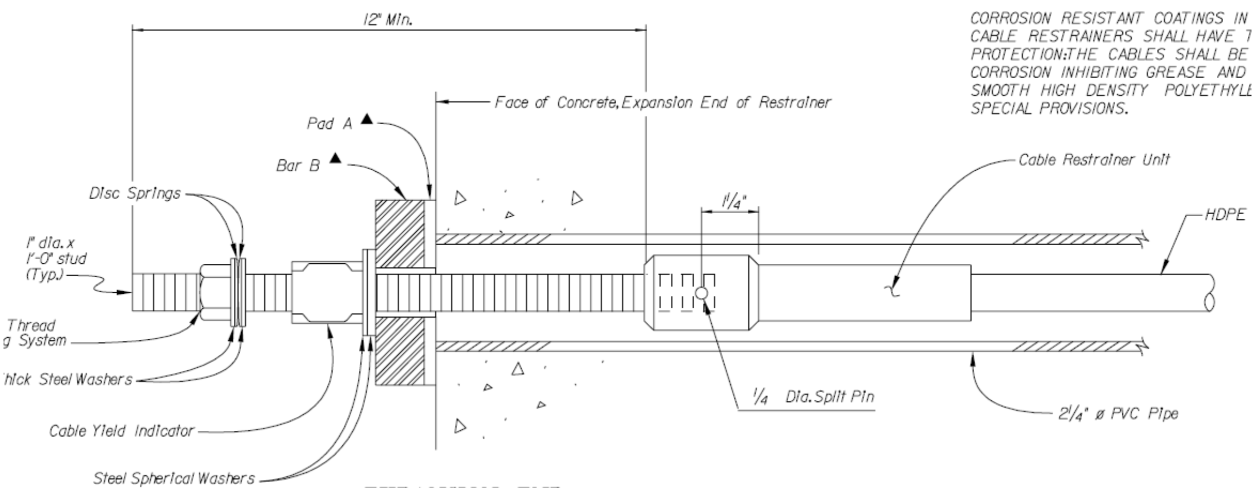

Figure 3.21: South Carolina DOT Cable Restrainer Unit

3.3.4 Missouri DOT Longitudinal Restrainer

Figure 3.22 shows the longitudinal restrainer detail used by Missouri DOT. Restrainer bars are located in the diaphragm on either side of a girder. An anchor plate is cast with the diaphragm on the girder with a fixed connection. For the expansion girder connection, an expansion gap for temperature is allowed before a bearing plate and nut assembly is welded to the bar. The bar passes through a PVC sleeve before being attached to the opposite anchor plate. During a seismic event, the restrainer bar would engage and transfer forces into the diaphragm. The diaphragm would transfer the forces into the deck and girders, and the girders would transfer the forces into the bent cap through the connection discussed earlier and detailed in Figure 3.12. (Missouri DOT, Bridge Design Manual Section 6.1, 2002). This restrainer bar connection was considered easier to design and construct than the South Carolina connection, so it was used to create a design for ALDOT. However, the bearing pad connection would still be unable to transfer the longitudinal forces into the bent cap. This issue will be addressed after the restrainer design is discussed.

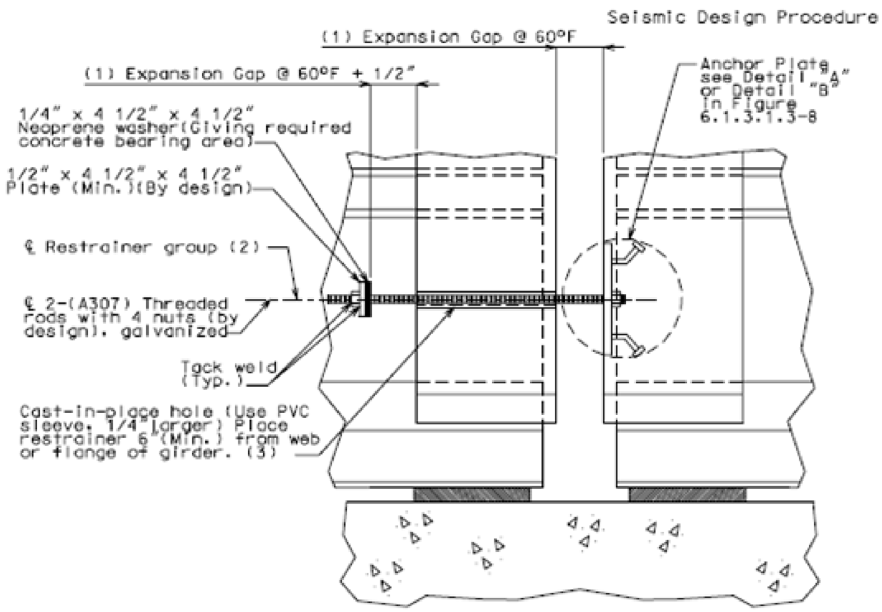

Figure 3.22: Missouri DOT Longitudinal Restrainer

3.3.5 Longitudinal Restrainer Recommendation

After deciding to base a new restrainer design on the Missouri DOT restrainer, the design in Figure 3.23 and Figure 3.24 was proposed. It features two longitudinal bars per girder spanning between diaphragms. When the bars displace past the expansion gap distance, steel plates at the end of the bears will engage the diaphragms and prevent additional longitudinal movement. The forces generated in the bars will be transferred into the diaphragms. The diaphragms would transfer the loads to the bridge deck and girders.

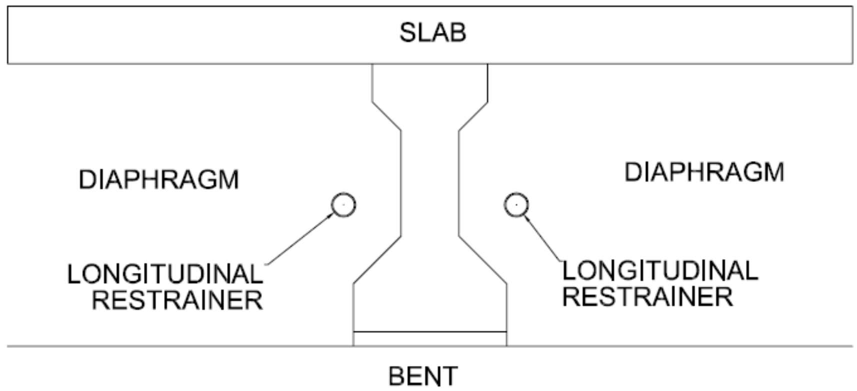

Figure 3.23: Proposed Longitudinal Restrainer Connection (Cross-Section)

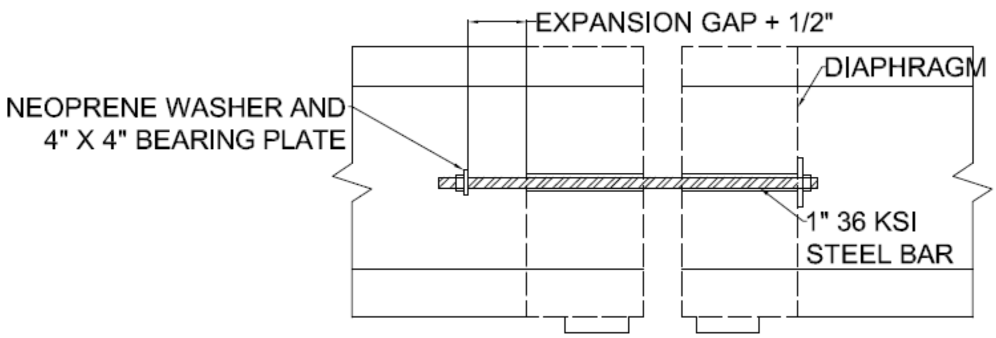

Figure 3.24: Proposed Longitudinal Restrainer (Elevation)

The main concern with this design was the resistance of the diaphragms to the restrainer force. The 12 inch diaphragms currently used by ALDOT at the bent webwall needed to be checked to ensure they could resist the loads. The two limit states studied were two-way concrete shear and local yielding of the steel bearing plate. The AISC Manual (2005) and LRFD Specifications (2009) were used in this design. This connection was designed to resist the maximum force from the bridges studied, which was 49 kips per girder, or 24.5 kips per restrainer rod. These forces were determined in the design of the bridges, which is discussed in Chapter 4. For the specific design force calculations, refer to the appendix for the specific bridge. It was calculated that 1 inch steel bars with a tensile strength of 36 ksi would be adequate for the design. Using 4,000 psi concrete, 60 ksi steel and a steel bearing plate with 4 inch sides, the two limit states were found to be adequate to resist the 24.5 kip load, as seen in Table 3.1. These calculations can be seen in Appendix A.

Table 3.1: Longitudinal Restrainer Design

Limit State	Design Strength (kip)
Local Yielding	1296.0
Punching Shear	29.7

After designing the restrainer, the load path from the diaphragm to the girder and bridge deck was analyzed because the forces in the diaphragm would be transferred into the bridge deck and girders through the reinforcing steel. It was necessary to analyze the diaphragms and make sure the steel would be able to transfer the loads without failing in shear. Figure 3.25 shows the typical detail used by ALDOT for an intermediate webwall at Scarham Creek. ALDOT is currently using a diaphragm thickness of 12 inches for the webwall at the bearings for all bridges in the state. The diaphragm between two girders is what was analyzed. A model was created for each bridge diaphragm using SAP2000 to determine where the restrainer rods could be placed where the loads would distribute to all the steel evenly without failure. The steel capacity was determined using Equation 3.2, where the area of the steel and strength of steel were the variables.

$$V_S = 0.6 * A_v * f_Y \qquad \text{Equation 3.2}$$

Figure 3.26 illustrates the model used for the Scarham Creek Bridge diaphragms. The thickness of these models was taken as 12 inches. When compared with the webwall detail used by ALDOT for the same bridge, it can be seen that the diaphragm between two girders is what was modeled. The dark plus signs represent where the reinforcing steel is present and will transfer the loads into the deck or girders. However, a real bridge concrete diaphragm is cast integrally with the bridge deck, so some of the force could be transferred into the bridge deck through the concrete. This analysis assumed that the restrainers would not be designed for a serviceability limit state, but for an ultimate strength limit state, to prevent span loss from occurring. For this reason, it was assumed that when the restrainers engaged, the concrete between the diaphragm and deck would be cracked and not transfer any load. The reinforcing steel would transfer all of the forces. But, some friction between the diaphragm and the girders on the side and deck on the top is still expected to occur as a result of the longitudinal movement of the diaphragm. This will dissipate some of the energy from the longitudinal restrainers. So the surrounding concrete, which in the model was represented by the grid points not labeled with a plus sign, was modeled as springs in order to reduce the amount of force transferred into the steel. The amount of force the concrete would transfer was a point of uncertainty. After discussion, a conservative assumption was made that the concrete would take 10% of the force from the restrainers through friction. Another limit state that was considered was out-of-plane bending. But, since the concrete was assumed to be cracked at the joint between the deck and diaphragm, it was assumed that the shear limit state would control, so out-of-plane bending was not checked for these diaphragms.

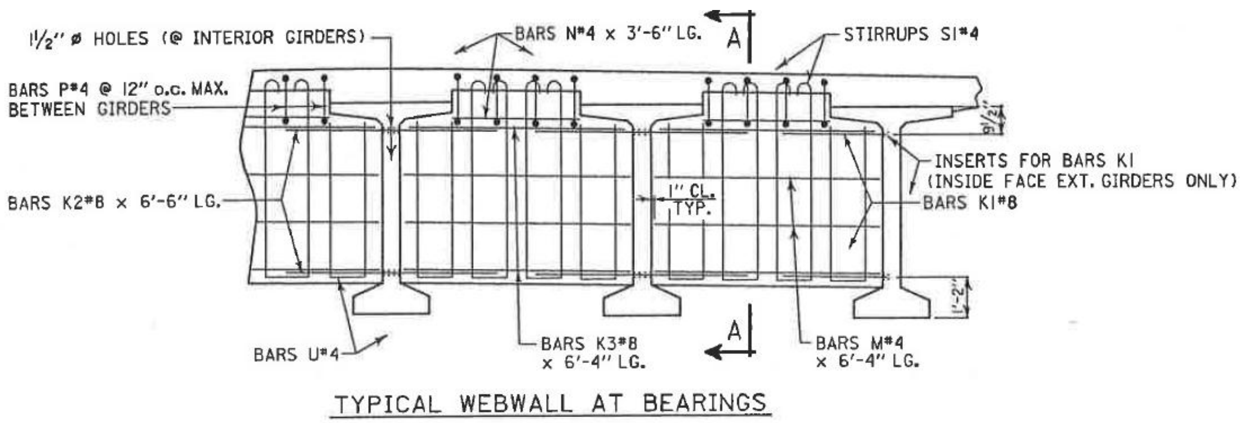

Figure 3.25: ALDOT Typical Webwall Detail (Scarham Creek)

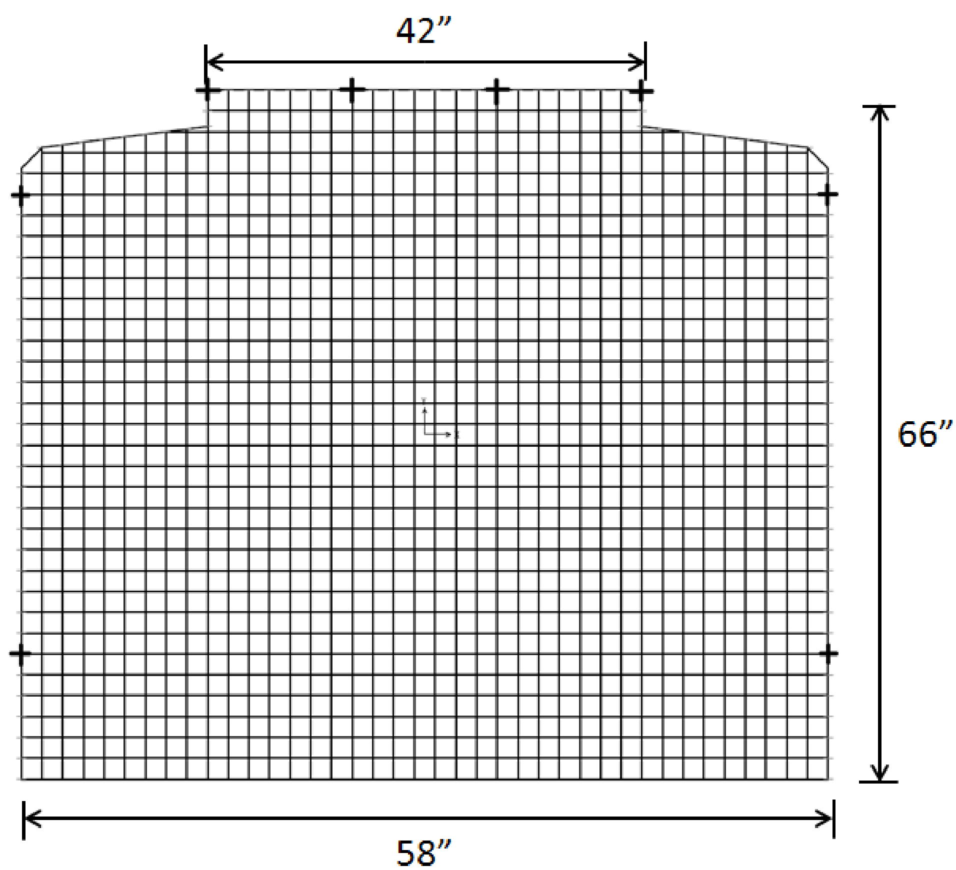

Figure 3.26: Diaphragm Model (Scarham Creek)

Once the model was created, the restrainer forces were moved symmetrically around the diaphragm until the forces in the steel were below the established thresholds from Equation 3.2. One of the goals of this design was to determine if the restrainer bars could be placed at a standard location in all of the bridge diaphragms without requiring any additional reinforcement. In order to determine this, five bridges that used different girder types and spacing were analyzed separately and the locations of the bar restrainers in their diaphragms was determined. Figure 3.27 shows the forces resulting from the loading of the Scarham Creek diaphragm. The restrainers had to be placed 10 inches below the bottom of the deck so the forces would spread evenly among the four steel connections at the top. This even spread was desirable in all of the diaphragms so that all of the steel carried smaller amounts instead of one or two carrying the entire load. However, the bar locations were different for the other diaphragms studied. Two additional diaphragm studies will be discussed to provide a better understanding of how the restrainer locations were different.

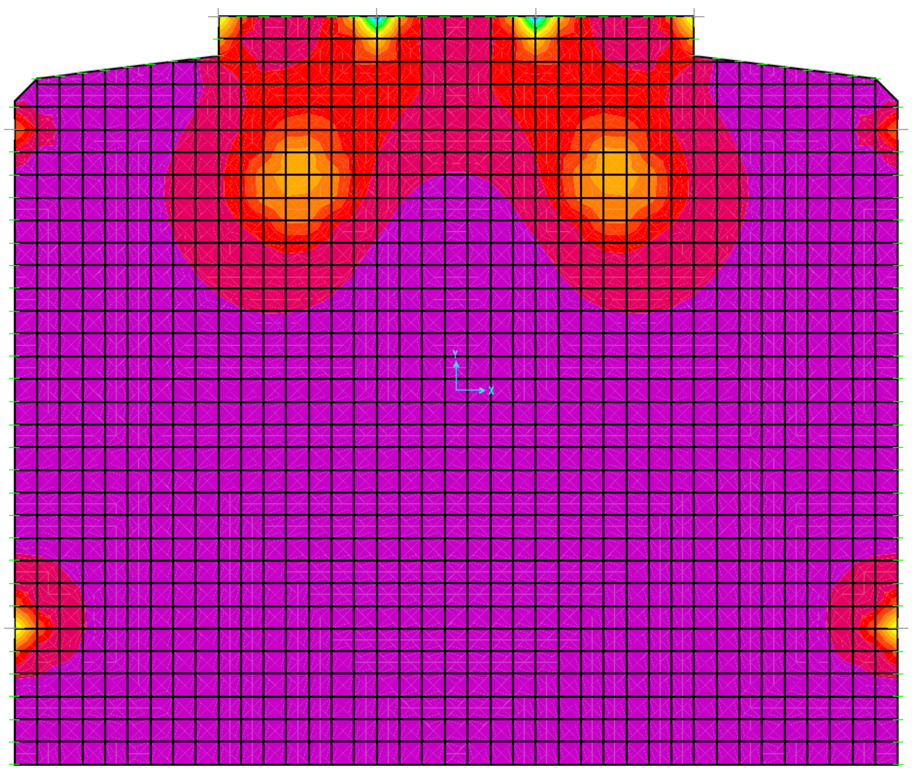

Figure 3.27: Scarham Creek Diaphragm Model Shear Forces

The second bridge diaphragm that was studied was the Mobile County bridge, which used BT-72 girders spaced at 6 feet and had a diaphragm height of 36 inches. It had four steel connections extending from the deck into the diaphragm and four longitudinal bar locations that transferred load to the girders. By comparison, the Scarham Creek bridge also used BT-72 girders, but with 7 foot spacing between them and a diaphragm height of 61.5 inches. It also had four steel connections extending from the deck into the diaphragm and four longitudinal bar locations along the side. Figure 3.28 shows the results from the analysis of the Mobile County bridge diaphragm. For this particular diaphragm, the bars had to be at mid-height of the diaphragm (18 inches from the bottom of the bridge deck) and between the top steel for the entire load to be transferred and none of the bars to be over capacity. It could not have the restrainer bars located at 10 inches like in the Scarham diaphragm because the top steel would be over capacity. Even though the same girder types were used, the restrainer bar locations were different. Flexure was also checked for this diaphragm, but it did not control the design.

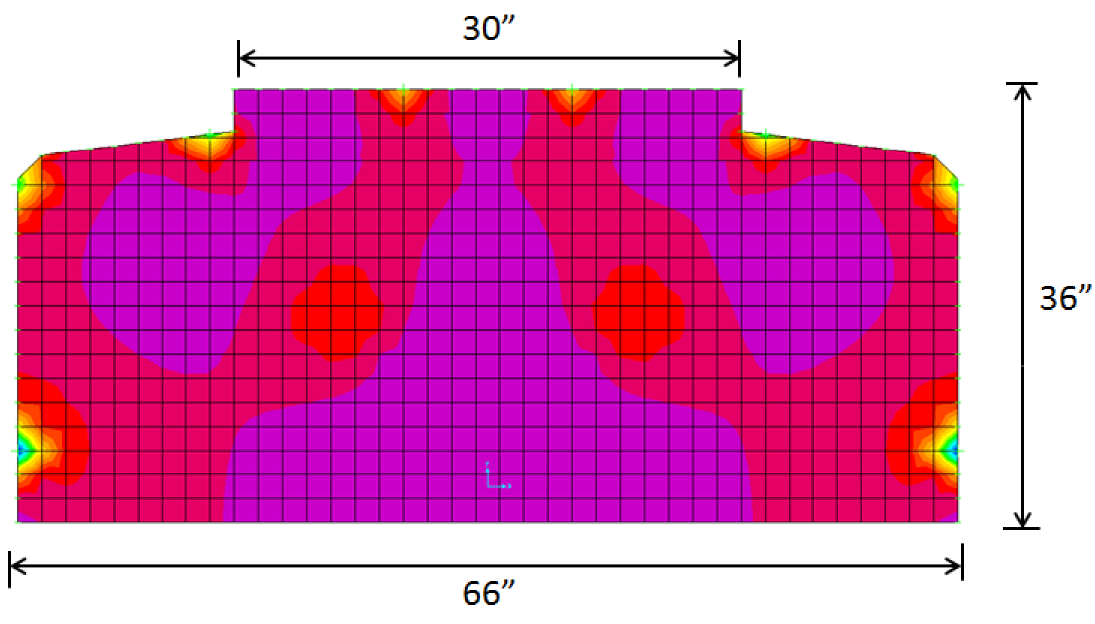

Figure 3.28: Mobile County Bridge Diaphragm Model Shear Forces

The third bridge diaphragm studied was Bent Creek Road Bridge, which uses BT-54 girders spaced at 5.33 feet with a diaphragm height of 54 inches. It had six places where reinforcing steel connected the diaphragm with the deck and two longitudinal reinforcing bar locations that transferred loads to the girders. For this bridge diaphragm, seen in Figure 3.29, the bars had to be placed at least 12 inches below the bottom of the bridge deck so that some of the forces would be carried by the longitudinal steel. It also had to be between the top reinforcement so that the forces spread out between them. Just among these three diaphragms, the location of the restrainer bars was different. The location is determined based on the geometry of the diaphragm, specifically the height, and the number of reinforcing steel locations on the top and side of the diaphragm. Since the heights and steel amounts varied for each diaphragm, the restrainer bar locations could not be at the same location. So if the longitudinal restrainer details were used by ALDOT, the diaphragms would have to be analyzed for each bridge. This was one of the reasons this design was not recommended.

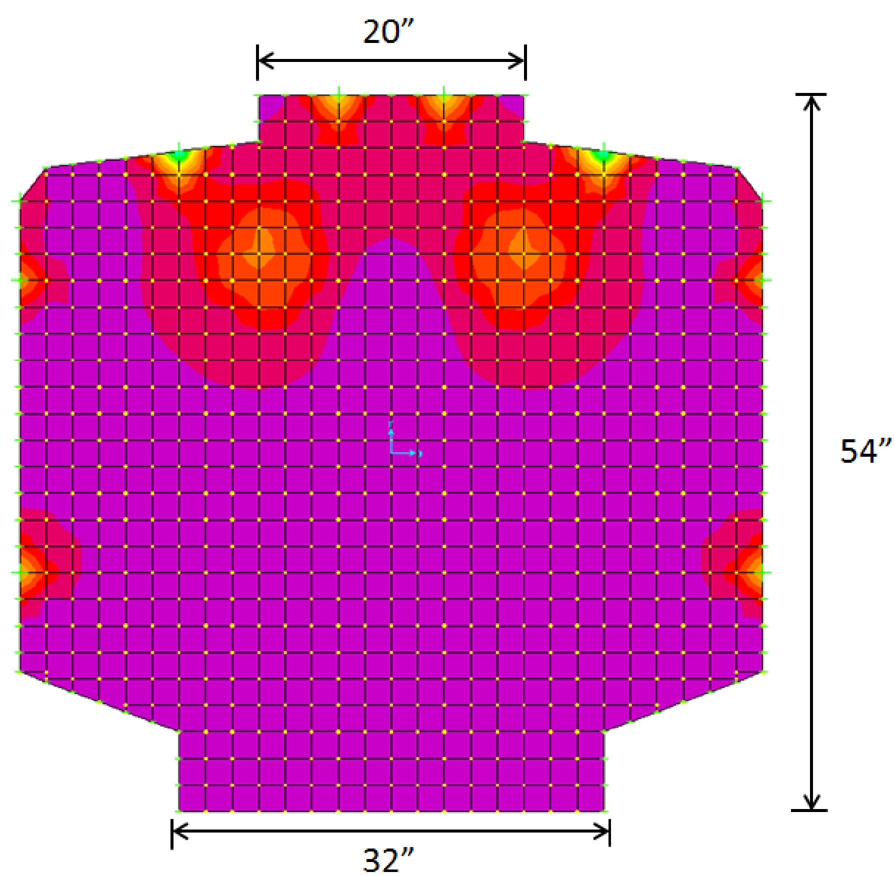

Figure 3.29: Bent Creek Road Diaphragm Model Shear Forces

The final design step was to make sure connection provided a complete load path. The force would be transferred into the bridge deck and girders, but would still need to travel through the connections before going into the bent cap. Because the original connection was still to be used, there was no longitudinal load path for the forces at the connection. The displacement of the bridge girders would be reduced, but the connection would not be able to transfer the longitudinal forces into the bent cap. The idea of allowing the girders to transfer the forces through friction between the girders and bent cap was presented, but Article 4.13.1 in the Guide Specifications does not allow friction to be considered as an effective restrainer, so this idea would require additional effort in detailing and design. After discussion with ALDOT, the longitudinal restrainer option was not recommended because the original problem of an incomplete load path in the longitudinal direction still existed. Instead, it was decided to provide additional seat width and allow the girders to move in the longitudinal direction after the connection slipped.

3.4 Extended Seat Width

As discussed earlier, a second option exists to prevent span loss. By extending the minimum seat width for the girders, more room can be provided for the girders to displace once the connection slips to prevent unseating. This technique is utilized by Illinois DOT as discussed earlier. The equation used by Illinois was one of two alternate equations that was compared with the current seat width calculations in the Guide Specifications to determine if they provided more seat width for the bridges studied.

The current method of calculating the seat width uses Equation 3.3 from the Guide Specifications in Article 4.12.2. It is based on the span length, column height, and skew of the bridge. 100% of this equation is required to be supplied in SDC A and 150% is required to be supplied for SDC B, C, and D.

$$N = (8 + 0.02L + 0.08H) * (1 + 0.000125S^2) \qquad \text{Equation 3.3}$$

The first alternative was Equation 3.1 which is used by the Illinois DOT as shown earlier in the chapter. This equation was selected because the Illinois earthquake resisting system design strategy is similar to the current design strategy in this thesis, which is to allow the girders to “ride out” the design earthquake. This equation is based on research performed by the Applied Technology Council (ATC) and the Multidisciplinary Center for Earthquake Engineering Research (MCEER) in 2003. It gives a better estimation of the expected displacements and deformations that occur at the seat (ATC/MCEER Joint Venture, 2003). Instead of multiplying the seat width by 1.5 for SDC B, which is the procedure found in the Guide Specifications, the multiplier is based on the expected spectral acceleration coefficient at a 1-second period, S_{D1}. As such, the seat width can vary for different sites in SDC B. The largest and lowest values of S_{D1} (0.15 and 0.30) will be used to find the seat width using this method and compared to the results from the other methods. The equation was converted from metric units into English units in Equation 3.4.

$$N = \left(0.10 + 0.0017L + 0.007H + 0.05\sqrt{H}\sqrt{1 + (2\frac{B}{L})^2}\right) * \left(\frac{1+1.25F_vS_1}{\cos(\alpha)}\right) \qquad \text{Equation 3.1}$$

$$N = \left(4 + 0.02L + 0.08H + 1.09\sqrt{H}\sqrt{1 + (2\frac{B}{L})^2}\right) * \left(\frac{1+1.25S_{D1}}{\cos(\alpha)}\right) \qquad \text{Equation 3.4}$$

The second alternative would be to perform a rigorous analysis that is required for SDC D. Article 4.12.3 in the Guide Specifications provides a minimum seat width equation, represented as Equation 3.5, for SDC D that uses the expected displacement demand instead of the column height and span length. The expected displacement demand was calculated for each bridge in SDC B using the structural analysis and computer bridge models in Chapter 4. The calculated seat width from this equation is not allowed to be less than 24 inches.

$$N = \left(4 + 1.65 * \Delta_{EQ}\right) * (1 + .00025 * S^2) \geq 24 \qquad \text{Equation 3.5}$$

Seat widths for each bent of each bridge studied in SDC B were calculated using each of the three equations. These seat widths were compared to determine which provided the greatest seat length. The results in Table 3.2 show that the maximum seat width depended on the S_{D1} coefficient for that particular site. In SDC B, it can range from 0.15 to 0.30. At 0.15, Equation 3.3 in the Guide Specification controls. But at 0.30, the Equation 3.4 from the ATC/MCEER study controls. This is because the multiplier for the Guide Specification equation is 1.5 for all sites in SDC B, and the multiplier for the ATC 49 Equation varies based on S_{D1}. The equation for SDC D did not control because all the calculated longitudinal displacements for these bridges were less than 1 inch (with one exception), so only small seat widths were determined. Technically these cannot be less than 24 inches, but in order to show the effect of the small displacements, values less than 24 inches were shown.

Table 3.2: Minimum Seat Width Calculations

Equation	Minimum Seat Lengths (in)								
	Bent Creek Road	Norfolk Southern Railroad	Little Bear Creek Bent 2	Little Bear Creek Bent 3	Oseligee Creek Bent 2	Oseligee Creek Bent 3	Scarham Creek Bent 2	Scarham Creek Bent 3	Scarham Creek Bent 4
Guide Spec SDC B	18.5	19.2	17.3	17.9	16.5	17.5	20.0	23.0	19.8
Guide Spec SDC D	5.7	5.7	5.7	5.7	6.1	6.3	5.7	5.7	5.7
ATC 49 (S_{D1} = 0.15)	17.1	18.2	14.3	15.8	15.2	17.4	19.9	25.1	19.5
ATC 49 (S_{D1} = 0.30)	19.8	21.1	16.6	18.3	17.6	20.1	23.1	29.1	22.6

As the table shows, for all but one of the bents, assuming S_{D1} equals 0.30 gave the most conservative value for minimum seat width. The current Guide Specification controlled the minimum seat width for bent 2 of Little Bear Creek bridge. This is a result of the small column heights at this bent. But, because the ATC 49 equation is designed to give a better estimation of the seat displacement, and because the minimum seat width obtained from this equation is only one inch less than the current specifications, this anomaly was not considered important. Equation 3.4 was selected to be recommended assuming S_{D1} equals 0.30 because it would be the upper limit for SDC B and result in a larger value than the Guide Specifications equation. Since the equation has been researched by ATC and MCEER and designed to give a better estimation of the deformations and displacements at the seat and is currently in use by Illinois DOT, it is reasonable to assume that this equation will provide enough seat width to prevent the girders from unseating during a design earthquake.

3.5 Conclusion

This task of the thesis was necessary because it was unknown if the current superstructure-to-substructure connection was adequate to resist the calculated horizontal design forces. After analysis, it was determined that it was adequate in the transverse direction, but not in the longitudinal direction, so a complete load path did not exist between the superstructure and substructure and a new connection design was necessary. Several options were investigated and designed, but ALDOT chose to keep the original connection design and allow the girders to move in the longitudinal direction after the connection slipped. This would be accomplished by providing additional seat width in the longitudinal direction using Equation 3.4 described above. Since the original connection design will continue to be used, the clip angles and anchor bolts will also have to be checked to ensure they can withstand the horizontal design forces. They will be checked in Chapter 4 for each bridge in SDC B to show if the connection is adequate.

Chapter 4: Bridge Design Standards

4.1 Introduction

Another objective of this thesis was to determine if standard design details and drawings could be created for bridges in Seismic Design Categories (SDC) A and B. As discussed in chapter 2, the Guide Specifications contain updated seismic hazard maps that have higher expected ground motions than the maps in the Standard Specifications. These greater accelerations, along with changes to the bridge design resulting from additional research in earthquake engineering, have resulted in changes in the minimum details and seismic design procedures for bridges. By redesigning multiple bridges in each SDC that had previously been designed under the Standard Specifications and comparing the column details, the change in the design details could be shown and standard details could be developed. Along with the standard details, design sheets for each bridge were developed to provide examples of the new seismic design procedures. In the previous study by Coulston and Marshall (2011), design sheets and standards for three bridges in SDC B were created. These design sheets were updated to include changes in the Guide Specification from the 2009 edition to the revised 2011 edition, and design sheets for two additional bridges in SDC B were created. Design sheets for SDC A, which has two subclasses, were developed using the same revised 2011 edition. ALDOT supplied the design drawings for each bridge studied in this thesis designed using the Standard Specifications, as well as a foundation report. While the expected ground accelerations a bridge would be expected to experience is typically determined from the Guide Specifications seismic hazard maps, this thesis used values that allowed different bridges to be placed in the SDC of choice.

This allowed some bridges to be designed in multiple SDCs in order to show the difference in the details resulting from the two design categories. The procedure for determining the SDC as well as the differences between the categories will be discussed further in this chapter.

The first subclass of SDC A, termed SDC A1 throughout this thesis, classifies bridges in seismic regions that are not likely to experience substantial ground accelerations and do not require minimum details. The two bridges designed in SDC A1 include the following: County Road 39 Bridge over CSX in Mobile County and Stave Creek Bridge in Clarke County. The design calculations and design sheets can be found in Appendices B and C**.**

The second subclass of SDC A, termed SDC A2, classifies bridges in low seismic regions that are not likely to experience plastic forces, but still require minimum detailing. The following four bridges were designed in SDC A2: Bent Creek Road Bridge in Lee County, Bridge over Norfolk Southern Railroad in Etowah County, Oseligee Creek Bridge in Etowah County, and Stave Creek Bridge in Clarke County. The Stave Creek Bridge was also designed in the SDC A1 category. All of the calculations for these details can be found in Appendices D-G**.**

Finally, five bridges were redesigned in SDC B, including the three designed under the previous study. SDC B bridges are in a moderate seismic hazard and must be designed using additional analysis techniques and must also satisfy minimum detailing. The analysis of all five of the bridges was completed using computer software, with the results recorded in the design sheets. The design sheets and supplemental design data for these five bridges can be found in Appendices H-Q. The five bridges include the following: Bridge over Little Bear Creek in Franklin County, Bent Creek Road Bridge over I-85 in Lee County, Oseligee Creek Bridge in Chambers County, Bridge over Norfolk Southern Railroad in Etowah County, and Scarham Creek Bridge in Marshall County. Bent Creek Road Bridge, Oseligee Creek Bridge and Bridge over Norfolk Southern Railroad had also been designed as SDC A2 so their design details could be compared.

All bridge design sheets can be found in the Appendices and were created using Mathcad (PTC, 2007). The first step was to input the given bridge information at the beginning of the sheet, including the length of the bridge, span lengths, deck thickness and widths, girder cross sectional areas, etc. Other information needed for specific articles or bridge components, such as reinforcement type and spacing, were input at that location in the sheet. All of the input variables were notated with a green background and all output information necessary for design was notated with a yellow background. This allows the variables to be quickly located and changed during the design. The steps in the design sheets were laid out in the same order as the design charts in the Guide Specification. Each specific article used either in the Guide Specifications or LRFD Specifications was cited. Each step of the design process will be discussed below.

4.2 SDC Determination

The first step in the design process is to determine the Seismic Design Category (SDC) of the bridge. The SDC will determine what type of analysis and detailing is necessary for the bridge. Chapter 3 of the Guide Specifications lists the steps involved in determining the design category. The soil site class is determined first. The site class plays a large role in the determination of the SDC, as a change from one class to the other can result in a change in the SDC. Site classes range from A (hard rock) to F (poor soil such as stiff clay) and are determined using either soil shear wave velocity, uncorrected blow counts, or undrained shear strengths. However, it should be noted that site classes A and B cannot be verified without performing a shear wave velocity test. Table 3.4.2.1-1 in the Guide Specifications is used to determine the appropriate site class.

The next step is to determine the response spectra from national ground motion maps. The AASHTO Ground Motion Calculator (AASHTO, 2007) was used to determine these ground accelerations. The latitude and longitude of the bridge site, along with the site class of the soil, is input into the program and the acceleration coefficient, A_s, design spectral acceleration coefficient at 1-sec period, S_{D1}, and design spectral acceleration coefficient at 0.2-sec period, S_{DS}, is output. If the longitude and latitude are not known, the zip code of the area can be used, but the spectral coefficients will not be as precise. Once S_{D1} is known, the seismic design category can be determined according to Table 4.1.

Table 4.1: SDC Category Determination

Value of S_{D1}	SDC
$S_{D1} < 0.10g$	A1
$0.10g \leq S_{D1} < 0.15g$	A2
$0.15g \leq S_{D1} < 0.30g$	B
$0.30g \leq S_{D1} < 0.50g$	C
$0.50g < S_{D1}$	D

In order to show the significance of the site class, the following three maps were created for Alabama. The AASHTO Ground Motion Calculator program (2007) was used to find the highest spectral accelerations for each county in Alabama. The SDC was determined for each county using three different site classes. The maximum spectral acceleration for each county in the northern half of the state was assumed to occur at either the northeast or northwest corner of the county since the maximum accelerations in the state are in the northeast and northwest corners. For the southern counties, the maximum spectral acceleration was assumed to occur at the northernmost point of the county. The results can be seen in Figure 4.1, Figure 4.2, and Figure 4.3. The entire state is classified as SDC A1 for soil site class B. This would also mean that soil site class A would result in the entire state being classified as SDC A1. For soil site class C, the northern part of the state is classified as SDC A2, with one county being in SDC B. Finally, for soil site class D, the majority of the state is at least SDC A2, with the northern part of the state being SDC B and the southern part of the state still in SDC A1. The changes in the soil site class can have a significant effect on the determination of the SDC, which affects the design of a bridge. It is recommended to use the soil shear wave velocity test to verify soil site class A or B at the site because it would result in the bridge being in SDC A1, generating a more economical design. It should be noted that these maps are only an estimation of the spectral accelerations in each county. Certain sites may have higher values than the average value assumed over the county.

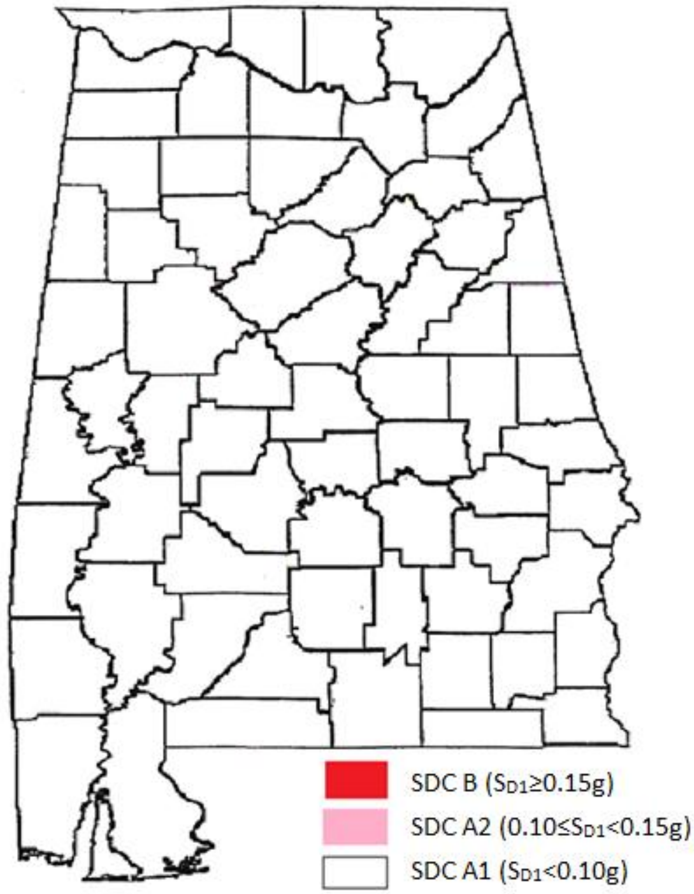

Figure 4.1: Alabama SDC Map for Soil Site Class B

Figure 4.2: Alabama SDC Map for Soil Site Class C

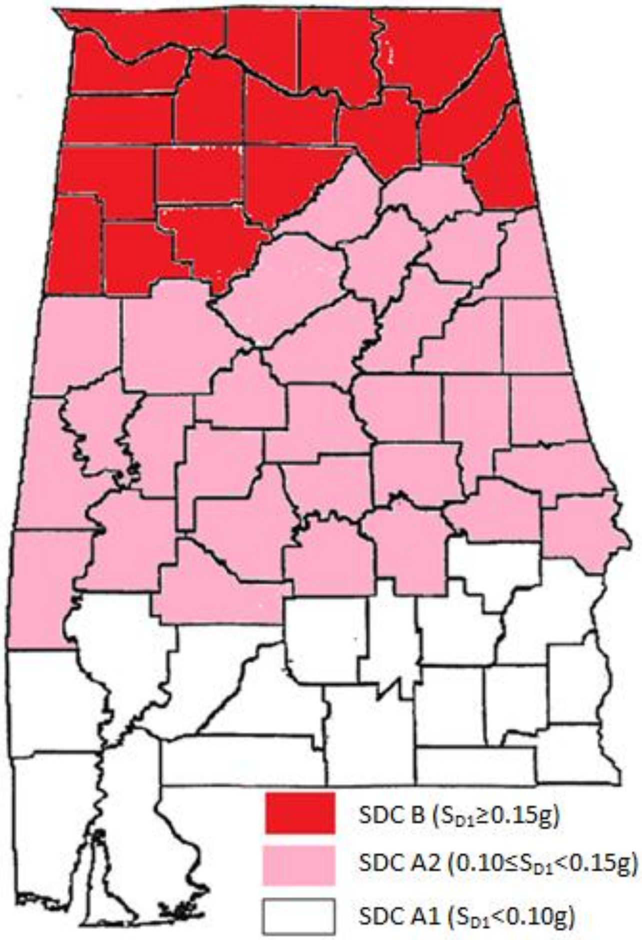

Figure 4.3: Alabama SDC Map for Soil Site Class D

4.3 Guide Specification Design Process for SDC A1

The design process for SDC A1 will be discussed first. SDC A1 is the lowest design category in the Guide Specifications, and bridges in this category are expected to experience low seismic forces. It does not require additional structural analysis or minimum detailing. The expected horizontal design forces are minimal and used only for designing the superstructure-to-substructure connection and the column for shear. These design forces are calculated as a percentage of the total tributary weight resisted at a bent. The minimum support length is also calculated as a part of the design. The steps involved with this design category will be discussed next.

4.3.1 Determine Vertical Reactions at Bent

The first step in calculating the horizontal design force is to determine the vertical reaction at the bent. This is accomplished by finding the tributary area of the bent, the total dead weight of the bridge in that tributary area, and the uniform live load acting on the area. The dead weight of the bridge includes the weight of the deck, girders, piers, columns, and guard rails. The uniform live load consists of a 0.64 kip per linear foot per lane load that is applied simultaneously with the dead load. The LRFD Specifications, in Article C3.4.1, recommends including 50% of this live load in the vertical reaction calculations, but does not require it. It does require the bridge owner to determine the live load factor, γ_{EQ}, on a project specific basis. This live load factor determines what percentage of the live load is to be included in the weight calculations. For bridges in high traffic areas, such as major highways in large city centers, it is recommended to include at least half of the live load, because it is possible for that bridge to experience live loads during a seismic event. Once the live load is determined, it is multiplied by the number of design lanes and the tributary length of the bent. The total vertical reaction is the sum of the dead and live load resisted by the bent. This thesis will show two horizontal design forces for all bridges in SDC A, one that includes the 0.50 live load factor and one that includes a factor of zero, so that no live load is considered. A comparison between these two design forces will show if the live load factor has a significant effect.

4.3.2 Determine Design Forces

Using the vertical reaction at the bent, the horizontal design forces are calculated using Article 4.6 of the Guide Specifications. This article details the seismic design requirements for bridges in SDC A. The horizontal design force is used to design the columns for shear as well as the connection between the superstructure and substructure. For column shear, the vertical reaction is divided by the number of columns at the bent to represent the amount of load each column will resist. For the connection, the vertical reaction is divided among the number of connections, which is equal to the number of girders at the bent. The horizontal design forces presented in this thesis will be the connection design forces. The design force is then multiplied by either 0.15 or 0.25 times the vertical reaction at the bent depending on the acceleration coefficient at the site. The acceleration coefficient (A_s) is calculated when the SDC is determined, as discussed earlier. For sites with an acceleration coefficient less than 0.05g, the design force is 0.15 times the tributary weight. For all other sites, the design force is 0.25 times the vertical reaction. This difference in design forces is only possible in SDC A1 because the ground accelerations in SDC A2 will be above 0.05g. The reason for the difference is the Guide Specifications recognize that since seismic forces in some parts of the country are very small, the seismic design forces will also be small (AASHTO, 2011). All the design forces are multiplied by a factor of 1.0 in accordance with the load combinations found in the LRFD Specification. Table 4.2 shows the relationship between the acceleration coefficient and horizontal design force. The Standard Specifications require 0.20 times the vertical reactions for all sites in SDC A. It does not allow for a different force in low seismic regions.

Table 4.2: Design Force Multiplier

A_s	Force
<0.05g	15%
≥0.05g	25%

4.3.3 Determine Minimum Support Lengths

Support lengths are the length of overlap between the girder and pier or abutment seat. The minimum support length must be provided to accommodate differential movement between the superstructure and the substructure. These displacements occur during a design earthquake and are typically conservative. However, providing the minimum support length alone does not guarantee the girder will remain seated during an earthquake, especially if it is larger than the design earthquake. Providing seat widths larger than the minimum or using restrainer bars and cables can limit the displacement if unseating is a concern. Article 4.12 in the Guide Specifications uses Equation 4.1 to determine the minimum support length. Currently, ALDOT uses this equation to determine the minimum support lengths, but in chapter 3 of this thesis, it was recommended to use Equation 4.2 from the ATC-49 study (ATC/MCEER Joint Venture, 2003) to determine the minimum seat length because it will give a larger seat width. The Standard Specification uses Equation 4.1 in both SDC A and B. In this thesis, the minimum seat lengths for bridges in SDC A1 will be calculated using Equation 4.2 and compared with the results from Equation 4.1, which represent the minimum seat length from the Standard Specifications.

$$N = (8 + 0.02L + 0.08H) * (1 + 0.000125S^2) \qquad \text{Equation 4.1}$$

$$N = \left(4 + 0.02L + 0.08H + 1.09\sqrt{H}\sqrt{1 + (2\frac{B}{L})^2}\right) * \left(\frac{1+1.25S_{D1}}{\cos(\alpha)}\right) \qquad \text{Equation 4.2}$$

4.3.4 Minimum Column Detailing

Once the design forces and minimum support lengths are determined, no further analysis is required for SDC A1. For bridges in this category, the bridge is not expected to experience forces that will result in the formation of plastic hinges. Therefore, the minimum design details are not required. The design force is used to design the superstructure to substructure connection and the remainder of the substructure. Article 8.6.1 of the Guide Specifications allows for the use of the LRFD Specifications to design the column for the areas outside of the plastic hinge region. For SDC A1, there is no plastic hinge region, so the LRFD Specifications are used to design the transverse reinforcement for the column.

4.3.4.1 Design of Reinforcement outside Plastic Hinge Region

The detailing for transverse reinforcement outside of the plastic hinge region is not mentioned in the Guide Specifications because the equations for determining concrete capacity used in the Guide Specification are not meant to be used outside of the plastic hinge region. They include the expected concrete behavior as the hinge region becomes plastic, which will not occur outside of the plastic hinge zone. Therefore, the LRFD Specifications are used to design the shear reinforcement outside of the plastic hinge region. The shear reinforcement must be checked to ensure that it provides greater resistance than the expected horizontal design force in the column. Equations 4.3 and 4.4 from Article 5.8.3.3 in the LRFD Specifications are used to determine the shear capacity of the transverse reinforcement and the concrete. Once the design is satisfied for strength, three spacing requirements are checked. These spacing requirements could control the design and must be checked. The first requirement can be found in Article 5.8.2.5 of the LRFD Specifications and is a minimum amount of transverse reinforcement. It is only required when the factored load is greater than half of the factored resistance by the concrete section and prestressing steel (if present). It is intended to provide reinforcement in regions where there is a significant chance of diagonal cracking (AASHTO, 2009). If it is determined that this minimum reinforcement is required, then Equation 4.5 is used to determine the minimum area of transverse reinforcement. This equation in the LRFD Specifications is different than the equation found in the Standard Specifications. It results in a larger minimum area of transverse steel in the column. Article 8.19.1.2 of the Standard Specification uses Equation 4.6 to find the minimum area. The value is a constant, 0.05 ksi. Article 5.8.2.5 in the LRFD Specifications uses Equation 4.5, and the coefficient is a function of the compressive strength of concrete. For 4,000 psi concrete, the value is 0.0632 ksi.

The second check is the maximum spacing check found in LRFD article 5.8.2.7. This check addresses the need for tighter spacing if the section experiences very high shear stress. Most sections will not experience very high shear stress, so this requirement will not typically control the design. The final check is an ALDOT standard maximum spacing of 12 inches. In the event that the column is not required to meet the minimum area of transverse reinforcement requirement, this 12 inch maximum spacing will likely control.

$$V_c = 0.0316 * \beta * \sqrt{f_c'} * b_v * d_v \qquad \text{Equation 4.3}$$

$$V_s = \frac{A_v * f_y * d_v * \cot(\theta)}{s} \qquad \text{Equation 4.4}$$

$$A_{v,min} = 0.0316 * \sqrt{f_c'} * \frac{b_v * s}{f_y} \qquad \text{Equation 4.5}$$

$$A_{v,min} = 0.05 * \frac{b * s}{f_y} \qquad \text{Equation 4.6}$$

Another factor that would affect the spacing of the reinforcement would be the requirement of cross-ties. LRFD Article 5.10.6.3 requires the use of cross-ties in rectangular columns to ensure that no longitudinal bar is more than 2 feet from a restrained bar. However, for all of the bridges in this study, no columns were large enough for this requirement to be necessary. Therefore, this requirement did not control the design.

4.4 Bridge Design Examples in SDC A1

The design procedure in the Guide Specifications for SDC A1 was used to redesign two bridges previously designed under the Standard Specifications. These bridges were supplied by ALDOT and are conventional bridges in the "other" category as described in Chapter 2, making them applicable to the Guide Specifications. One bridge was designed with an acceleration coefficient less than 0.05g and the other with an acceleration coefficient greater than 0.05g to show how the lower accelerations affect the design of the bridge, as well as highlight the differences between the Standard Specifications and Guide Specifications for bridges in each. For each bridge, design sheets were created with references to specific articles in the Guide Specifications or LRFD Specifications and can be seen in Appendix B and C. Notes and other information necessary to the understanding of a certain variable were also noted. Since the purpose of these designs is to determine if a standard set of drawings and details can be identified for these bridges, design data is established for each bent of a bridge. This information will be summarized for each bridge. The two bridges include County Road 39 Bridge over CSX in Mobile County and Stave Creek Bridge in Clarke County.

4.4.1 County Road 39 Bridge

County Road 39 crosses over CSX railroad and US Highway 90 in Mobile County. The overpass has two bridges designed to carry traffic in both the northbound and southbound directions. Each bridge is similarly designed, but the deck of the southbound bridge flares from a width of 54.75 feet at the second pier to 66 feet at the north abutment. The northbound bridge deck remains constant at a width of 54.75 feet. Because the northbound bridge is closest to the conventional bridge definition, it was chosen to be redesigned instead of the southbound bridge. It is a four span bridge with three equal spans of 135 feet and one unequal span of 80 feet at the north end of the bridge. The three equal spans support the 7-inch concrete deck with 9 BT-72 Girders. The unequal span supports the deck with 9 Type III girders. The three bridge piers are 53' x 4' x 4.5' and are supported by three rectangular columns 3.75 feet in diameter with 2 inches of concrete cover. The columns are longitudinally reinforced with 16 #11 bars and transversely with #4 ties uniformly spaced at 12 inches from the bottom of the pier cap to the top of the foundation. The average clear height of each bent was measured from the bottom of the pier cap to the top of the pile cap foundation. The average clear height is 23.6 feet for Bent 2, 28.84 feet for Bent 3, and 26.6 feet for Bent 4. All columns are supported on pile caps with dimensions of 8.6' x 8' x4.5' and each pile cap is supported by nine HP 12x53 driven steel piles. All design calculations can be found in Appendix B.

The first step is finding the vertical reaction at each of the bridge bents. The uniform live load on the bridge, discussed in LRFD 3.6.1.2.4, over the 4 design lanes was 1.28 kips per linear foot. The dead weight included the deck, girders, pier, columns, and guard rails. The total loads were determined using the tributary area of the bents. Table 4.3 compares the design forces when the 0.5 live load factor is used. It shows that using the 0.5 live load factor increases the design forces by 10%.

Table 4.3: Mobile County Bridge Design Force Live Load Factor Comparison

Bent	Design Force with γ_{EQ} (kips)	Design Force without γ_{EQ} (kips)	Percent Difference
2	32.0	29.1	9.9%
3	32.0	29.1	9.9%
4	25.4	23.1	10.0%

Once the vertical reactions were calculated, the design forces for each column were calculated. The acceleration coefficient for this bridge was 0.045g. Since it was less than the 0.05g limit found in Article 4.6, the horizontal design forces were 15% of the vertical reactions. The design forces using the Standard Specifications were 20% of the vertical reactions. As seen in Table 4.4, the design forces are reduced by 25% in the Guide Specifications.

Table 4.4: Mobile County Bridge Design Force Specification Comparison

Bent	Vertical Reaction (kips)	Guide Spec Design Force (kips)	Standard Spec Design Force (kips)	Percent Difference
2	1948	32.0	42.6	-25.0%
3	1948	32.0	42.6	-25.0%
4	1400	25.4	33.9	-25.0%

The minimum support lengths were calculated next. They were different for each bridge bent because of the difference in heights of each bent. Equation 4.2 was used to calculate the new minimum seat lengths and Equation 4.1 was used to calculate the Standard Specifications seat lengths. At each bent, the new lengths were 31-36% greater than those required by the Standard Specifications. Table 4.5 shows the minimum lengths for each bent.

Table 4.5: Mobile County Bridge Minimum Support Lengths

Bent	New Design Minimum Support Length (in)	Standard Spec Minimum Support Length (in)	Percent Difference
2	16.6	12.6	31.7%
3	17.8	13.1	35.9%
4	16.7	12.3	35.8%

For SDC A1, no structural analysis is necessary and the detailing requirements of SDC A2 and B do not apply. The design of the column outside of the plastic hinge zone is accomplished using the LRFD Specifications. #4 ties were used to remain consistent with the previous design. The tie spacing was controlled by 12 inch ALDOT standard. Since the calculated shear was less than half of the nominal shear resistance of the concrete, the minimum area of transverse reinforcement was not required to be satisfied for any of the bents. This resulted in the same amount of transverse reinforcement being required for the designs since the Standard Specifications design also used ties spaced at 12 inches. The results from the redesign of the column can be seen in Table 4.6. Figure 4.4 compares the final design details from the Guide Specifications and Standard Specifications at bent 2. The details for bents 3 and 4 will be similar, except for a different column height, so they are not shown. The only changes in this design were the decrease in the horizontal design force and the increase in the minimum seat width.

Table 4.6: Mobile County Bridge Design Summary

	Bent 2		Bent 3		Bent 4	
	Standard Specification	Guide Specification	Standard Specification	Guide Specification	Standard Specification	Guide Specification
Column Height (in)	283	283	346	346	319	319
Tie Size	#4	#4	#4	#4	#4	#4
Tie Spacing (in)	12	12	12	12	12	12
Number of Ties	24	24	29	29	27	27
Area of Steel (in^2)	4.8	4.8	5.8	5.8	5.4	5.4
Percent Difference	0%		0%		0%	

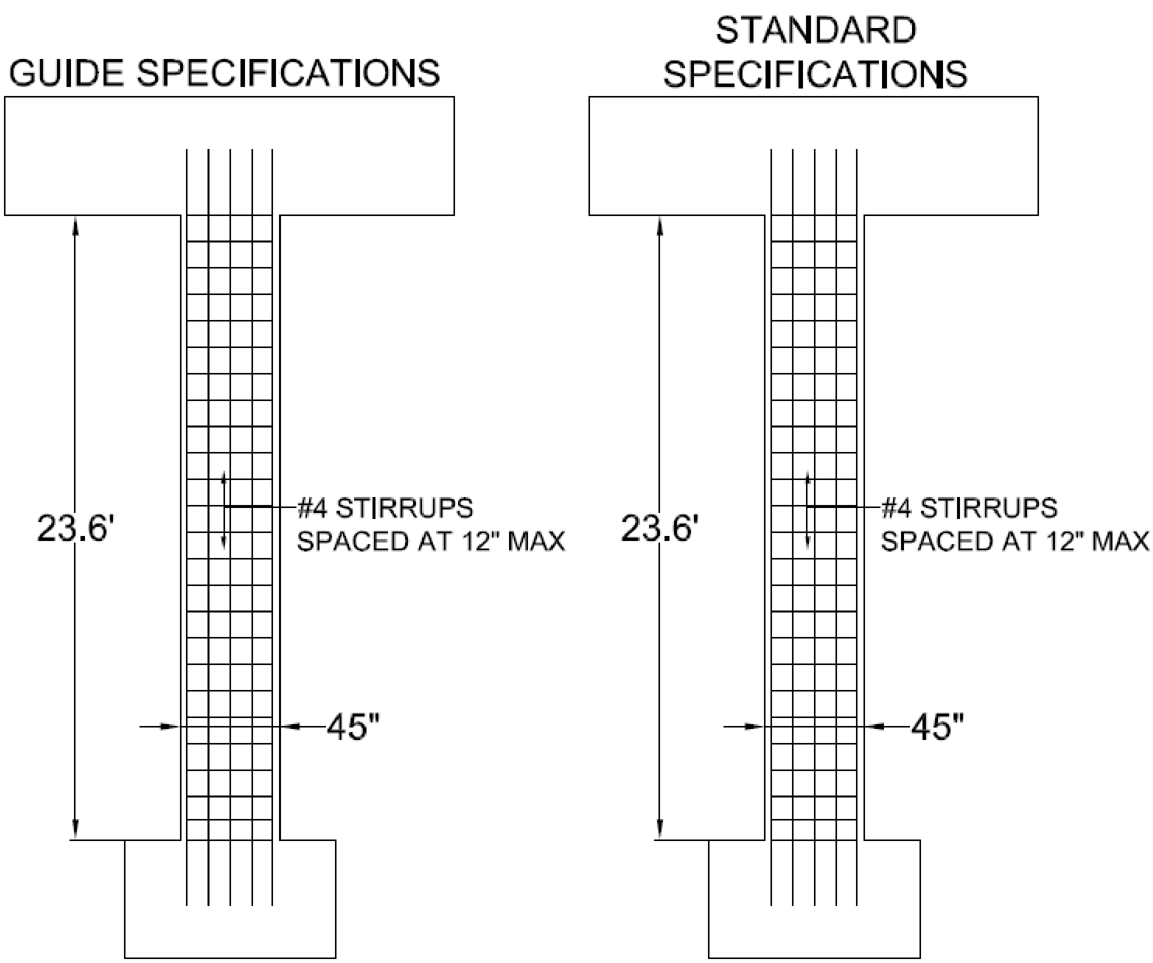

Figure 4.4: Mobile County Bridge Bent 2 Final Design Details

4.4.2 Stave Creek Bridge

State Road 69 crosses over Stave Creek in Clarke County. The overpass has two bridges designed to carry traffic in both the northbound and southbound directions. It is a three span bridge with the two end spans 40 feet long and middle span 85 feet long. The 7-inch concrete deck is a constant 42.75 feet in width and supported by 6 Type I girders in the end spans and 6 Type III girders in the middle span. The two bridge piers are not rectangular because of the different girder types. They are 40 feet long, 4 feet wide, and have depths of 3.75 feet and 5.4 feet. The depths change at approximately 2 feet of width. The piers are supported by two square columns 3 feet in width with 2 inches of concrete cover. The columns are reinforced longitudinally with twelve #11 bars and transversely with #4 ties spaced uniformly at 12 inches from the bottom of the pier cap to the top of the foundation. The average clear height of the columns in Bent 2 is 10.2 feet and for the columns in Bent 3 is 14.34 feet. All columns are supported on 7' x 6.5' x 4.5' pile caps and the pile caps are supported on five HP 12x53 driven steel piles. All design calculations can be found in Appendix C.

The first step is determining the vertical reaction at each of the bridge bents. The uniform live load on the bridge, discussed in LRFD 3.6.1.2.4, over the 3 design lanes was 0.96 kips per linear foot. The dead weight included the deck, girders, pier, columns, and guard rails. The total loads were determined using the tributary area of the bents. Because the bridge was symmetric, the vertical reactions of the bents were equal. Table 4.7 compares the connection design forces when the live load factor is considered and not considered. As the table shows, the design forces increased by 11% when the live load factor of 0.5 was included.

Table 4.7: Stave Creek Bridge Design Force Live Load Factor Comparison (SDC A1)

Bent	Design Force with γ_{EQ} (kips)	Design Force without γ_{EQ} (kips)	Percent Difference
2	25.2	22.7	11.0%
3	25.2	22.7	11.0%

Once the vertical reactions were calculated, the horizontal design forces were calculated for each column. The acceleration coefficient for this bridge was 0.086g, greater than the 0.05g limit found in Article 4.6, so the horizontal design forces were 25% of the vertical reactions. The design forces from the Standard Specification were 20% of the vertical reactions. The design forces can also be found in Table 4.8, displaying the design forces, shows that the Guide Specifications resulted in a 25% increase in the horizontal design forces.

Table 4.8: Stave Creek Bridge Vertical Reactions and Design Forces Comparison (SDC A1)

Bent	Vertical Reaction (kips)	Guide Spec Design Force (kips)	Standard Spec Design Force (kips)	Percent Difference
2	604	25.2	20.1	25.0%
3	604	25.2	20.1	25.0%

The minimum support lengths were calculated next. They were different for each bridge bent because of the difference in clear heights of each bent. The support lengths from the Standard Specifications were calculated using Equation 4.1 and the recommended design support lengths were calculated using Equation 4.2. The new support lengths are greater than the Standard Specifications support lengths by 14-23%, as seen in Table 4.9.

Table 4.9: Stave Creek Bridge Minimum Support Lengths Comparison (SDC A1)

Bent	New Design Minimum Support Length (in)	Standard Spec Minimum Support Length (in)	Percent Difference
2	11.5	10.1	13.9%
3	12.8	10.4	23.1%

For SDC A1, the detailing requirements of SDC A2 and B do not apply. The LRFD Specifications were used to design the transverse reinforcement in the columns since there is no plastic hinge zone. #4 ties were used to remain consistent with the current design. The tie spacing was controlled by the minimum area of transverse reinforcement requirements instead of the shear capacity of the ties, which decreased the maximum spacing to 10 inches. This spacing decrease resulted in a 20% increase in the number of ties at each bent compared to the Standard Specification design. The results from the redesign of the column can be seen in Table 4.10. Figure 4.5 and Figure 4.6 compare the final design details from the Guide Specifications and the Standard Specifications.

Table 4.10: Stave Creek Bridge Design Summary (SDC A1)

	Bent 2		**Bent 3**	
	Standard Specification	**Guide Specification**	**Standard Specification**	**Guide Specification**
Column Height (in)	120	120	168	168
Tie Size	#4	#4	#4	#4
Tie Spacing (in)	12	10	12	10
Number of Ties	10	12	14	17
Area of Steel (in^2)	2	2.4	2.8	3.4
Percent Difference	20.0%		21.4%	

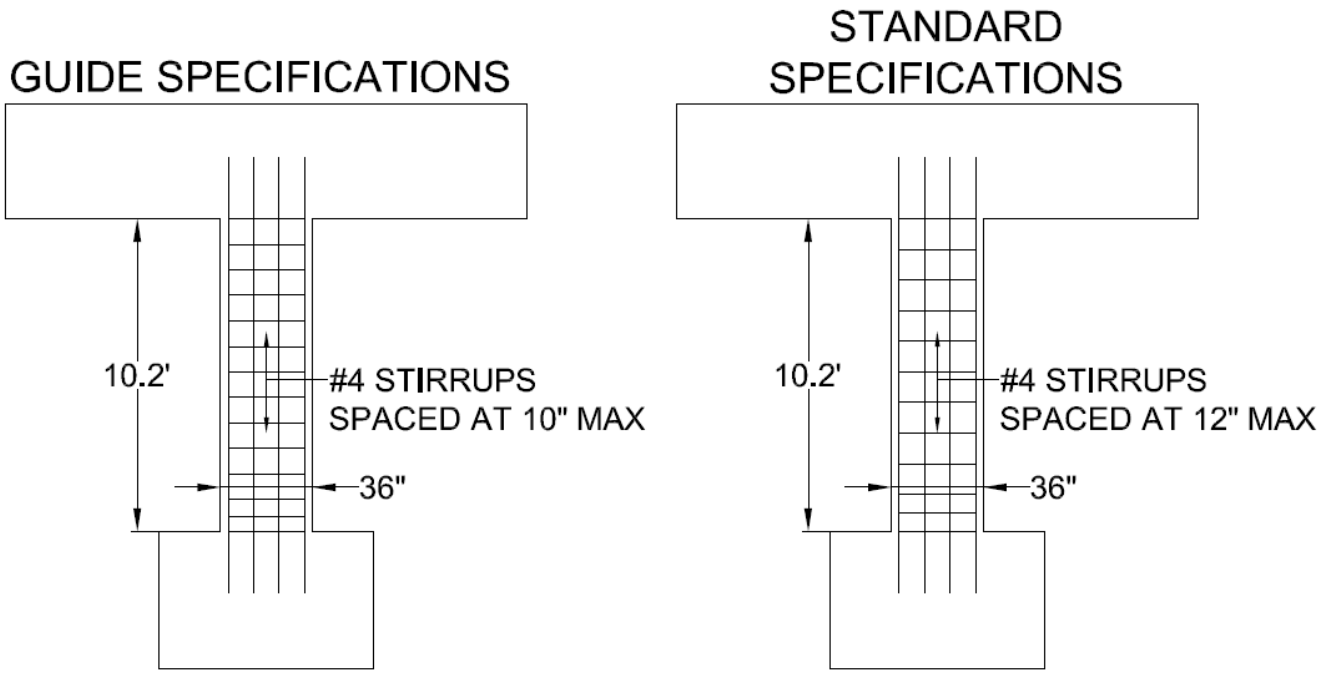

Figure 4.5: Stave Creek Bridge Bent 2 Final Design Details (SDC A1)

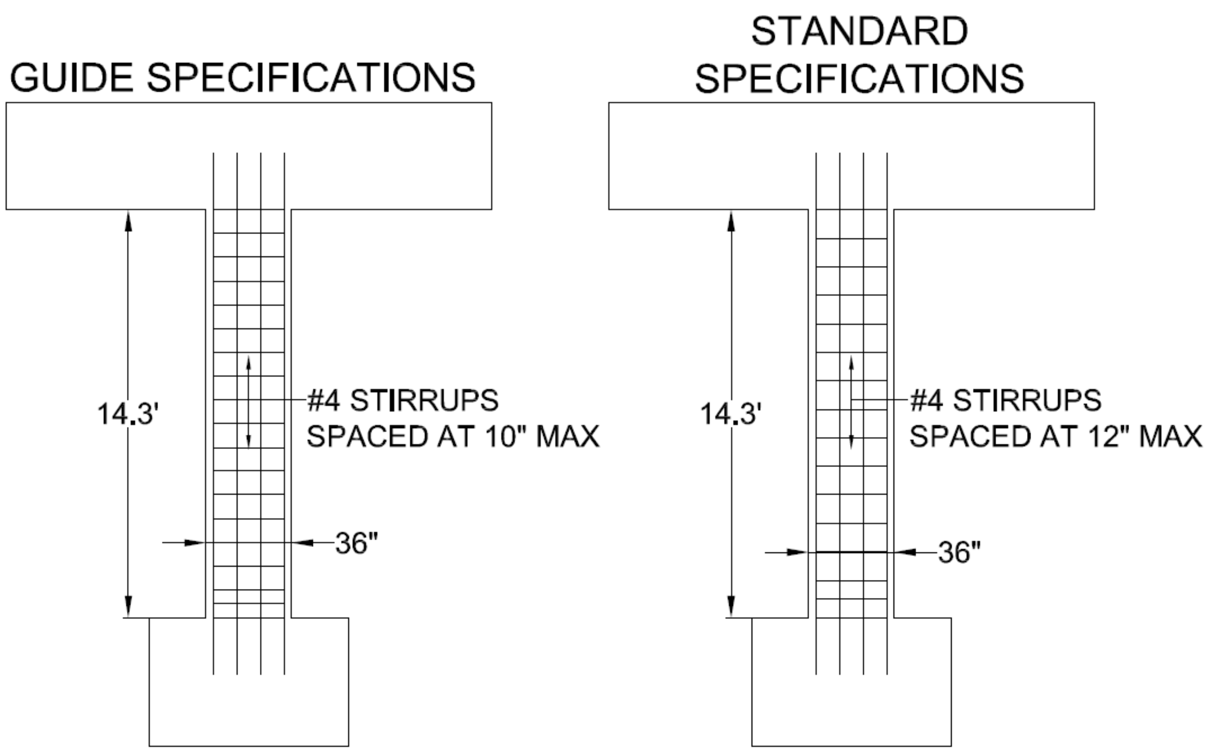

Figure 4.6: Stave Creek Bridge Bent 3 Final Design Details (SDC A1)

4.4.3 Summary of Differences in SDC A1

The changes in bridge design from the Standard Specification to the Guide Specification in SDC A1 were different for lower and higher seismic regions. For very low seismic regions ($A_S < 0.05g$), the design forces decreased by 25%. For other regions in SDC A1 ($A_S \geq 0.05g$), the design forces were increased by 25%. The design forces increased because the changes in the seismic hazard maps resulted in higher ground accelerations than those used in the Standard Specifications. However, the Guide Specifications recognizes that bridges in areas of low seismicity will not experience very high seismic design forces and reduces them accordingly. Another change was that the new seat width equation resulted in greater seat widths for both bridges studied, which was expected since it was designed to give larger seat widths than the equation used in the Standard Specifications and Guide Specifications.

The other change between the two specifications was not related to seismic design. The amount of transverse reinforcement was the same for Mobile County Bridge but different for Stave Creek Bridge. When it did change, it was the result of the minimum area of transverse reinforcement equation in the LRFD Specifications requiring a tighter spacing than that required by a similar equation in the Standard Specifications. This equation was not required for the Mobile County Bridge because the nominal shear resistance of the concrete was twice as large as the expected shear. Though it only affected one of the bridges, this minimum area check is still an important change because it could control the spacing of the ties in the columns. Therefore, it is possible that the transverse reinforcement spacing requirements outside of the plastic hinge zone for bridges designed using the LRFD Specifications will be tighter than the Standard Specifications. However, there are some options that can be used to increase the spacing to 12 inches. Using cross-ties will increase the area of shear reinforcement at each tie level, which would allow the spacing to be increased. Similarly, using a larger size reinforcing bar would also increase the area and spacing of reinforcement. These options could be used if 12 inch spacing was more desirable.

4.5 Guide Specification Design Process for SDC A2

Bridges in SDC A that are expected to experience moderate seismic forces are classified as SDC A2. The possibility exists that these bridges will experience seismic forces that result in plastic hinging and, therefore, require the same minimum detailing from SDC B so that the hinges form at the top and bottom of the column in the transverse direction, and at the bottom in the longitudinal direction. However, there is no structural analysis required. Like SDC A1, the design forces are determined using simplified relationships between the vertical reaction at a bent and the expected ground acceleration. The design steps for SDC A2 bridges will be discussed next.

4.5.1 Determine Vertical Reactions at Bent

Like SDC A1, the first step in calculating the horizontal design force is to determine the vertical reaction at the bent. This is accomplished by finding the tributary area of the bent, the total dead weight of the bridge in that tributary area, and the uniform live load acting on the area. The dead weight of the bridge includes the weight of the deck, girders, piers, columns, and guard rails. The uniform live load consists of a 0.64 kip per linear foot per lane load that is applied simultaneously with the dead load. The LRFD Specifications, in Article C3.4.1, recommends including 50% of this live load in the vertical reaction calculations, but does not require it. It does require the bridge owner to determine the live load factor, γ_{EQ}, on a project specific basis.

This live load factor determines what percentage of the live load is to be included in the weight calculations. For bridges in high traffic areas, such as major highways in large city centers, it is recommended to include at least half of the live load, because it is possible for that bridge to experience live loads during a seismic event. Once the live load is determined, it is multiplied by the number of design lanes and the tributary length of the bent. The total vertical reaction is the sum of the dead and live load resisted by the bent. This thesis will show two horizontal design forces for all bridges in SDC A, one that includes the 0.50 live load factor and one that does not. A comparison between these two design forces will show if the live load factor has a significant effect.

4.5.2 Determine Design Forces

Using the vertical reaction at the bent, the horizontal design forces are calculated using Article 4.6 of the Guide Specifications. This article details the seismic design requirements for bridges in SDC A. The design force is used to design the column for shear and the connection between the superstructure and substructure. For column shear, the vertical reaction is divided by the number of columns at the bent to represent the amount of load each column will resist. For the connection, the vertical reaction is divided by the number of connections, which is equal to the number of girders at the bent. The design force is then multiplied by 0.25 times the vertical reaction at the bent. Unlike SDC A1, it is not possible for the design force to be 0.15 times the vertical reaction at the bent because A_S will not be below 0.05g, which is the limit for the lower design force in Article 4.6. The Standard Specifications require 0.20 times the vertical reactions for all sites in SDC A.

4.5.3 Determine Minimum Support Lengths

Support lengths are the length of overlap between the girder and pier or abutment seat. The minimum support length must be provided to accommodate differential movement between the superstructure and the substructure. These displacements occur during a design earthquake and are typically conservative. However, providing the minimum support length alone does not guarantee the girder will remain seated during an earthquake, especially if it is larger than the design earthquake. Providing seat widths larger than the minimum or using restrainer bars and cables can limit the displacement if unseating is a concern. Article 4.12 in the Guide Specifications uses Equation 4.1 to determine the minimum support length. Currently, ALDOT uses this equation to determine the minimum support lengths, but in chapter 3 of this thesis, it was recommended to use Equation 4.2 from the ATC-49 study (ATC/MCEER Joint Venture, 2003) to determine the minimum seat length because it will give a larger seat width. The Standard Specification uses Equation 4.1 in both SDC A and B. In this thesis, the minimum seat lengths for bridges in SDC A1 will be calculated using Equation 4.2 and compared with the results from Equation 4.1, which represent the minimum seat length from the Standard Specifications. Because the new equation uses the spectral acceleration, S_{D1}, in the multiplier factor, for SDC A2 greater seat widths can be expected since these bridges will have higher accelerations than those in SDC A1.

$$N = (8 + 0.02L + 0.08H) * (1 + 0.000125S^2) \qquad \text{Equation 4.1}$$

$$N = \left(4 + 0.02L + 0.08H + 1.09\sqrt{H}\sqrt{1 + (2\frac{B}{L})^2}\right) * \left(\frac{1+1.25S_{D1}}{\cos(\alpha)}\right) \qquad \text{Equation 4.2}$$

4.5.4 Minimum Column Detailing

Once the minimum seat widths and horizontal design forces are calculated, the minimum detailing requirements of SDC B must be met, according to Article 8.2. These include the minimum shear reinforcement of Article 8.6.5 and the minimum requirements for lateral reinforcement in Article 8.8.9. This shear reinforcement is to extend over the entire plastic hinge length determined in Article 4.11.7. These details will allow the column to be ductile and form plastic hinges in the high moment regions if the bridge experiences high seismic forces.

4.5.4.1 Plastic Hinge Length

The plastic hinge length (PHL) is the assumed length of the column where the plastic hinge will form and is designed to be at the top of the column and the bottom of the column for bending in the transverse direction and at the bottom of the column for bending in the longitudinal direction, where the column meets the foundation, although the location of the plastic hinge at the bottom can vary depending on the soil and foundation type. For the bridges in SDC A2, the plastic hinge was assumed to form at the connection between the column and foundation element. These locations occur at the point of maximum moment and shear in the column. These flexural areas allow the bridge to dissipate energy. The shear reinforcement helps confine the concrete and prevent buckling of the longitudinal reinforcement, as well as increase the shear resistance of the section, which decreases the possibility of a brittle failure that will not allow the column to dissipate energy and remove vertical capacity. Article 4.11.7 in the Guide Specification defines the PHL to be the largest of three lengths (AASHTO, 2011):

- 1.5 times the largest cross-sectional dimension in the direction of bending

- The region of the column where the moment demand exceeds 75% of the maximum plastic moment
- The analytical plastic hinge length, L_p

The largest cross-sectional dimension will be either the diameter of a circular column or the width in the direction of bending of a rectangular column. The maximum plastic moment is determined by a moment-axial load interaction diagram. For this project, the software program spColumn was used (StructurePoint, 2012). The dimensions of the column and the reinforcement layout are input into the program, and the maximum moment is determined from the resulting moment interaction diagram. Once the maximum plastic moment is determined, the moment diagram from the computer analysis software can be used to determine the length of the column where the moment exceeds 75% of the plastic moment. The analytical plastic hinge length is determined in Article 4.11.6 using Equation 4.7. This equation is specifically for reinforced concrete columns framing into a footing, integral bent cap, oversized shaft, and cased shaft, which meets the criteria for this project.

$$L_p = 0.08 * L + 0.15 * f_{ye} * d_{bl} \geq 0.3 * f_{ye} * d_{bl} \qquad \text{Equation 4.7}$$

In most cases, the PHL is controlled by the 1.5 times the gross cross-sectional dimension. This can result in a large PHL for large columns and since the PHL is at the top and bottom of the column, the entire column could be considered to be within the plastic hinge. This makes it difficult to satisfy the lap splicing requirements found in Article 8.8.3 for the longitudinal column reinforcement. The splicing is required to be outside of the plastic hinge length. Failure to do so could lead to undesirable seismic performance because the splice would be subject to plastic forces and deformations, which could lead to a reduced effective plastic hinge length and severe local curvature demand (AASHTO, 2011). While this article only applies to SDC C and

D, the commentary recommends that they also be applied to SDC B. While these splicing requirements are not required in SDC A2, the designer should consider their effects.

An alternative to this PHL is given in Article 8.2 of the Guide Specifications. This article allows the use of Article 5.10.11.4.1e in the LRFD Specifications to calculate the length. These requirements are easier to determine and do not require any computer software. The maximum of three limits is taken as the PHL (AASHTO, 2007):

- The largest cross-sectional dimension
- One-sixth the clear height of the column
- 18 inches

The largest cross-sectional dimension will be either the diameter of a circular column or the largest width of a rectangular column. The clear height of the column depends on the foundation type and geometry. For three of the bridges, driven piles were used as the foundations. It was assumed the plastic hinge would form at the column to pile cap connection, and the column height was taken from the bottom of the pier cap to the top of the pile cap. However, one of the bridges used drilled shafts as the foundation. The drilled shaft was the same diameter as the column, so because of the similar geometry and relatively small amount of soil able to resist flexure of the column and drilled shaft, it was conservatively assumed that the point of fixity of the column to be at the rock line. It is important to understand how the column and foundation will interact in order to determine where the plastic hinge is likely to form. Using the maximum of these three values will typically result in a smaller PHL than that found in the Guide Specifications. This will allow for a greater length of column for splicing. For the bridges in SDC A2, both values will be checked.

The LRFD Specifications, in Article 5.10.11.4.3, discuss an extension of the plastic hinge length into the cap beam or the foundation (pile cap or drilled shaft). The extension length is an extra length over which the ties from the plastic hinge zone span. The spacing of these ties is the same required in the plastic hinge zone. This is an extra measure to ensure the plastic hinge forms at the top or bottom of the column. Article 5.10.11.4.1e in the LRFD Specifications requires the extension length to be the maximum of the following:

- One-half of the column diameter
- 15 inches

This extension is only required in SDC C and D, but it is recommended in SDC B. This extension length is not found in the Guide Specifications, but since the plastic hinge zone requirements for SDC A2 include the same detailing from SDC B, the extension length will be calculated for all four bridges, but will not be provided in the design drawings.

4.5.4.2 Transverse Reinforcement inside the Plastic Hinge Zone

Once the plastic hinge length is determined, the size and spacing of the transverse reinforcement within the PHL can be determined. For SDC A2, the minimum ratios of transverse reinforcement in Article 8.6.5 and the requirements of Article 8.8.9 must be met. The ratios are calculated in Article 8.6.2, and must be greater than or equal to 0.003 for spirals in circular columns and greater than or equal to 0.002 for rectangular columns. Article 8.8.9 lists standard tie requirements that will ensure the lateral support is supplied to the longitudinal reinforcement. These requirements will not be discussed, with the exception of the maximum spacing requirements inside the plastic hinge regions. The maximum spacing is to be the smaller of the following:

- One-fifth the least dimension of the cross-section for columns
- Six times the nominal diameter of the longitudinal reinforcement
- 6 inches

If the longitudinal reinforcement is at least a #9 bar and the column size is at least 30 inches, as all of the bridges in this study are, then the 6 inch maximum spacing controls. However, the spacing must still satisfy the minimum ratios. Once this ratio and all the requirements of Article 8.8.9 have been satisfied, the detailing within the PHL is finished.

4.5.4.3 Transverse Reinforcement outside the Plastic Hinge Zone

The detailing for transverse reinforcement outside of the plastic hinge region in SDC A2 is the same as SDC A1. The LRFD Specifications were used to design the shear reinforcement outside of the plastic hinge zone. The shear reinforcement must be checked to ensure that it provides greater resistance than the expected horizontal design force in the column. Equations 4.3 and 4.4 from Article 5.8.3.3 in the LRFD Specifications are used to determine the shear capacity of the transverse reinforcement and the concrete. Once the design is satisfied for strength, three spacing requirements are checked. These spacing requirements could control the design and must be checked. The first requirement can be found in Article 5.8.2.5 of the LRFD Specifications and is a minimum amount of transverse reinforcement. It is only required when the factored load is greater than half of the factored resistance by the concrete section and prestressing steel (if present). It is intended to provide reinforcement in regions where there is a significant chance of diagonal cracking (AASHTO, 2009). If it is determined that this minimum reinforcement is required, then Equation 4.5 is used to determine the minimum area of transverse reinforcement. This equation in the LRFD Specifications is different than the equation found in the Standard Specifications. It results in a larger minimum area of transverse steel in the column. Article 8.19.1.2 of the Standard Specification uses Equation 4.6 to find the minimum area. The value is a constant, 0.05 ksi. Article 5.8.2.5 in the LRFD Specifications uses Equation 4.5, and the coefficient is a function of the compressive strength of concrete. For 4,000 psi concrete, the value is 0.0632 ksi. The difference between the values shows that the LRFD Specifications will result in a higher minimum area of reinforcement compared to the Standard Specifications.

The second check is the maximum spacing check found in LRFD article 5.8.2.7. This check addresses the need for tighter spacing if the section experiences very high shear stress. Most sections will not experience very high shear stress, so this requirement will not typically control the design. The final check is an ALDOT standard maximum spacing of 12 inches. In the event that the column is not required to meet the minimum area of transverse reinforcement requirement, this 12 inch maximum spacing will likely control.

$$V_c = 0.0316 * \beta * \sqrt{f_c'} * b_v * d_v \qquad \text{Equation 4.3}$$

$$V_s = \frac{A_v * f_y * d_v * \cot(\theta)}{s} \qquad \text{Equation 4.4}$$

$$A_{v,min} = 0.0316 * \sqrt{f_c'} * \frac{b_v * s}{f_y} \qquad \text{Equation 4.5}$$

$$A_{v,min} = 0.05 * \frac{b * s}{f_y} \qquad \text{Equation 4.6}$$

Another factor that would affect the spacing of the reinforcement would be the requirement of cross-ties. LRFD Article 5.10.6.3 requires the use of cross-ties in rectangular columns to ensure that no longitudinal bar is more than 2 feet from a restrained bar. However, for all of the bridges in this study, no columns were large enough for this requirement to be necessary. Therefore, this requirement did not control the design.

4.6 Bridge Design Examples in SDC A2

Four bridges in SDC A2 were redesigned using the Guide Specifications. These bridges were supplied by ALDOT and are conventional bridges in the "other" category as described earlier, making them applicable to the Guide Specifications. One of the bridges was also redesigned in the SDC A1 category. The differences between the two designs will be discussed to show how SDC A1 and SDC A2 are different. Three other bridges will be redesigned as SDC B bridges for similar purposes, but comparisons will not be mentioned in this section. For those comparisons, refer to the "Bridge Design Examples in SDC B" chapter of the thesis. For each bridge, design sheets were created with references to specific articles in the Guide Specifications or LRFD Specifications. Notes and other information necessary to the understanding of a certain variable were also recorded. Since the purpose of these redesigns is to determine if a standard set of drawings and details can be identified for these bridges, design data is established for each bent of a bridge. This information will be summarized for each bridge. The four bridges to be redesigned include the following: Stave Creek Bridge in Clarke County, Bent Creek Road Bridge in Lee County, Bridge over Norfolk Southern Railroad in Etowah County, and Oseligee Creek Bridge in Etowah County.

4.6.1 Stave Creek Bridge

This bridge has been previously designed in the SDC A1 section and will be compared to it in order to determine the differences in design. The designs from the Standard Specification to the Guide Specification will also be compared. Stave Creek Bridge is in Clarke County and carries State Road 69 over Stave Creek. The overpass has two bridges designed to carry traffic in both the northbound and southbound directions. It is a three span bridge with the two end spans 40 feet long and middle span 85 feet long. The 7 inch concrete deck is a constant 42.75 feet in width and supported by 6 Type I girders in the end spans and 6 Type III girders in the middle span. The two bridge piers are not rectangular because of the different girder types. They are 40 feet long, 4 feet wide, and have depths of 3.75 feet and 5.4 feet. The depths change at approximately 2 feet of width. The piers are supported by two square columns 3 feet in width with 2 inches of concrete cover. The columns are reinforced longitudinally with 12 #11 bars and transversely with #4 ties spaced uniformly at 12 inches from the bottom of the pier cap to the top of the foundation. The average clear height of the columns in Bent 2 is 10.2 feet and for the columns in Bent 3 is 14.34 feet. All columns are supported on 7' x 6.5' x 4.5' pile caps and the pile caps are supported on five HP 12x53 driven steel piles. All design calculations for this bridge can be found in Appendix D.

The first step is determining the vertical reaction at each of the bridge bents. The uniform live load on the bridge, discussed in LRFD 3.6.1.2.4, over the 3 design lanes was 0.96 kips per linear foot. Since the tributary area of the bents was equal, the vertical reactions at each bent will be equal. Since the horizontal design force is 25% of the vertical reaction, these design forces are equal to the design forces from SDC A1. As Table 4.11 shows, including the 0.50 live load factor increases the design forces by 11%.

Table 4.11: Stave Creek Bridge Design Force Live Load Factor Comparison (SDC A2)

Bent	Design Force with γ_{EQ} (kips)	Design Force without γ_{EQ} (kips)	Percent Difference
2	25.2	22.7	11.0%
3	25.2	22.7	11.0%

Once the vertical reactions were calculated, the horizontal design forces were calculated. For SDC A2, the design forces are 25% of the vertical reaction. The design forces in the

Standard Specification are 20% of the vertical reaction, meaning the Guide Specification results in a 25% increase in the horizontal design forces, as seen in found in Table 4.12.

Table 4.12: Stave Creek Bridge Vertical Reactions and Design Forces Comparison (SDC A2)

Bent	Vertical Reaction (kips)	Guide Spec Design Force (kips)	Standard Spec Design Force (kips)	Percent Difference
2	604	25.2	20.1	25.0%
3	604	25.2	20.1	25.0%

The minimum support lengths were determined next. They were different for each bridge bent because of their difference in clear heights. The support lengths from the Standard Specifications were calculated using Equation 4.1 and the recommended design support lengths were calculated using Equation 4.2. Table 4.13 shows the minimum support lengths for each bent required by each specification. The seat length increases by 16-26%. The new design seat length is greater for the Stave Creek Bridge bents designed in SDC A2 (compared to SDC A1) because the spectral acceleration values are slightly greater.

Table 4.13: Stave Creek Bridge Minimum Seat Width Comparison (SDC A2)

Bent	New Design Minimum Support Length (in)	Standard Spec Minimum Support Length (in)	Percent Difference
2	11.8	10.1	16.8%
3	13.1	10.4	25.9%

Once the design forces and seat widths were calculated, the transverse reinforcement was designed. Table 4.14 shows the results from the design. Both bents had the same plastic hinge lengths, tie sizes, and tie spacing. The plastic hinge length was determined to be 36 inches for each bent. The width of the columns controlled the plastic hinge length since the columns were relatively short. The spacing inside the plastic hinge zones was controlled by the reinforcement ratio, and a maximum spacing of 5 inches was determined to satisfy the ratio. The spacing outside of the plastic hinge zone was 10 inches, controlled by the minimum area of transverse reinforcement, which was required for these columns. A 12 inch maximum spacing throughout the entire column was required by the Standard Specifications. So, the tighter spacing resulted in a 40-45% increase in the number of ties required. Figure 4.7 and Figure 4.8 compare the final design details from the Standard Specifications and the Guide Specifications.

Table 4.14: Stave Creek Bridge Design Summary

	Bent 2		Bent 3	
	Standard Specification	Guide Specification	Standard Specification	Guide Specification
Column Height (in)	122	122	172	172
Tie Size	#4	#4	#4	#4
Plastic Hinge Length (in)	-	36	-	36
PHL Spacing (in)	-	5	-	5
Spacing outside PHL (in)	12	10	12	10
Number of Ties	11	16	15	21
Area of Steel (in^2)	2.2	3.2	3	4.2
Percent Difference	45.5%		40.0%	

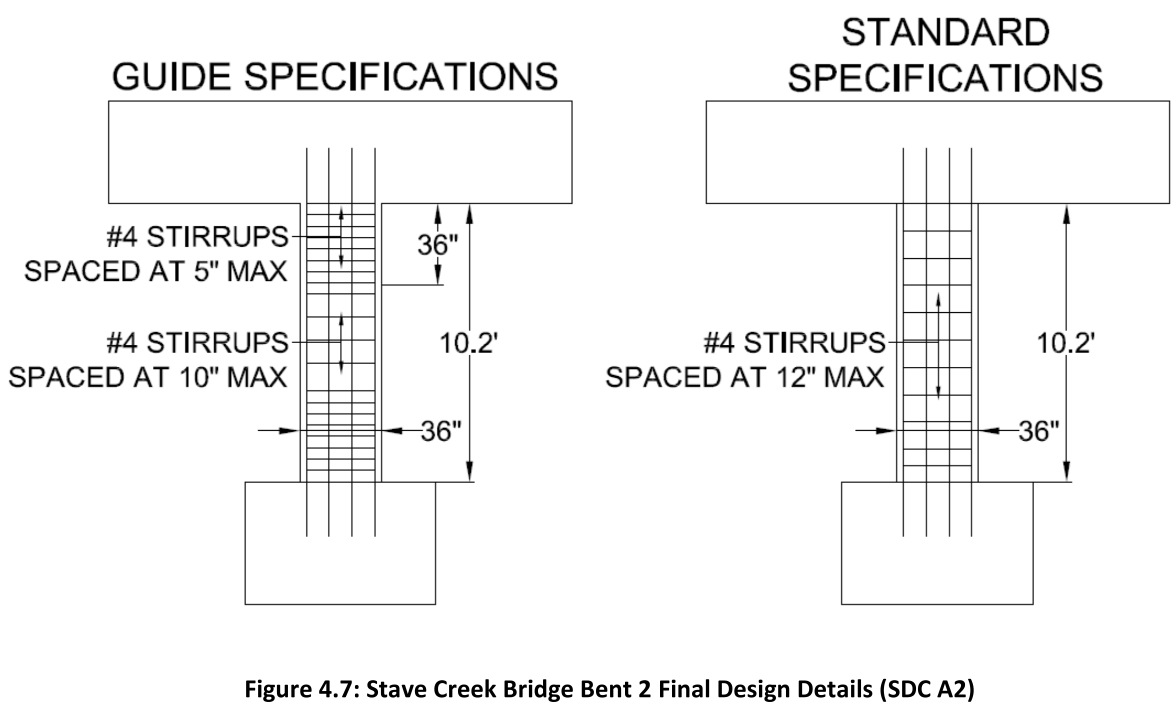

Figure 4.7: Stave Creek Bridge Bent 2 Final Design Details (SDC A2)

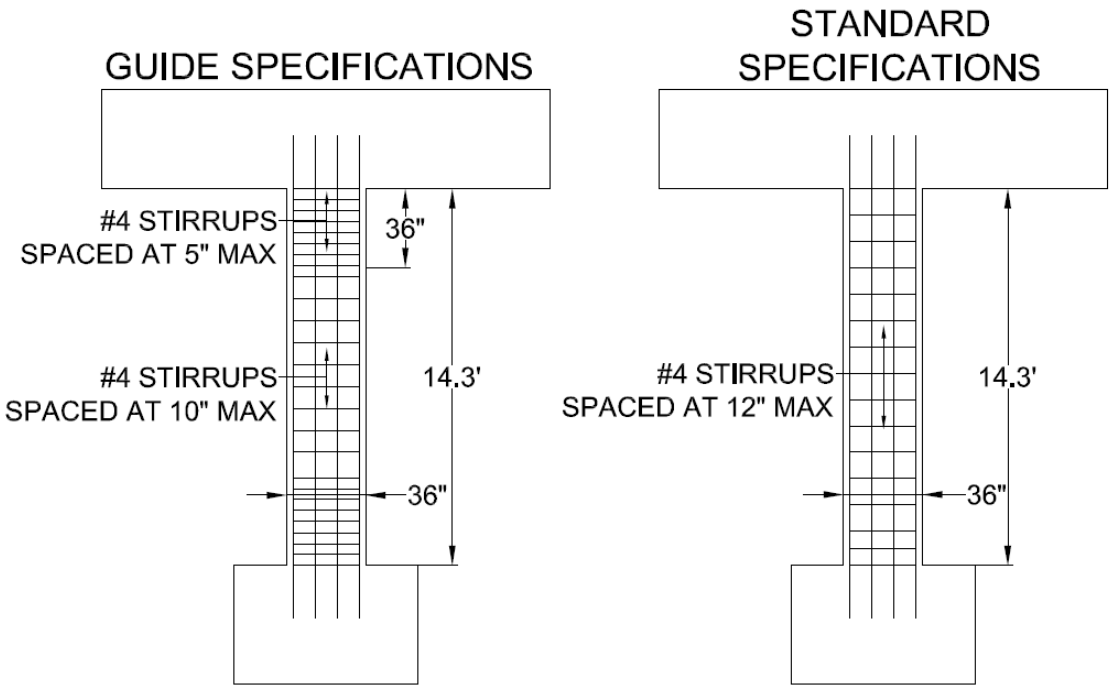

Figure 4.8: Stave Creek Bridge Bent 3 Final Design Details (SDC A2)

One change from SDC A1 to SDC A2 was the addition of the plastic hinge zone. As Table 4.15 shows, requiring tighter tie spacing over a portion of the column results in a 25-30% increase in the number of ties. The design forces stayed the same, but the minimum seat widths as determined by the recommended equation increased slightly as a result of higher expected spectral accelerations.

Table 4.15: Stave Creek SDC A1 and A2 Design Comparison

	Bent 2		Bent 3	
	Guide SDC A1	**Guide SDC A2**	**Guide SDC A1**	**Guide SDC A2**
Number of Ties	12	16	17	21
Area of Steel (in^2)	2.4	3.2	3.4	4.2
Percent Difference	33.3%		23.5%	

4.6.2 Bent Creek Road Bridge

The next bridge to be designed was Bent Creek Road Bridge in Lee County. It is a five-lane bridge that crosses over Interstate 85 with two spans of 135 feet. Each span is comprised of 15 modified BT-54 girders spaced approximately 5.33 feet apart that support a 6 inch concrete deck that is 80.75 feet wide. The only bridge pier is 79' x 4' x 4.5' and supported by five square columns 3.5 feet in width. The columns are reinforced longitudinally with 12 #11 bars and transversely with #4 ties uniformly spaced at 12 inches from the bottom of the bent to the top of the pile cap foundation. The average clear height of the columns is 20.1 feet. The bridge is supported on driven piles. The pile cap is 8.5' x 8' x 4.5' and each pile cap is supported by 9 HP 12x52 steel piles. The design calculations for this bridge can be found in Appendix E.

The first step is determining the vertical reaction at the bridge bent. The uniform live load on the bridge, discussed in LRFD 3.6.1.2.4, over the 6 design lanes was 1.92 kips per linear foot. The total loads were determined using the tributary area of the bent. The horizontal design forces including the live load factor of 0.50 were compared with the design forces with no live load considered. As Table 4.16 shows, the design forces increased by 10% with the addition of the live load.

Table 4.16: Bent Creek Road Bridge Design Force Live Load Factor Comparison (SDC A2)

Bent	Design Force with γ_{EQ} (kips)	Design Force without γ_{EQ} (kips)	Percent Difference
2	47.5	43.2	9.9%

Once the vertical reaction was found, the horizontal design forces were calculated. For SDC A2, the vertical reactions are 25% of the vertical reactions. These forces were 25% greater than those calculated using the Standard Specification, where the design force is 20% of the vertical reaction. Table 4.17 compares the two design forces.

Table 4.17: Bent Creek Road Bridge Vertical Reaction and Design Forces (SDC A2)

Bent	Vertical Reaction (kips)	Guide Spec Design Force (kips)	Standard Spec Design Force (kips)	Percent Difference
2	2852.2	47.5	38.0	25.0%

The next step was to calculate the minimum seat widths. Equation 4.1 and Equation 4.2 were used to calculate the seat widths according to the Standard Specifications and the new design recommendation, respectively. The new design equation resulted in a 30% longer seat width, as seen in Table 4.18.

Table 4.18: Bent Creek Road Bridge Minimum Seat Width Comparison (SDC A2)

Bent	New Design Minimum Support Length (in)	Standard Spec Minimum Support Length (in)	Percent Difference
2	16.4	12.3	30.1%

The design of the transverse reinforcement in the columns was completed next. Table 4.19 summarizes the results from the design. The plastic hinge length was controlled by the width of the column and determined to be 42 inches. The spacing inside the plastic hinge zones was controlled by the reinforcement ratio, and a maximum spacing of 4 inches was determined to satisfy this ratio. The spacing outside of the plastic hinge zone was 9 inches. The minimum area of transverse reinforcement check in the LRFD Specifications controlled this spacing. The Standard Specifications design only required 12 inch spacing. Using the Guide Specification resulted in an 85% increase in the number of ties required, both from the tighter spacing of ties in the plastic hinge zone and the tighter spacing of ties outside of the plastic hinge zone.

Table 4.19: Bent Creek Road Bridge Design Summary (SDC A2)

	Bent 2	
	Standard Specification	Guide Specification
Column Height (in)	240	240
Tie Size	#4	#4
Plastic Hinge Length (in)	-	36
PHL Spacing (in)	-	4
Spacing outside PHL (in)	12	9
Number of Ties	20	37
Area of Steel (in^2)	4	7.4
Percent Difference	85.0%	

The major differences between the two design specifications were the design forces and spacing of ties. The design forces increased by 25% and required approximately 85% more ties because of the tighter spacing requirements. This was due to the addition of the plastic hinge zone, which requires tight spacing, and the increase in spacing outside of the plastic hinge zone from the minimum area requirements. The only thing not affected was the minimum seat width, which was the same. Figure 4.9 shows the design details from each specification. While this design used #4 ties to maintain consistency with the Standard Specifications design, another option that would increase the spacing would be to use cross-ties or a larger bar size. This would maintain the same amount of reinforcing steel, but allow for larger spacing between ties.

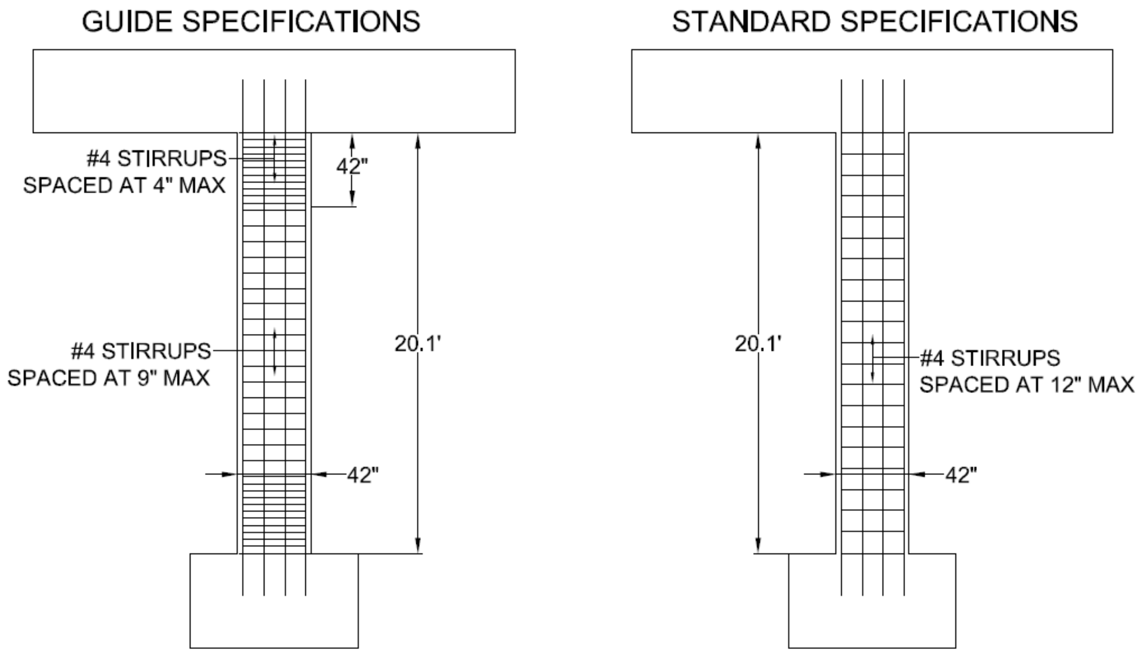

Figure 4.9: Bent Creek Road Bridge Bent 2 Final Design Details (SDC A2)

4.6.3 Bridge over Norfolk Southern Railroad

The third bridge to be designed in SDC A2 was the Bridge over Norfolk Southern Railroad. The southbound I-59 bridge in Etowah County is a two lane bridge that crosses over a Norfolk Southern railroad line and a state highway. It is a two span bridge with unequal span lengths of 125 feet and 140 feet. Nine modified BT-54 girders support a 6 inch concrete deck that is 46.75 feet wide. The only bridge pier is 53' x 4.5' x 4' and supported by three square columns 3.5 feet in width. The columns are reinforced longitudinally with twelve #11 bars and transversely with #4 ties uniformly spaced at 12 inches from the bottom of the bent to the top of the pile cap foundation. The average clear height of the columns is 25.25 feet. The bridge is supported on driven piles. The pile cap is 8.5' x 8' x 4.5' and each pile cap is supported by 7 HP 12x53 steel piles. Appendix F contains the design calculations for this bridge.

The first step was to determine the horizontal design forces at the bridge bent. This live load was calculated using 3 design lanes. The horizontal design forces were determined with and without the live load factor and are compared in Table 4.20. Including the live load increased the design forces by almost 8% for this bridge.

Table 4.20: Norfolk Southern Bridge Design Force Live Load Factor Comparison (SDC A2)

Bent	Design Force with γ_{EQ} (kips)	Design Force without γ_{EQ} (kips)	Percent Difference
2	49.1	45.5	7.9%

Once the vertical reaction was found, the horizontal design forces were calculated. For SDC A2, the design forces are 25% of the vertical reactions according to the Guide Specifications. In the Standard Specifications, the horizontal design forces are 20% of the vertical reaction. Table 4.21 shows that using the Guide Specifications increased the forces by 25%.

Table 4.21: Norfolk Southern Bridge Design Force Comparison (SDC A2)

Bent	Vertical Reaction (kips)	Guide Spec Design Force (kips)	Standard Spec Design Force (kips)	Percent Difference
2	1766	49.1	39.3	25.0%

The next step was to calculate the minimum seat width. Equation 4.1 was used to calculate the seat width for the Standard Specification and Equation 4.2 was used to calculate the new recommended seat width. As Table 4.22 shows, the new seat length is nearly 45% greater than the seat length provided by the Standard Specifications.

Table 4.22: Norfolk Southern Bridge Minimum Seat Width Comparison (SDC A2)

Bent	New Design Minimum Support Length (in)	Standard Specification Minimum Support Length (in)	Percent Difference
2	18.4	12.7	44.9%

Next, the design of the transverse reinforcement in the columns was completed. Table 4.23 summarizes the results from the design. The plastic hinge length was controlled by the height of the column and determined to be 50.5 inches. The spacing inside the plastic hinge zones was controlled by the reinforcement ratio, and a maximum spacing of 4 inches was determined to satisfy this ratio. The spacing outside of the plastic hinge zone was 9 inches. The minimum area of transverse reinforcement check in the LRFD Specifications was required for this bent and it controlled the spacing. Using the Guide Specification resulted in an 85% increase in the number of ties required because of the tighter spacing. An option that could be used to increase the spacing would be to use cross-ties or increase the tie size.

Table 4.23: Bridge over Norfolk Southern Railroad Design Summary (SDC A2)

	Bent 2	
	Standard Specification	**Guide Specification**
Column Height (in)	303	303
Tie Size	#4	#4
Plastic Hinge Length (in)	-	50.5
PHL Spacing (in)	-	4
Spacing outside PHL (in)	12	9
Number of Ties	26	48
Area of Steel (in^2)	5.2	9.6
Percent Difference	84.6%	

The major differences between the two design specifications were the design forces, minimum seat widths, and amount of transverse reinforcement. The design forces increased by 25% and the new design needed approximately 85% more ties. This was due to the addition of the plastic hinge zone and the increase in spacing outside of the plastic hinge zone from the minimum area requirements. The minimum seat width increased because of the new equation that is used. Because the bent was very tall, the change in the seat length was greater than in any of the previously studied bridges. Figure 4.10 shows the design details from each specification.

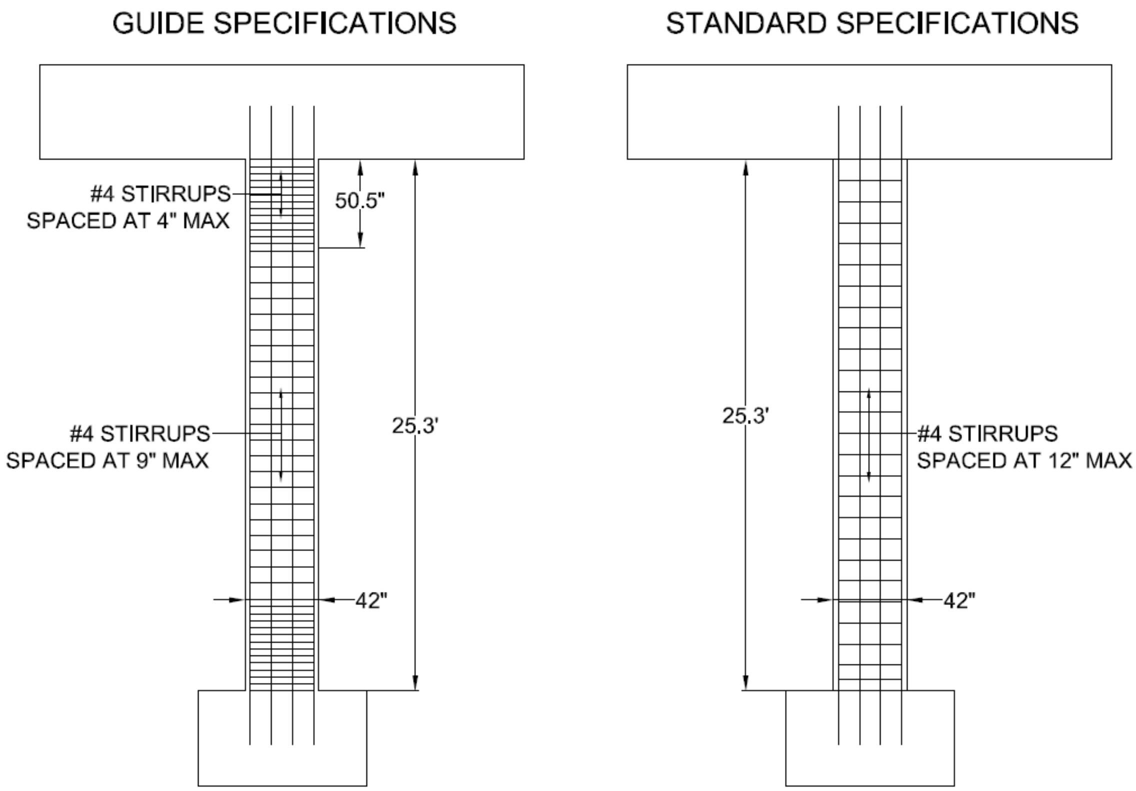

Figure 4.10: Bridge over Norfolk Southern Railroad Final Design Details (SDC A2)

4.6.4 Oseligee Creek Bridge

The final SDC A2 bridge to be re-designed was Oseligee Creek Bridge. This two lane bridge carries County Road 1289 over Oseligee Creek in Chambers County. It is a three span bridge with equal span lengths of 80 feet. The 7 inch concrete deck is supported by 4 Type III girders. The two bridge piers are 30’ x 4’ x 5’ and supported by two circular columns 3.5 feet in diameter with 3 inches of concrete cover. The columns are reinforced longitudinally with 12 #11 bars and transversely with #5 hoops uniformly spaced at 12 inches from the bottom of the pier cap to the rock line. The average clear height of Bent 2 is 17.93 feet and 25.83 feet for Bent 3.

All columns are supported on drilled shafts 3.5 feet in diameter with concrete cover of 3 inches. Because the column and drilled shaft were the same diameter with no clear transition between them, it was unknown where the plastic hinge would form. It was assumed that the soil would not provide enough lateral reinforcement alone to force the plastic hinge to form at the ground line, so the plastic hinge was designed to form at the rock line. For this reason, the height of the columns used for the plastic hinge calculation was assumed to be from the bottom of the bent cap to the rock line. The design calculations for this bridge can be seen in Appendix G.

The first step in calculating the horizontal design force was determining the vertical reaction at each of the bridge bents. Each bent was similar, with the same dead weight and tributary area, so the vertical reactions were the same. The horizontal design forces including and excluding the live load factor were compared. As Table 4.24 shows, including the live load increased the design forces by 9%.

Table 4.24: Oseligee Creek Bridge Design Force Live Load Factor Comparison (SDC A2)

Bent	Design Force with γ_{EQ} (kips)	Design Force without γ_{EQ} (kips)	Percent Difference
2	38.8	35.6	8.9%
3	38.8	35.6	8.9%

Once the vertical reactions were found, the design forces were calculated. The Guide Specifications require the horizontal design forces to be 25% of the vertical reactions for bridges in SDC A2. The Standard Specifications requires the design forces to only be 20% of the vertical reactions. Table 4.25 shows that the design forces increased by 25% when the Guide Specification was used.

Table 4.25: Oseligee Creek Bridge and Design Force Comparison (SDC A2)

Bent	Vertical Reaction (kips)	Guide Spec Design Force (kips)	Standard Spec Design Force (kips)	Percent Difference
2	621	38.8	31.0	25.0%
3	621	38.8	31.0	25.0%

The next step was to calculate the minimum seat widths. Equation 4.1 and 4.2 were used to calculate the seat widths. As the results in Table 4.26 show, the new minimum seat lengths are 32-42% greater than those required by the Standard Specifications.

Table 4.26: Oseligee Creek Bridge Minimum Support Lengths Comparison (SDC A2)

Bent	New Design Minimum Support Length (in)	Standard Spec Minimum Support Length (in)	Percent Difference
2	14.6	11.0	32.7%
3	16.6	11.7	41.9%

The design of the transverse reinforcement in the columns was completed next. Table 4.27 summarizes the results from the design. The plastic hinge length for Bent 2 was 42 inches, controlled by the diameter of the column. Bent 3 was controlled by the height of the column and determined to be 51.7 inches. The design using the Standard Specifications used a #5 bar as the hoop size, but during the re-design, it was determined that a #4 hoop could be used at the maximum spacing, which inside the plastic hinge zones was controlled by the reinforcement ratio and was 6 inches. The spacing outside of the plastic hinge zone was 12 inches. The minimum area of transverse reinforcement check in the LRFD Specifications was not required because the shear resistance of the concrete was twice as large as the expected shear. Using a #4 hoop at the maximum spacing actually reduced the amount of transverse reinforcement by 11% for each of the bents.

Table 4.27: Oseligee Creek Bridge Design Summary (SDC A2)

	Bent 2		Bent 3	
	Standard Specification	Guide Specification	Standard Specification	Guide Specification
Column Height (in)	215	215	310	310
Hoop Size	#5	#4	#5	#4
Plastic Hinge Length (in)	-	42	-	51.7
PHL Spacing (in)	-	6	-	6
Spacing outside PHL (in)	12	12	12	12
Number of Hoops	18	25	26	36
Area of Steel (in^2)	5.6	5.0	8.1	7.2
Percent Difference	-10.7%		-11.1%	

There were three significant differences between the two designs: increase in design forces, increase in the minimum seat length, and decrease in the amount of transverse reinforcement. Interestingly, the design forces increased by 20% but the amount of transverse reinforcement was actually reduced by 11%, even with the addition of the plastic hinge zone. In the Standard Specifications design, #5 hoops were used. However, in the re-design it was determined #4 hoops could be used with the maximum spacing of the reinforcement. So even though the number of hoops increased, the area of the reinforcement decreased. Since only the seismic load condition was used in the re-design, it is possible that another load case resulting in a higher shear force controlled the original bridge design performed using the Standard Specifications, which required #5 hoops. The minimum seat width also increased, which is the result of the using the recommended minimum seat width equation. Figure 4.11 and Figure 4.12 show the design details from each specification.

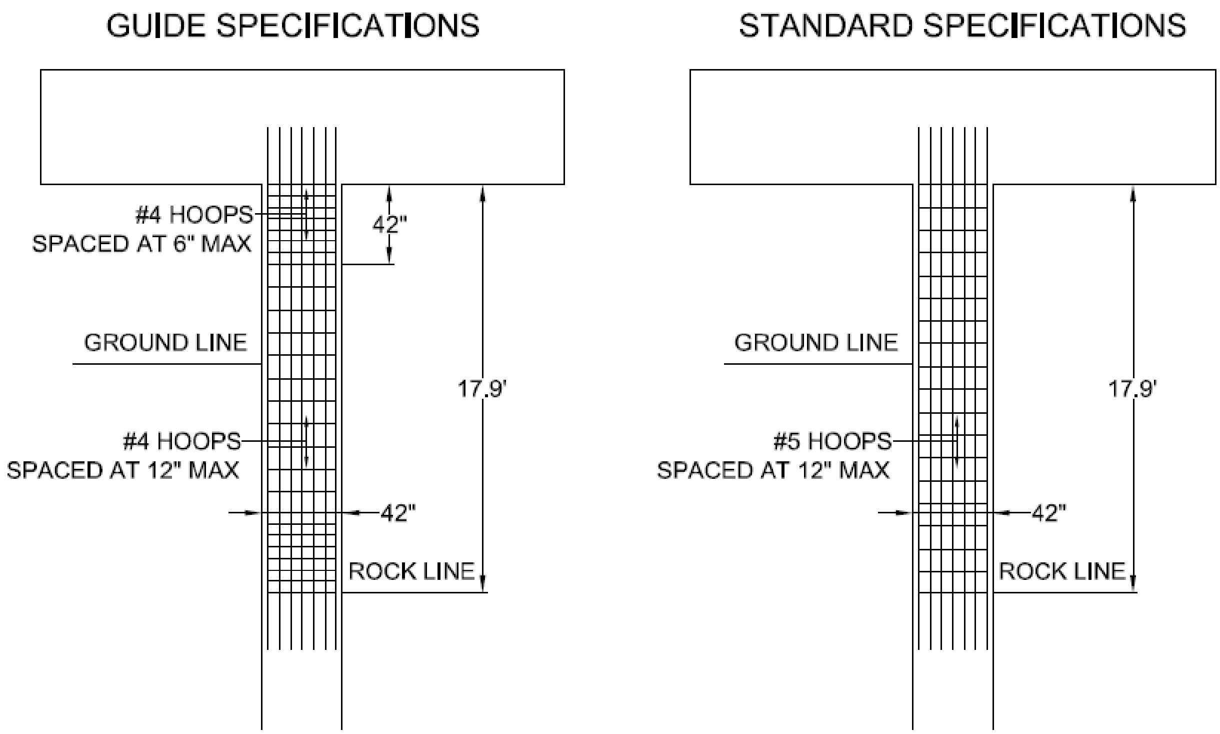

Figure 4.11: Oseligee Creek Bridge Bent 2 Final Design Details (SDC A2)

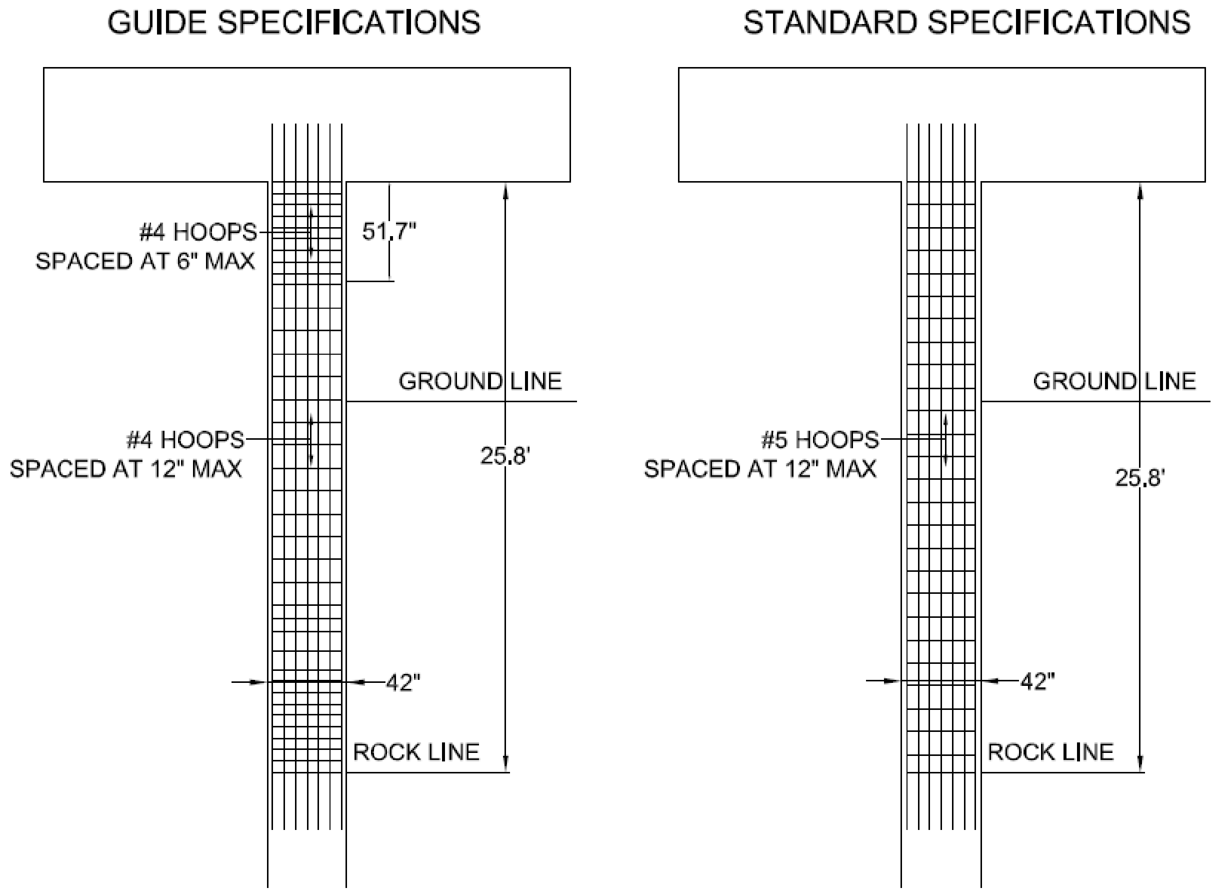

Figure 4.12: Oseligee Creek Bridge Bent 3 Final Design Details (SDC A2)

4.6.5 Summary of Differences in SDC A2

In SDC A2, the differences between the specifications are the increased horizontal design forces, change in the amount of transverse reinforcement, and the increased minimum seat length. The design force increased by 25% because SDC A2 requires the design force to be 25% of the vertical reaction, while the Standard Specification only requires 20%. The amount of transverse reinforcement increased by 40-85% for three of the bridges because the addition of the plastic hinge zone required more ties to satisfy the minimum ratios. The Standard Specification does not require a plastic hinge zone for bridges in SDC A, so the reinforcement is allowed to be spaced further apart. For all the bridges in this study, 12 inch uniform spacing was used for the

Standard Specification design and the Guide Specifications require a maximum spacing of 6 inches in the plastic hinge zone. The one exception came at Oseligee Creek Bridge, where #4 hoops were able to be used instead of #5 hoops as specified in the original design. And even with more hoops required because of the plastic hinge zone, the overall area of transverse reinforcement decreased. It is expected, however, that when the plastic hinge zone is required the amount of transverse reinforcement will increase because a larger number of ties or hoops will be needed. Taller columns will require more reinforcement because of the larger length over which the more tightly spaced ties or hoops will span. The minimum seat width increased in the range of 16-45%, depending on the height of the bridge and the expected spectral acceleration at the bridge site. This is a direct result of using the recommended ATC-49 equation, which gives a better estimation of the displacement of the girder during a seismic event.

There were three changes from SDC A1 to SDC A2. The first was the addition of the plastic hinge zone. This resulted in an increase in the amount of transverse reinforcement as mentioned earlier. The second was a slight increase in the minimum seat width. This is because the spectral accelerations for sites in SDC A2 are higher than sites in SDC A1. Since Equation 4.2 uses the spectral acceleration in the calculation of the minimum seat length, a higher value will give a higher minimum seat length. The third change was an increase in the horizontal design forces. They were required to be 25% of the vertical reaction because, unlike SDC A1 where they were 15% of the vertical reaction, bridges in SDC A2 will not experience low seismic forces ($A_S < 0.05g$).

4.7 Guide Specification Design Process for SDC B

SDC B bridges are expected to experience moderate seismic forces that will cause plastic hinges to form in the columns. These forces cannot be estimated using the simple relationships from SDC A. Additional structural analysis is required to determine the shear forces in the columns at individual bents during a design earthquake. These bents must be designed to resist the shear forces and moments. Minimum detailing is also required in this design category to ensure that the hinges form in the top and bottom of the column in the transverse direction and only in the bottom in the longitudinal direction. The design steps for this SDC are discussed below.

4.7.1 Create a Design Response Spectrum

Bridges in SDC B require a design response spectrum in order to calculate the horizontal design forces. The response spectrum is created from the three spectral accelerations, A_S, S_{D1}, and S_{DS}, calculated when determining the SDC. Article 3.4.1 in the Guide Specification outlines the steps to create the design spectrum. Figure 4.13 illustrates the three-point method found in the article, with T_o and T_S calculated from the three spectral acceleration values.

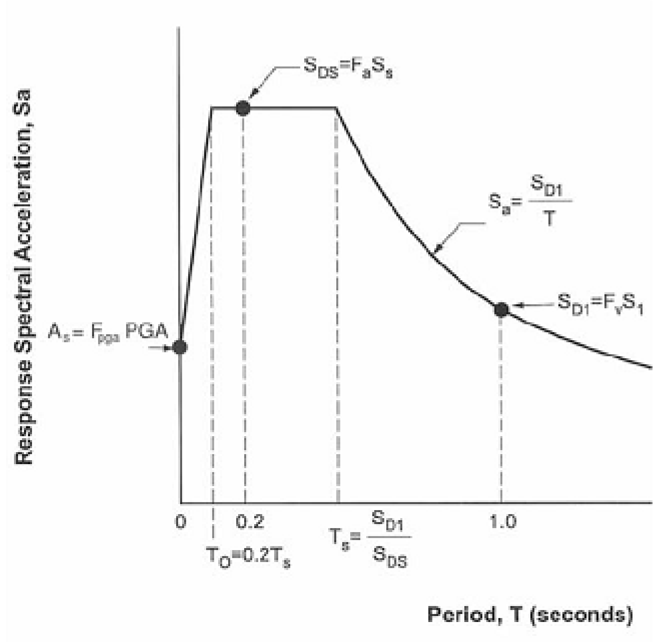

Figure 4.13: Design Response Spectrum, Construction Using Three-Point Method

4.7.2 Create and Analyze Bridge Model

The design forces will be the lesser the elastic forces and the plastic forces. The plastic forces will be determined at a later step. The elastic forces are determined from a bridge model and structural analysis. An equivalent static earthquake loading factor is determined from the structural analysis and design response spectrum. This factor is multiplied by the forces in the model to determine the elastic forces. Each of the five SDC B bridges in the project was modeled using the structural analysis software, CSI Bridge 15 (Computer and Structures Inc., 2012). The three bridges from the previous study, Little Bear Creek, Oseligee Creek, and Scarham Creek, had already been modeled so new models of those bridges were not necessary. Once the model is created, a structural analysis method is performed on the model to determine the displacements. The Guide Specifications allow for the use of either an Equivalent Static Analysis (ESA) or Elastic Dynamic Analysis (EDA). It recommends using the ESA if the bridge is regular and EDA if it is not. Bridge regularity is defined as having fewer than 7 spans, no abrupt or unusual change in geometry and satisfying the requirements in Table 4.28 (Guide Specifications Table 4.2-3). Regular bridges typically respond in their fundamental mode of vibration, and the procedures in an ESA are calibrated for that specific response. For the bridges in this study, ESA methods were used because all of the bridges in this project were “regular” bridges. The EDA provides a much better model for inelastic behavior by better representing inelastic elements and secondary modal responses. However, if it is used, the bridge model should be based on cracked section properties for concrete components and secant stiffness coefficients for foundations and abutments (AASHTO, 2011).

Table 4.28: Regular Bridge Requirements

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle (curved bridge)	30^0	30^0	30^0	30^0	30^0
Maximum span length ratio from span-to-span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span-to-span (excluding abutments)	-	4	4	3	2

Two different ESA options that are acceptable are the uniform load method and single-mode spectral method. The uniform load method is simpler, but it can overestimate the lateral forces in the abutment by as much as 100% (AASHTO, 2011). The uniform load method procedure is described in article C5.4.2 of the Guide Specification. This analysis should be completed in each direction (transverse and longitudinal) because the results will need to be combined. This method places a uniform load of 1 kip/in along the entire length of the bridge and determines the maximum displacement along the bridge length. The maximum displacement is used to calculate the bridge lateral stiffness as seen in Equation 4.8. The period of the bridge is calculated using Equation 4.9. Using the bridge period and response spectrum, the equivalent static earthquake loading is calculated using Equation 4.10.

$$K = \frac{p_o * L}{v_{s,max}} \qquad \text{Equation 4.8}$$

$$T_m = 2 * \pi * \sqrt{\frac{W}{K * g}} \qquad \text{Equation 4.9}$$

$$\rho_e = \frac{S_a * W}{L} \qquad \text{Equation 4.10}$$

Another analysis procedure, the single-mode spectral method, is described in article 4.7.4.3.2b of the LRFD Specifications. This is a more complicated analysis, but can be used to determine more accurate design forces if the results from the uniform load method are too conservative. This analysis should also be done in both the transverse and longitudinal direction, just like the uniform load method. The procedures are similar, but generalized functions are used to describe the displacement instead of a maximum value. The first step is building a bridge model and applying a uniform load of 1 kip/in. The displacement of the bridge is calculated as a function along the entire length of the bridge. A program such as Microsoft Excel can be used to input the displacements along the length of the bridge and generate a function from a graph. Three factors are determined from this displacement function as seen in Equations 4.11, 4.12, and 4.13: α, generalized flexibility, β, generalized participation, and γ, generalized mass.

$$\alpha = \int_0^L v_s(x) * dx \qquad \text{Equation 4.11}$$

$$\beta = \int_0^L w(x) * v_s(x) * dx \qquad \text{Equation 4.12}$$

$$\gamma = \int_0^L w(x) * {v_s}^2(x) * dx \qquad \text{Equation 4.13}$$

The period of the bridge is determined using Equation 4.14 and the equivalent static earthquake load is determined using Equation 4.15. In the LRFD Specifications, the variable C_{sm} is equal to S_a used in the Guide Specifications.

$$T_m = 2 * \pi * \sqrt{\frac{\gamma}{p_o * g * \alpha}} \quad \text{Equation 4.14}$$

$$p_e(x) = \frac{\beta * C_{sm}}{\gamma} * w(x) * v_s(x) \quad \text{Equation 4.15}$$

The equivalent static earthquake loading factor, ρ_e, represents the response of the bridge in the primary mode of vibration. Both the transverse and longitudinal directions have their own factor since the response of the bridge is different in each direction. This factor is used to determine the bridge displacement demand as well as the design forces. The design forces are determined by multiplying the appropriate equivalent static earthquake loading by the forces from the model (either longitudinal or transverse). The displacement demand will be discussed in the next section.

4.7.3 Bridge Capacity vs. Displacement

Article 4.8 in the Guide Specification requires a capacity displacement check to be satisfied for bridges in SDCs B, C, and D. The bridge is required to have a larger displacement capacity than displacement demand at each of the bents. This ensures that the bridge can achieve its inelastic deformation capacity (AASHTO, 2011). Since the bridge is designed to be ductile, it is assumed that the bridge will be able to carry load without failure through the entire demand displacement. But the capacity of the bridge must be greater than the demand for this to be true. Equation 4.16 shown below is used to determine the capacity of the bridge based on the geometry and clear height of the columns for bridges in SDC B. It is only intended for determining displacement capacities of single and multiple reinforced concrete column bridges with clear heights greater than 15 feet, plastic hinging occurring above ground, and where fusing of the superstructure and substructure during a design earthquake is not expected (AASHTO, 2011). The five bridges studied in SDC B were assumed to have the plastic hinging occur either where the column was connected to the foundation or where the foundation reached the rock line, which was below ground, and would violate the requirements for use of the equations. However, the Guide Specifications specifically allow for these equations to be used for bridges with a plastic hinge occurring below ground where the column connects with the foundation. Equation 4.17 requires a factor for column end restraint condition (Λ), which for this project was assumed to be fixed at the top and bottom for movement in the transverse direction and pinned at one end for movement in the longitudinal direction. The Guide Specifications provides guidance if a different end restraint condition exists. If a bridge does not satisfy the requirements for use of Equation 4.16, a Nonlinear Static Procedure or "pushover" analysis, mentioned in Article 4.8.2, is to be used. Also, if any of the bent displacements are greater than the capacities of Equation 4.16, a pushover analysis could be performed on the model using the computer software.

$$\Delta_C^L = 0.12 * H_o * (-1.27 \ln(x) - 0.32) \geq 0.12 * H_o \qquad \text{Equation 4.16}$$

$$x = \frac{\Lambda * B_o}{H_o} \qquad \text{Equation 4.17}$$

The demand displacement of the bridge is determined in each orthogonal direction at each bent from the bridge model and structural analysis. Once the static analysis is performed, the displacements of the bent in each orthogonal direction are recorded and multiplied by the equivalent static earthquake load and short period magnification factor. The short period magnification factor is determined in Article 4.3.3, and corrects the displacement determined from an elastic analysis for bridges in a short period range as determined from the response spectrum. The expected displacement of the bents is determined using Equations 4.18 and 4.19. Article 4.4 in the Guide Specification requires the use of two unique load cases to capture the expected displacement of the bridge based on the uncertainty of earthquake motions and simultaneous earthquake forces in two perpendicular horizontal directions. Equations 4.18 and 4.19 determine the displacement by taking the square root sum of the squares of 100% of the absolute value of seismic displacements in one direction (either longitudinal or transverse) with 30% of the absolute value of seismic displacements in the other orthogonal direction. The larger of the two displacements is taken as the expected displacement of the bent. If all bents in a bridge have a higher capacity than demand, then detailing of the reinforcement in each column can begin. If a single bent does not satisfy the displacement demands, a pushover analysis can be performed, the capacity equations from SDC C can be used, or the dynamic characteristics of the bridge can be modified. If these equations for SDC C are used, the bridge must be designed according to SDC C requirements. This project did not deal with this method, instead using a pushover analysis if a bent did not meet the capacity requirements.

$$\Delta D = \sqrt{(1 * \Delta D_{LONG})^2 + (0.3 * \Delta D_{TRAN})^2} \qquad \text{Equation 4.18}$$

$$\Delta D = \sqrt{(1 * \Delta D_{TRAN})^2 + (0.3 * \Delta D_{LONG})^2} \qquad \text{Equation 4.19}$$

4.7.4 Column Seismic Detailing

Once the capacity of a bridge bent is confirmed, the reinforcement for each column can be detailed. For SDC B, there is minimum detailing that must be met within the plastic hinge region as well as detailing for reinforcement outside of the plastic hinge region. The first step is to determine the plastic hinge length (PHL) for each individual column.

4.7.4.1 Plastic Hinge Length

The plastic hinge length (PHL) is the assumed length of the column where the plastic hinge will form and is designed to be at the top of the column and the bottom of the column, where the column meets the foundation, although the location of the plastic hinge at the bottom can vary depending on the soil and foundation type. For the bridges in SDC B, the plastic hinge was assumed to form at the top of the column and at the bottom of the column, at the connection with the foundation element, in the transverse direction, but only at the bottom of the column in the longitudinal direction. The minimum detailing requirements increase the amount of shear reinforcement, which helps confine the concrete and prevent buckling of the longitudinal reinforcement, as well as give the section more shear resistance, decreasing the possibility of a brittle failure that will not allow the column to dissipate energy. Article 4.11.7 in the Guide Specification provides the PHL to be the largest of three lengths (AASHTO, 2011):

- 1.5 times the largest cross-sectional dimension in the direction of bending
- The region of the column where the moment demand exceeds 75% of the maximum plastic moment
- The analytical plastic hinge length, L_p

The largest cross-sectional dimension will be either the diameter of a circular column or the largest width of a rectangular column. The maximum plastic moment is determined by a moment-axial force interaction diagram. For this project, the software program spColumn was used (StructurePoint, 2012). The dimensions of the column and the reinforcement layout are input into the program, and the maximum moment is determined from the resulting moment interaction diagram. Once the maximum moment is determined, the moment diagram from the computer analysis software can be used to determine the length of the column where the moment exceeds 75% of the plastic moment. The analytical plastic hinge length is determined in Article 4.11.6 using Equation 4.7. This equation is specifically for reinforced concrete columns framing into a footing, integral bent cap, oversized shaft, and cased shaft, which meets the criteria for this project.

$$L_p = 0.08 * L + 0.15 * f_{ye} * d_{bl} \geq 0.3 * f_{ye} * d_{bl} \qquad \text{Equation 4.7}$$

In most cases, the PHL is controlled by the 1.5 times the gross cross-sectional dimension. This can result in a large PHL for large columns and since the PHL is at the top and bottom of the column, the entire column could be considered to be within the plastic hinge. This makes it difficult to meet the splicing requirements found in Article 8.8.3 for the longitudinal column reinforcement. The splicing is required to be outside of the plastic hinge length. Failure to do so could lead to undesirable seismic performance because the splice would be subject to plastic forces and deformations, which could lead to a reduced effective plastic hinge length and severe local curvature demand (AASHTO, 2011). While this article only applies to SDC C and D, the commentary recommends that they also be applied to SDC B.

Article 8.8.9 in the Guide Specifications gives an alternative PHL that can be used in SDC B that is calculated using Article 5.10.11.4.1e of the LRFD Specification. These requirements are easier to determine and do not require any computer software. The maximum of three limits is taken as the PHL (AASHTO, 2007):

- The largest cross-sectional dimension
- One-sixth the clear height of the column
- 18 inches

The largest cross sectional dimension will be either the diameter of a circular column or the largest width of a rectangular column. The clear height of the column depends on the foundation type and geometry. The foundations from the three bridges in the previous project were drilled shafts. For two of the bridges, Little Bear Creek Bridge and Scarham Creek Bridge, the drilled shafts were six inches wider than the columns, and the plastic hinge was assumed to form at the transition between the two. The clear height was taken from the bottom of the bent cap to this transition point. However, for Oseligee Creek Bridge, the drilled shaft was the same size as the columns, and it was unknown if the plastic hinge would form at the transition point because there was no change in stiffness between the two members. Therefore, it was assumed the plastic hinge would form at the rock line because below the drilled shaft would be unable to displace below the rock line. The clear height of these columns was measured from the bottom of the bent cap to the rock line. For the other two bridges, driven piles were used as the foundations. It was assumed the plastic hinge would form at the column to pile cap connection, and the column height was taken from the bottom of the pier cap to the top of the pile cap. It is important to understand how the column and foundation will interact in order to determine where the plastic hinge is likely to form. Using the plastic hinge length from the LRFD Specifications will typically result in a smaller PHL than that found in the Guide Specifications. This will allow for a greater length of column for splicing and fewer confinement ties. For the bridges in SDC B, both values will be checked.

The LRFD Specifications, in Article 5.10.11.4.3, discuss an extension of the plastic hinge length into the cap beam or the foundation (pile cap or drilled shaft). The Guide Specifications do not specifically mention this extension length, but since it is mentioned in the same article as the LRFD plastic hinge length, it will be considered appropriate for use. This extension is only required in SDC C and D, but it is recommended in SDC B. The extension length is an extra length over which the ties from the plastic hinge zone extend. The spacing of these ties is equal to the spacing from the plastic hinge zone. This is an added measure to protect the elements adjacent to the plastic hinge. Article 5.10.11.4.1e in the LRFD Specifications requires the extension length to be the maximum of the following:

- One-half of the column diameter

- 15 inches

The extension length will be calculated and shown in the details for each bridge in SDC B, but it should be noted that the inclusion of this length in the design is at the Owner's discretion and not required.

4.7.4.2 Transverse Reinforcement inside Plastic Hinge Zone

Once the plastic hinge length is determined, the size and spacing of the transverse reinforcement within the plastic hinge length can be determined. Unlike SDC A2, the flexure and shear demands in the column are used, along with the minimum ratios, to determine the spacing. The column will be designed for the maximum expected forces in the plastic hinge, and the minimum ratios will be checked. In order to determine the design forces, Article 8.3.2 states that for SDC B "the design forces shall be the lesser of the forces resulting from the overstrength plastic hinging moment capacity or unreduced elastic seismic forces in columns or pier walls" (AASHTO, 2011). The elastic seismic forces come directly from the software analysis, multiplied by the equivalent static earthquake load factor. For SDC B, the plastic moment capacity comes from the moment-axial force interaction diagram computed earlier. The Guide Specifications allow for the use of the nominal plastic moment from the interaction diagram instead of the idealized capacity because the inelastic demands should be small (AASHTO, 2011). This plastic moment must still be multiplied by an overstrength factor to account for material strength variations between the column and adjacent members. A shear force is calculated from this overstrength plastic moment, and the lesser of the plastic shear force and elastic shear force is used to design the transverse reinforcement. Article 8.6 recommends designing for the plastic force whenever possible, but does not require it. In this project, the lesser of the elastic forces and plastic forces will be used in the design.

Once the design forces have been determined, the concrete shear capacity and steel reinforcement shear capacities are determined according to Articles 8.6.2 through 8.6.4. These equations are based on the degradation of the concrete shear capacity within the plastic hinge region (AASHTO, 2011). To determine these capacities, column dimensions and reinforcement size and spacing must be known. A computer based design sheet can be used to easily allow for an iterative process. The Guide Specifications does give some guidance to the size of ties and the spacing of ties. Article 8.8.9 requires at least #4 bars to be used for transverse reinforcement if #9 bars or smaller are used as longitudinal reinforcement, and at least #5 bars for transverse reinforcement if #10 bars or greater are used as longitudinal reinforcement. The article also has maximum spacing requirements. These requirements indicate that the maximum spacing of transverse reinforcement cannot be greater than the smallest of the following:

- One-fifth the least dimension of the cross-section for columns
- Six times the nominal diameter of the longitudinal reinforcement
- 6 inches

If the longitudinal reinforcement is at least a #9 bar and the column size is at least 30 inches, as all of the bridges in this study are, then the 6 inch maximum spacing controls. Once the concrete and shear capacities are determined to be greater than the demand, the minimum ratio of transverse reinforcement in Article 8.6.5 must be checked. It requires a minimum ratio of transverse reinforcement, as calculated in Article 8.6.2, of greater than or equal to 0.003 for spirals in circular columns and greater than or equal to 0.002 for rectangular columns. Once this minimum requirement has been satisfied and the transverse reinforcement is determined to provide sufficient capacity, the detailing within the PHL is finished.

4.7.4.3 Transverse Reinforcement outside Plastic Hinge Zone

The detailing requirements inside the plastic hinge region are specifically outlined in the Guide Specifications. However, the equations for determining concrete capacity used in the Guide Specification are not meant to be used outside of the plastic hinge region because they include the expected concrete behavior as the hinge region becomes plastic. Therefore, the LRFD Specifications are used to design the shear reinforcement outside of the plastic hinge region. The shear force to be used in the design is the same shear force calculated in the static analysis. Article 5.8.3.3 in the LRFD Specifications is used to determine the capacity of the column. Since these requirements are different than those used in the Guide Specifications they should not be used to calculate the concrete capacity within the plastic hinge zone. Equations 4.3 and 4.4 are used determine the shear capacity of the transverse reinforcement and the concrete. These equations are specific to the bridges studied and should be checked against the other methods for calculating shear capacity in the LRFD Specifications. Once the design is satisfied for strength, three spacing requirements are checked. The reinforcement size is already known from the plastic hinge zone calculations, but the spacing of reinforcement is determined from the capacity equations and the limit checks. The first requirement can be found in Article 5.8.2.5 of the LRFD Specifications and is a minimum amount of transverse reinforcement. It is only required when the factored load is greater than half of the factored resistance by the concrete section and prestressing steel (if present). It is intended to provide reinforcement in regions where there is a significant chance of diagonal cracking (AASHTO, 2009). If it is determined that this minimum reinforcement is required, then Equation 4.5 is used to determine the minimum area of transverse reinforcement. This equation in the LRFD Specifications is different than the equation found in the Standard Specifications. It results in a larger minimum area of transverse steel in the column. Article 8.19.1.2 of the Standard Specification uses Equation 4.6 to find the minimum area. The value is a constant, 0.05 ksi. Article 5.8.2.5 in the LRFD Specifications uses Equation 4.5, and the coefficient is a function of the compressive strength of concrete. For 4,000 psi concrete, the value is 0.0632 ksi. The difference between the values shows that the LRFD Specifications will result in a higher minimum area of reinforcement compared to the Standard Specifications.

The second check is the maximum spacing check found in LRFD article 5.8.2.7. This check addresses the need for tighter spacing if the section experiences very high shear stress. Most sections will not experience very high shear stress, so this requirement will not typically control the design. The final check is an ALDOT standard maximum spacing of 12 inches. In the event that the column is not required to meet the minimum area of transverse reinforcement requirement, this 12 inch maximum spacing will likely control.

$$V_c = 0.0316 * \beta * \sqrt{f_c'} * b_v * d_v \qquad \text{Equation 4.3}$$

$$V_s = \frac{A_v * f_y * d_v * \cot(\theta)}{s} \qquad \text{Equation 4.4}$$

$$A_{v,min} = 0.0316 * \sqrt{f_c'} * \frac{b_v * s}{f_y} \qquad \text{Equation 4.5}$$

$$A_{v,min} = 0.05 * \frac{b * s}{f_y} \qquad \text{Equation 4.6}$$

Another factor that would affect the spacing of the reinforcement would be the requirement of cross-ties. LRFD Article 5.10.6.3 requires the use of cross ties in rectangular columns to ensure that no longitudinal bar is more than 2 feet from a restrained bar. However, for all of the bridges in this study, no columns were large enough for this requirement to be necessary. Therefore, this requirement did not control the design.

4.7.4.4 Longitudinal Reinforcement

The longitudinal reinforcement is designed using the moment-axial force interaction diagrams for the columns. For this project, the longitudinal reinforcement in the original designs was used in the new designs. This reinforcement was checked using the moment-axial force interaction diagrams to determine if the column capacity is greater than the load demand. The load demands are calculated from the bridge model and structural analysis. Multiple load cases need to be analyzed, as discussed in Article 4.4 of the Guide Specifications. The axial load is determined by taking the largest axial force from the dead load and adding it to the largest axial force from the combination of earthquake loads multiplied by the equivalent static earthquake load. In order to determine the maximum moment, the maximum and minimum dead load cases should be considered because the axial load can affect the moment capacity. The most severe axial loads and moments should be input into the spColumn software to determine if the column capacity is sufficient to resist the loads.

Once the capacity of the columns is ensured, a minimum and maximum ratio check is to be performed. Articles 8.8.1 and 8.8.2 detail these checks. The maximum check, found in Article 8.8.1, requires the area of longitudinal reinforcement to be equal to or less than 4% of the gross area of the column. Limiting the amount of longitudinal reinforcement increases the ductility of the column. The minimum check in Article 8.8.2 requires that the longitudinal reinforcement area be greater than or equal to 0.7% of the gross area. This check is done to “avoid a sizable difference between the flexural cracking and yield moments” (AASHTO, 2011). Once these checks are satisfied, the longitudinal reinforcement design is finished.

4.8 SDC B Design Examples

The design procedure in the Guide Specifications for SDC B was used to redesign five bridges previously designed under the Standard Specifications. These bridges were supplied by ALDOT and are conventional bridges in the “other” category as described earlier, making them applicable to the Guide Specifications. For each bridge, design sheets were created with references to specific articles in the Guide Specifications or LRFD Specifications. Since the purpose of these redesigns is to determine if a standard set of drawings and details can be identified for these bridges, design data is established for each bent of a bridge. This information was summarized for each bridge. Under the previous study by Coulston and Marshall (2011), three bridges in SDC B were redesigned using both the Guide Specifications and the LRFD Specifications. These three bridges are included in the five bridges to be redesigned in this project so that all the bridges will have been designed using the most recent edition of the Guide Specifications. The two new bridges to be redesigned are Bent Creek Road Bridge over I-85 in Lee County and the Bridge over Norfolk Southern Railroad in Etowah County. The three bridges previously designed include Bridge over Little Bear Creek in Franklin County, Oseligee Creek Bridge in Chambers County, and Scarham Creek Bridge in Marshall County.

The superstructure-to-substructure connection must be investigated for bridges in this SDC. As discussed in chapter 3, the current connection is to be used for all bridges, and any longitudinal forces will be dealt with by allowing the girders to move and “ride out” the earthquake without unseating. This is accomplished by providing greater seat widths than provided by the equations in the Guide Specifications. However, in the transverse direction, the connection needs to be checked to ensure it can transfer the loads into the substructure. Article 4.11 in the Guide Specifications requires those elements "not participating as part of the primary energy-dissipating system" to be capacity protected, meaning they must be designed for the maximum expected forces (AASHTO, 2011). These forces are determined from a pushover analysis. The clip angles and anchor bolts from this connection were designed for each bridge based on these forces. The results from the pushover analysis, as well as the design of the transverse connection, will be discussed for each bridge.

4.8.1 Bent Creek Road over I-85

This bridge was already designed in the SDC A2 category and will be re-designed in SDC B to compare the two designs determine if it is more economical to design the bridge as a SDC B bridge. Bent Creek Road bridge is a five-lane bridge that crosses over Interstate 85 in Lee County. It is a two span bridge with equal span lengths of 135 feet comprised of 15 modified BT-54 girders spaced approximately 5.33 feet apart and supports a 6 inch concrete deck that is 80.75 feet wide. The only bridge pier is 79' x 4' x 4.5' and supported by five rectangular square columns 3.5 feet in width. The columns are reinforced longitudinally with 12 #11 bars and transversely with #4 ties uniformly spaced at 12 inches from the bottom of the bent to the top of the pile cap foundation. The average clear height of the columns is 20.1 feet. The bridge is supported on driven piles. The pile cap is 8.5' x 8' x 4.5' and each pile cap is supported by 9 HP 12x52 steel piles. Figure 4.14 shows a 3D view of the bridge as modeled in SAP2000. All design calculations can be seen in Appendix H and the moment-axial force interaction diagrams for the columns can be seen in Appendix I.

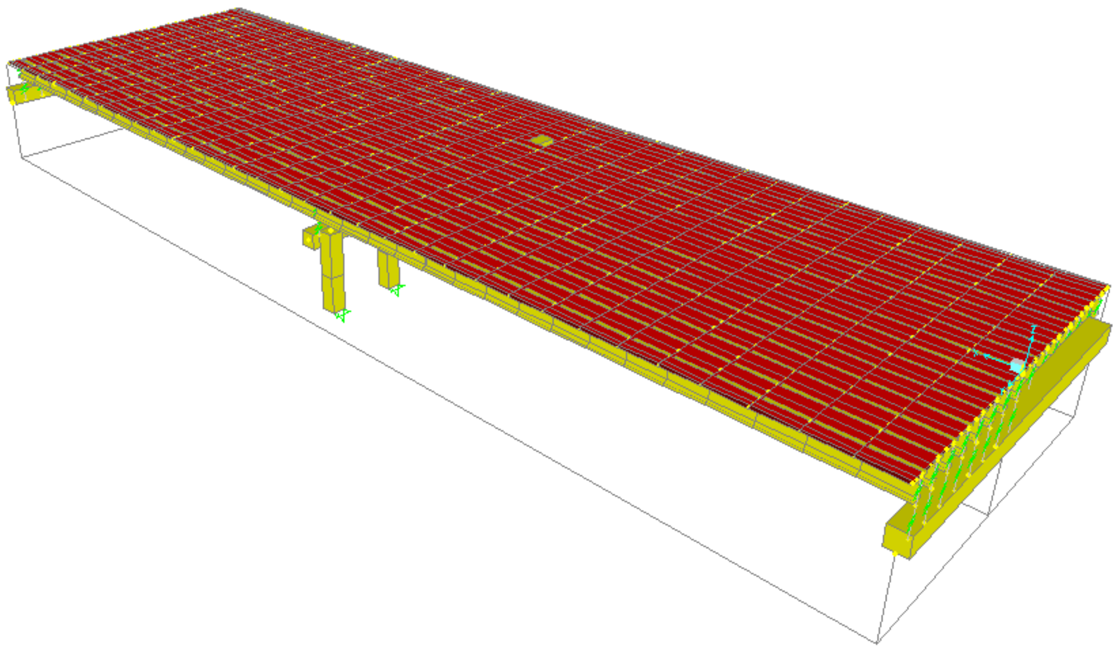

Figure 4.14: SAP2000 3D Model of Bent Creek Road Bridge

The first step was to determine if the bridge capacity was greater than the demand. Table 4.29 shows the results from this analysis. The bridge model was used to analyze the bridge and determine the displacements of the bents. The uniform load method was used to determine the equivalent static earthquake loading factor, which, along with the short period magnification factor, was multiplied by the bent displacements in each direction to determine the expected displacement at the bent. The capacity of the bridge bent was determined in each direction using Equations 4.16 and 4.17. The largest displacement from the square root sum of the squares (SRSS) of the two orthogonal displacements was compared to the smallest capacity. As the table shows, the capacity was greater than demand, so this bent passed the demand/capacity check and could be designed.

Table 4.29: Analysis Results for Bent Creek Road Bent 2

	Transverse	Longitudinal
Displacement at Bent from Model	3.012"	0.052"
Expected Displacement at Bent	0.862"	0.078"
Bent Capacity	2.448"	4.567"
SRSS Displacement	0.863"	

A pushover analysis of the bridge was also performed. The design force for the connection was determined using the expected transverse displacement of the bent calculated in the structural analysis as described above. Figure 4.15 shows that, for this bridge, the base shear was 620 kips and since the bridge had 15 girder connections, the connection design force was 41.3 kips. This force was used to design the clip angles and anchor bolts, which will be discussed below.

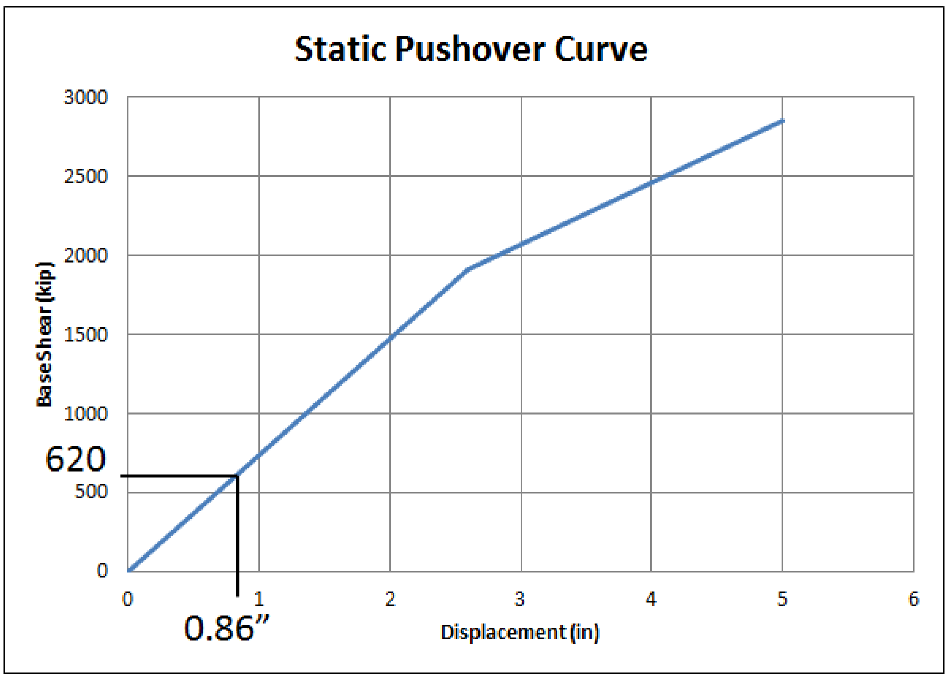

Figure 4.15: Static Pushover Curve for Bent Creek Road Bridge Bent 2

Using the design force from the pushover analysis, the transverse clip angles and anchor bolts were designed. The LRFD Specifications and AISC Specifications were used to design them. The specific articles are referenced in the design which can be seen in the appendices. The clip angle size was chosen from the original connection and block shear, tension and shear capacities of the angles were checked against the design force. It was assumed that one of the angles would have to resist the entire design force because the other angle would not be able to transfer a tensile force. Table 4.30 shows the capacity of the clip angle for these three limit states. For this design force, the clip angle was acceptable. The anchor bolt was designed for shear, bearing, tension, and combined tension and shear. Like the clip angle, only one anchor bolt was assumed to resist the load since only one clip angle would be able to transfer load. It was determined that an ASTM A307 Class C bolt with a diameter of 1.75 inches would be required for this connection.

Table 4.30: Capacity of the Steel Clip Angle

Limit State	Capacity (kips)
Block Shear	156
Tension	118
Shear	130

Once the capacity check was satisfied and the connection design completed, the minimum seat widths were calculated. The ATC-49 equation, Equation 4.2, was used to calculate the minimum seat widths and Equation 4.1 was used to find the Standard Specifications minimum seat width. As recommended in chapter 3, S_{D1} was taken to be 0.30 for all of the bridges in SDC B. This resulted in a 70% increase in the minimum seat length required, as seen in Table 4.31.

Table 4.31: Bent Creek Road Bridge Seat Width Specification Comparison (SDC B)

Specification	Standard	New Design
Minimum Seat Width (in)	12.3	19.8
Percent Difference	70.0%	

The column design was completed next. The longitudinal reinforcement satisfied both checks, and the column capacity was acceptable. Table 4.32 shows the results from this design analysis. The plastic hinge length from the LRFD Specifications was used, because it resulted in a 50% decrease of the plastic hinge length calculated using the Guide Specifications. The length was calculated to be 42 inches, with an extension length of 21 inches. The column length outside of the plastic hinge region that could be used for splicing was 156 inches or about 13 feet. Similar to SDC A2, the new design requires 95% more ties than the original design under the Standard Specifications. Figure 4.16 shows the differences between the two specifications using the design details.

Table 4.32: Bent Creek Road Bent 2 Design Results (SDC B)

	Bent 2	
Plastic Hinge Length (in)	42	
Extension Length (in)	21	
Available Splice Length (in)	156	
Tie Size	#4	
Specification	Standard	Guide
Spacing within PHL (in)	-	4
Spacing outside PHL (in)	12	9
Total Number of Ties	20	37
Area of Ties (in^2)	4	7.8
Percent Difference	95.0%	

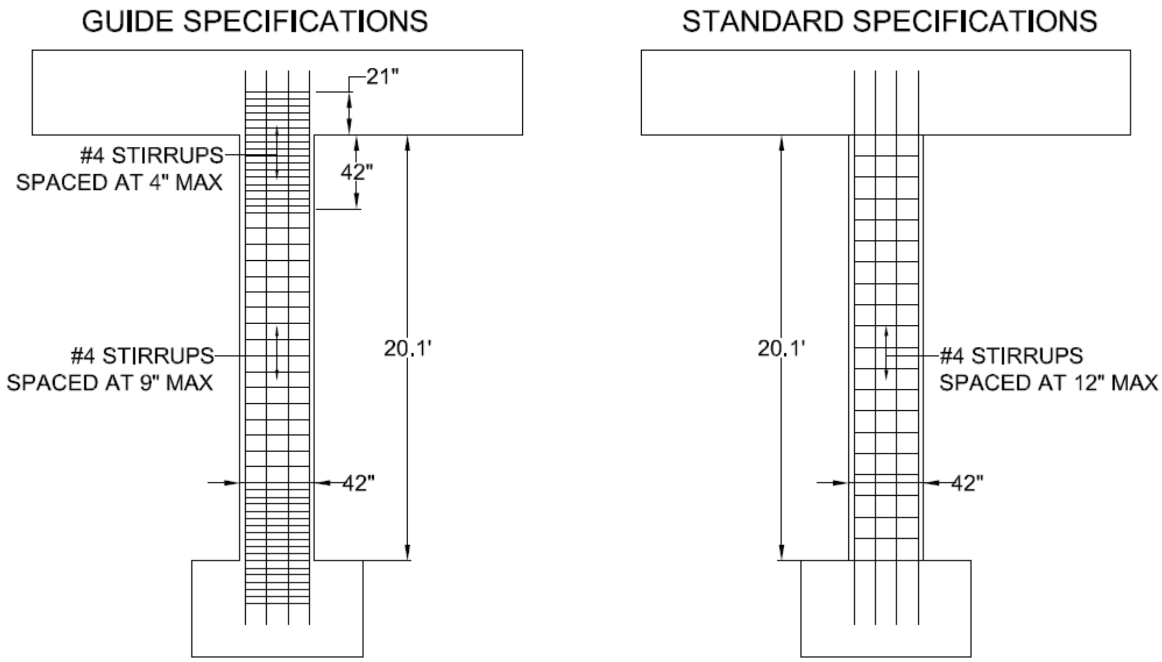

Figure 4.16: Bent Creek Road Bridge Bent 2 Final Design Details (SDC B)

When compared with the same design in SDC A2, the only differences are the horizontal design force and minimum seat width. With the exception of the extension length, which is not required for SDC B and therefore not included in the reinforcement calculation, the amount of transverse reinforcement was the same in both categories. The horizontal design force from SDC A2 that was compared did not include the live load because it resulted in the smaller force. As Table 4.33 shows, the horizontal design force was determined to be 4.4% less for SDC B than for SDC A2. For this bridge, it would be more economical to perform a structural analysis to determine the horizontal design forces for the connection. The minimum seat width is almost 21% greater in SDC B than in SDC A2, because the new equation increases the seat width for higher SDC because of the increase in expected spectral acceleration. The amount of transverse reinforcement did not change, because both categories satisfy the same minimum detailing requirements.

Table 4.33: Bent Creek Road SDC A2 and SDC B Design Comparison

	SDC A2	SDC B
Design Force (kip)	43.2	41.3
Percent Difference	-4.4%	
Minimum Seat Width (in)	16.4	19.8
Percent Difference	20.7%	
Number of Ties	37	37
Percent Difference	0.0%	

4.8.2 I-59 Bridge over Norfolk Southern Railroad

This bridge was the second SDC A2 bridge redesigned as an SDC B bridge. The designs will be compared to determine if it is more economical to design the bridge as a SDC B bridge. The southbound I-59 bridge in Etowah County is a two lane bridge that crosses over a Norfolk Southern railroad line and a state highway. It is a two span bridge with unequal span lengths of 125 feet and 140 feet. Nine modified BT-54 girders support a 6 inch concrete deck that is 46.75 feet wide. The only bridge pier is 53’ x 4.5’ x 4’ and supported by three square columns 3.5 feet in width. The columns are reinforced longitudinally with 12 #11 bars and transversely with #4 ties uniformly spaced at 12 inches from the bottom of the bent to the top of the pile cap foundation. The average clear height of the columns is 25.25 feet. The bridge is supported on driven piles. The pile cap is 8.5’ x 8’ x 4.5’ and each pile cap is supported by 7 HP 12x53 steel piles. Figure 4.17 shows the 3D model of the bridge used in the structural analysis. All design calculations can be found in Appendix J and the moment-axial force interaction diagrams can be seen in Appendix K.

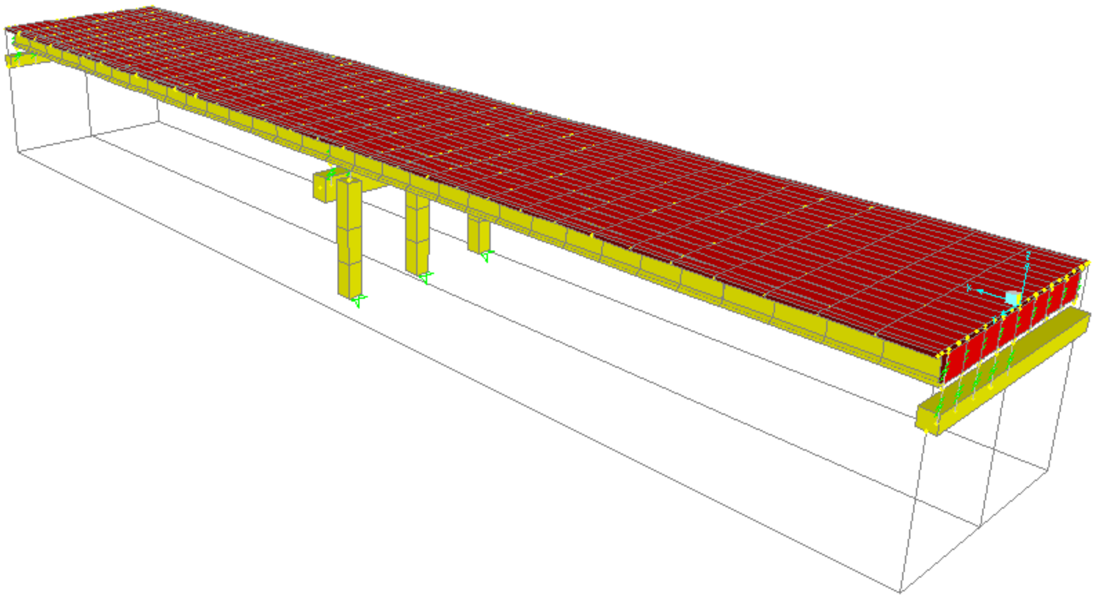

Figure 4.17: SAP2000 3D Model of Bridge over Norfolk Southern RR

The capacity of the bridge was checked first. All results from the capacity analysis can be found in Table 4.34. The bridge model was used to determine the demand displacements at each of the bents, as well as the equivalent static earthquake loading following the uniform load method. The equivalent static earthquake loading factor in each direction was multiplied by the short period magnification factor to determine the expected displacement at the bent. The capacity of the bridge bent was determined in each direction using Equations 4.16 and 4.17. The largest displacement from the square root sum of the squares (SRSS) of the two orthogonal displacements was compared to the smallest capacity. As the table shows, the capacity was greater than demand, so this bent passed the demand/capacity check and could be designed.

Table 4.34: Analysis Results for Bridge over Norfolk Southern Railroad Bent 2

	Transverse	Longitudinal
Displacement at Bent from Model	5.601"	0.042"
Expected Displacement at Bent	0.788"	0.241"
Bent Capacity	3.967"	6.634"
SRSS Displacement	0.788"	

A pushover analysis of the bridge was performed next. The static pushover curve can be seen in Figure 4.18. The design force for the connection was determined using the expected transverse displacement of the bent calculated in the structural analysis as described above. For this bridge, the base shear was 200 kips, and since the bridge had 9 girder connections, the connection design force was 22.2 kips. This force was used to design the clip angles and anchor bolts, which will be discussed below.

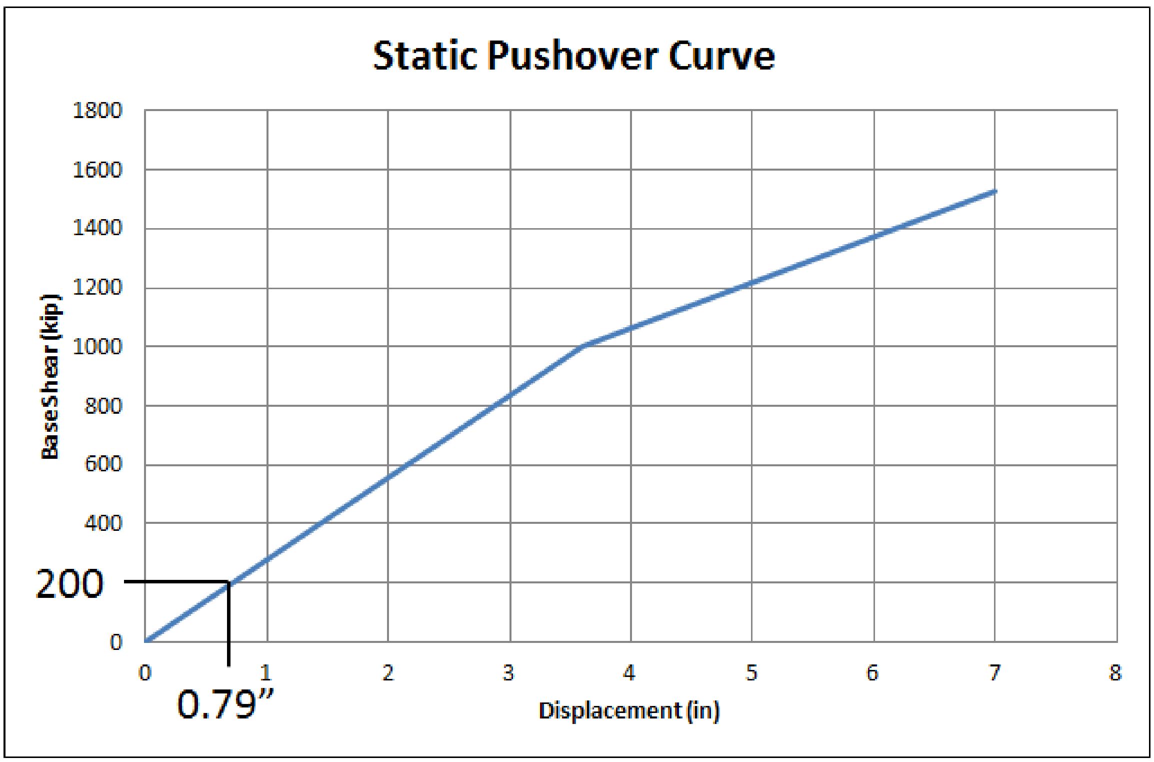

Figure 4.18: Static Pushover Curve for the Bridge over Norfolk Southern Railroad Bent 2

The design force from the pushover analysis was used to design the clip angles and anchor bolts. Since the clip angles were adequate for a force of 41.3 kips, used in the Bent Creek Road Bridge above, they would also be adequate for the force of 22.2 kips. ASTM A307 Class C anchor bolts were used in the design, and it was determined they would have to be 1.375 inches in diameter to resist the connection. This is smaller than the diameter determined above, and it can be seen that the anchor bolts should be designed for each bridge.

The minimum seat width was calculated once the capacity check and connection design were completed. The comparison can be seen in Table 4.35. The seat width is increased by 68% using the new equation.

Table 4.35: Norfolk Southern Bridge Seat Width Specification Comparison (SDC B)

Specification	Standard	New Design
Minimum Seat Width (in)	12.7	21.3
Percent Difference	67.7%	

The column design was completed next. The longitudinal reinforcement was sufficient for the expected loading, and both longitudinal checks were satisfied. Table 4.36 shows the final results from this design. The plastic hinge length was calculated to be 50.5 inches using the LRFD Specifications. For these columns, one-sixth of the column height controlled the hinge length instead of the width of the column. However, the length was still almost 25% less than that calculated by the Guide Specifications. The column length outside of the plastic hinge zone available for splicing was approximately 202 inches or 16.5 feet. The extension length was 21 inches, controlled by one-half of the column width. When compared to the original design using the Standard Specifications, the main difference is the increase in the amount of transverse reinforcement. Like SDC A2, more ties are required because of the plastic hinge zone and the stricter minimum area requirements. The seat width is also required to be 68% larger than the

Standard Specification seat width. Figure 4.19 compares the design details between the two specifications.

Table 4.36: Bridge over Norfolk Southern RR Bent 2 Design Results

	Bent 2	
Plastic Hinge Length (in)	50.5	
Extension Length (in)	21	
Available Splice Length (in)	202	
Specification	Standard	Guide
Spacing within PHL (in)	-	4
Spacing outside PHL (in)	12	9
Total Number of Ties	26	48
Area of Ties (in^2)	5.2	9.6
Percent Difference	84.6%	

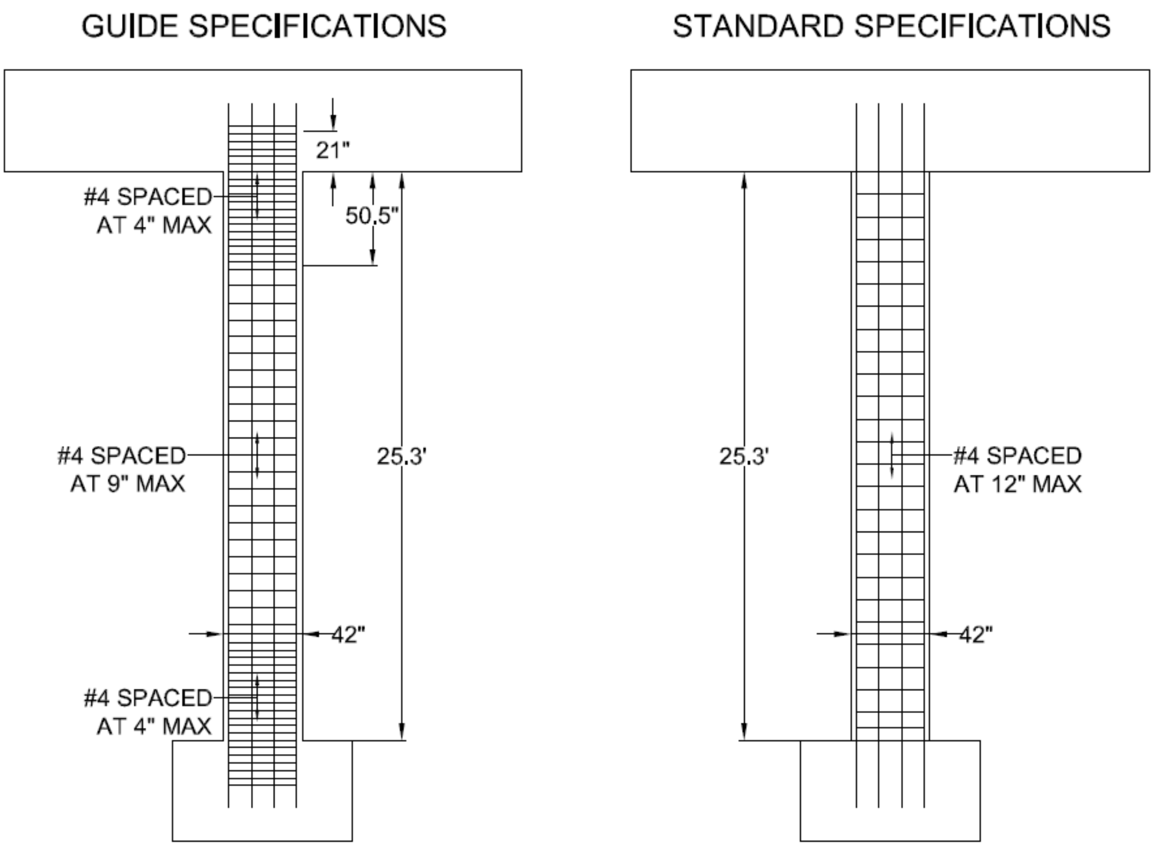

Figure 4.19: Bridge over Norfolk Southern Railroad Final Design Details (SDC B)

Table 4.37 compares the designs in SDC A2 and SDC B, and it can be seen that the horizontal design force for the connection is 50% smaller than the design force in SDC A2 that does not use the live load factor. For this bridge, it is more economical to perform a structural analysis to determine the horizontal design forces. The minimum seat width is larger in the SDC B design because the spectral acceleration value is higher in SDC B than in SDC A2. Even though the SDC B design force is half of the SDC A design force, the amount of transverse reinforcement does not change because the same minimum requirements still apply.

Table 4.37: Bridge over Norfolk Southern Railroad SDC A2 and SDC B Comparison

	SDC A2	**SDC B**
Design Force (kip)	45.5	22.2
Percent Difference	-51.2%	
Minimum Seat Width (in)	18.4	21.3
Percent Difference	15.8%	
Number of Ties	48	48
Percent Difference	0.0%	

4.8.3 Oseligee Creek Bridge

Oseligee Creek Bridge is the final bridge that was designed in both SDC A2 and SDC B. The SDC B design will be compared to the Standard Specification design to show the differences between the Standard Specification and Guide Specification in SDC B and it will be compared to the Guide Specification SDC A2 design to determine if it is more economical to design the bridge as SDC B instead of SDC A2. This bridge carries two lanes of County Road 1289 over Oseligee Creek in Chambers County. It has three spans of equal lengths of 80 feet. The 7 inch concrete deck is supported by 4 Type III girders. The two bridge piers are 30’ x 4’ x 5’ and supported by two circular columns 3.5 feet in diameter with 3 inches of concrete cover. The columns are reinforced longitudinally with 12 #11 bars and transversely with #5 hoops uniformly spaced at 12 inches from the bottom of the pier cap to the rock line. The average clear height of Bent 2 is 17.93 feet and 25.83 feet for Bent 3. All columns are supported on drilled shafts 3.5 feet in diameter with concrete cover of 3 inches. Because no clear transition between the drilled shaft and the column existed, it was unknown where the plastic hinge would form. It was assumed that the soil would not provide enough lateral reinforcement alone to force the plastic hinge to form at the ground line, so the plastic hinge was designed to form at the rock line. For this reason, the height of the columns used for the plastic hinge calculation was assumed to be from the bottom of the bent cap to the rock line. Figure 4.20 shows the 3D model of the bridge used in the structural analysis. All the design calculations can be seen in Appendix L and the moment-axial force interaction diagrams can be seen in Appendix M.

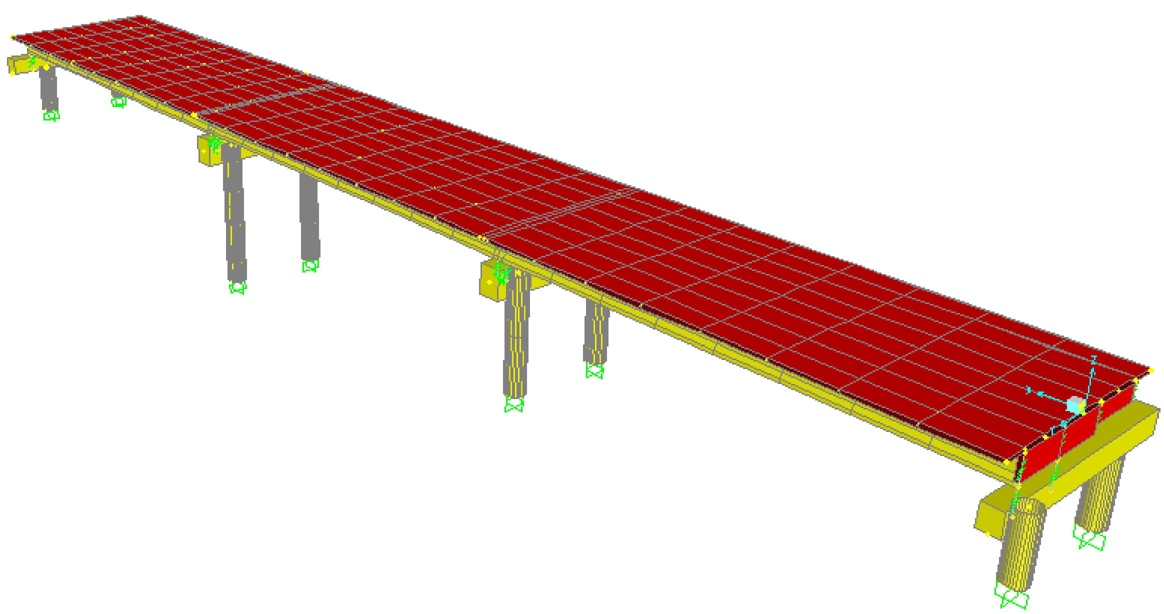

Figure 4.20: SAP2000 3D Model of Oseligee Creek Bridge

The first step was to determine the demand displacements and compare them to the bridge capacity. The SAP2000 bridge model and uniform load method were used to determine the displacement at each bent. Table 4.38 lists the results from the capacity analysis. The expected displacement was determined by multiplying the bent displacement from the model by the equivalent static earthquake load and short period magnification factor. The largest displacement from the square root sum of the squares of the two orthogonal displacements was compared to the smallest capacity. As the table shows, the capacity was greater than demand for both bents, so this bridge satisfied the capacity check.

Table 4.38: Displacement Results for Oseligee Creek Bridge

	Bent 2		**Bent 3**	
	Transverse	**Longitudinal**	**Transverse**	**Longitudinal**
Displacement at Bent from Model	2.081"	1.346"	2.90"	1.437"
Expected Displacement at Bent	0.446”	0.359”	0.621”	0.383”
Bent Capacity	1.833”	3.777”	4.149”	6.878”
SRSS Displacement	0.458”		0.632”"	

Once the capacity check was satisfied, a pushover analysis of the bridge was performed to determine the sequence of plastic hinging as well as determine the connection design forces. Figure 4.21 shows the static pushover curve for bent 3 of this bridge. The greatest displacement occurred at this bent. The connection design force was determined using the expected transverse displacement of the bent calculated in the structural analysis mentioned above. The base shear at the expected displacement of 0.50” was 173 kips, which works out to 43.3 kips per connection. This force was used to design the clip angles and anchor bolts, which will be discussed below.

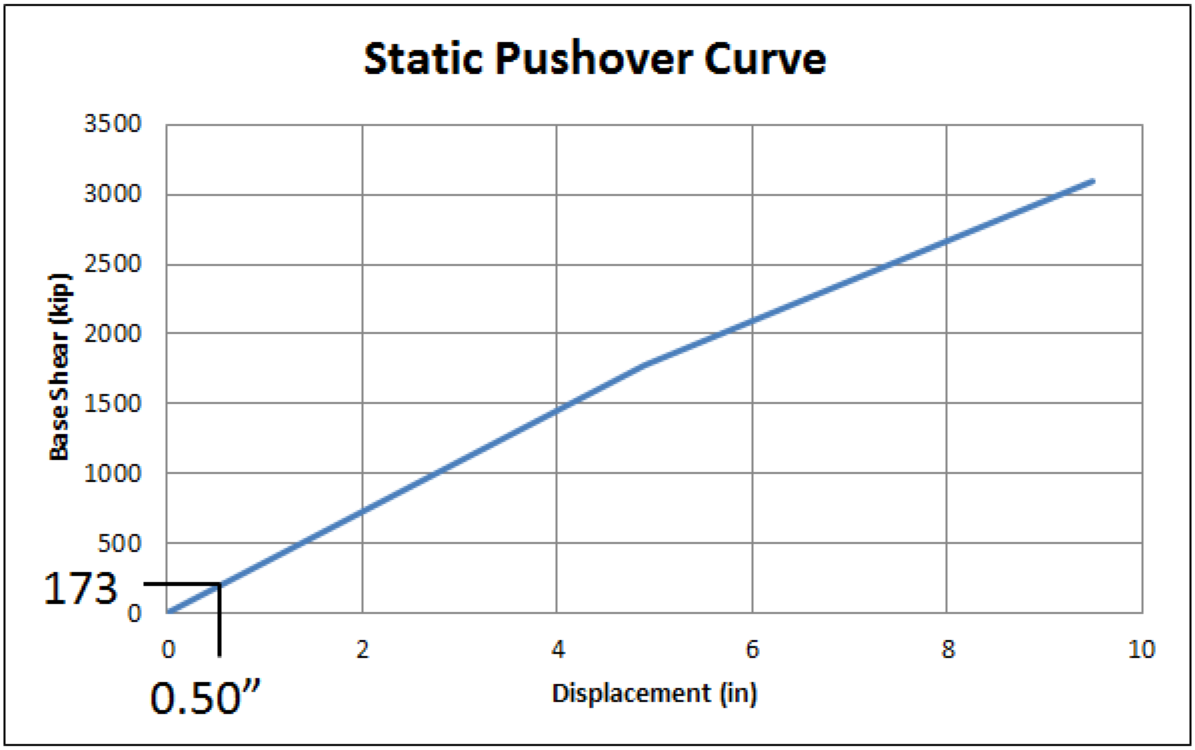

Figure 4.21: Static Pushover Curve for Oseligee Creek Bridge Bent 3

Using the design force from the pushover analysis, the transverse clip angles and anchor bolts were designed. The clip angle size was chosen from the original connection and block shear, tension and shear capacities of the angles were checked against the design force of 43.3 kips. Since this was larger than the previous connection design forces, the clip angles had to be checked. It was assumed that one of the angles would have to resist the entire design force because the other angle would not be able to transfer a tensile force. Table 4.39 shows the capacities of the clip angle as determined above using the design checks, and it can be seen that the clip angles can withstand the design force. The anchor bolt was designed for shear, bearing, tension, and combined tension and shear. Like the clip angle, only one anchor bolt was used to resist the loads. It was determined that an ASTM A307 Class C bolt with a diameter of 1.75

inches would be required for this connection. Since the bolt size is designed using the horizontal design force, the bolt should be specifically designed for each bridge.

Table 4.39: Capacity of the Steel Clip Angle

Limit State	Capacity (kips)
Block Shear	156
Tension	118
Shear	130

The minimum seat width was calculated for each bent using Equation 4.2 once the connection design was completed. The results, seen in Table 4.40 show that the minimum seat width using the new equation is 60-70% greater than the seat width calculated using the Standard Specifications. Bent 3 is a taller by almost 8 feet, and the effect of the height on the seat width can also be seen, since the minimum seat length is 2.5 inches greater for the taller column.

Table 4.40: Oseligee Creek Bridge Seat Width Specification Comparison (SDC B)

	Bent 2		**Bent 3**	
Specification	Standard	New Design	Standard	New Design
Minimum Seat Width (in)	11.0	17.6	11.7	20.1
Percent Difference	60%		71.8%	

The next step was to design the columns. The plastic hinge length was determined for each bent using both the Guide and LRFD Specifications to show the advantages of using the LRFD Specifications. These lengths can be seen in Table 4.41. As it shows, the plastic hinge lengths from the LRFD Specification is less than the Guide Specification length. This results in a larger length of column available for splicing to occur. For these columns, this length increased by 2 to 3.5 feet. The advantage of using the LRFD Specifications is that a shorter hinge length is required and having a shorter plastic hinge reduces the total number of ties and increases the length over which splicing may occur. The LRFD Specifications also allow for an extension of the plastic hinge length into the connecting member in order to ensure the formation of a plastic hinge by increasing the shear resistance of the section. The extension length was 21 inches for both bents.

Table 4.41: Oseligee Creek Plastic Hinge Length Comparison (SDC B)

	Bent 2		Bent 3	
Specification	Guide	LRFD	Guide	LRFD
Plastic Hinge Length (in)	63	42	63	51.7
Available Splice Length (in)	89	131	183	206
Extension Length (in)	-	21	-	21
% Difference PHL	-33.3%		-17.9%	

The LRFD plastic hinge length was used throughout the remainder of this design. It is important to note that the diameter of the columns controlled the hinge length in Bent 2 but one-sixth of the column height controlled for Bent 3. This shows that the hinge lengths can vary for different columns supporting a bridge. And it can also vary for different columns at a bent if the columns differ significantly in height.

The longitudinal reinforcement was determined to be sufficient for the loads. The maximum and minimum longitudinal reinforcement checks were also satisfied, so the transverse reinforcement was designed next. Table 4.42 shows the final design of transverse reinforcement using both the Standard and Guide Specifications. #4 ties were used for the transverse reinforcement. Using the Guide Specifications, a maximum spacing of 6 inches was required inside the plastic hinge. The extension length, which is not required for SDC B but recommended, required the same maximum spacing as the PHL, which was 6 inches. The maximum spacing outside of the plastic hinge length was determined to be 12 inches using the Standard Specification and 9 inches using the Guide Specification. This spacing resulted in approximately 60% more reinforcement in the Guide Specification design than the Standard

Specification design. This is typical, because the addition of the plastic hinge length requires tighter hoop spacing. Figure 4.22 and Figure 4.23 show two details of a column at Bents 2 and 3, respectively, using each of the design specifications. The spacing of the reinforcement can be seen, as well as the plastic hinge zone and extension length.

Table 4.42: Oseligee Creek Final Design Comparison (SDC B)

	Bent 2		**Bent 3**	
Specification	Standard	Guide	Standard	Guide
Spacing within PHL (in)	-	6	-	6
Spacing outside PHL (in)	12	12	12	12
Total Number of Hoops	18	29	26	41
Area of Hoops (in^2)	5.6	5.8	8.1	8.2
% Difference	3.6%		1.2%	

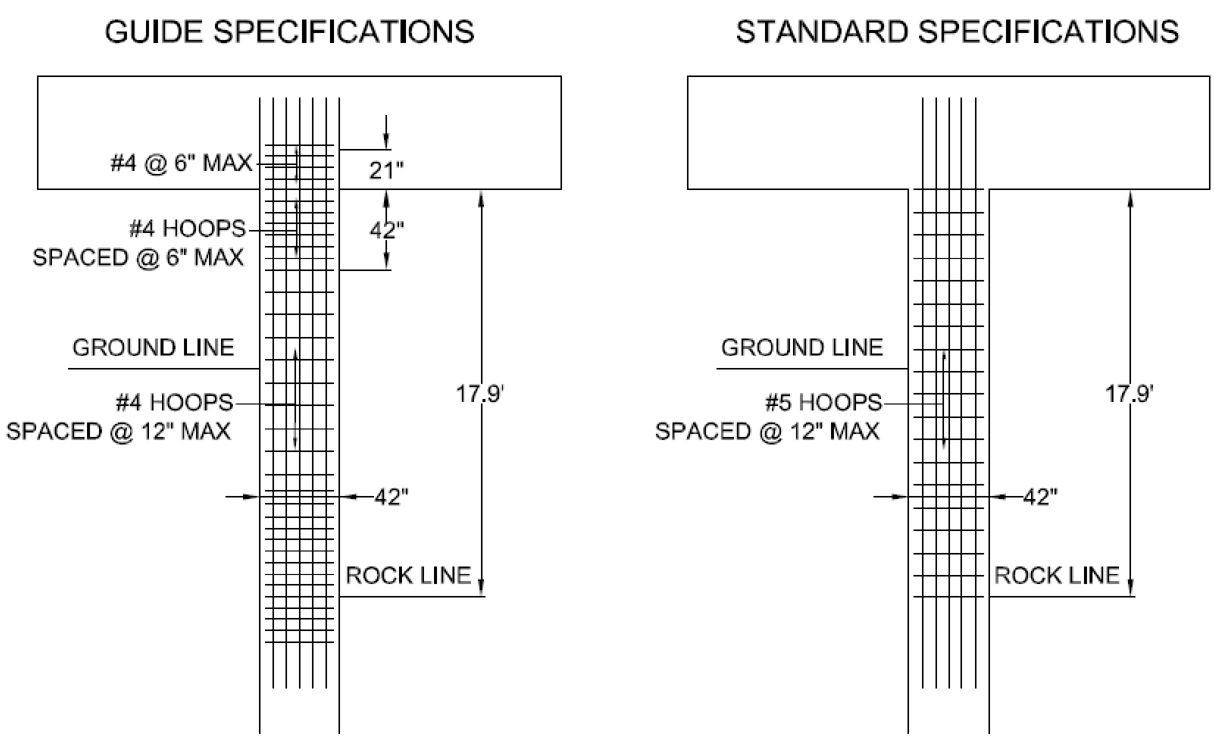

Figure 4.22: Oseligee Creek Bent 2 Final Design Details (SDC B)

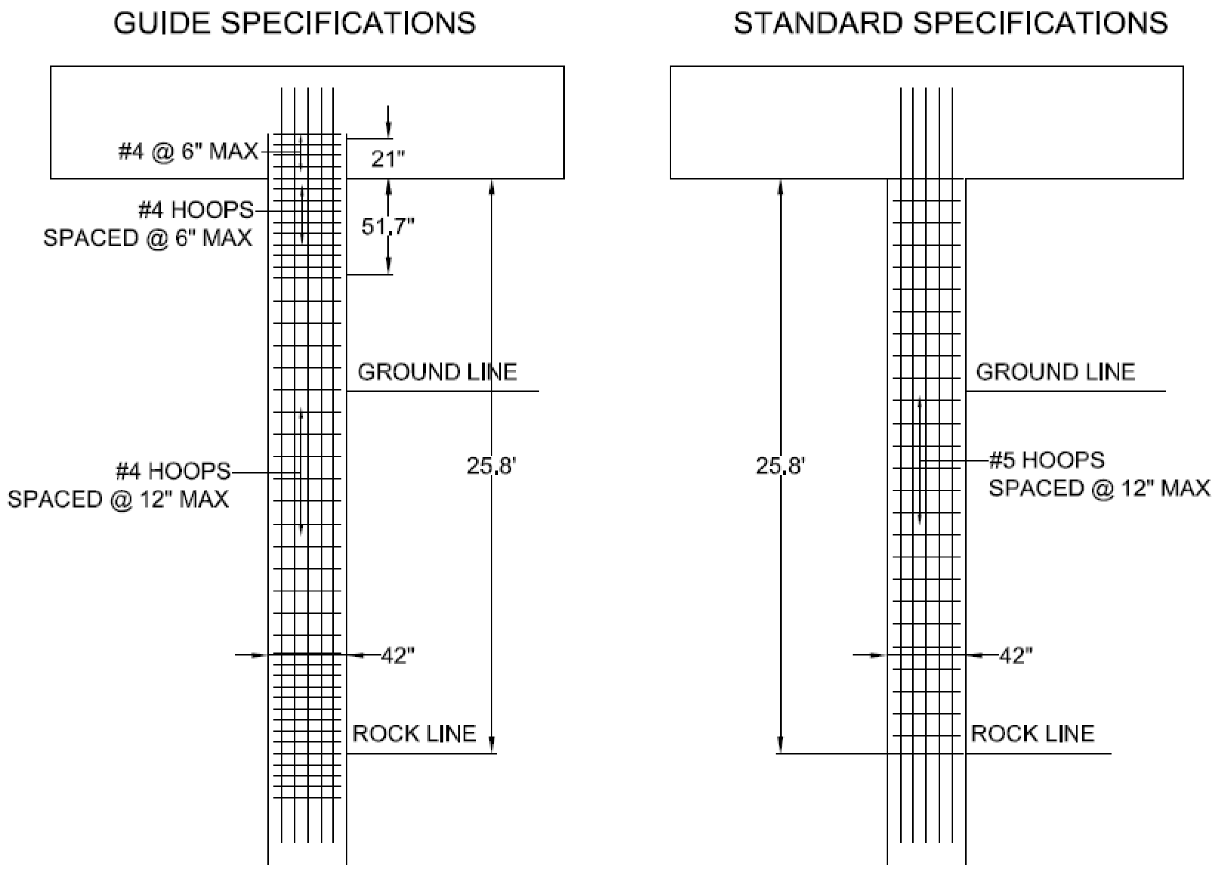

Figure 4.23: Oseligee Creek Bent 3 Final Design Details (SDC B)

The designs from SDC A2 and SDC B were compared in Table 4.43. The connection design forces in SDC B are 21.6% larger than the horizontal design force in SDC A2 that does not include the live load factor, meaning that performing a more rigorous analysis on the bridge does not guarantee smaller design forces. Therefore, it cannot be recommended to create a bridge model and perform a structural analysis for the sole purpose of getting lower design forces. The minimum seat width increased by about 20%, because SDC B bridges have higher spectral accelerations than SDC A2 bridge sites. The amount of transverse reinforcement did not change in the designs. This comparison shows that for this bridge, it would not be economical to design the bridge as SDC B because higher horizontal design forces would be required.

Table 4.43: Oseligee Creek Bridge SDC A2 and SDC B Comparison

	Bent 2		Bent 3	
	SDC A2	SDC B	SDC A2	SDC B
Design Force (kip)	35.6	43.3	35.6	43.3
% Difference	21.6%		21.6%	
Minimum Seat Width (in)	14.6	17.6	16.6	20.1
% Difference	20.5%		21.1%	
Number of Hoops	29	29	41	41
% Difference	0.0%		0.0%	

4.8.4 Little Bear Creek Bridge

Little Bear Creek Bridge was designed as SDC B to show the differences between designs from the Standard Specifications and Guide Specifications in SDC B. This bridge carries the two lanes of State Road 24 over Little Bear Creek in Franklin County. It is a three span bridge with spans of unequal lengths. The outer span lengths are 85 feet and the interior span is 130 feet. The outer spans support the 7 inch concrete deck with 6 Type III Girders and the interior span supports the deck with 6 BT-72 Girders. The two bridge piers are 40’ x 5’ x 7’ and supported by two circular columns 4.5 feet in diameter with 3 inches of concrete cover. The columns are reinforced longitudinally with 24 #11 bars and transversely with #5 hoops uniformly spaced at 12 inches from the bottom of the pier cap to the top of the foundation. The average clear height of Bent 2 is 12.06 feet and 16.88 feet for Bent 3. All columns are supported on drilled shafts 5 feet in diameter. The concrete cover of the drilled shafts is 6 inches but the longitudinal reinforcement in the drilled shaft still aligns with the longitudinal reinforcement of the column. Figure 4.24 shows the 3D model of the bridge used in the structural analysis. The design calculations for this bridge can be seen in Appendix N and the moment-axial force interaction diagrams can be seen in Appendix O.

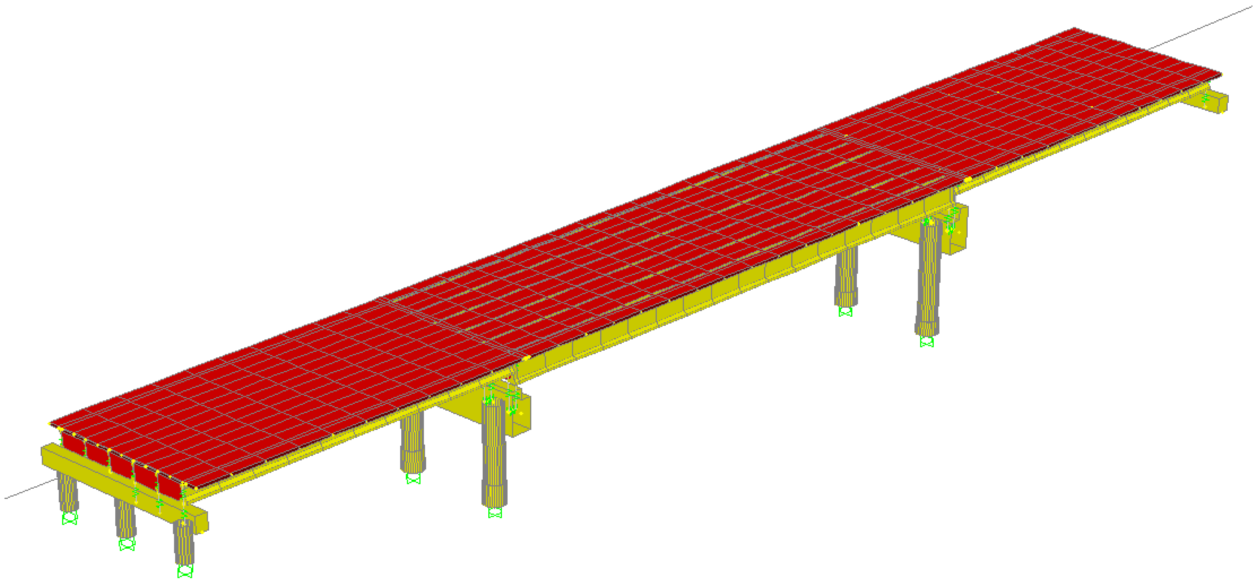

Figure 4.24: SAP2000 3D Model of Little Bear Creek Bridge

The first step was to perform the displacement capacity check. The SAP2000 bridge model and the uniform load method were used to determine the maximum displacements of the bridge. Table 4.44 lists the results from the capacity analysis. The expected displacement was determined by multiplying the bent displacement from the model by the equivalent static earthquake load and short period magnification factor. The largest displacement from the square root sum of the squares of the two orthogonal displacements was compared to the smallest capacity. For bent 2, the capacity in the longitudinal direction was smaller than the displacement demand. However, the Guide Specifications has a minimum value that can be taken as the bent capacity, which in this case was greater than the demand displacement. As the table shows, the capacity was greater than demand for both bents, so this bridge satisfied the capacity check.

Table 4.44: Displacement Results for Little Bear Creek Bridge

	Bent 2		Bent 3	
	Transverse	Longitudinal	Transverse	Longitudinal
Displacement at Bent from Model	0.795"	0.257"	2.241"	0.370"
Expected Displacement at Bent	0.183"	0.142"	0.516"	0.257"
Bent Capacity	1.35"	0.075"	0.97"	2.753"
Bent Capacity Lower Limit	1.448"		2.026"	
SRSS Displacement	0.188"		0.519"	

Once the capacity check was satisfied, a pushover analysis was performed to determine the connection design forces. Figure 4.25 shows the static pushover curve for Bent 3, which had the greatest expected displacement in the transverse direction. The displacement from the pushover analysis was less than the displacement from the elastic displacement of the bent in the structural analysis, so it was used as the displacement. As the figure shows, the design connection force was 400 kips, or 66.7 kips per connection.

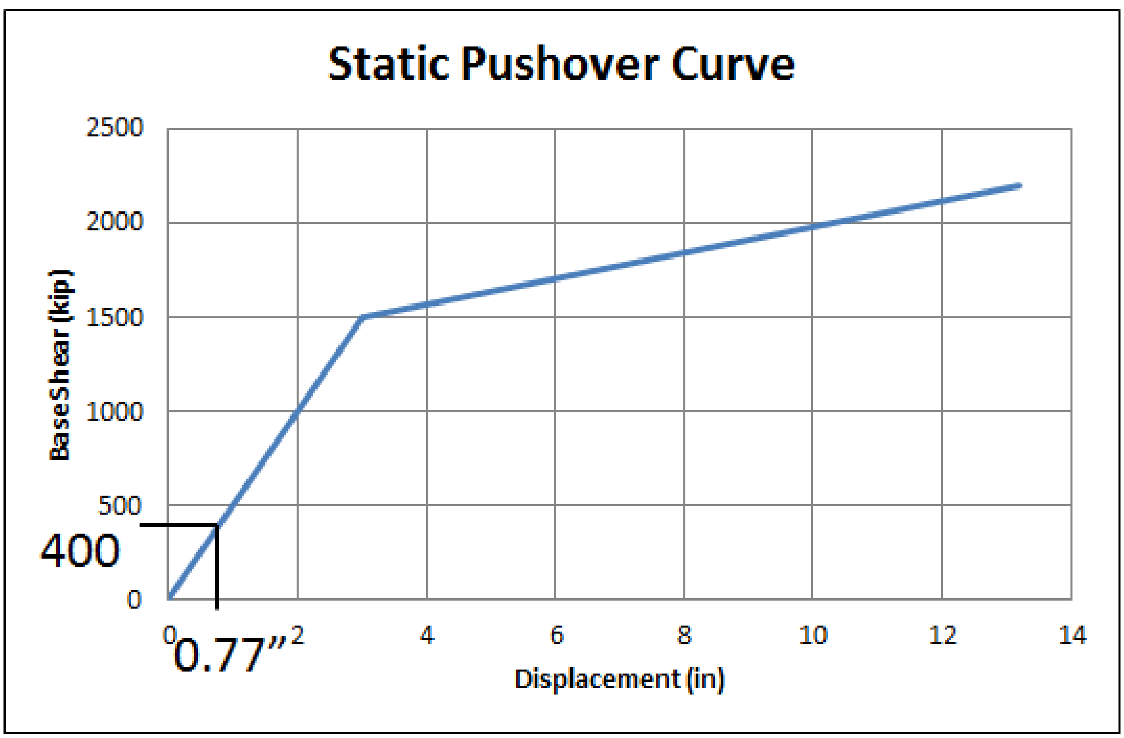

Figure 4.25: Static Pushover Curve for Little Bear Creek Bridge Bent 3

Using the design force of 66.7 kips from the pushover analysis, the transverse clip angles and anchor bolts were designed. The clip angle size was chosen from the original connection and block shear, tension and shear capacities of the angles were checked against the design force. Since this was larger than the previous connection design forces, the clip angles had to be checked. It was assumed that one of the angles would have to resist the entire design force because the other angle would not be able to transfer a tensile force because of the screw caps. Table 4.45 shows the capacities of the clip angle as determined above using the design checks, and it can be seen that the clip angles can withstand the design force. The anchor bolt was designed for shear, bearing, tension, and combined tension and shear. Like the clip angle, only one anchor bolt was used to resist the loads. It was determined that an ASTM A307 Class C bolt with a diameter of 2.25 inches would be required for this connection. This bolt size is different than the previous two connection designs. This shows that the bolt should be specifically designed for each bridge.

Table 4.45: Capacity of the Steel Clip Angle

Limit State	Capacity (kips)
Block Shear	156
Tension	118
Shear	130

Once the capacity check and connection design were completed, the bent was designed. First, the minimum seat width was calculated for each bent using Equation 4.2. The results, seen in Table 4.46, show that the results from Equation 4.2 are approximately 50% greater than Equation 4.1, which is used in the Standard Specifications.

Table 4.46: Little Bear Creek Bridge Seat Width Specification Comparison

	Bent 2		Bent 3	
Specification	Standard	Guide	Standard	Guide
Minimum Seat Width (in)	11.1	16.3	11.5	18.0
% Difference	46.8%		56.5%	

The next step was to design the columns. The minimum and maximum longitudinal reinforcement checks were satisfied and the longitudinal reinforcement was determined to be sufficient for the loads from the moment-axial force interaction diagram. The plastic hinge length was determined for each bent using both the Guide and LRFD Specifications and can be seen in Table 4.47. As it shows, the plastic hinge lengths from the LRFD Specification is less than the Guide Specification length. This results in a larger length of column available for splicing to occur. For Bent 2, there was no splice length because the plastic hinge extended the entire length of the column. Using the LRFD Specifications, however, allowed for a 36 inch section over which splicing could occur. For Bent 3, this available splice length increased by 4.5 feet. The advantage of using the LRFD Specifications for the plastic hinge length is includes having a shorter plastic hinge length, which results in a larger length over which splicing may occur. The LRFD Specifications also allow for an extension of the plastic hinge length into the connecting member in order to better ensure the formation of a plastic hinge by increasing the shear resistance of the section. This extension length was 27 inches.

Table 4.47: Little Bear Creek Plastic Hinge Length Comparison (SDC B)

	Bent 2		Bent 3	
Specification	Guide	LRFD	Guide	LRFD
Plastic Hinge Length (in)	81	54	81	54
Available Splice Length (in)	0	36	40	94
Extension Length (in)	-	27	-	27
% Difference PHL	-33.3%		-33.3%	

The LRFD plastic hinge length was used throughout the remainder of this design. It is important to note that the diameter of the columns controlled the plastic hinge lengths in both bents. This shows that for short columns, the hinge length will be controlled by the diameter of the columns.

The transverse reinforcement was designed next using both the Standard and Guide Specifications. Table 4.48 shows the results of the final designs. The spacing of reinforcement inside the hinge length was determined using #5 hoops. The Guide Specifications determined a maximum spacing of 6 inches inside the plastic hinge. The extension length, which is not required for SDC B but recommended, was 27 inches long with the same hoop spacing that was in the plastic hinge length. The maximum spacing outside of the plastic hinge length was determined to be 12 inches using the Standard Specification and 10 inches using the LRFD Specification. The Guide Specification design resulted in an increase of 65-80% of hoops compared to the original design. This shows that bridges requiring plastic hinges will need more transverse reinforcement. Figure 4.26 and Figure 4.27 show two details of a column at Bents 2 and 3 using both design specifications. The spacing of the reinforcement can be seen, as well as the plastic hinge zone and extension length.

Table 4.48: Little Bear Creek Final Design Comparison (SDC B)

	Bent 2		**Bent 3**	
Specification	Standard	Guide	Standard	Guide
Spacing within PHL (in)	-	6	-	6
Spacing outside PHL (in)	12	10	12	10
Total Number of Hoops	12	22	17	28
Area of Hoops (in^2)	3.72	6.82	5.27	8.68
% Difference	83.3%		64.7%	

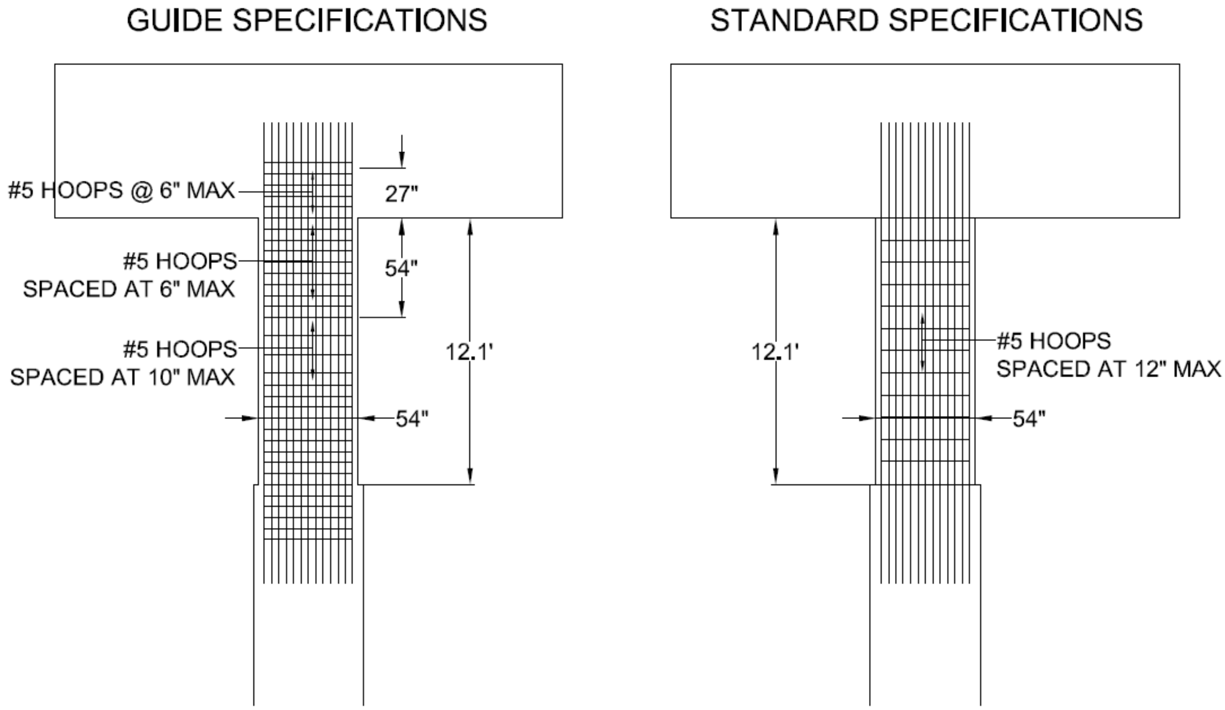

Figure 4.26: Little Bear Creek Bridge Bent 2 Final Design Details

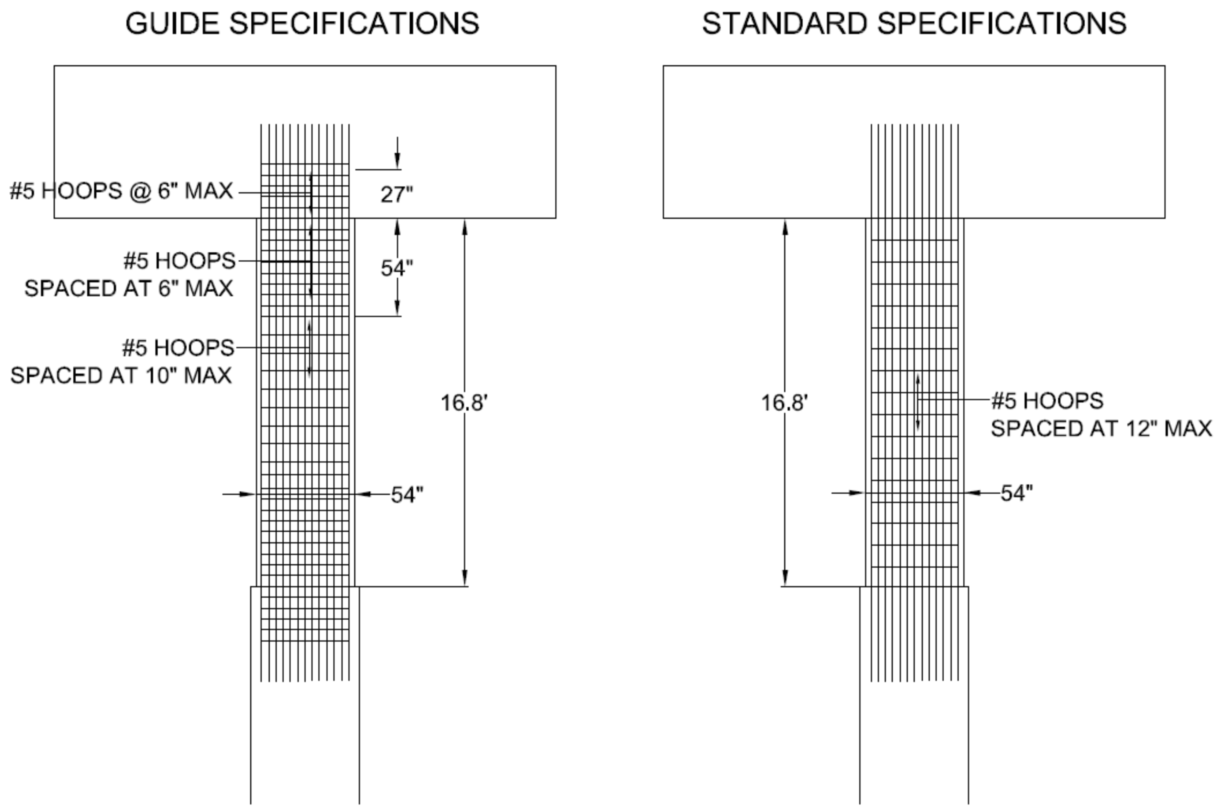

Figure 4.27: Little Bear Creek Bridge Bent 3 Final Design Details

4.8.5 Scarham Creek Bridge

Scarham Creek Bridge is the last bridge designed in SDC B. It differs from the other bridges because it is designed with struts at mid-height of the columns at each bent. These struts are required to provide stability and load transfer. These struts will also be redesigned since they play an important role in the behavior of the substructure. The bridge is two lanes and carries State Route 75 over Scarham Creek in Marshall County. It is a four span bridge with equal span lengths of 130 feet. The 7 inch concrete deck is supported by 6 BT-72 girders. The bridge pier at bents 2 and 4 are 40' x 5.5' x 7.5' and the pier at bent 3 is 40' x 6.5' x 7.5.' Bents 2 and 4 are supported by two circular columns 5 feet in diameter with 3 inches of concrete cover. Bent 3 is supported by two circular columns 6 feet in diameter with 3 inches of concrete cover. All columns are supported on drilled shafts, which are six inches larger in diameter than the columns. It is assumed that the plastic hinge will form at this transition, so the clear height of the columns is measured from the bottom of the bent cap to the transition between the column and drilled shaft. The average height of columns is 34.02 feet at Bent 2, 59.17 feet at Bent 3, and 32.16 feet at Bent 4. Because of the height of the columns, struts are provided at approximately mid-height of the columns and span the full length between columns with a thickness of 3.5 feet. The strut at bents 2 and 4 are 6 feet deep and 10 feet deep at bent 3. Figure 4.28 shows the 3D model of the bridge used in the structural analysis. The design calculations can be seen in Appendix P and the moment-axial force interaction diagrams can be seen in Appendix Q.

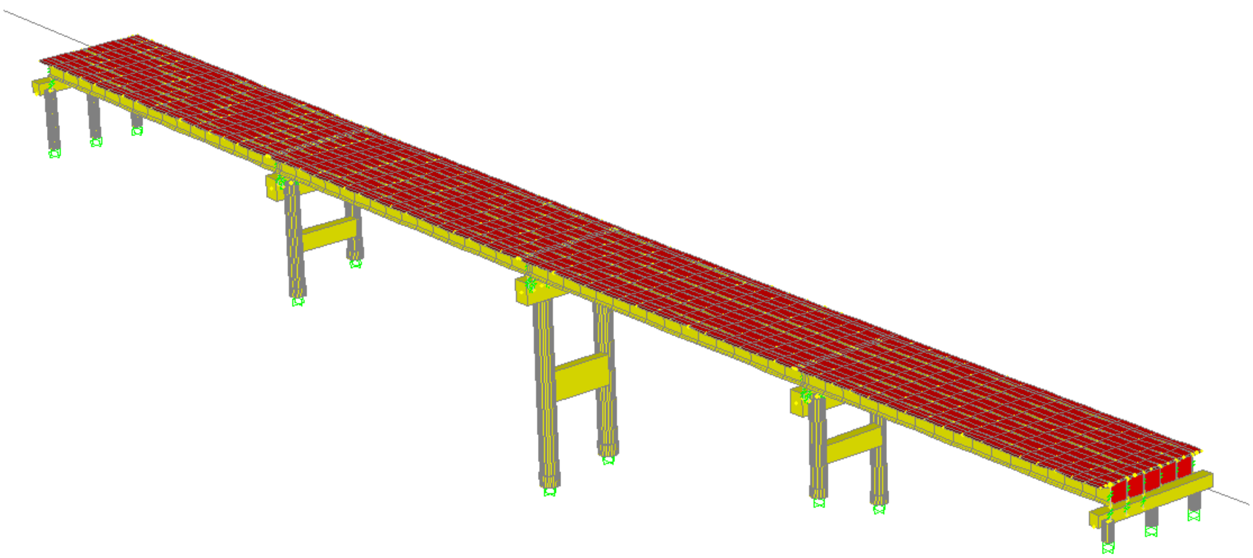

Figure 4.28: SAP2000 3D Model of Scarham Creek Bridge

The capacity check was completed first. However, the capacity equations for SDC B could not be used because of the struts. A pushover analysis was performed to verify the capacity of the columns for the expected displacements. The results from the pushover analysis performed using the computer software can be seen in Table 4.49. Since all three bents have greater capacities than demand in each orthogonal direction, the bridge satisfies the capacity check.

Table 4.49: Pushover Analysis Results for Scarham Creek Bridge

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

A static pushover curve, seen in Figure 4.29, was developed for Bent 3, where the largest displacement demand occurs. This curve was used to determine the horizontal design force for the connection as well as the sequence of plastic hinging. From the SAP2000 model, it could be seen that the plastic hinges in the bottom of the column and in the struts formed at the same time. At the time of the pushover analysis was completed, these were the only two hinges that had activated. This suggests that the struts were too large, because the struts should be the first to yield in order to protect the columns. If the struts were smaller, it would allow them to yield first and form plastic hinges, which would dissipate more energy from the system and protect the columns.

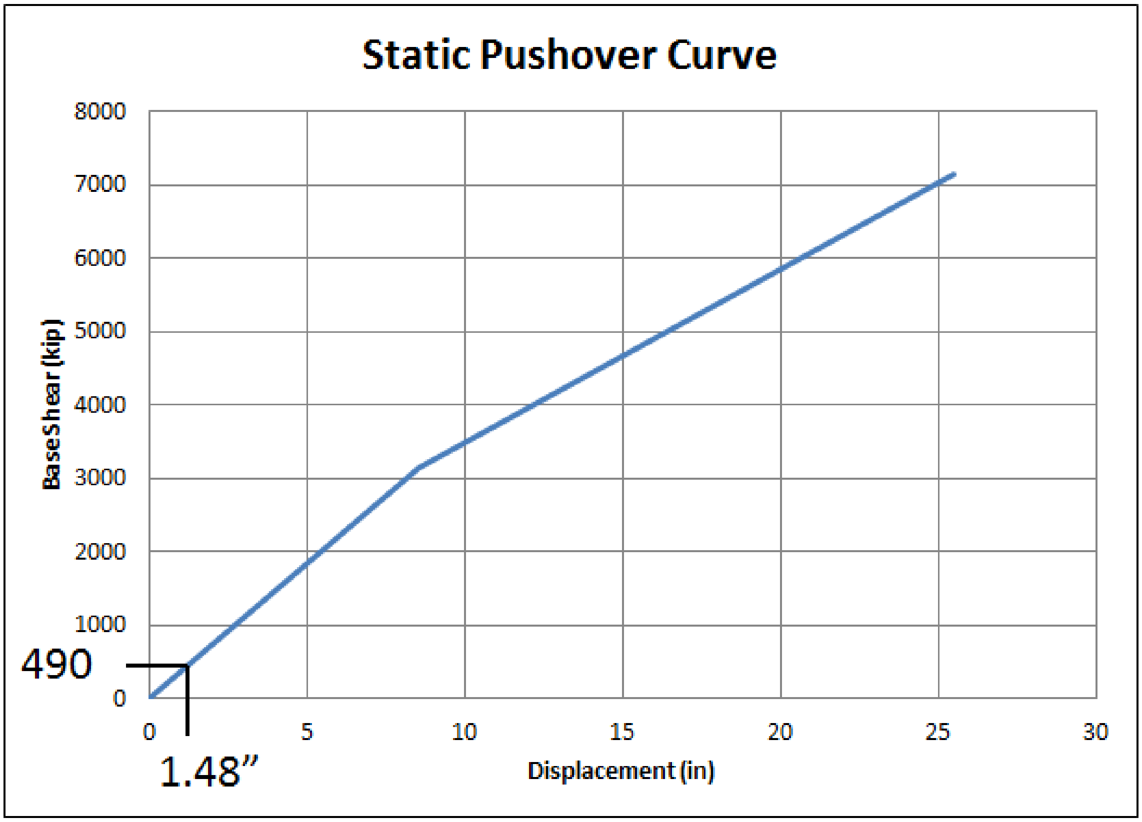

Figure 4.29: Static Pushover Curve for Scarham Creek Bridge Bent 3

The horizontal connection force was determined to be 81.67 kips. This comes from the 490 kips found in the graph above divided by 6 girders. The clip angles were designed to resist this force, and based on Table 4.50, were determined adequate. The anchor bolts were also designed. Using ASTM A307 Class C grade bolts, it was determined that bolts with a diameter of 2.5 inches would be adequate to resist the loads. In all of the bridges in SDC B, the clip angles were adequate to resist the loads, but the anchor bolts were all different sizes, ranging from 1.25 inches to 2.5 inches.

Table 4.50: Capacity of the Steel Clip Angle

Limit State	Capacity (kips)
Block Shear	156
Tension	118
Shear	130

The next step was to calculate the minimum seat widths. Equation 4.2, the ATC-49 equation, was used to calculate the new design seat widths and was compared with the seat widths from the Standard Specifications, found using Equation 4.1. Table 4.51 shows the minimum seat widths. For bents 2 and 4, the seat widths increased by 78%. But for bent 3, the seat width was almost double what was required by the Standard Specifications. This is because the columns in bent 3 are very tall.

Table 4.51: Scarham Creek Minimum Seat Width Comparison

	Bent 2		Bent 3		Bent 4	
Specification	Standard	Guide	Standard	Guide	Standard	Guide
Minimum Seat Width (in)	13.3	23.7	15.3	30.0	13.2	23.2
% Difference	78.2%		96.1%		75.8%	

The next step was to design the columns and struts. The design of the columns will be discussed first. The plastic hinge lengths were calculated using the Guide and LRFD Specifications in order to discuss their effect on the amount of splice length in the column. Table 4.52 displays the plastic hinge lengths, available splice lengths, and extension lengths for each bent. At Bents 2 and 4, the LRFD plastic hinge length was approximately 25% shorter than the hinge length from the Guide Specifications. The plastic hinge length was controlled by the column height instead of the column diameter. The available splice length was calculated a little differently than for the other bridges. Because of the presence of the struts, it was assumed splicing could not occur at a location where the strut connected to the column, which further shortened the splice length. However, because all of the columns were tall, there was still quite a bit of length over which splicing could occur. By using the LRFD plastic hinge length, the splicing length increased by about 2 feet for both Bents 2 and 4. Bent 3 was unique because it was very tall, and because of its height, the Guide Specifications hinge length was shorter than the LRFD hinge length. This is only bent in any of the bridges studied in this thesis where the plastic hinge length from the Guide Specifications was shorter. Therefore, it should be noted that for extremely tall columns, it is recommended to check the plastic hinge lengths from both the LRFD Specifications and Guide Specifications. The length available for splicing was also larger using the Guide Specifications, allowing for ten more inches. The extension lengths were also calculated to be 30 inches for Bents 2 and 4, and 36 inches for Bent 3. These lengths were controlled by the column diameters.

Table 4.52: Scarham Creek Plastic Hinge Length Comparison

	Bent 2		**Bent 3**		**Bent 4**	
Specification	Guide	LRFD	Guide	LRFD	Guide	LRFD
Plastic Hinge Length (in)	90	68	108	118	90	64.3
Available Splice Length (in)	78	100	187	177	66.5	92.7
Extension Length (in)	-	30	-	36	-	30
% Difference PHL	-24.4%		9.3%		-28.6%	

Once the plastic hinge lengths were determined, the transverse reinforcement was designed and the results can be seen in Table 4.53. The design forces were determined from the structural analysis and uniform load method. #6 hoops were used in the columns so that an accurate comparison with the original design by the Standard Specifications could be made. Bents 2 and 4 required the same maximum hoop spacing of 6 inches in the plastic hinge zone and 12 inches outside of it. This resulted in an approximately 33% increase in the number of hoops in both bents. Bent 3 required a maximum spacing of 6 inches in the plastic hinge zone and 10 inches outside of it, increasing the number of hoops by 43% compared to the Standard Specifications. Once again, the redesign of this bridge shows that using the Guide Specifications will require more ties because of the tighter spacing requirements.

Table 4.53: Scarham Creek Final Column Design Summary

	Bent 2		Bent 3		Bent 4	
Specification	Standard	Guide	Standard	Guide	Standard	Guide
Spacing within PHL (in)	-	6	-	6	-	6
Spacing outside PHL (in)	12	12	12	10	12	12
Total Number of Hoops	34	46	60	86	33	44
Area of Hoops (in^2)	14.96	20.24	26.4	37.84	14.52	19.36
% Difference	35.3%		43.3%		33.3%	

The struts were designed next. Table 4.54 shows the final design results for the struts. Because the struts at Bents 2 and 4 were of equal geometry, their design will be the same. The plastic hinge lengths for the struts were determined using the Guide Specifications to be 72 inches for Struts 2 and 4, and 120 inches for Strut 3. The depth of the strut controlled the length of the plastic hinge. #5 ties were used as transverse reinforcement for Struts 2 and 4. The maximum spacing was 4 inches inside the plastic hinge zone and 14 inches outside. This resulted in a 120% increase in the amount of shear reinforcement in the strut. For Strut 3, #6 ties were used. The maximum spacing requirements using #5 ties was determined to be 2 inches. It was determined that this spacing was too small to allow the concrete to be consolidated. If a self-consolidating concrete is used, #5 ties may be a possibility. Using #6 ties, the maximum spacing inside the plastic hinge length was 3.5 inches, and since the plastic hinge length covered the entire length of the strut, this spacing was used across its entire length. Because of the use of a larger tie size and the much tighter spacing, the amount of transverse reinforcement increased by 390%. Another option that could be used to increase the spacing of the ties would be to use cross-ties. Adding two additional vertical legs to the strut at Bent 3 would allow the spacing to be increased to the 6 inch maximum, which would make the reinforcement cage less congested and allow the concrete to be more easily consolidated. The depth of the struts contributes to the length of the plastic hinge zone. If the struts were smaller, the plastic hinge zone would be smaller and the amount of transverse reinforcement would be significantly smaller. Figure 4.30, Figure 4.31, and Figure 4.32 show the final design details for each bent using both the Standard Specification and Guide Specification for the columns and struts.

Table 4.54: Scarham Creek Final Strut Design Summary

	Strut 2		Strut 3		Strut 4	
Plastic Hinge Length (in)	72		120		72	
Specification	Standard	Guide	Standard	Guide	Standard	Guide
Spacing within PHL (in)	-	4	-	3.5	-	4
Spacing outside PHL (in)	12	14	12	18	12	14
Total Number of Ties	19	42	18	62	19	42
Area of Ties (in^2)	5.89	13.02	5.58	27.28	5.89	13.02
% Difference	121.1%		388.9%		121.1%	

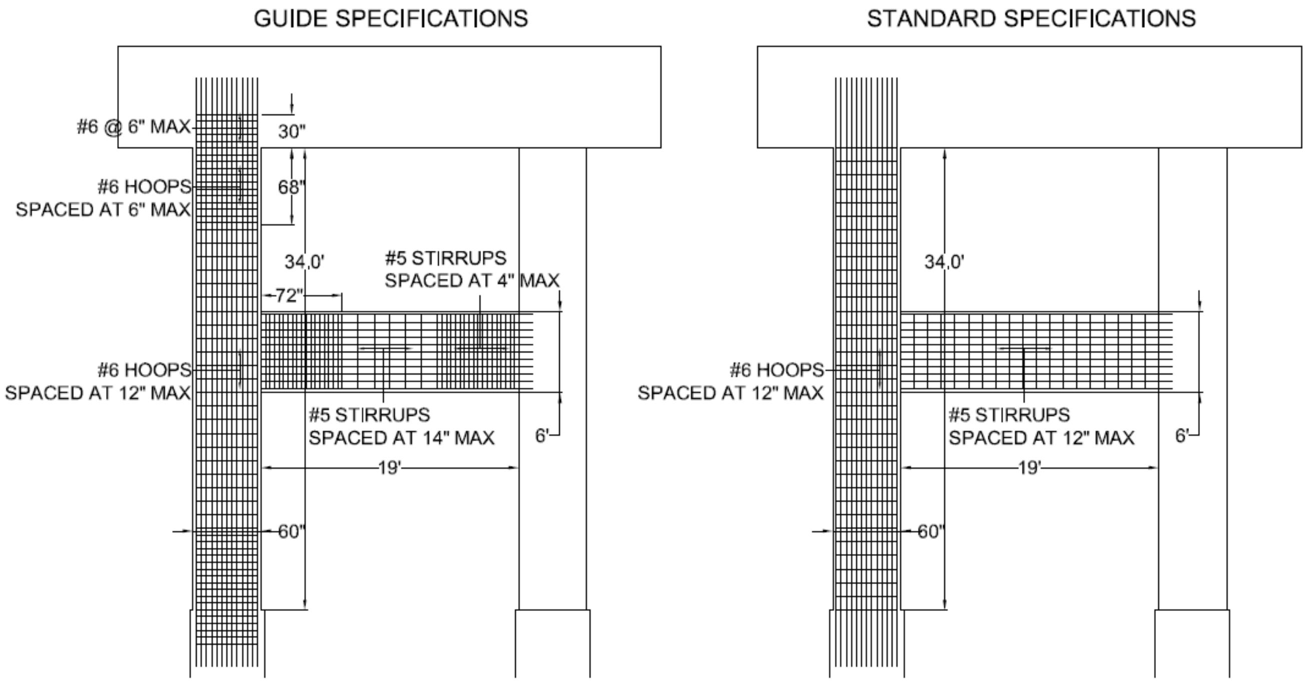

Figure 4.30: Scarham Creek Bridge Bent 2 Final Design Details

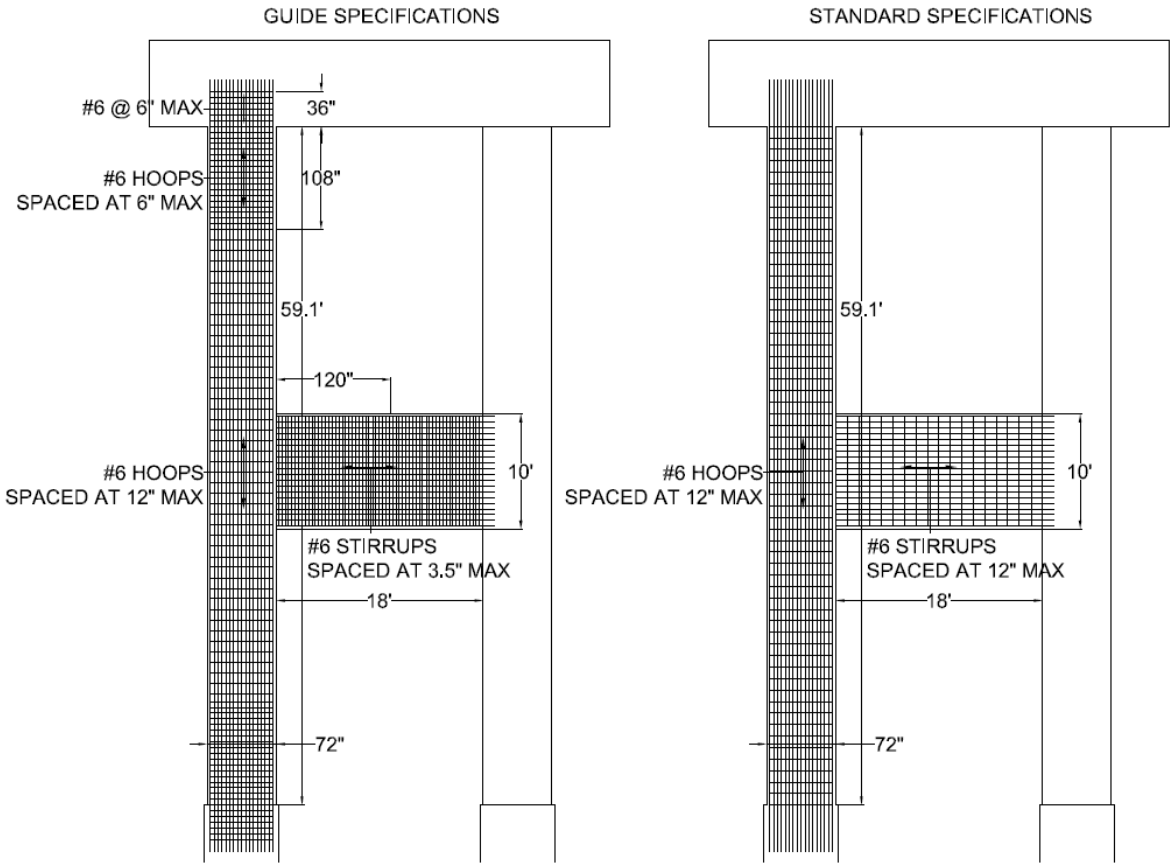

Figure 4.31: Scarham Creek Bridge Bent 3 Final Design Details

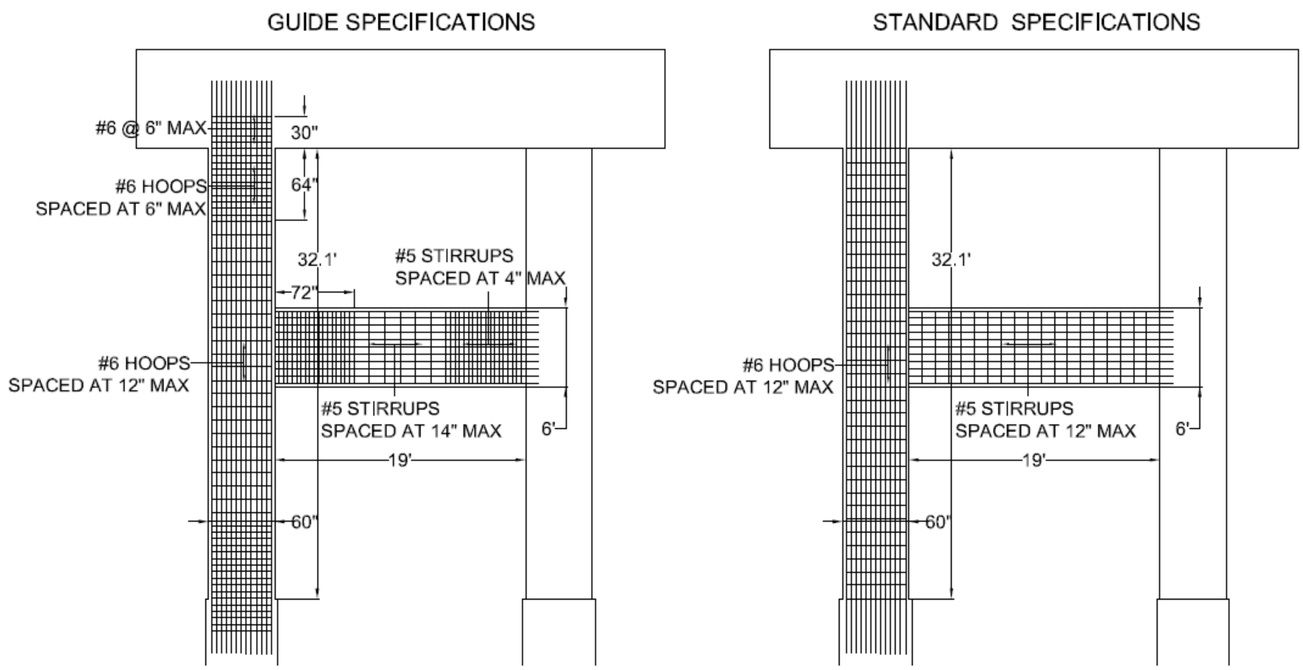

Figure 4.32: Scarham Creek Bridge Bent 4 Final Design Details

4.8.6 Summary of Differences in SDC B

In SDC B, designing by the Guide Specification resulted in many changes compared to the Standard Specification design. The most significant was the addition of the plastic hinge zone, which resulted in higher amounts of transverse reinforcement. The spacing inside of the plastic hinge zone could not be greater than 6 inches, and the spacing outside of the plastic hinge zone was either equal to or smaller than the spacing from the original designs. Another change was the larger minimum seat widths required by the recommended equation from the ATC-49 study, notated in Equation 4.2. All five bridges required a greater seat width than that required by the Standard Specifications. This is because the new equation is designed to give a better estimation of the displacement of the seat, which turns out to be larger. This change affected all bridges in SDC B.

The new designs also showed that using the LRFD Specifications to determine the plastic hinge length results in smaller hinge lengths, which decreased the amount of transverse reinforcement required and increased the length of the column over which splicing can occur. The one exception was for the very tall columns in Bent 3 of Scarham Creek Bridge. At this bent, the Guide Specifications actually resulted in a smaller hinge length. So while it is recommended to use the LRFD Specifications for the plastic hinge length, the Guide Specifications should be checked, especially for tall columns.

Three of the SDC A2 bridges were redesigned as SDC B bridges to determine if the horizontal design forces from a structural analysis method found in the Guide Specifications were smaller than those determined by the simple equations of SDC A2. This was the case in two of the bridges. The horizontal design forces for Bent Creek Road Bridge and the Bridge over Norfolk Southern Railroad were reduced when a structural analysis was completed. But the design forces for Oseligee Creek Bridge increased by 20%. Therefore, it cannot be assumed that a bridge in SDC B will have lower horizontal design forces than a bridge in SDC A. The other change from SDC A2 to SDC B was the increase in minimum seat width. The ATC-49 seat width equation uses the spectral acceleration to multiply the seat width. Since, by definition, SDC B sites have a higher spectral acceleration than SDC A sites, the minimum seat width will be greater in SDC B. The amount of transverse reinforcement did not change for the bridges studied with the exception of Oseligee Creek Bridge. Smaller transverse reinforcing bars were able to be used and even with the tighter spacing, the amount of reinforcement only increased by 1-3%.

4.9 Design Standards

Design standards were created by comparing the designs from bridges in each SDC. The procedures of the Guide Specifications were used to design these bridges. By designing multiple bridges, multiple designs could be checked against the standard details to ensure that a variety of bridges would satisfy the criteria, instead of the few that were designed. These new design standards and details will be discussed below.

4.9.1 Design Standards for SDC A1

SDC A1 is for bridges in low seismic hazard areas ($S_{D1} < 0.10$). There are three changes in the design to these bridges: an increase in the seat width, a change in the horizontal design forces and a possible decreased spacing of the transverse reinforcement. The seat length is calculated using Equation 4.2, which is the recommended ATC-49 equation. This equation results in a greater seat length than that calculated by the Standard Specifications as well as the Guide Specifications, as discussed in chapter 3. The design force is changed based on the expected ground acceleration at the site. For bridges in areas where the ground acceleration is less than 0.05g, the horizontal design force is 15% of the vertical reaction carried by the bent being designed. Otherwise, the horizontal design force is 25% of the vertical reaction carried by the bent. The vertical reaction includes the dead weight of the bridge tributary to the bent. It can also include the live load tributary to the bent at the discretion of the Owner. Choosing to include the live load will increase the design forces by approximately 10%. Since the live load is not required by the Specifications and choosing not to include it would decrease the horizontal design force, it is recommended that it not be included on every bridge. However, it should be considered for bridge that could experience a significant live load presence during an earthquake.

$$N = \left(4 + 0.02L + 0.08H + 1.09\sqrt{H}\sqrt{1 + (2\frac{B}{L})^2}\right) * \left(\frac{1+1.25S_{D1}}{\cos(\alpha)}\right) \quad \text{Equation 4.2}$$

The final difference was a possible increase in the amount of transverse reinforcement. This change resulted from a new design equation in the LRFD Specifications for the required minimum amount of transverse reinforcement, which is used to design the reinforcement outside of the plastic hinge zone. Equation 4.5 can be seen below, and is applicable only if the design procedures used in this project are used, which are detailed in Article 5.8.3.4 of the LRFD Specifications. In the equation, the spacing is the variable that will be changed until the area of reinforcement supplied is greater than the minimum area required. This is only required when the design shear force in the column is greater than half of the factored shear resistance of the concrete. Only one of the bridges in this SDC required the minimum amount of reinforcement. If the minimum reinforcement equation is not required, a 12 inch ALDOT standard will control the spacing outside of the plastic hinge zone. If tight spacing outside of the hinge zone is a problem, cross-ties can be used to allow for the same amount of reinforcement at a larger spacing. For aid with determining the required spacing when the minimum requirements must be satisfied, Table 4.55 and Figure 4.33 were developed. For a given column width or diameter and known size of transverse reinforcement, the maximum spacing can be determined from the graph. The table can be used to find a specific value if it cannot be obtained from the graph. It should be noted that these aids do not include the effects of cross-ties, and are only applicable to columns with 4,000 psi concrete and 60 ksi reinforcing steel.

$$A_{v,min} = 0.0316 * \sqrt{f_c'} * \frac{b_v * s}{f_y} \quad \text{Equation 4.5}$$

Table 4.55: Maximum Spacing Requirements outside of Plastic Hinge Zone

Column Width or Diameter (in)	Maximum Spacing (in)	
	#4 Bars	#5 Bars
24	16.0	24.5
30	13.0	20.0
36	11.0	16.5
42	9.5	14.0
48	8.0	12.5
54	7.0	11.0
60	6.5	10.0
66	6.0	9.0
72	5.5	8.5
78	5.0	8.0
84	4.5	7.0

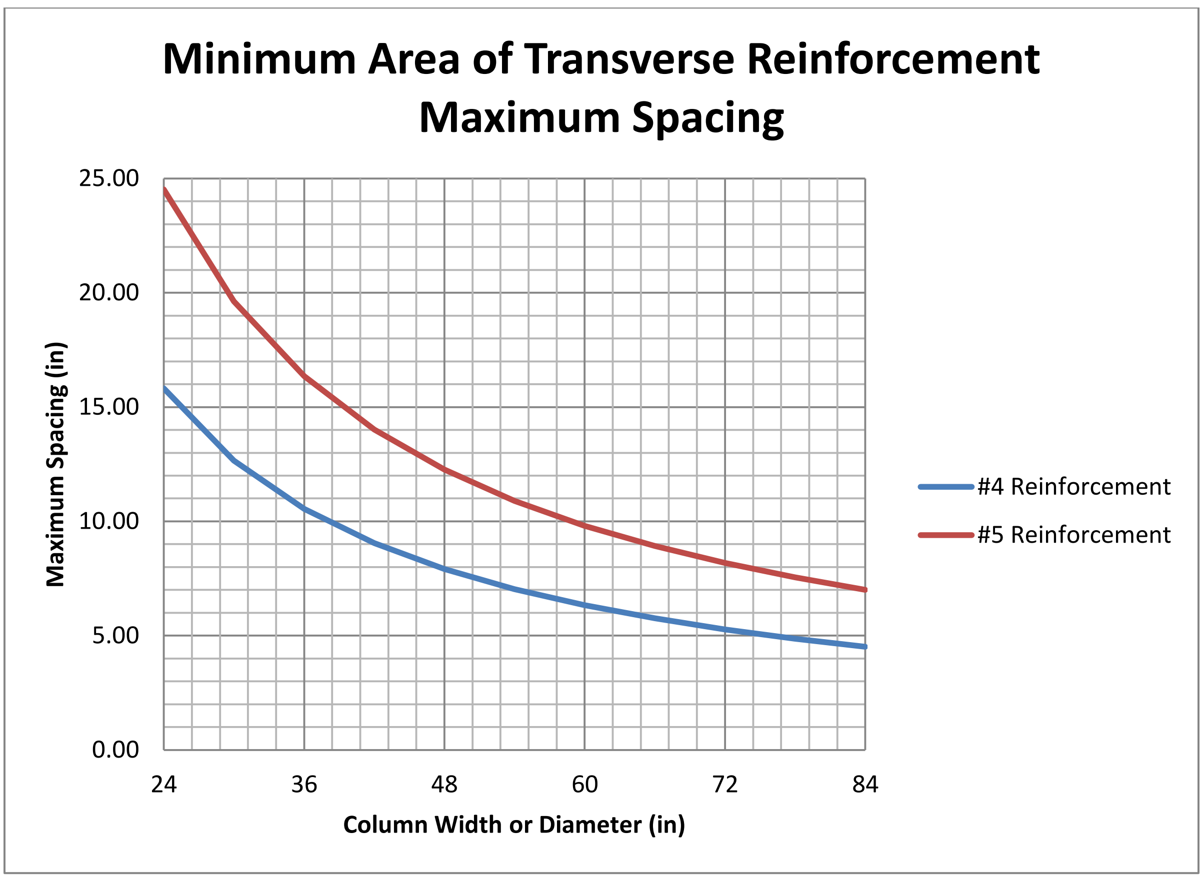

Figure 4.33: Maximum Spacing Requirements outside of Plastic Hinge Zone

4.9.2 Design Standards for SDC A2

SDC A2 bridges are still in areas of low seismicity ($0.10 \le S_{D1} < 0.15$) but with a greater possibility of experiencing forces that could cause plastic behavior to occur in the column. The changes to this design category from the Standard Specification reflect this possibility. They include an increased minimum seat width, the addition of the plastic hinge zone, and smaller spacing of the reinforcement inside the hinge zone. The seat widths are increased because Equation 4.2 is used to determine them. By increasing the seat width, the girders are provided with more room to "ride out" a design earthquake, as discussed in chapter 3. The plastic hinge zone is calculated using the LRFD Specifications because it resulted in a smaller hinge length. However, for very tall columns, the length from the Guide Specifications may control and should be checked. The plastic hinge length is determined to be the maximum of the following:

- The largest cross-sectional dimension
- One-sixth the clear height of the column
- 18 inches

The spacing of the transverse reinforcement inside the plastic hinge zone is only required to satisfy minimum ratios and not shear capacity equations. The minimum ratio for circular columns is 0.002 and for rectangular columns is 0.003. Article 8.6.2 in the Guide Specifications shows how to calculate these ratios. Once a reinforcement size has been chosen, the spacing of the reinforcement will affect the ratio. Figure 4.34 and Figure 4.35 have been developed to provide standard design drawings for bridges in SDC A2. They are applicable to bridges with largest column widths or diameters less than or equal to 6 feet. The plastic hinge length is based on the LRFD Specifications, so it is recommended to check the Guide Specifications hinge length if the columns are very tall. Because none of the rectangular columns in this study were large enough to require the use of cross-ties, the design drawings were developed using only one tie around the outside of the reinforcement. The transverse reinforcement spacing is based on the ratios, and the values given will satisfy them. The longitudinal reinforcement is not shown because it is determined on a project specific basis.

The reinforcement spacing outside of the plastic hinge zone is controlled either by the minimum area of transverse reinforcement requirement, discussed in the design standards for SDC A1, or by the 12 inch ALDOT standard. Therefore, those spacing requirements should also be checked. Figure 4.33 and Table 4.55 are recommended to be used when the minimum requirements are necessary.

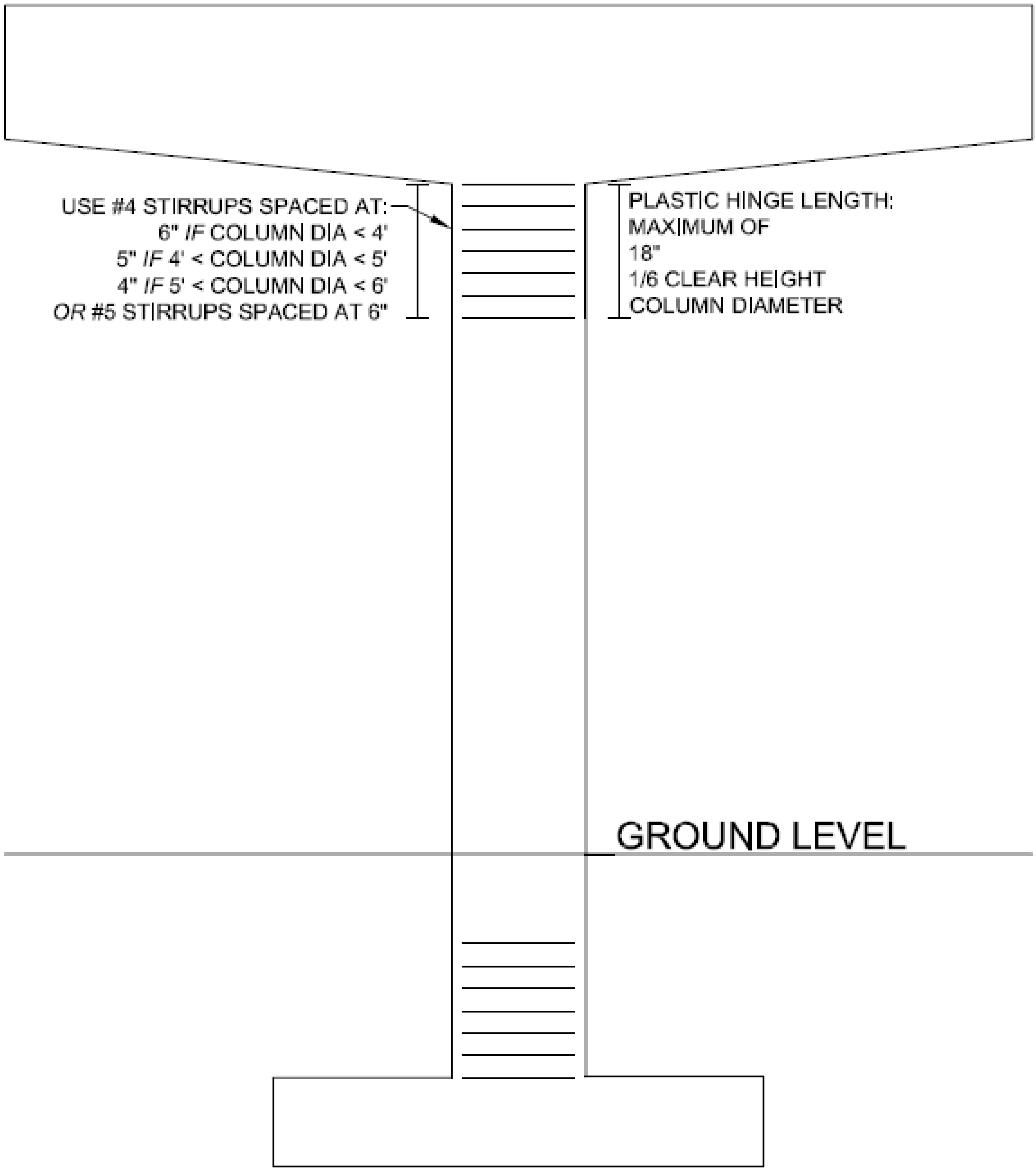

Figure 4.34: Standard Details for Circular Columns in SDC A2

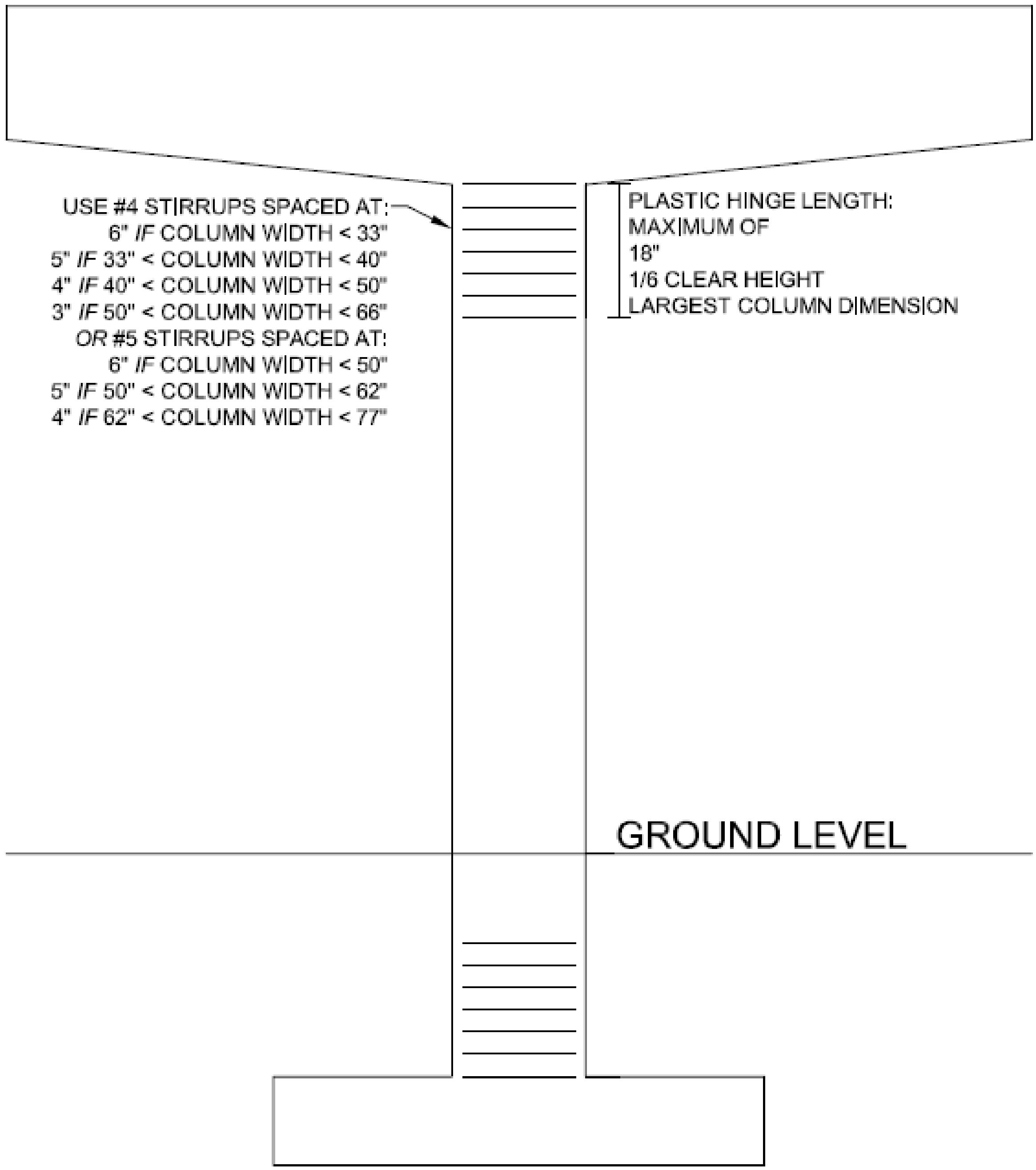

Figure 4.35: Standard Details for Rectangular Columns in SDC A2

4.9.3 Design Standards for SDC B

SDC B bridges are expected to experience moderate seismic forces ($0.15 \leq S_{D1} < 0.30$). These forces may be large enough that the columns must be designed with plastic hinges in order to dissipate the energy. Many changes were made to the design procedures for this category including the need for a bridge model and structural analysis to determine the horizontal design forces, an increase in the minimum seat width, and the recommendation of an extension of the plastic hinge zone into the bent cap or footing.

When the horizontal design forces from the rigorous structural analysis in SDC B were compared with the horizontal forces from the simple relationships in SDC A2, it was discovered that the structural analysis resulted in lower forces in only two out of the three bridges. So it cannot be assumed that performing a structural analysis will result in lower design forces.

As discussed in chapter 3, the purpose of the superstructure-to-substructure connection was to transfer forces in the transverse direction and allow the girders to move in the longitudinal direction by providing greater seat width. For bridges in SDC B, the minimum seat width is increased by using Equation 4.2. This is by design since the superstructure-to-substructure connection does not provide a complete load path in the longitudinal direction and must have additional room to move during a design earthquake. The greater seat widths prevent them from becoming unseated. And since the original connection was to be used, the components of the connection that contribute to the resistance in the transverse direction were checked against the calculated capacity design forces. The clip angles were determined to be adequate for the largest forces encountered, and the diameter of the anchor bolts was increased until it was also adequate to resist the forces. However, the anchor bolt diameters were different for each bridge, and it is recommended they be designed on a per bridge basis, as indicated on the current connection details.

The plastic hinge length is determined in the same manner as in SDC A2, but the transverse reinforcement must resist the shear forces in the cross section as well as satisfy the minimum ratios. For all five bridges studied, however, the minimum ratios still controlled the spacing of the transverse reinforcement in the plastic hinge length. The reinforcement outside of the hinge length was still controlled by either the minimum area of reinforcement check, if required, mentioned in the standards for SDC A1 or the 12 inch ALDOT standard. The extension of the plastic hinge zone is recommended by the Guide Specifications to increase the shear capacity of the cross section and allow the plastic hinge to form at the top of the column. This extension length should have the same transverse reinforcement spacing that is in the plastic hinge zone. This thesis recommends the use of the extension length in bridges in SDC B.

Figure 4.36 and Figure 4.37 were developed as standard details for SDC B. Like SDC A2, none of the rectangular columns in the bridges studied were large enough to require cross-ties, so the standard details were developed using only one tie to surround the longitudinal reinforcement in the plastic hinge zone. These details are similar to the details in SDC A2, except for the addition of the extension length. They are only applicable for columns with a width or diameter less than or equal to 72 inches.

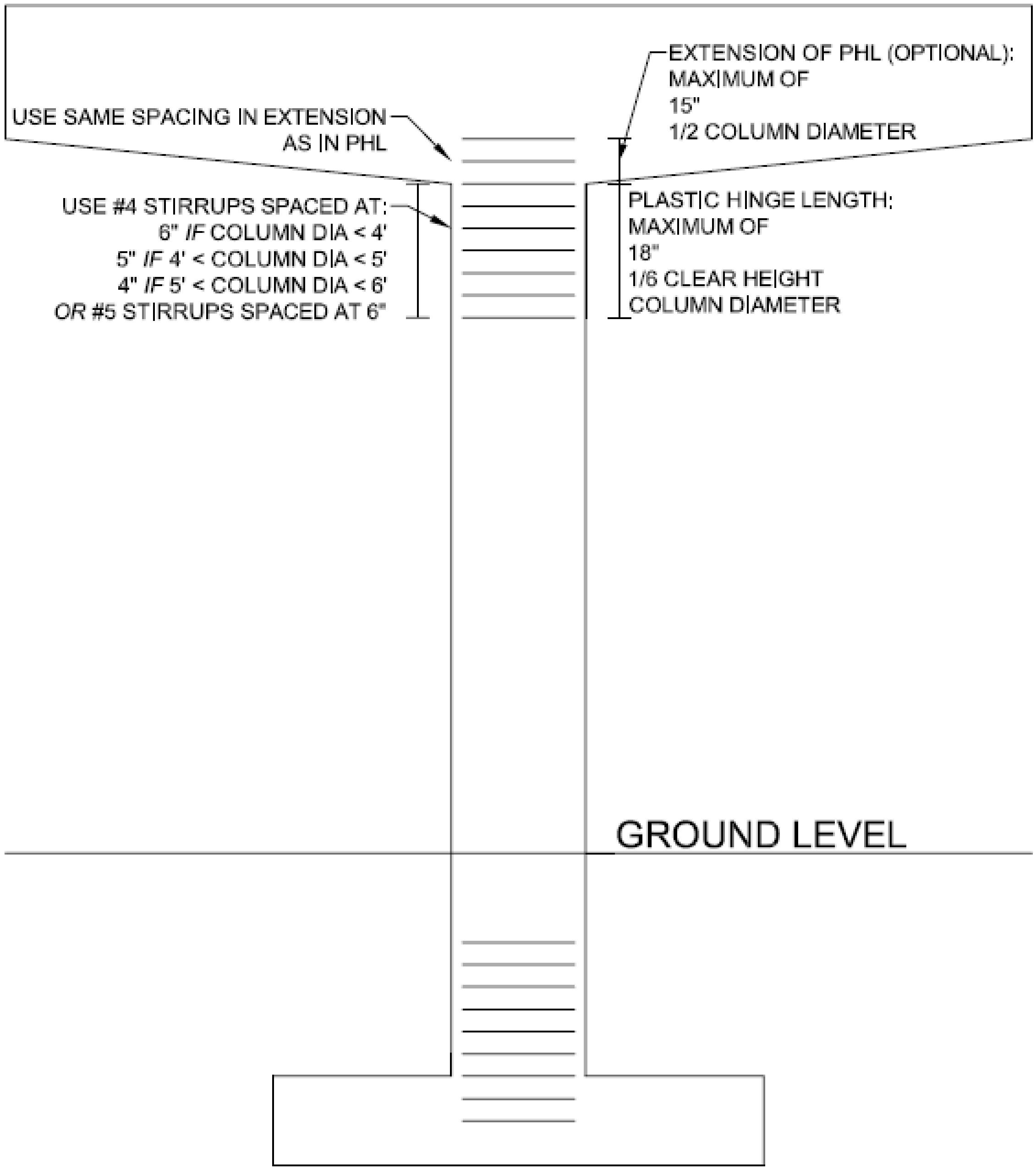

Figure 4.36: Standard Details for Circular Columns in SDC B

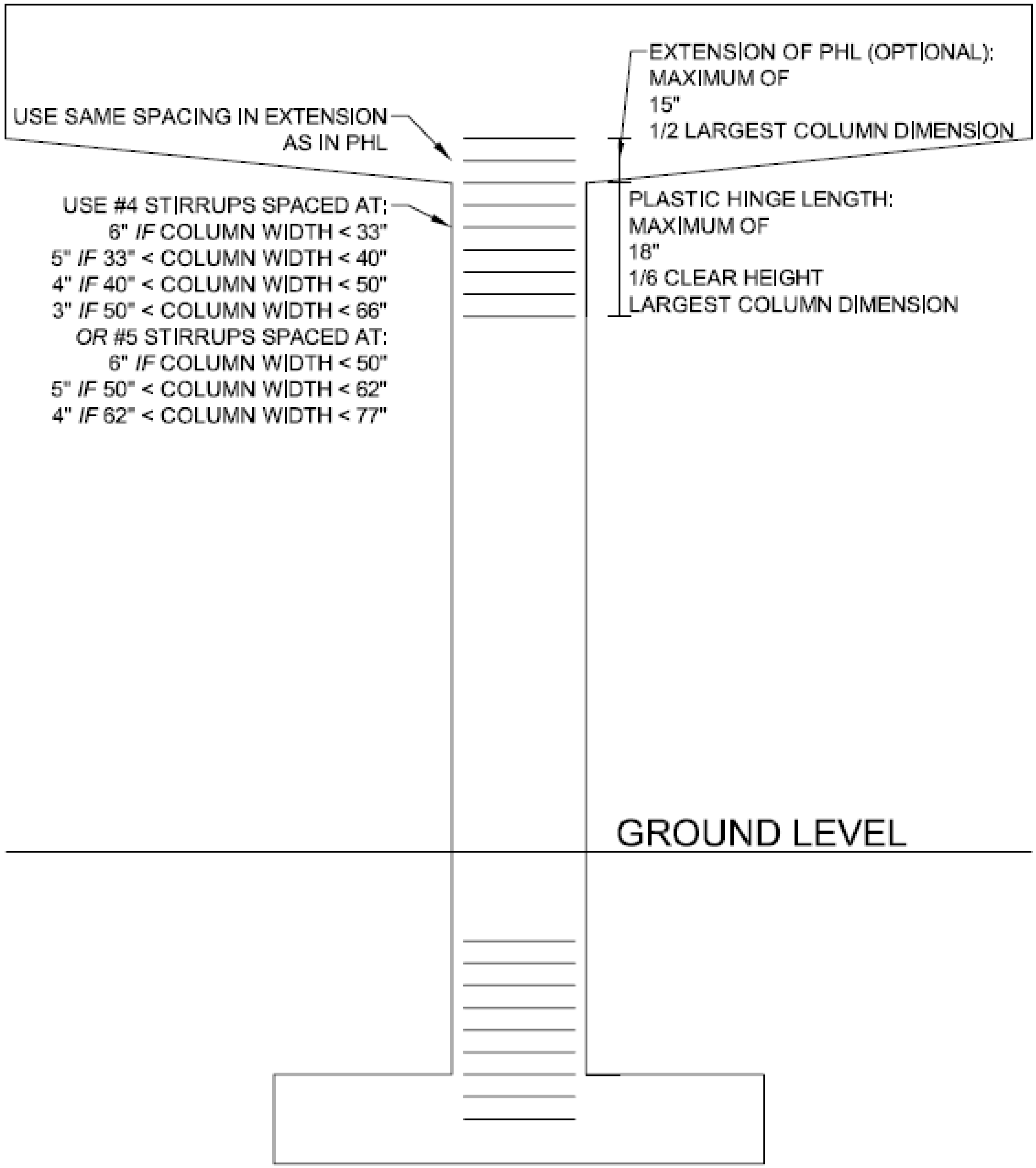

Figure 4.37: Standard Details for Rectangular Columns in SDC B

4.10 Conclusion

The purpose of this task was to develop new seismic design standards and details for bridges in the state of Alabama in Seismic Design Category A and B. These new standards are based on the Guide Specifications. This was accomplished by redesigned bridges in each SDC and comparing the new designs with the old designs under the Standard Specifications. 11

different bridges were re-designed by the Guide Specifications and compared with their designs using the Standard Specifications. The changes between the designs were used to develop design standards for each SDC.

For all of the bridges, the use of the ATC-49 equation (Equation 4.2) to determine the minimum seat length resulted in larger seat widths than those required by the Standard Specifications. The difference between the minimum lengths increased as the SDC increased, and specifically as the spectral accelerations within each SDC increased. The results from the research conducted in chapter 3 suggested that larger seat widths should be provided to allow the girders more room to displace in the longitudinal direction. The 11 bridges studied in this chapter proved that using the new equation did increase the minimum seat width.

The two bridges designed in SDC A1 showed that the horizontal design forces were different than they were in the Standard Specifications. The design force was either 15% or 25% of the vertical reaction resisted by the bent depending on the ground acceleration at the site, whereas in the Standard Specification, it was always 20%. The only other change in this SDC was a change in the LRFD Specifications that increased the amount of transverse steel in columns. This change affected all bridges that were designed, not just those in SDC A1.

One of the issues that was raised was the inclusion of the live load in the determination of the horizontal design force. The LRFD Specifications suggest including 50% of the live load at the Owner's discretion, but if the live load was included it would increase the horizontal design force, albeit only on the order of 10%. It was recommended not to include the live load in the design force calculation on every bridge, but to consider it on bridges that experience a significant live load presence throughout its service life. In summary for SDC A1, it was recommended to calculate the horizontal design force, minimum seat width, and maximum spacing of transverse reinforcement for the column. These three design steps controlled the design for the two bridges studied.

The bridges designed in SDC A2 showed an increase in the amount of transverse reinforcement required for the columns because of the requirement that the plastic hinge zone be detailed. The transverse reinforcement in the plastic hinge zone also had to satisfy minimum ratios found in the Guide Specifications. Once these minimum ratios were satisfied the plastic hinge zone design was completed. The horizontal design forces were calculated to be 25% of the vertical reaction in all cases, which resulted in higher forces than in the Standard Specifications. In summary for SDC A2, it was recommended to calculate the horizontal design force, minimum seat width, plastic hinge length, maximum spacing within the plastic hinge length, and the maximum spacing of transverse reinforcement outside of the plastic hinge length. These design steps were easily calculated and did not require any computer analysis of the bridge.

The biggest changes occurred in SDC B. Unlike the Standard Specifications, which simply required that the columns be designed to resist the expected loads, the Guide Specifications required the bridge displacement capacity to be greater than the expected displacement. In order to accomplish this, a computer model was built and a structural analysis was run. Once the capacity was confirmed, minimum detailing requirements had to be met as well as checking that the column section in the plastic hinge zone was capable of resisting the expected shear forces. However, for the five bridges studied, the minimum ratios from the detailing controlled the transverse reinforcement design instead of the strength. An additional extension length was recommended by the LRFD Specifications to promote the forming of a plastic hinge at the top and bottom of the column and protect the elements around the hinge. While not required for SDC B, it was recommended to use this extension length.

Another question that arose concerned the use of structural analysis to get smaller horizontal design forces for the connections and columns. Three bridges were designed in both SDC A2 and SDC B categories, and for only two of them were the design forces from SDC B lower than those for SDC A2. Therefore, it was not recommended to attempt a more complicated structural analysis to get smaller design forces.

Finally, the original superstructure-to-substructure connection used by ALDOT and discussed in chapter 3 was to be used. The longitudinal direction was allowed to displace and greater seat widths were provided to accommodate the movement, but the transverse direction needed to be analyzed to determine if it was adequate to resist the design forces. So for the five bridges studied in SDC B, the transverse connection was designed, and it was determined that the clip angles were adequate to resist the largest horizontal design force of 82 kips. The anchor bolts were also designed, but they varied in diameter from 1.25 inches to 2.5 inches. Therefore, as long as the anchor bolts were designed, it was recommended that the current connection be used as the superstructure-to-substructure connection since the minimum seat lengths provided were expected to provide enough room for the girders to move and the clip angles would provide enough resistance to transfer the forces into the substructure.

In summary for SDC B, it was recommended to first model the bridge in a structural analysis software package, such as SAP2000 or CSI Bridge, and determine the bridge displacement capacity and column and connection design forces. Next, the plastic hinge length, extension length, and spacing of the transverse reinforcement based on the minimum ratios were to be calculated, and then the section was checked to ensure it could resist the forces from the structural analysis. The transverse reinforcement outside of the plastic hinge zone was designed next and, finally, the minimum seat width was calculated and anchor bolts for the connection were designed. While this SDC does require the use of computer software and analysis, the design sheets and design aids created in this thesis provide guidance on how to accomplish certain design steps, as well as examples. The standard details and designs developed in this chapter are not intended to be used in lieu of designing the bridge, but do provide a starting point where designers can begin.

Chapter 5: Conclusion and Recommendations

5.1 Introduction

This objective of this thesis was to update the seismic standards for bridge design in the state of Alabama. With the transition of design from the Standard Specification to the new LRFD Specification, ALDOT wanted to know the changes that would occur in bridge design as a result. These changes are due mainly to the research in seismic hazard mapping and earthquake engineering that have been incorporated into the LRFD Specifications, but not the Standard Specifications. The seismic hazard maps in the Guide Specifications have a higher return period than the maps in the Standard Specifications, meaning that bridges must be designed to experience larger seismic forces. This increase in forces must be dealt with by the designer. This particular thesis dealt with the changes to bridges in low to moderate seismic regions, SDC A and SDC B, as well as the changes in the superstructure to substructure connection.

5.2 Superstructure-to-Substructure Connection

The superstructure-to-substructure connection was analyzed because it was unknown if the current connection was adequate to resist the expected horizontal design forces. In this study, it was resist that the current connection used by ALDOT did not provide a complete load path in the longitudinal direction, so a new connection was designed that would provide the load path. However, it was eventually decided to continue using the original connection and allow the girders to move after the connection slipped. The connection needed to be analyzed in both orthogonal directions to ensure that it was acceptable. The results from this study include the following:

- Using Equation 5.1 to determine the minimum seat width was found to be acceptable for estimating the minimum seat width in the longitudinal direction and ensuring the girders had enough room to "ride out" the design earthquake.
- It was determined that for bridges in SDC B, the S_{D1} value used in Equation 5.1 should be taken as 0.3 in order to provide a greater seat width than that provided by the Guide Specifications.
- The connection was determined adequate in the transverse connection because the steel clip angles and anchor bolts were designed to resist the largest horizontal loads from the SDC B bridges studied.
- The anchor bolts were recommended to be designed for each bridge, since the diameter of the bolts depended on the expected horizontal force.

Equation 5.1 can be seen below. This equation was created through research conducted by the Applied Technology Council and Multidisciplinary Center for Earthquake Engineering Research (ATC/MCEER Joint Venture, 2003) that resulted in a better estimation of the seat width demand for girders.

$$N = \left(4 + 0.02L + 0.08H + 1.09\sqrt{H}\sqrt{1 + (2\frac{B}{L})^2}\right) * \left(\frac{1+1.25S_{D1}}{\cos(\alpha)}\right) \qquad \text{Equation 5.1}$$

5.3 Bridge Design Standards

Once the superstructure-to-substructure connection was analyzed, design standards were developed for bridges in SDC A and B. These standards were developed by re-designing multiple bridges in each SDC and observing the differences in the final design between the two specifications. SDC A was split into two categories representative of the expected spectral accelerations, A1 and A2. The design standards for bridges in SDC A1 included only designing the connection for the horizontal design forces, supplying the minimum seat width using Equation 5.1, and designing the transverse reinforcement for the column. Once these standards were met, the design for bridges in SDC A1 was completed. Bridges in SDC A2 were still expected to experience low seismic forces, but had the possibility of experiencing plastic forces, and thus required to satisfy the minimum detailing requirements of SDC B. The design standards for bridges in SDC A2 included designing the connection for the horizontal design forces, determining the plastic hinge length, calculating the spacing of reinforcement within the hinge, supplying the minimum seat length using Equation 5.1, designing the transverse reinforcement outside of the plastic hinge length. Standard design details for bridges in SDC A2 were developed to aid the designer with these calculations. Bridges in SDC A did not require any structural analysis.

However, for SDC B, a computer model and structural analysis were required to be completed in order to determine the bridge displacement capacity and column design forces. These bridges were expected to experience plastic forces, so the columns were designed to allow plastic hinges to form in order to dissipate the energy from the expected design earthquakes. The other design standards included calculating the plastic hinge length using the LRFD Specifications, detailing the transverse reinforcement inside this length using the minimum ratios of the Guide Specifications, supplying the minimum seat length calculated from Equation 5.1 and designing the transverse reinforcement outside of the plastic hinge length. Standard design details for bridges in SDC B were developed to aid the designer with these calculations. One additional recommendation was made for bridges in SDC B: to use the extension length suggested in the LRFD Specifications to promote the formation of plastic hinges.

Other recommendations that were made concerning the seismic design of bridges include the following:

- Use a soil shear wave velocity test to verify soil site class of A or B at a bridge site to decrease the SDC of a bridge.
- The live load factor from the LRFD Specifications should not be used when calculating the horizontal design force in SDC A. However, it should be considered for high traffic bridges that constantly experience a full live load, such as in a major city center.
- The plastic hinge length should be determined using the LRFD Specifications because it results in a smaller hinge length, which allows a greater length of the column over which splicing can occur. For extremely tall columns, however, the length in the Guide Specifications should be checked.
- Cross-ties should be used to increase the spacing of transverse reinforcement outside of the plastic hinge length if it is determined that using only one tie around the perimeter of the longitudinal reinforcement results in very tight spacing.
- Smaller struts should be used in bridges with very tall columns to allow the struts to yield first and protect the columns.

5.4 Future Research

Future research should be conducted to study the effects of the transition from the Standard Specifications to the Guide Specifications for all types of bridges. This thesis focused on precast concrete bridges in SDC A and B, but these bridges do not reflect all of the bridges in the state. Since the LRFD Specifications are required to design all bridges in Alabama, research should be conducted to analyze how these new design specifications affect other types of bridges, including steel bridges in all seismic design categories. Future research could also address the change in the LRFD Specifications of the use of the soil site class in the determination of the SDC. Site Class A and B cannot be verified without using the results of a soil shear wave velocity test. If a correlation between shear wave velocity of soil and other parameters, such as standard blow count or undrained shear strength, can be determined, these site classes could be used, which could reduce the SDC for a bridge. Finally, the location of plastic hinges in drilled shafts was an area of uncertainty in this thesis. There was not a significant change in the stiffness of the column versus the drilled shafts, so a conservative assumption was made to determine the column height, which was to assume the column height to be from the bent cap to the rock line. Because the column height can affect the plastic hinge lengths, if a more reasonable assumption could be determined through additional research, the plastic hinge length could be reduced. The transition to the LRFD Specifications for bridge design marks a significant change in the philosophy of bridge engineering for the state of Alabama. This thesis, and future research, provides tools and information that help alleviate this transition, but it is still the responsibility of the Engineer to understand the design process and ensure the safety of a bridge.

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Appendix A: Connection Design

ORIGIN := 1

$$\text{Check}(\text{demand}, \text{capacity}) := \begin{array}{|l} \text{"OK"} \quad \text{if demand} \le \text{capacity} \\ \text{"NOT GOOD"} \quad \text{otherwise} \end{array}$$

ALDOT Current Connection Steel Angle Design Check

Vcolbent := 10(

LRFD Article 6.5.4.2: Resistance Factors

$\phi_t := 0.8$	Tension for A307
$\phi_s := 0.75$	Shear for A307
$\phi_{bs} := 0.8($	Block Shear
$\phi_{bb} := 0.8($	Bolts Bearing
$\phi_{sc} := 0.85$	Shear Connectors
$\phi_f := 1.0($	Flexure
$\phi_{sangle} := 1.0($	Shear for the Angle

Bolt Properties

Fub := 58	ksi	Strength of Cap Screw (It is assumed that ASTM A307 Grade C bolt is used)
$Dia_b := \frac{7}{8}$	in	Diameter of Cap Screw

INPUT

Angle Properties

Fy := 36	ksi	Yield Stress of the Angle
Fu := 58	ksi	Ultimate Stress of the Angle
t := 1.0(	in	Thickness of Angle
h := 6	in	Height of the Angle
w := 6	in	Width of the Angle
l := 12	in	Length of the Angle

INPUT

$k := 1.5$	in	Height of the Bevel
$distanchorhole := 4$	in	Distance from the vertical leg to the center of the hole. This is the location of the holes.
$diahole := Dia_b + \frac{1}{8} = 1$	in	Diameter of bolt hole
$BLSHlength := 6$	in	Block Shear Length
$BLSHwidth := 2$	in	Block Shear Width
$Ubs := 1.0$		Shear Lag Factor for Block Shear
$a := 2$	in	Distance from the center of the bolt to the edge of plate
$b := 3.5$	in	Distance from center of bolt to toe of fillet of connected part
$Lc := 2$	in	Clear dist. between the hole and the end of the member

Shear Force per Angle:

$$Vangle := \frac{Vcolbent}{2} = 50 \quad \text{kips}$$

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

AISC J4: Block Shear

$$Agv := t \cdot BLSHlength = 6 \quad \text{in}^2$$

$$Anv := t \cdot (BLSHlength - 0.5diahole) = 5.5 \quad \text{in}^2$$

$$Ant := t \cdot (BLSHwidth - 0.5diahole) = 1.5 \quad \text{in}^2$$

AISC Eq. J4-5

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

$$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 216.6 \quad \text{kips}$$

$\phi bsRn := \phi_{bs} \cdot Rn = 173.28$ kips

BlockShearCheck := ShearCheck(ϕbsRn, Vangle) = "OK"

AISC D2: Tension Member

$Ut := 0.6$

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

$Ant := t \cdot [w - (1 \cdot diahole)] = 5$ in^2

$Ae := Ant \cdot Ut = 3$ in^2 ***AISC Eq. D3-1***

$\phi tPn := \phi_t \cdot Fub \cdot Ae = 139.2$ kips ***AISC Eq. D2-2***

$TensionCheck_{AISC}$:= ShearCheck(ϕtPn, Vangle) = "OK"

AISC G: Shear Check

$Cv := 1.0$

$Aw := t \cdot w = 6$ in^2

$\phi sangleVn := \phi_{sangle} \cdot 0.6 Fy \cdot Aw \cdot Cv = 129.6$ kips ***AISC Eq. G2-1***

ShearAngleCheck := ShearCheck(ϕsangleVn, Vangle) = "OK"

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

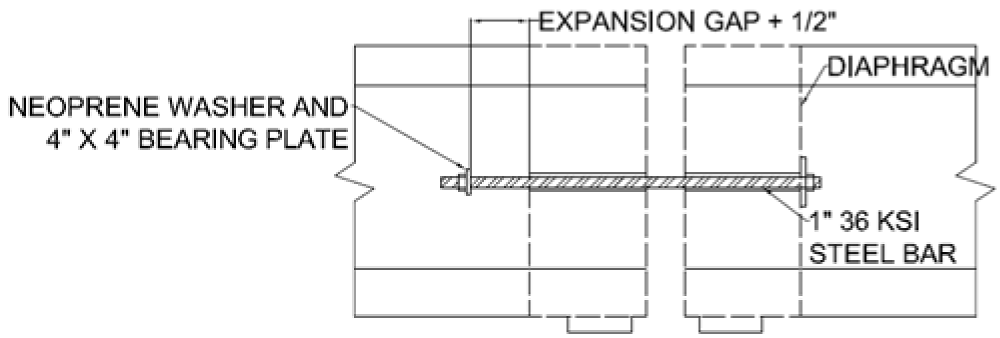
Summary

BlockShearCheck = "OK"

$TensionCheck_{AISC}$ = "OK"

ShearAngleCheck = "OK"

Longitudinal Restrainer Design Check



The two limit states were local yielding of the steel bearing plate and two way shear of the restrainer through the diaphragm. The largest restrainer force was calculated to be one-half of the design force because two restrainers would be present at each girder

$R_u := 24.5$ kips

Local Yielding ***AISC J7***

$b := 4$	in	Width of Square Bearing Plate
$F_y := 60$	ksi	Yield Strength of Bearing Plate
$A := b^2 = 16$	in^2	Area of Bearing Plate
$\phi := 0.75$		Resistance Factor for Local Yielding

$\phi R_{nYield} := \phi \cdot 1.8 F_y \cdot A = 1296$ kips ***AISC Equation J7-1***

Two Way Shear ***LRFD 5.13.3.6.3***

$f_c := 4000$	psi	Compressive Strength of Concrete
$d := 6$	in	Distance to Reinforcement from Extreme Compression Fiber
$A_s := 0.31$	in^2	Area of Reinforcing Steel
$s := 12$	in	Spacing of Reinforcing Steel

$f_y := 60$ ksi Yield Strength of Reinforcing Steel

$\phi := 0.9$ Resistance Factor for Shear ***LRFD 5.5.4.2.1***

$b_o := (d + b)\cdot 4 = 40$ in Perimeter of Critical Section

$V_c := 0.0632\left(\sqrt{\frac{f_c}{1000}}\cdot b_o \cdot d\right) = 30.336$ kips Shear Resistance of Concrete ***LRFD 5.13.3.6.3-3***

$V_s := \frac{A_s \cdot f_y \cdot d}{s} = 9.3$ kips Shear Resistance of Steel ***LRFD 5.13.3.6.3-4***

$\phi R_{nShear} := \phi \cdot (V_c + V_s) = \blacksquare$ kips

$\phi R_n := \min(\phi R_{nYield}, \phi R_{nShear}) = 35.672$ kips

$\text{Check}(R_u, \phi R_n) = \text{"OK"}$

Seat Width Calculations

The minimum seat width was calculated for each bent using three different equations. The longitudinal displacement was not greater than 1 inch for any bridge except Oseligee Creek Bridge, so 1 inch was used as a minimum.

N1 will be the equation used in Guide Specification for SDC B
N2 will be the equation used in the Guide Specifications for SDC D
N3 will be the equation used in the MCEER/ATC-49 study with $S_{D1} = 0.15$
N4 will be the equation used in the MCEER/ATC-49 study with $S_{D1} = 0.30$

$S_{D1a} := 0.15$

$S_{D1b} := 0.30$

Bent Creek Road

$L := 135$ ft Largest Span Length

$B := 80.75$ ft Deck With

$HCol_{Bent2} := 20$ ft Average Column Height at Bent 2

$Skew := 0$ deg Skew of Bridge

$\Delta_{Bent2} := 1$ in Longitudinal Displacement of Bridge Bent

Bent 2

$$N1_2 := 1.5(8 + 0.02L + 0.08HCol_{Bent2})\cdot\left(1 + 0.000125Skew^2\right) = 18.45 \quad in$$

$$N2_2 := (4 + 1.65\Delta_{Bent2})\cdot\left(1 + 0.00025Skew^2\right) = 5.65 \quad in$$

$$N3_2 := \left[4 + .02L + .08HCol_{Bent2} + 1.09\sqrt{HCol_{Bent2}}\cdot\sqrt{1 + \left(2\cdot\frac{3}{8}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1a}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 17.092 \quad in$$

$$N4_2 := \left[4 + .02L + .08HCol_{Bent2} + 1.09\sqrt{HCol_{Bent2}}\cdot\sqrt{1 + \left(2\cdot\frac{3}{8}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1b}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 19.791 \quad in$$

Norfolk Southern RR Bridge

$L := 140$	ft	Largest Span Length
$B := 46.75$	ft	Deck With
$HCol_{Bent2} := 25$	ft	Average Column Height at Bent 2
$Skew := 0$	deg	Skew of Bridge
$\Delta_{Bent2} := 1$	in	Longitudinal Displacement of Bridge Bent

Bent 2

$$N1_2 := 1.5(8 + 0.02L + 0.08HCol_{Bent2})\cdot\left(1 + 0.000125Skew^2\right) = 19.2 \quad in$$

$$N2_2 := (4 + 1.65\Delta_{Bent2})\cdot\left(1 + 0.00025Skew^2\right) = 5.65 \quad in$$

$$N3_2 := \left[4 + .02L + .08HCol_{Bent2} + 1.09\sqrt{HCol_{Bent2}}\cdot\sqrt{1 + \left(2\cdot\frac{B}{L}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1a}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 18.233 \quad in$$

$$N4_2 := \left[4 + .02L + .08HCol_{Bent2} + 1.09\sqrt{HCol_{Bent2}}\cdot\sqrt{1 + \left(2\cdot\frac{B}{L}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1b}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 21.111 \quad in$$

Little Bear Creek

$L := 130$	ft	Largest Span Length
$B := 42.75$	ft	Deck With
$HCol_{Bent2} := 12$	ft	Average Column Height at Bent 2
$HCol_{Bent3} := 17$	ft	Average Column Height at Bent 3
$Skew := 0$	deg	Skew of Bridge
$\Delta_{Bent2} := 1$	in	Longitudinal Displacement of Bridge Bent
$\Delta_{Bent3} := 1$	in	Longitudinal Displacement of Bridge Bent

Bent 2

$$N1_2 := 1.5\left(8 + 0.02L + 0.08HCol_{Bent2}\right)\cdot\left(1 + 0.000125Skew^2\right) = 17.34 \quad \text{in}$$

$$N2_2 := \left(4 + 1.65\Delta_{Bent2}\right)\cdot\left(1 + 0.00025Skew^2\right) = 5.65 \quad \text{in}$$

$$N3_2 := \left[4 + .02L + .08HCol_{Bent2} + 1.09\sqrt{HCol_{Bent2}}\cdot\sqrt{1 + \left(2\cdot\frac{B}{L}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1a}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 14.344 \quad \text{in}$$

$$N4_2 := \left[4 + .02L + .08HCol_{Bent2} + 1.09\sqrt{HCol_{Bent2}}\cdot\sqrt{1 + \left(2\cdot\frac{B}{L}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1b}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 16.609 \quad \text{in}$$

Bent 3

$$N1_3 := 1.5\left(8 + 0.02L + 0.08HCol_{Bent3}\right)\cdot\left(1 + 0.000125Skew^2\right) = 17.94 \quad \text{in}$$

$$N2_3 := \left(4 + 1.65\Delta_{Bent3}\right)\cdot\left(1 + 0.00025Skew^2\right) = 5.65 \quad \text{in}$$

$$N3_3 := \left[4 + .02L + .08HCol_{Bent3} + 1.09\sqrt{HCol_{Bent3}}\cdot\sqrt{1 + \left(2\cdot\frac{B}{L}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1a}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 15.84 \quad \text{in}$$

$$N4_3 := \left[4 + .02L + .08HCol_{Bent3} + 1.09\sqrt{HCol_{Bent3}}\cdot\sqrt{1 + \left(2\cdot\frac{B}{L}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1b}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 18.341 \quad in$$

Oseligee Creek

$L := 80$	ft	Largest Span Length
$B := 32.75$	ft	Deck With
$HCol_{Bent2} := 18$	ft	Average Column Height at Bent 2
$HCol_{Bent3} := 26$	ft	Average Column Height at Bent 3
$Skew := 0$	deg	Skew of Bridge
$\Delta_{Bent2} := 1.3$	in	Longitudinal Displacement of Bridge Bent
$\Delta_{Bent3} := 1.4$	in	Longitudinal Displacement of Bridge Bent

Bent 2

$$N1_2 := 1.5(8 + 0.02L + 0.08HCol_{Bent2})\cdot\left(1 + 0.000125Skew^2\right) = 16.56 \quad in$$

$$N2_2 := (4 + 1.65\Delta_{Bent2})\cdot\left(1 + 0.00025Skew^2\right) = 6.145 \quad in$$

$$N3_2 := \left[4 + .02L + .08HCol_{Bent2} + 1.09\sqrt{HCol_{Bent2}}\cdot\sqrt{1 + \left(2\cdot\frac{3}{8}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1a}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 15.224 \quad in$$

$$N4_2 := \left[4 + .02L + .08HCol_{Bent2} + 1.09\sqrt{HCol_{Bent2}}\cdot\sqrt{1 + \left(2\cdot\frac{3}{8}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1b}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 17.628 \quad in$$

Bent 3

$$N1_3 := 1.5(8 + 0.02L + 0.08HCol_{Bent3})\cdot\left(1 + 0.000125Skew^2\right) = 17.52 \quad in$$

$$N2_3 := (4 + 1.65\Delta_{Bent3})\cdot\left(1 + 0.00025Skew^2\right) = 6.31 \quad in$$

$$N3_3 := \left[4 + .02L + .08HCol_{Bent3} + 1.09\sqrt{HCol_{Bent3}}\cdot\sqrt{1 + \left(2\cdot\frac{3}{8}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1a}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 17.37 \quad in$$

$$N4_3 := \left[4 + .02L + .08HCol_{Bent3} + 1.09\sqrt{HCol_{Bent3}} \cdot \sqrt{1 + \left(2 \cdot \frac{3}{8}\right)^2}\right] \cdot \left(\frac{1 + 1.25 S_{D1b}}{\cos\left(\frac{Skew \cdot \pi}{180}\right)}\right) = 20.113 \quad in$$

Scarham Creek

$L := 130$	ft	Largest Span Length
$B := 40$	ft	Deck With
$HCol_{Bent2} := 34$	ft	Average Column Height at Bent 2
$HCol_{Bent3} := 59$	ft	Average Column Height at Bent 3
$HCol_{Bent4} := 32$	ft	Average Column Height at Bent 4
$Skew := 0$	deg	Skew of Bridge
$\Delta_{Bent2} := 1$	in	Longitudinal Displacement of Bridge Bent
$\Delta_{Bent3} := 1$	in	Longitudinal Displacement of Bridge Bent
$\Delta_{Bent4} := 1$	in	Longitudinal Displacement of Bridge Bent

Bent 2

$$N1_2 := 1.5\left(8 + 0.02L + 0.08HCol_{Bent2}\right) \cdot \left(1 + 0.000125Skew^2\right) = 19.98 \quad in$$

$$N2_2 := \left(4 + 1.65\Delta_{Bent2}\right) \cdot \left(1 + 0.00025Skew^2\right) = 5.65 \quad in$$

$$N3_2 := \left[4 + .02L + .08HCol_{Bent2} + 1.09\sqrt{HCol_{Bent2}} \cdot \sqrt{1 + \left(2 \cdot \frac{B}{L}\right)^2}\right] \cdot \left(\frac{1 + 1.25 S_{D1a}}{\cos\left(\frac{Skew \cdot \pi}{180}\right)}\right) = 19.93 \quad in$$

$$N4_2 := \left[4 + .02L + .08HCol_{Bent2} + 1.09\sqrt{HCol_{Bent2}} \cdot \sqrt{1 + \left(2 \cdot \frac{B}{L}\right)^2}\right] \cdot \left(\frac{1 + 1.25 S_{D1b}}{\cos\left(\frac{Skew \cdot \pi}{180}\right)}\right) = 23.076 \quad in$$

Bent 3

$$N1_3 := 1.5\left(8 + 0.02L + 0.08HCol_{Bent3}\right) \cdot \left(1 + 0.000125Skew^2\right) = 22.98 \quad in$$

$$N2_3 := \left(4 + 1.65\Delta_{Bent3}\right)\cdot\left(1 + 0.00025Skew^2\right) = 5.65 \quad in$$

$$N3_3 := \left[4 + .02L + .08HCol_{Bent3} + 1.09\sqrt{HCol_{Bent3}}\cdot\sqrt{1 + \left(2\cdot\frac{B}{L}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1a}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 25.117 \quad in$$

$$N4_3 := \left[4 + .02L + .08HCol_{Bent3} + 1.09\sqrt{HCol_{Bent3}}\cdot\sqrt{1 + \left(2\cdot\frac{B}{L}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1b}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 29.082 \quad in$$

Bent 4

$$N1_4 := 1.5\left(8 + 0.02L + 0.08HCol_{Bent4}\right)\cdot\left(1 + 0.000125Skew^2\right) = 19.74 \quad in$$

$$N2_4 := \left(4 + 1.65\Delta_{Bent4}\right)\cdot\left(1 + 0.00025Skew^2\right) = 5.65 \quad in$$

$$N3_4 := \left[4 + .02L + .08HCol_{Bent4} + 1.09\sqrt{HCol_{Bent4}}\cdot\sqrt{1 + \left(2\cdot\frac{B}{L}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1a}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 19.475 \quad in$$

$$N4_4 := \left[4 + .02L + .08HCol_{Bent4} + 1.09\sqrt{HCol_{Bent4}}\cdot\sqrt{1 + \left(2\cdot\frac{B}{L}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1b}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 22.55 \quad in$$

Appendix B: County Road 39 Bridge SDC A1

Designer: Jordan Law
Project Name: County Road 39
Job Number: ST-049-039-001
Date: 5/24/2012

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

Coordinates: 30.512N, 88.227W

Soil Site Class: D

Superstructure Type: AASTHO BT-72 girders for long spans
AASTHO Type III girders for simple span

Substructure Type: Rectangular columns supported on piles

Abutment Type: Abutment beam supported on piles

Note: ***Input** all of the below information.*

The designer should input any information that can be used to calculate the dead weight of the bridge, including but not limited to length of bridge, column height(s), deck thickness, bent volume(s), and guard rail volume(s). Also, information about foundations should also be included if the bridge is classified as SDC B.

INPUT

$fc := 400($ psi

$fye := 6000($ psi

$\rho_{conc} := 0.0868 \ \frac{lb}{in^3}$

$g := 386.4$

$A_s := .045$

$S_{D1} := .074$

$S_{DS} := .10($

SDC := "A"

Length of Bridge (ft)	$L := 485$	ft
Skew of Bridge (degrees)	$Skew := 0$	degrees
Span Length 1 (ft)	$Span1 := 135$	ft
Span Length 2 (ft)	$Span2 := 8($	ft
Deck Thickness (in)	$t_{deck} := 7$	in
Deck Width (ft)	$DeckWidth := 54.75$	ft

Number of Bridge Girders $N := 9$

Girder Type III X-Sectional Area (in^2) $GirderIIIArea := 559.5$ in^2

Bulb Tee Girder X-Sectional Area (in^2) $BulbTeeArea := 767$

Bent 2 and 3 Volume (ft^3) $BentVolume_{23} := 4.5 \cdot 4 \cdot 53 = 954$ ft^3

Bent 4 Volume (ft^3) $BentVolume_{4} := 4.5 \cdot 4 \cdot 53 = 954$ ft^3

Guard Rail Area (in^2) $GuardRailArea := 310$ in^2

Bent 2, 3, 4 Column Width (in) $Columnwidth_{Bent234} := 45$ in

Number of Columns per Bent $N_{col} := 3$

The column height is measured from the bottom of the bent to the top of the pile footing. Other options include measuring from the top of the bent to the ground surface or to a change in diameter (if possible). If the plastic hinge location (at the bottom of the column) is known, then the column height should be measured from the bottom of the bent to the known hinge point.

Average Column Height Bent 2 (ft) $ColumnHeight_{Bent2} := 23.606$ ft

Average Column Height Bent 3 (ft) $ColumnHeight_{Bent3} := 28.843$ ft

Average Column Height Bent 4 (ft) $ColumnHeight_{Bent4} := 26.63$ ft

Bent Column Area (in^2) $Acolumn_{Bent234} := Columnwidth_{Bent234}^{2} = 2.025 \times 10^{3}$ in^2

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

$L := L \cdot 12 = 5.82 \times 10^{3}$ in

$Span1 := Span1 \cdot 12 = 1.62 \times 10^{3}$ in

$Span2 := Span2 \cdot 12 = 960$ in

$DeckWidth := DeckWidth \cdot 12 = 657$ in

$BentVolume_{23} := BentVolume_{23} \cdot 12^{3} = 1.649 \times 10^{6}$ in^3

$BentVolume_{4} := BentVolume_{4} \cdot 12^{3} = 1.649 \times 10^{6}$ in^3

$ColumnHeight_{Bent2} := ColumnHeight_{Bent2} \cdot 12 = 283.272$ in

$ColumnHeight_{Bent3} := ColumnHeight_{Bent3} \cdot 12 = 346.116$ in

$ColumnHeight_{Bent4} := ColumnHeight_{Bent4} \cdot 12 = 319.56$ in

Find Vertical Reactions at Each Bent:

Live Loads assumed to be present during an earthquake (see ***LRFD Article 3.4.1***)

$$\text{Num\_Lanes} := \text{trunc}\left(\frac{\frac{\text{DeckWidth}-2\cdot 1.375}{12}}{12}\right) = 4$$

Number of Lanes On Bridge (Design Lane Width of 10 ft) See ***LRFD 3.6.1.2.4***

$\gamma_{EQ} := 0.5$ ***LRFD Specificaiton C3.4.1 (Extreme Case I)*** **INPUT**

The γ_{EQ} value is to be determined on a project-specific basis. In the standard specification, a value of 0.0 was used, however, the LRFD Specification recommends a value of 0.5. See LRFD Article C3.4.1 under "EXTREME EVENT I"

$\text{LL\_design} := 0.64 \ \frac{\text{klf}}{\text{lane}}$ ***LRFD Specification 3.6.1.2.4***

$Q := \text{LL\_design}\cdot\gamma_{EQ} = 0.32 \ \frac{\text{klf}}{\text{lane}}$

$\text{LL\_foot} := Q\cdot\text{Num\_Lanes} = 1.28$ klf Live Load per linear foot of deck (includes all lanes)

Note: If the Vertical Reactions at each bent are already known, input them below, otherwise the sheet will calculate vertical reactions based on the given information above.

$DL_{Bent2} :=$ ▪ kip $LL_{Bent2} :=$ ▪ kip

$DL_{Bent3} :=$ ▪ kip $LL_{Bent3} :=$ ▪ kip **INPUT**

$DL_{Bent4} :=$ ▪ kip $LL_{Bent4} :=$ ▪ kip

$VR_{Bent2} := DL_{Bent2} + LL_{Bent2} =$ ▪ kip

$VR_{Bent3} := DL_{Bent3} + LL_{Bent3} =$ ▪ kip

$VR_{Bent4} := DL_{Bent4} + LL_{Bent4} =$ ▪ kip

The weight calculation takes into account the entire dead weight of the structure, including the deck, bents, abutments, columns, girders, and railings. Any other expected dead loads should also be included.

$$W := \frac{\rho_{conc} \cdot \begin{pmatrix} L \cdot t_{deck} \cdot DeckWidth \ldots \\ + 2 \cdot BentVolume_{23} + BentVolume_{4} \ldots \\ + 3 \cdot Acolumn_{Bent234} \cdot ColumnHeight_{Bent2} \ldots \\ + 3Acolumn_{Bent234} \cdot ColumnHeight_{Bent3} \ldots \\ + 3Acolumn_{Bent234} \cdot ColumnHeight_{Bent4} \ldots \\ + 3 \cdot Span1 \cdot N \cdot GirderIIIArea + Span2 \cdot N \cdot BulbTeeArea + 2 \cdot GuardRailArea \cdot L \end{pmatrix}}{1000}$$

$W = 6266.326$ kips

To determine the vertical reaction at the bent, the bents tributary area will be calculated and multiplied by the total weight. A similar calculation will be done for the live load. This vertical reaction will be used to determine the connection force (below).

$BentTribLength_{23} := \frac{Span1}{12} = 135$ ft

$BentTribLength_{4} := \frac{\frac{Span1 + Span2}{2}}{12} = 107.5$ ft

$BentTribArea_{23} := \frac{Span1}{L} = 0.278$ Percent of Area Tributary to Bent

$BentTribArea_{4} := \frac{\frac{Span1 + Span2}{2}}{L} = 0.222$

$DL_{Bent23} := BentTribArea_{23} \cdot W = 1744.235$ kip

$LL_{Bent23} := BentTribLength_{23} \cdot LL\_foot = 172.8$ kip

$DL_{Bent4} := BentTribArea_{4} \cdot W = 1388.928$ kip

$LL_{Bent4} := BentTribLength_{4} \cdot LL\_foot = 137.6$ kip

$VR_{Bent23} := DL_{Bent23} + LL_{Bent23} = 1917.035$ kip

$VR_{Bent4} := DL_{Bent4} + LL_{Bent4} = 1526.528$ kip

$VR_{Bent2} := VR_{Bent23}$ $VR_{Bent3} := VR_{Bent23}$ $VR_{Bent4} := VR_{Bent4}$

<u>Steps for Seismic Design</u>

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

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

Article 3.4: Determine Deisgn Response Spectrum

Article 3.5: Determine SDC

Article 4.6: Determine Design Forces

Article 4.12: Determine Minimum Support Length

Article 8.2: Column Detailing

Articles 3.4 and 3.5 have already been determined from the "SDC Classification" sheet. Make sure the four values (A_s, S_{DS}, S_{D1}, and SDC) have been input above.

Bent 2 Design

Reinforcement Information ***Guide Article 8.6.5***

The designer should input all information concerning the longitudinal and transverse reinforcement of the column, specifically within the plastic hinge zone. Both circular and rectangular columns are allowed.

$d_{bl} := 1.41$	in	Diameter of Longitudinal Reinforcement
Stirrup := "#4"		Stirrup Size
$A_{sp} := .20$	in^2	Area of Transverse Reinforcement
$D_{sp} := 0.625$	in	Diameter of Transverse Reinforcement
Dprime := 0	in	Diameter of Spiral or Hoop for Circular Columns
s := 8	in	Pitch of Spiral or Spacing of Hoops/Ties for Circular Columns
b := $Columnwidth_{Bent234}$	in	Width of Rectangular Column
Cover := 2	in	Column Concrete Cover

INPUT

Article 4.6: Determine Design Forces

The Guide Specification requires only a minimum design force for SDC A. This design force is based on the tributary dead load and live load assumed to be present during an earthquake.

$$\text{DesignForce}(A_s) := \left| \begin{array}{l} A \leftarrow A_s \\ a \leftarrow 0.15 \text{ if } A < 0.05 \\ a \leftarrow 0.25 \text{ if } A \geq 0.05 \\ a \end{array} \right.$$

$$\text{VR\_Multiplier} := \text{DesignForce}(A_s) = 0.15$$

$$\text{HorizontalDesignForce} := \frac{VR_{Bent2} \cdot \text{VR\_Multiplier}}{N} = 31.951 \qquad \text{kip}$$

$$\text{HorizontalDesignForce2} := \frac{DL_{Bent23} \cdot \text{VR\_Multiplier}}{N} = 29.071 \qquad \text{kip}$$

Article 4.12: Determine Minimum Support Length

$$L := \text{BentTribLength}_{23} = 135$$

$$H := \frac{\text{ColumnHeight}_{Bent2}}{12} = 23.606$$

Standard Specifications

$$N_{2Stan} := (8 + 0.02 \cdot L + 0.08 \cdot H) \cdot \left(1 + 0.000125 \cdot \text{Skew}^2\right) = 12.588 \qquad \text{in}$$

ATC-49 Equation (New Design)

$$N_2 := \left[4 + .02 \cdot L + .08 \cdot H + 1.09 \cdot \sqrt{H} \cdot \sqrt{1 + \left(2 \cdot \frac{3}{8}\right)^2}\right] \cdot \left(\frac{1 + 1.25 \cdot S_{D1}}{\cos\left(\frac{\text{Skew} \cdot \pi}{180}\right)}\right) = 16.615 \qquad \text{in}$$

Article 8.2: Column Detailing

Note: If SD1 is greater than or equal to 0.10, the minimum requirements from SDC B must be met. Otherwise, no minimum column detailing is required and the checks below can be ignored.

$$\text{SD1Check}(SD1) := \left| \begin{array}{l} a \leftarrow \text{"Minimum SDC B Detailing Required" if } SD1 \geq 0.10 \\ a \leftarrow \text{"No SDC B Detailing Required" otherwise} \\ a \end{array} \right.$$

$$\text{Bent2} := \text{SD1Check}(S_{D1}) = \text{"No SDC B Detailing Required"}$$

LRFD 5.8.3.3 Nominal Shear Resistance — *Guide Article 8.6.1*

$$V_u := \text{HorizontalDesignForce} \cdot \frac{N}{N_{col}} = 95.852 \qquad \text{kips}$$

$$\phi_s := 0.9$$

$\beta := 2.0$

LRFD Article 5.8.3.4.1

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

$Dr := b - Cover - D_{sp} - \frac{d_{bl}}{2} = 41.67$ in

$de := b - Cover = 43$ in

LRFD Eq. 5.8.2.9-1

$dv := 0.9 de = 38.7$ in

$V_c := 0.0316 \beta \cdot \sqrt{\frac{fc}{1000}} b \cdot dv = 220.126$ kips

LRFD Eq. 5.8.3.3-3

$V_s := \frac{2A_{sp} \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{s} = 116.1$ kips

LRFD Eq. 5.8.3.3-4

$V_n := \phi_s \cdot .25 fc \cdot b \cdot dv = 1.567 \times 10^6$ kips

$\phi Vn := \min\left[V_n, \left(V_c + V_s\right) \cdot \phi_s\right] = 302.603$ kips

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

$Shearcheck2 := ShearCheck\left(\phi Vn, V_u\right) = \text{"OK"}$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

$s :=$

LRFD Eq. 5.8.2.5-1

$Avmin := 0.0316 \sqrt{\frac{fc}{1000}} \cdot \frac{b \cdot s}{\frac{fye}{1000}} = 0.379$ in^2

$Av := 2A_{sp} = 0.4$ in^2

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

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

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.

LRFD 5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

$$vu := \frac{V_u}{\phi_s \cdot b \cdot dv} = 0.061 \quad \text{ksi}$$ ***LRFD Eq. 5.8.2.9-1***

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

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

$$MaxSpacing := floor(spacingProgram(vu, dv, fc)) = 24 \quad \text{in}$$

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$Spacecheck(MaxSpacing, s, V_u, V_c) := \begin{array}{|l} a \leftarrow min(MaxSpacing, 12) \ \text{ if } V_u \le 0.5 0.9 V_c \\ a \leftarrow min(s, MaxSpacing, 12) \ \text{ otherwise} \\ a \end{array}$$

$$s := Spacecheck(MaxSpacing, s, V_u, V_c) = 12 \quad \text{in}$$

Design Summary - Bent 2

Stirrup = "#4"

$s = 12$ in

$N_2 = 16.615$ in

Design Check Summary - Bent 2

Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran= "OK"	Minimum shear reinforcement outside hinge zone

Bent 3 Design

Reinforcement Information

Guide Article 8.6.5

The designer should input all information concerning the longitudinal and transverse reinforcement of the column, specifically within the plastic hinge zone. Both circular and rectangular columns are allowed.

INPUT

$d_{bl} := 1.41$	in	Diameter of Longitudinal Reinforcement
Stirrup := "#4"		Stirrup Type
$A_{sp} := .20$	in^2	Area of Transverse Reinforcement
$D_{sp} := 0.625$	in	Diameter of Transverse Reinforcement
Dprime := 0	in	Diameter of Spiral or Hoop for Circular Columns
$s := 8$	in	Pitch of Spiral or Spacing of Hoops/Ties for Circular Columns
$b := Columnwidth_{Bent234}$	in	Width of Rectangular Column
Cover := 2	in	Column Concrete Cover

Article 4.6: Determine Design Forces

The Guide Specification requires only a minimum design force for SDC A. This design force is based on the tributary dead load and live load assumed to be present during an earthquake.

$$\text{DesignForce}(A_s) := \left| \begin{array}{l} A \leftarrow A_s \\ a \leftarrow 0.15 \text{ if } A < 0.05 \\ a \leftarrow 0.25 \text{ if } A \geq 0.05 \\ a \end{array} \right.$$

$$\text{VR\_Multiplier} := \text{DesignForce}(A_s) = 0.15$$

$$\text{HorizontalDesignForce} := \frac{VR_{Bent3} \cdot \text{VR\_Multiplier}}{N} = 31.951 \qquad \text{kip}$$

$$\text{HorizontalDesignForce2} := \frac{DL_{Bent23} \cdot \text{VR\_Multiplier}}{N} = 29.071 \qquad \text{kip}$$

Article 4.12: Determine Minimum Support Length

$$L := \text{BentTribLength}_{23} = 135$$

$$H := \frac{\text{ColumnHeight}_{Bent3}}{12} = 28.843$$

Standard Specifications

$$N_{3Stan} := (8 + 0.02 \cdot L + 0.08 \cdot H) \cdot \left(1 + 0.000125 \cdot \text{Skew}^2\right) = 13.007 \qquad \text{in}$$

ATC-49 Equation (New Design)

$$N_3 := \left[4 + .02 \cdot L + .08 \cdot H + 1.09 \cdot \sqrt{H} \cdot \sqrt{1 + \left(2 \cdot \frac{3}{8}\right)^2}\right] \cdot \left(\frac{1 + 1.25 \cdot S_{D1}}{\cos\left(\frac{\text{Skew} \cdot \pi}{180}\right)}\right) = 17.835 \qquad \text{in}$$

Article 8.2: Column Detailing

Note: If SD1 is greater than or equal to 0.10, the minimum requirements from SDC B must be met. Otherwise, no minimum column detailing is required and the checks below can be ignored.

$$\text{Bent3} := \text{SD1Check}(S_{D1}) = \text{"No SDC B Detailing Required"}$$

LRFD 5.8.3.3* Nominal Shear Resistance** ***Guide Article 8.6.1

$$V_u := \text{HorizontalDesignForce} \cdot \frac{N}{N_{col}} = 95.852 \qquad \text{kips}$$

$$\phi_s := 0.9$$

$\beta := 2.0$

LRFD Article 5.8.3.4.1

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

$Dr := b - Cover - D_{sp} - \frac{d_{bl}}{2} = 41.67$ in

$de := b - Cover = 43$ in

LRFD Eq. 5.8.2.9-1

$dv := 0.9de = 38.7$ in

$V_c := 0.0316\beta \cdot \sqrt{\frac{fc}{1000}} b \cdot dv = 220.126$ kips

LRFD Eq. 5.8.3.3-3

$V_s := \frac{2A_{sp} \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{s} = 116.1$ kips

LRFD Eq. 5.8.3.3-4

$V_n := \phi_s \cdot .25 fc \cdot b \cdot dv = 1.567 \times 10^6$ kips

$\phi Vn := \min\left[V_n, \left(V_c + V_s\right) \cdot \phi_s\right] = 302.603$ kips

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

$Shearcheck2 := ShearCheck\left(\phi Vn, V_u\right) = \text{"OK"}$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

LRFD Eq. 5.8.2.5-1

$Avmin := 0.0316 \sqrt{\frac{fc}{1000}} \cdot \frac{b \cdot s}{\frac{fye}{1000}} = 0.379$ in^2

$Av := 2A_{sp} = 0.4$ in^2

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

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

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.

LRFD 5.8.2.7 Maximum Spacing of Transverse Reinforcement

$$vu := \frac{V_u}{\phi_s \cdot b \cdot dv} = 0.061 \quad \text{ksi}$$

LRFD Eq. 5.8.2.9-1

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

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

$$\text{MaxSpacing} := \text{floor}(\text{spacingProgram}(vu, dv, fc)) = 24 \quad \text{in}$$

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$\text{Spacecheck}(\text{MaxSpacing}, s, V_u, V_c) := \left| \begin{array}{l} a \leftarrow \min(\text{MaxSpacing}, 12) \quad \text{if } V_u \le 0.5\,0.9\,V_c \\ a \leftarrow \min(s, \text{MaxSpacing}, 12) \quad \text{otherwise} \\ a \end{array} \right.$$

$s := \text{Spacecheck}(\text{MaxSpacing}, s, V_u, V_c) = 12$ in

Design Summary - Bent 3

Stirrup = "#4"

$s = 12$ in

$N_3 = 17.835$ in

Design Check Summary - Bent 3

Shearcheck2 = "OK" Shear capacity outside hinge zone > Vn

MinimumTran = "OK" Minimum shear reinforcement outside hinge zone

Bent 4 Design

Reinforcement Information ***Guide Article 8.6.5***

The designer should input all information concerning the longitudinal and transverse reinforcement of the column, specifically within the plastic hinge zone. Both circular and rectangular columns are allowed.

$d_{bl} := 1.41$	in	Diameter of Longitudinal Reinforcement
Stirrup := "#4"		Stirrup Type
$A_{sp} := .20$	in^2	Area of Transverse Reinforcement
$D_{sp} := 0.625$	in	Diameter of Transverse Reinforcement
Dprime := 0	in	Diameter of Spiral or Hoop for Circular Columns
$s := 8$	in	Pitch of Spiral or Spacing of Hoops/Ties for Circular Columns
$b := \text{Columnwidth}_{Bent234}$	in	Width of Rectangular Column
Cover := 2	in	Column Concrete Cover

INPUT

Article 4.6: Determine Design Forces

The Guide Specification requires only a minimum design force for SDC A. This design force is based on the tributary dead load and live load assumed to be present during an earthquake.

$$\text{DesignForce}(A_s) := \left| \begin{array}{l} A \leftarrow A_s \\ a \leftarrow 0.15 \text{ if } A < 0.05 \\ a \leftarrow 0.25 \text{ if } A \geq 0.05 \\ a \end{array} \right.$$

$$\text{VR\_Multiplier} := \text{DesignForce}(A_s) = 0.15$$

$$\text{HorizontalDesignForce} := \frac{VR_{Bent4} \cdot \text{VR\_Multiplier}}{N} = 25.442 \qquad \text{kip}$$

$$\text{HorizontalDesignForce2} := \frac{DL_{Bent4} \cdot \text{VR\_Multiplier}}{N} = 23.149 \qquad \text{kip}$$

Article 4.12: Determine Minimum Support Length

$$L := \text{BentTribLength}_4 = 107.5$$

$$H := \frac{\text{ColumnHeight}_{Bent4}}{12} = 26.63$$

Standard Specifications

$$N_{4Stan} := (8 + 0.02 L + 0.08 H) \cdot \left(1 + 0.000125 \text{Skew}^2\right) = 12.28 \qquad \text{in}$$

ATC-49 Equation (New Design)

$$N_4 := \left[4 + .02 L + .08 H + 1.09\sqrt{H} \cdot \sqrt{1 + \left(2 \cdot \frac{3}{8}\right)^2}\right] \cdot \left(\frac{1 + 1.25 S_{D1}}{\cos\left(\frac{\text{Skew} \cdot \pi}{180}\right)}\right) = 16.728 \qquad \text{in}$$

Article 8.2: Column Detailing

Note: If SD1 is greater than or equal to 0.10, the minimum requirements from SDC B must be met. Otherwise, no minimum column detailing is required and the checks below can be ignored.

$$\text{Bent4} := \text{SD1Check}(S_{D1}) = \text{"No SDC B Detailing Required"}$$

LRFD 5.8.3.3* Nominal Shear Resistance** ***Guide Article 8.6.1

$$V_u := \text{HorizontalDesignForce} \cdot \frac{N}{N_{col}} = 76.326 \qquad \text{kips}$$

$$\phi_s := 0.9$$

$$\beta := 2.0$$

LRFD Article 5.8.3.4.1

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

$Dr := b - Cover - D_{sp} - \frac{d_{bl}}{2} = 41.67$ in

$de := b - Cover = 43$ in

LRFD Eq. 5.8.2.9-1

$dv := 0.9de = 38.7$ in

$V_c := 0.0316\beta \cdot \sqrt{\frac{fc}{1000}} b \cdot dv = 220.126$ kips

LRFD Eq. 5.8.3.3-3

$V_s := \frac{2A_{sp} \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{s} = 116.1$ kips

LRFD Eq. 5.8.3.3-4

$V_n := \phi_s \cdot .25 fc \cdot b \cdot dv = 1.567 \times 10^6$ kips

$\phi Vn := \min\left[V_n, \left(V_c + V_s\right) \cdot \phi_s\right] = 302.603$ kips

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

$\text{Shearcheck2} := \text{ShearCheck}\left(\phi Vn, V_u\right) = \text{"OK"}$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

LRFD Eq. 5.8.2.5-1

$Avmin := 0.0316 \sqrt{\frac{fc}{1000}} \cdot \frac{b \cdot s}{\frac{fye}{1000}} = 0.379$ in^2

$Av := 2A_{sp} = 0.4$ in^2

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

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

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.

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

$$vu := \frac{V_u}{\phi_s \cdot b \cdot dv} = 0.049 \qquad \text{ksi}$$

LRFD Eq. 5.8.2.9-1

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

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

$$\text{MaxSpacing} := \text{floor}(\text{spacingProgram}(\text{vu}, \text{dv}, \text{fc})) = 24 \qquad \text{in}$$

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$\text{Spacecheck}\left(\text{MaxSpacing}, s, V_u, V_c\right) := \begin{array}{|l} a \leftarrow \min(\text{MaxSpacing}, 12) \quad \text{if } V_u \le 0.5\,0.9 V_c \\ a \leftarrow \min(s, \text{MaxSpacing}, 12) \quad \text{otherwise} \\ a \end{array}$$

$$s := \text{Spacecheck}\left(\text{MaxSpacing}, s, V_u, V_c\right) = 12 \qquad \text{in}$$

Design Summary - Bent 4

Stirrup = "#4"

$s = 12$ in

$N_4 = 16.728$ in

Design Check Summary - Bent 4

Shearcheck2 = "OK" Shear capacity outside hinge zone > Vn

MinimumTran= "OK" Minimum shear reinforcement outside hinge zone

Appendix C: Stave Creek Bridge SDC A1

Designer: Jordan Law

$ORIGIN := 1$

Project Name: Stave Creek Bridge
Job Number: BR-0069 (501)
Date: 5/24/2012
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.

<u>Project Known Information</u>

Coordinates: 31.551N, 87.930W

Soil Site Class: D

Superstructure Type: AASTHO Type I girders for end spans
AASTHO Type III girders for middle span

Substructure Type: Rectangular columns supported on piles

Abutment Type: Abutment beam supported on drilled shafts

Note: ***Input*** *all of the below information*.

The designer should input any information that can be used to calculate the dead weight of the bridge, including but not limited to length of bridge, column height(s), deck thickness, bent volume(s), and guard rail volume(s). Also, information about foundations should also be included if the bridge is classified as SDC B.

INPUT

$fc := 4000$ psi

$fye := 60000$ psi

$\rho_{conc} := 0.0868 \ \frac{lb}{in^3}$

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

$A_s := .076$

$S_{D1} := .086$

$S_{DS} := .104$

$SDC := "A"$

Length of Bridge (ft)	$L := 165$	ft
Skew of Bridge (degrees)	$Skew := 0$	degrees
End Spans (ft)	$EndSpan := 40$	ft
Middle Span (ft)	$MidSpan := 85$	ft
Deck Thickness (in)	$t_{deck} := 7$	in
Deck Width (ft)	$DeckWidth := 42.75$	ft

Number of Bridge Girders $N := 6$

I-Girder (AASHTO Type III) X-Sectional Area (in^2) $IGirderIIIArea := 559.5$ in^2

I-Girder (AASHTO Type I) X-Sectional Area (in^2) $IGirderIArea := 276$ in^2

Guard Rail Area (in^2) $GuardRailArea := 310$ in^2

Bent Volume (ft^3) $BentVolume := 40 \cdot (3.75 \cdot 4 + 1.583 \cdot 1.65) = 704.478$ ft^3

Column Diameter (in) $Columnwidth := 36$ in

Number of Columns per Bent $N_{col} := 2$

Drilled Shaft Diameter (in) $DSdia := 60$ in

Drilled Shaft Abutment Diameter (in) $DSabutdia := 42$ in

The column height is measured from the bottom of the bent to the top of the pile footing. Other options include measuring from the top of the bent to the ground surface or to a change in diameter
(if possible). If the plastic hinge location (at the bottom of the column) is known, then the column height should be measured from the bottom of the bent to the known hinge point.

Average Column Height for Bent 2 (ft) $ColumnHeight_{Bent2} := 10.20$ ft

Average Column Height for Bent 3 (ft) $ColumnHeight_{Bent3} := 14.34$ ft

Column Area (in^2) $Acolumn := Columnwidth^2 = 1.296 \times 10^3$ in^2

Drilled Shaft Area (in^2) $Adrilledshaft := \frac{DSdia^2 \cdot \pi}{4} = 2.827 \times 10^3$ 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.

$EndSpan := EndSpan \cdot 12 = 480$ in

$MidSpan := MidSpan \cdot 12 = 1.02 \times 10^3$ in

$L := L \cdot 12 = 1.98 \times 10^3$ in

$DeckWidth := DeckWidth \cdot 12 = 513$ in

$BentVolume := BentVolume \cdot 12^3 = 1.217 \times 10^6$ in^3

$ColumnHeight_{Bent2} := ColumnHeight_{Bent2} \cdot 12 = 122.448$ in

$ColumnHeight_{Bent3} := ColumnHeight_{Bent3} \cdot 12 = 172.08$ in

Find Vertical Reactions at Each Bent:

Live Loads assumed to be present during an earthquake (see ***LRFD Article 3.4.1***)

$$Num\_Lanes := trunc\left(\frac{\frac{DeckWidth - 2 \cdot 1.375}{12}}{12}\right) = 3$$

Number of Lanes On Bridge (Design Lane Width of 10 ft) See ***LRFD 3.6.1.2.4***

$\gamma_{EQ} := 0.5$ ***LRFD Specificaiton C3.4.1 (Extreme Case I)*** **INPUT**

The γ_{EQ} value is to be determined on a project-specific basis. In the standard specification, a value of 0.0 was used, however, the LRFD Specification recommends a value of 0.5. See LRFD Article C3.4.1 under "EXTREME EVENT I"

$LL\_design := 0.64 \ \frac{klf}{lane}$ ***LRFD Specification 3.6.1.2.4***

$Q := LL\_design \cdot \gamma_{EQ} = 0.32 \ \frac{klf}{lane}$

$LL\_foot := Q \cdot Num\_Lanes = 0.96$ klf Live Load per linear foot of deck (includes all lanes)

Note: If the Vertical Reactions at each bent are already known, input them below, otherwise the sheet will calculate vertical reactions based on the given information above.

$DL_{Bent2} :=$ ▪ kip $LL_{Bent2} :=$ ▪ kip

$DL_{Bent3} :=$ ▪ kip $LL_{Bent3} :=$ ▪ kip **INPUT**

$VR_{Bent2} := DL_{Bent2} + LL_{Bent2} =$ ▪ kip

$VR_{Bent3} := DL_{Bent3} + LL_{Bent3} =$ ▪ kip

The weight calculation takes into account the entire dead weight of the structure, including the deck, bents, abutments, columns, girders, and railings. Any other expected dead loads should also be included.

$$W := \frac{\rho_{conc} \cdot \left(\begin{array}{l} L \cdot t_{deck} \cdot DeckWidth + 2 \cdot BentVolume + 2 \cdot Acolumn \cdot ColumnHeight_{Bent2} \ldots \\ + 2 \cdot Acolumn \cdot ColumnHeight_{Bent3} + 2 EndSpan \cdot N \cdot IGirderIArea \ldots \\ + N \cdot MidSpan \cdot IGirderIIIArea + 2 \cdot GuardRailArea \cdot L \end{array}\right)}{1000}$$

$W = 1436.686$ kips

Note: An elevation view of the bridge shows that the tributary area for Bents 2 and 3 are identical, and therefore the tributary weights will be equal. The information below should be adjusted for different bridges.

$$BentTribLength := \frac{\frac{EndSpan + MidSpan}{2}}{12} = 62.5 \qquad ft$$

$$BentTribArea := \frac{\frac{EndSpan + MidSpan}{2}}{L} = 0.379$$ Percent of Area Tributary to Bent

$$DL_{Bent} := BentTribArea \cdot W = 544.199 \qquad kip$$

$$LL_{Bent} := BentTribLength \cdot LL\_foot = 60 \qquad kip$$

$$VR_{Bent} := DL_{Bent} + LL_{Bent} = 604.199 \qquad kip$$

$$VR_{Bent2} := VR_{Bent} \qquad VR_{Bent3} := VR_{Bent}$$

Steps for Seismic Design

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

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

Article 3.4: Determine Deisgn Response Spectrum

Article 3.5: Determine SDC

Article 4.6: Determine Design Forces

Article 4.12: Determine Minimum Support Length

Article 8.2: Column Detailing

Articles 3.4 and 3.5 have already been determined from the "SDC Classification" sheet. Make sure the four values (A_S, S_{DS}, S_{D1}, and SDC) have been input above.

Bent 2 Design

The designer should input all information concerning the longitudinal and transverse reinforcement of the column, specifically within the plastic hinge zone. Both circular and rectangular columns are allowed.

Reinforcement Information

$d_{bl} := 1.41$	in	Diameter of Longitudinal Reinforcement
Stirrup := "#4"		Stirrup Type
$A_{sp} := .20$	in^2	Area of Transverse Reinforcement
$D_{sp} := 0.625$	in	Diameter of Transverse Reinforcement
Dprime := 0	in	Diameter of Spiral or Hoop for Circular Columns **INPUT**
s := 10	in	Pitch of Spiral or Spacing of Hoops/Ties for Circular Columns
b := Columnwidth	in	Width of Rectangular Column
Cover := 2	in	Column Concrete Cover

Article 4.6: Determine Design Forces

The Guide Specification requires only a minimum design force for SDC A. This design force is based on the tributary dead load and live load assumed to be present during an earthquake.

$$\text{DesignForce}(A_s) := \left| \begin{array}{l} A \leftarrow A_s \\ a \leftarrow 0.15 \text{ if } A < 0.05 \\ a \leftarrow 0.25 \text{ if } A \geq 0.05 \\ a \end{array} \right.$$

$$\text{VR\_Multiplier} := \text{DesignForce}(A_s) = 0.25$$

$$\text{HorizontalDesignForce} := \frac{VR_{Bent2} \cdot \text{VR\_Multiplier}}{N} = 25.175 \qquad \text{kip}$$

$$\text{HorizontalDesignForce2} := \frac{DL_{Bent} \cdot \text{VR\_Multiplier}}{N} = 22.675 \qquad \text{kip}$$

Article 4.12: Determine Minimum Support Length

$$L := \text{BentTribLength} = 62.5$$

$$H := \frac{\text{ColumnHeight}_{Bent2}}{12} = 10.204$$

Standard Specifications

$$N_{2Stan} := (8 + 0.02 L + 0.08 H) \cdot \left(1 + 0.000125 \text{Skew}^2\right) = 10.066 \qquad \text{in}$$

ATC-49 Equation (New Design)

$$N_2 := \left[4 + .02L + .08H + 1.09\sqrt{H}\cdot\sqrt{1 + \left(2\cdot\frac{3}{8}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 11.539 \qquad in$$

Article 8.2: Column Detailing

Note: If SD1 is greater than or equal to 0.10, the minimum requirements from SDC B must be met. Otherwise, no minimum column detailing is required and the checks below do not apply.

$$SD1Check(SD1) := \left| \begin{array}{l} a \leftarrow \text{"Minimum SDC B Detailing Required" if } SD1 \geq 0.10 \\ a \leftarrow \text{"No Minimum SDC B Detailing Required" otherwise} \\ a \end{array} \right.$$

$$SD1Check(S_{D1}) = \text{"No Minimum SDC B Detailing Required"}$$

LRFD 5.8.3.3 **Nominal Shear Resistance** ***Guide Article 8.6.1***

$$V_u := HorizontalDesignForce\cdot\frac{N}{N_{col}} = 75.525 \qquad kips$$

$$\phi_s := 0.9$$

$$\beta := 2.0$$

LRFD Article 5.8.3.4.1

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

$$Dr := b - Cover - D_{sp} - \frac{d_{bl}}{2} = 32.67 \qquad in$$

$$de := b - Cover = 34 \qquad in$$

LRFD Eq. 5.8.2.9-1

$$dv := 0.9de = 30.6 \qquad in$$

LRFD Eq. 5.8.3.3-3

$$V_c := 0.0316\beta\cdot\sqrt{\frac{fc}{1000}}\,b\cdot dv = 139.242 \qquad kips$$

LRFD Eq. 5.8.3.3-4

$$V_s := \frac{2A_{sp}\cdot\frac{fye}{1000}dv\cdot\cot(\theta)}{s} = 73.44 \qquad kips$$

$$V_n := \phi_s\cdot .25fc\cdot b\cdot dv = 9.914\times 10^5 \qquad kips$$

$$\phi Vn := \min\left[V_n, \left(V_c + V_s\right)\cdot\phi_s\right] = 191.414 \qquad kips$$

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

$$\text{Shearcheck2} := \text{ShearCheck}(\phi Vn, V_u) = \text{"OK"}$$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

LRFD Eq. 5.8.2.5-1

$$\text{Avmin} := 0.0316\sqrt{\frac{fc}{1000}} \cdot \frac{b \cdot s}{\frac{fye}{1000}} = 0.379 \qquad in^2$$

$$\text{Av} := 2A_{sp} = 0.4 \qquad in^2$$

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

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

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.

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

$$vu := \frac{V_u}{\phi_s \cdot b \cdot dv} = 0.076 \qquad ksi$$

LRFD Eq. 5.8.2.9-1

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

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

$MaxSpacing := floor(spacingProgram(vu, dv, fc)) = 24$ in

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$Spacecheck\left(MaxSpacing, s, V_u, V_c\right) := \begin{vmatrix} a \leftarrow min(MaxSpacing, 12) \;\; \text{if } V_u \le 0.5\, 0.9 V_c \\ a \leftarrow min(s, MaxSpacing, 12) \;\; \text{otherwise} \\ a \end{vmatrix}$$

$s := Spacecheck\left(MaxSpacing, s, V_u, V_c\right) = 10$ in

Design Summary - Bent 2

Stirrup = "#4"

$s = 10$ in

$N_2 = 11.539$ in

Design Check Summary - Bent 2

Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran = "OK"	Minimum shear reinforcement outside hinge zone

Bent 3 Design

The designer should input all information concerning the longitudinal and transverse reinforcement of the column, specifically within the plastic hinge zone. Both circular and rectangular columns are allowed.

Reinforcement Information

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

Stirrup := "#4"		Stirrup Type
A_{sp} := .20	in^2	Area of Transverse Reinforcement
D_{sp} := 0.625	in	Diameter of Transverse Reinforcement
Dprime:= 0	in	Diameter of Spiral or Hoop for Circular Columns **INPUT**
s := 10	in	Pitch of Spiral or Spacing of Hoops/Ties for Circular Columns
b := Columnwidth	in	Width of Rectangular Column
Cover := 2	in	Column Concrete Cover

Article 4.6: Determine Design Forces

The Guide Specification requires only a minimum design force for SDC A. This design force is based on the tributary dead load and live load assumed to be present during an earthquake.

$$VR\_Multiplier := DesignForce\left(A_s\right) = 0.25$$

$$HorizontalDesignForce := \frac{VR_{Bent3} \cdot VR\_Multiplier}{N} = 25.175 \quad \text{kip}$$

$$HorizontalDesignForce2 := \frac{DL_{Bent} \cdot VR\_Multiplier}{N} = 22.675 \quad \text{kip}$$

Article 4.12: Determine Minimum Support Length

$$L := BentTribLength = 62.5$$

$$H := \frac{ColumnHeight_{Bent3}}{12} = 14.34$$

Standard Specifications

$$N_{3Stan} := (8 + 0.02 \cdot L + 0.08 \cdot H) \cdot \left(1 + 0.000125 \cdot Skew^2\right) = 10.397 \quad \text{in}$$

ATC-49 Equation (New Design)

$$N_3 := \left[4 + .02 \cdot L + .08 \cdot H + 1.09 \cdot \sqrt{H} \cdot \sqrt{1 + \left(2 \cdot \frac{3}{8}\right)^2}\right] \cdot \left(\frac{1 + 1.25 \cdot S_{D1}}{\cos\left(\frac{Skew \cdot \pi}{180}\right)}\right) = 12.799 \quad \text{in}$$

Article 8.2: Column Detailing

Note: If SD1 is greater than or equal to 0.10, the minimum requirements from SDC B must be met. Otherwise, no minimum column detailing is required and the checks below do not apply.

$\text{SD1Check}(S_{D1}) = \text{"No Minimum SDC B Detailing Required"}$

LRFD 5.8.3.3* Nominal Shear Resistance** ***Guide Article 8.6.1

$V_u := \text{HorizontalDesignForce} \cdot \dfrac{N}{N_{col}} = 75.525$ kips

$\phi_s := 0.9$

$\beta := 2.0$ ***LRFD Article 5.8.3.4.1***

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

$Dr := b - Cover - D_{sp} - \dfrac{d_{bl}}{2} = 32.67$ in

$de := b - Cover = 34$ in ***LRFD Eq. 5.8.2.9-1***

$dv := 0.9 de = 30.6$ in

$V_c := 0.0316 \beta \cdot \sqrt{\dfrac{fc}{1000}} b \cdot dv = 139.242$ kips ***LRFD Eq. 5.8.3.3-3***

$V_s := \dfrac{2A_{sp} \cdot \dfrac{fye}{1000} dv \cdot \cot(\theta)}{s} = 73.44$ kips ***LRFD Eq. 5.8.3.3-4***

$V_n := \phi_s \cdot .25 fc \cdot b \cdot dv = 9.914 \times 10^5$ kips

$\phi Vn := \min\left[V_n, \left(V_c + V_s\right) \cdot \phi_s\right] = 191.414$ kips

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

$\text{Shearcheck2} := \text{ShearCheck}(\phi Vn, V_u) = \text{"OK"}$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

$$Avmin := 0.0316\sqrt{\frac{fc}{1000}}\cdot\frac{b\cdot s}{\frac{fye}{1000}} = 0.379 \qquad in^2$$

LRFD Eq. 5.8.2.5-1

$$Av := 2A_{sp} = 0.4 \qquad in^2$$

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

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

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.

LRFD 5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

$$vu := \frac{V_u}{\phi_s\cdot b\cdot dv} = 0.076 \qquad ksi$$

LRFD Eq. 5.8.2.9-1

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

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

$$MaxSpacing := floor(spacingProgram(vu, dv, fc)) = 24 \qquad in$$

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$\text{Spacecheck}(\text{MaxSpacing}, s, V_u, V_c) := \left| \begin{array}{l} a \leftarrow \min(\text{MaxSpacing}, 12) \ \text{ if } \ V_u \le 0.5 \, 0.9 V_c \\ a \leftarrow \min(s, \text{MaxSpacing}, 12) \ \text{ otherwise} \\ a \end{array} \right.$$

$$s := \text{Spacecheck}(\text{MaxSpacing}, s, V_u, V_c) = 10 \quad \text{in}$$

Design Summary - Bent 3

$\text{Stirrup} = \text{"\#4"}$

$s = 10$ in

$N_3 = 12.799$ in

Design Check Summary - Bent 3

$\text{Shearcheck2} = \text{"OK"}$ Shear capacity outside hinge zone > Vn

$\text{MinimumTran} = \text{"OK"}$ Minimum shear reinforcement outside hinge zone

Appendix D: Stave Creek Bridge SDC A2

Designer: Jordan Law

Project Name: Stave Creek Bridge

Job Number: BR-0069 (501)

Date: 5/24/2012

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.

ORIGIN := 1

<u>Project Known Information</u>

Coordinates: 31.551N, 87.930W

Soil Site Class: D

Superstructure Type: AASTHO Type I girders for end spans
AASTHO Type III girders for middle span

Substructure Type: Rectangular columns supported on piles

Abutment Type: Abutment beam supported on drilled shafts

Note: ***Input*** *all of the below information*.

The designer should input any information that can be used to calculate the dead weight of the bridge, including but not limited to length of bridge, column height(s), deck thickness, bent volume(s), and guard rail volume(s). Also, information about foundations should also be included if the bridge is classified as SDC B.

INPUT

$fc := 400($ psi

$fye := 6000($ psi

$\rho_{conc} := 0.0868 \ \frac{lb}{in^3}$

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

$A_s := .07($

$S_{D1} := .10\!\!\!($

$S_{DS} := .15\!\!\!($

SDC := "A"

Length of Bridge (ft)	$L := 16\!\!\!5$	ft
Skew of Bridge (degrees)	$Skew := 0$	degrees
End Spans (ft)	$EndSpan := 4($	ft
Middle Span (ft)	$MidSpan := 8\!\!\!5$	ft
Deck Thickness (in)	$t_{deck} := 7$	in
Deck Width (ft)	$DeckWidth := 42.7\!\!\!5$	ft

Number of Bridge Girders $N := 6$

I-Girder (AASHTO Type III) X-Sectional Area (in^2) $IGirderIIIArea := 559.5$ in^2

I-Girder (AASHTO Type I) X-Sectional Area (in^2) $IGirderIArea := 276$ in^2

Guard Rail Area (in^2) $GuardRailArea := 310$ in^2

Bent Volume (ft^3) $BentVolume := 40\cdot(3.75\cdot 4 + 1.583\cdot 1.65) = 704.478$ ft^3

Column Diameter (in) $Columnwidth := 36$ in

Number of Columns $N_{col} := 2$

Drilled Shaft Diameter (in) $DSdia := 60$ in

Drilled Shaft Abutment Diameter (in) $DSabutdia := 42$ in

The column height is measured from the bottom of the bent to the top of the pile footing. Other options include measuring from the top of the bent to the ground surface or to a change in diameter
(if possible). If the plastic hinge location (at the bottom of the column) is known, then the column height should be measured from the bottom of the bent to the known hinge point.

Average Column Height for Bent 2 (ft) $ColumnHeight_{Bent2} := 10.20$ ft

Average Column Height for Bent 3 (ft) $ColumnHeight_{Bent3} := 14.34$ ft

Column Area (in^2) $Acolumn := Columnwidth^2 = 1.296\times 10^3$ in^2

Drilled Shaft Area (in^2) $Adrilledshaft := \frac{DSdia^2\pi}{4} = 2.827\times 10^3$ 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.

$EndSpan := EndSpan\cdot 12 = 480$ in

$MidSpan := MidSpan\cdot 12 = 1.02\times 10^3$ in

$L := L\cdot 12 = 1.98\times 10^3$ in

$DeckWidth := DeckWidth\cdot 12 = 513$ in

$BentVolume := BentVolume\,12^3 = 1.217\times 10^6$ in^3

$ColumnHeight_{Bent2} := ColumnHeight_{Bent2} \cdot 12 = 122.448$ in

$ColumnHeight_{Bent3} := ColumnHeight_{Bent3} \cdot 12 = 172.08$ in

Find Vertical Reactions at Each Bent:

Live Loads assumed to be present during an earthquake (see ***LRFD Article 3.4.1***)

$$Num\_Lanes := trunc\left(\frac{\frac{DeckWidth - 2 \cdot 1.375}{12}}{12}\right) = 3$$

Number of Lanes On Bridge (Design Lane Width of 10 ft) See ***LRFD 3.6.1.2.4***

$\gamma_{EQ} := 0.5$ ***LRFD Specificaiton C3.4.1 (Extreme Case I)*** **INPUT**

The γ_{EQ} value is to be determined on a project-specific basis. In the standard specification, a value of 0.0 was used, however, the LRFD Specification recommends a value of 0.5. See LRFD Article C3.4.1 under "EXTREME EVENT I"

$LL\_design := 0.64 \ \frac{klf}{lane}$ ***LRFD Specification 3.6.1.2.4***

$Q := LL\_design \cdot \gamma_{EQ} = 0.32 \ \frac{klf}{lane}$

$LL\_foot := Q \cdot Num\_Lanes = 0.96$ klf Live Load per linear foot of deck (includes all lanes)

Note: If the Vertical Reactions at each bent are already known, input them below, otherwise the sheet will calculate vertical reactions based on the given information above.

$DL_{Bent2} :=$ ▮ kip $LL_{Bent2} :=$ ▮ kip

$DL_{Bent3} :=$ ▮ kip $LL_{Bent3} :=$ ▮ kip **INPUT**

$VR_{Bent2} := DL_{Bent2} + LL_{Bent2} =$ ▮ kip

$VR_{Bent3} := DL_{Bent3} + LL_{Bent3} =$ ▮ kip

The weight calculation takes into account the entire dead weight of the structure, including the deck, bents, abutments, columns, girders, and railings. Any other expected dead loads should also be included.

$$W := \frac{\rho_{conc} \cdot \begin{pmatrix} L \cdot t_{deck} \cdot DeckWidth + 2 \cdot BentVolume + 2 \cdot Acolumn \cdot ColumnHeight_{Bent2} \cdots \\ + 2 \cdot Acolumn \cdot ColumnHeight_{Bent3} + 2EndSpan \cdot N \cdot IGirderIArea \ldots \\ + MidSpan \cdot N \cdot IGirderIIIArea + 2 \cdot GuardRailArea \cdot L \end{pmatrix}}{1000}$$

$W = 1436.686$ kips

Note: An elevation view of the bridge shows that the tributary area for Bents 2 and 3 are identical, and therefore the tributary weights will be equal. The information below should be adjusted for different bridges.

$$BentTribLength := \frac{\frac{EndSpan + MidSpan}{2}}{12} = 62.5 \quad ft$$

$$BentTribArea := \frac{\frac{EndSpan + MidSpan}{2}}{L} = 0.379$$ Percent of Area Tributary to Bent

$$DL_{Bent} := BentTribArea \cdot W = 544.199 \quad kip$$

$$LL_{Bent} := BentTribLength \cdot LL\_foot = 60 \quad kip$$

$$VR_{Bent} := DL_{Bent} + LL_{Bent} = 604.199 \quad kip$$

$$VR_{Bent2} := VR_{Bent} \qquad VR_{Bent3} := VR_{Bent}$$

Steps for Seismic Design

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

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

Article 3.4: Determine Deisgn Response Spectrum

Article 3.5: Determine SDC

Article 4.6: Determine Design Forces

Article 4.12: Determine Minimum Support Length

Article 8.2: Column Detailing

Articles 3.4 and 3.5 have already been determined from the "SDC Classification" sheet. Make sure the four values (A_s, S_{DS}, S_{D1}, and SDC) have been input above.

Bent 2 Design

Reinforcement Information ***Guide Article 8.6.5***

The designer should input all information concerning the longitudinal and transverse reinforcement of the column, specifically within the plastic hinge zone. Both circular and rectangular columns are allowed.

Reinforcement Information

$d_{bl} := 1.41$	in	Diameter of Longitudinal Reinforcement
Stirrup := "#4"		Stirrup Type
$A_{sp} := .20$	in^2	Area of Transverse Reinforcement
$D_{sp} := 0.625$	in	Diameter of Transverse Reinforcement
Dprime := 0	in	Diameter of Spiral or Hoop for Circular Columns **INPUT**
s := 5	in	Spacing of Stirrups or Hoops/Ties
sNOhinge := 10	in	Pitch of Spiral or Spacing of Hoops/Ties outside PHL
b := Columnwidth	in	Width of Rectangular Column
Cover := 2	in	Column Concrete Cover

Article 4.6: Determine Design Forces

The Guide Specification requires only a minimum design force for SDC A. This design force is based on the tributary dead load and live load assumed to be present during an earthquake.

$$\mathrm{DesignForce}(A_s) := \left| \begin{array}{l} A \leftarrow A_s \\ a \leftarrow 0.15 \text{ if } A < 0.05 \\ a \leftarrow 0.25 \text{ if } A \ge 0.05 \\ a \end{array} \right.$$

$$\mathrm{VR\_Multiplier} := \mathrm{DesignForce}(A_s) = 0.25$$

$$\mathrm{HorizontalDesignForce} := \frac{VR_{Bent2} \cdot \mathrm{VR\_Multiplier}}{N} = 25.175 \qquad \mathrm{kip}$$

$$\mathrm{HorizontalDesignForce}_2 := \frac{DL_{Bent} \cdot \mathrm{VR\_Multiplier}}{N} = 22.675 \qquad \mathrm{kip}$$

Article 4.12: Determine Minimum Support Length

$$L := \mathrm{BentTribLength} = 62.5$$

$$H := \frac{\mathrm{ColumnHeight}_{Bent2}}{12} = 10.204$$

Standard Specifications

$$N_{2Stan} := (8 + 0.02L + 0.08H) \cdot \left(1 + 0.000125 \mathrm{Skew}^2\right) = 10.066$$

in

ATC-49 Equation (New Design)

$$N_2 := \left[4 + .02L + .08H + 1.09\sqrt{H}\cdot\sqrt{1+\left(2\cdot\frac{3}{8}\right)^2}\right]\cdot\left(\frac{1+1.25S_{D1}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 11.825$$

in

Article 8.2: Column Detailing

Note: If SD1 is greater than or equal to 0.10, the minimum requirements from SDC B must be met. Otherwise, no minimum column detailing is required and the checks below can be ignored.

```
SD1Check(SD1) := | a ← "Minimum SDC B Detailing Required" if SD1 ≥ 0.10
                 | a ← "No SDC B Detailing Required" otherwise
                 | a
```

$SD1Check(S_{D1})$ = "Minimum SDC B Detailing Required"

The Guide Specifications has a minimum shear reinforcement of 0.003 for spiral or circular hoop reinforced columns and 0.002 for ties in the direction of bending.

Calculate ρ_w:

$$\rho_w := \frac{2A_{sp}}{b\cdot s} = 0.002$$

Guide Equation 8.6.2-10

```
CheckReinforcement(ρw) := | a ← "PASS" if ρw ≥ 0.002
                          | a ← "FAIL" otherwise
                          | a
```

$ReinforcementCheck := CheckReinforcement(\rho_w) = \text{"PASS"}$

If the reinforcement check is not satisifed, the spacing between the hoops/ties or pitch of spirals can be reduced or the area of the reinforcement can be increased

Note (***Guide Article 8.8.9***): The Guide Specification has a maximum spacing of six inches within the plastic hinging zone (See below section regarding 'maximum spacing of lateral reinforcement within plastic hinging region' for more information).

All requirements in Artcile 8.8.9 must be 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 long 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

Plastic Hinging Region (4.11.7)

Guide Article C8.8.9

The length over which the transverse reinforcement calculated above is to extend over the plastic hinge length to be calculated. For SDC A, this region can be calulated using Article 5.10.11.4.1e of the LRFD Specifications, as it is here. The LRFD Specifications allows for a shorter plastic hinge.

LRFD Article 5.10.11.4.1e

$$\mathrm{PlasticHingeLength}(\mathrm{ColumnDia}, \mathrm{ColumnHeight}) := \left| \begin{array}{l} a \leftarrow \mathrm{ColumnDia} \\ b \leftarrow \frac{1}{6} \cdot \mathrm{ColumnHeigh} \\ c \leftarrow 18 \\ \mathrm{PHL} \leftarrow \max(a, b, c) \\ \mathrm{PHL} \end{array} \right.$$

$$\mathrm{PHL} := \mathrm{PlasticHingeLength}\left(\mathrm{Columnwidth}, \mathrm{ColumnHeight}_{\mathrm{Bent2}}\right) = 36 \qquad \mathrm{in}$$

Maximum Spacing of Lateral Reinforcement in Plastic Hinge Region ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

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

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

Check reinforcement spacing vs maximum allowed spacing:

$$\text{SpaceCheck}(s, \text{MaximumSpacing}) := \begin{array}{|l} a \leftarrow \text{"PASS"} \;\; \text{if } s \le \text{MaximumSpacing} \\ a \leftarrow \text{"SPACING GREATER THAN MAXIMUM"} \;\; \text{otherwise} \\ a \end{array}$$

$$\text{SpacingCheck} := \text{SpaceCheck}(s, \text{MaximumSpacing}) = \text{"PASS"}$$

LRFD 5.8.3.3* Nominal Shear Resistance** ***Guide Article 8.6.1

$$V_u := \text{HorizontalDesignForce} \cdot \frac{N}{N_{col}} = 75.525 \quad \text{kips}$$

$$\phi_s := 0.9$$

$$\beta := 2.0$$

LRFD Article 5.8.3.4.1

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

$$Dr := b - \text{Cover} - D_{sp} - \frac{d_{bl}}{2} = 32.67 \quad \text{in}$$

$$de := b - \text{Cover} = 34 \quad \text{in}$$

LRFD Eq. 5.8.2.9-1

$$dv := 0.9 de = 30.6 \quad \text{in}$$

LRFD Eq. 5.8.3.3-3

$$V_c := 0.0316\beta \cdot \sqrt{\frac{fc}{1000}} b \cdot dv = 139.242 \quad \text{kips}$$

LRFD Eq. 5.8.3.3-4

$$V_s := \frac{2A_{sp} \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{s} = 146.88 \quad \text{kips}$$

$$V_n := \phi_s \cdot .25 fc \cdot b \cdot dv = 9.914 \times 10^5 \quad \text{kips}$$

$$\phi Vn := \min\left[V_n, (V_c + V_s) \cdot \phi_s\right] = 257.51 \quad \text{kips}$$

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

$$\text{Shearcheck2} := \text{ShearCheck}(\phi Vn, V_u) = \text{"OK"}$$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

LRFD Eq. 5.8.2.5-1

$$\text{Avmin} := 0.0316\sqrt{\frac{fc}{1000}} \cdot \frac{b \cdot \text{sNOhinge}}{\frac{fye}{1000}} = 0.379 \qquad in^2$$

$$Av := 2A_{sp} = 0.4 \qquad in^2$$

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

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

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.

LRFD 5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

LRFD Eq. 5.8.2.9-1

$$vu := \frac{V_u}{\phi_s \cdot b \cdot dv} = 0.076 \qquad ksi$$

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

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

$MaxSpacing := floor(spacingProgram(vu, dv, fc)) = 24$ in

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$Spacecheck(MaxSpacing, s, V_u, V_c) := \left| \begin{array}{l} a \leftarrow min(MaxSpacing, 12) \quad \text{if } V_u \le 0.5 0.9 V_c \\ a \leftarrow min(s, MaxSpacing, 12) \quad \text{otherwise} \\ a \end{array} \right.$$

$sNOhinge := Spacecheck(MaxSpacing, sNOhinge, V_u, V_c) = 10$ in

There are no seismic foundation design requirements for SDC A

Design Summary

Stirrup = "#4"		Stirrup size of transverse reinforcement
$s = 5$	in	Spacing of stirrups in PHL
$sNOhinge = 10$	in	Spacing of stirrups outisde PHL
$PHL = 36$	in	Plastic Hinge Length
$N_2 = 11.825$	in	Minimum Seat Length

Design Check Summary

ReinforcementCheck = "PASS"	Reinforcement ratio
SpacingCheck = "PASS"	Max spacing of transverse reinforcement
Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran= "OK"	Minimum shear reinforcement outside hinge zone

Bent 3 Design

Reinforcement Information *Guide Article 8.6.5*

The designer should input all information concerning the longitudinal and transverse reinforcement of the column, specifically within the plastic hinge zone. Both circular and rectangular columns are allowed.

Reinforcement Information

$d_{bl} := 1.41$	in	Diameter of Longitudinal Reinforcement
Stirrup := "#4"		Stirrup Type
$A_{sp} := .20$	in^2	Area of Transverse Reinforcement
$D_{sp} := 0.625$	in	Diameter of Transverse Reinforcement
Dprime := 0	in	Diameter of Spiral or Hoop for Circular Columns **INPUT**
s := 5	in	Spacing of Stirrups or Hoops/Ties
sNOhinge := 10	in	Pitch of Spiral or Spacing of Hoops/Ties outside PHL
b := Columnwidth	in	Width of Rectangular Column
Cover := 2	in	Column Concrete Cover

Article 4.6: Determine Design Forces

The Guide Specification requires only a minimum design force for SDC A. This design force is based on the tributary dead load and live load assumed to be present during an earthquake.

$$\mathrm{DesignForce}(A_s) := \left| \begin{array}{l} A \leftarrow A_s \\ a \leftarrow 0.15 \text{ if } A < 0.05 \\ a \leftarrow 0.25 \text{ if } A \geq 0.05 \\ a \end{array} \right.$$

$$\mathrm{VR\_Multiplier} := \mathrm{DesignForce}(A_s) = 0.25$$

$$\mathrm{HorizontalDesignForce} := \frac{VR_{Bent3} \cdot \mathrm{VR\_Multiplier}}{N} = 25.175 \qquad \text{kip}$$

$$\mathrm{HorizontalDesignForce}_2 := \frac{DL_{Bent} \cdot \mathrm{VR\_Multiplier}}{N} = 22.675 \qquad \text{kip}$$

Article 4.12: Determine Minimum Support Length

$L := \mathrm{BentTribLength} = 62.5$

$$H := \frac{ColumnHeight_{Bent3}}{12} = 14.34$$

Standard Specifications

$$N_{3Stan} := (8 + 0.02 \cdot L + 0.08 \cdot H) \cdot \left(1 + 0.000125 \cdot Skew^2\right) = 10.397 \qquad \text{in}$$

ATC-49 Equation (New Design)

$$N_3 := \left[4 + .02 \cdot L + .08 \cdot H + 1.09 \cdot \sqrt{H} \cdot \sqrt{1 + \left(2 \cdot \frac{3}{8}\right)^2}\right] \cdot \left(\frac{1 + 1.25 S_{D1}}{\cos\left(\frac{Skew \cdot \pi}{180}\right)}\right) = 13.117 \qquad \text{in}$$

Article 8.2: Column Detailing

Note: If SD1 is greater than or equal to 0.10, the minimum requirements from SDC B must be met. Otherwise, no minimum column detailing is required and the checks below can be ignored.

$$SD1Check(SD1) := \left| \begin{array}{l} a \leftarrow \text{"Minimum SDC B Detailing Required" if } SD1 \geq 0.10 \\ a \leftarrow \text{"No SDC B Detailing Required" otherwise} \\ a \end{array} \right.$$

$$SD1Check\left(S_{D1}\right) = \text{"Minimum SDC B Detailing Required"}$$

The Guide Specifications has a minimum shear reinforcement of 0.003 for spiral or circular hoop reinforced columns and 0.002 for ties in the direction of bending.

Calculate ρ_w:

$$\rho_w := \frac{2A_{sp}}{b \cdot s} = 0.002$$

Guide Equation 8.6.2-10

$$CheckReinforcement\left(\rho_w\right) := \left| \begin{array}{l} a \leftarrow \text{"PASS" if } \rho_w \geq 0.002 \\ a \leftarrow \text{"FAIL" otherwise} \\ a \end{array} \right.$$

$$ReinforcementCheck := CheckReinforcement\left(\rho_w\right) = \text{"PASS"}$$

If the reinforcement check is not satisifed, the spacing between the hoops/ties or pitch of spirals can be reduced or the area of the reinforcement can be increased

Note (***Guide Article 8.8.9***): The Guide Specification has a maximum spacing of six inches within the plastic hinging zone (See below section regarding 'maximum spacing of lateral reinforcement within plastic hinging region' for more information).

All requirements in Artcile 8.8.9 must be 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 long 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

Plastic Hinging Region (4.11.7)

Guide Article C8.8.9

The length over which the transverse reinforcement calculated above is to extend over the plastic hinge length to be calculated. For SDC A, this region can be calulated using Article 5.10.11.4.1e of the LRFD Specifications, as it is here. The LRFD Specifications allows for a shorter plastic hinge.

LRFD Article 5.10.11.4.1e

$$PlasticHingeLength(ColumnDia, ColumnHeight) := \left| \begin{array}{l} a \leftarrow ColumnDia \\ b \leftarrow \frac{1}{6} \cdot ColumnHeigh \\ c \leftarrow 18 \\ PHL \leftarrow max(a, b, c) \\ PHL \end{array} \right.$$

$$PHL := PlasticHingeLength\left(Columnwidth, ColumnHeight_{Bent2}\right) = 36 \qquad in$$

Maximum Spacing of Lateral Reinforcement in Plastic Hinge Region ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

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

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

Check reinforcement spacing vs maximum allowed spacing:

$$\text{SpaceCheck}(s, \text{MaximumSpacing}) := \begin{array}{|l} a \leftarrow \text{"PASS"} \ \text{ if } \ s \le \text{MaximumSpacing} \\ a \leftarrow \text{"SPACING GREATER THAN MAXIMUM" otherwise} \\ a \end{array}$$

$$\text{SpacingCheck} := \text{SpaceCheck}(s, \text{MaximumSpacing}) = \text{"PASS"}$$

LRFD 5.8.3.3* Nominal Shear Resistance** ***Guide Article 8.6.1

$$V_u := \text{HorizontalDesignForce} \cdot \frac{N}{N_{col}} = 75.525 \qquad \text{kips}$$

$$\phi_s := 0.9$$

$$\beta := 2.0$$

LRFD Article 5.8.3.4.1

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

$$Dr := b - \text{Cover} - D_{sp} - \frac{d_{bl}}{2} = 32.67 \qquad \text{in}$$

$$de := b - \text{Cover} = 34 \qquad \text{in}$$

LRFD Eq. 5.8.2.9-1

$$dv := 0.9\,de = 30.6 \qquad \text{in}$$

LRFD Eq. 5.8.3.3-3

$$V_c := 0.0316\beta \cdot \sqrt{\frac{fc}{1000}}\, b \cdot dv = 139.242 \qquad \text{kips}$$

LRFD Eq. 5.8.3.3-4

$$V_s := \frac{2A_{sp} \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{s} = 146.88 \qquad \text{kips}$$

$$V_n := \phi_s \cdot .25\, fc \cdot b \cdot dv = 9.914 \times 10^5 \qquad \text{kips}$$

$$\phi Vn := \min\left[V_n, (V_c + V_s) \cdot \phi_s\right] = 257.51 \qquad \text{kips}$$

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

$$Shearcheck2 := ShearCheck(\phi Vn, V_u) = \text{"OK"}$$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

LRFD Eq. 5.8.2.5-1

$$Avmin := 0.0316\sqrt{\frac{fc}{1000}} \cdot \frac{b \cdot sNOhinge}{\frac{fye}{1000}} = 0.379 \qquad in^2$$

$$Av := 2A_{sp} = 0.4 \qquad in^2$$

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

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

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.

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

$$vu := \frac{V_u}{\phi_s \cdot b \cdot dv} = 0.076 \qquad ksi$$

LRFD Eq. 5.8.2.9-1

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

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

$MaxSpacing := floor(spacingProgram(vu, dv, fc)) = 24$ in

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$Spacecheck(MaxSpacing, s, V_u, V_c) := \left| \begin{array}{l} a \leftarrow min(MaxSpacing, 12) \;\; \text{if } V_u \le 0.50.9V_c \\ a \leftarrow min(s, MaxSpacing, 12) \;\; \text{otherwise} \\ a \end{array} \right.$$

$sNOhinge := Spacecheck(MaxSpacing, sNOhinge, V_u, V_c) = 10$ in

Design Summary

Stirrup = "#4"		Stirrup size of transverse reinforcement
$s = 5$	in	Spacing of stirrups in PHL
$sNOhinge = 10$	in	Spacing of stirrups outisde PHL
$PHL = 36$	in	Plastic Hinge Length
$N_3 = 13.117$	in	Minimum Seat Length

Design Check Summary

ReinforcementCheck = "PASS"	Reinforcement ratio
SpacingCheck = "PASS"	Max spacing of transverse reinforcement
Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran= "OK"	Minimum shear reinforcement outside hinge zone

Appendix E: Bent Creek Road Bridge SDC A2

Designer: Jordan Law
Project Name: Bent Creek Road Bridge
Job Number:STPOA-9032 (600)
Date: 6/4/2012
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.

$ORIGIN := 1$

Project Known Information

Coordinates: 32.605N, -85.428W

Soil Site Class: D

Superstructure Type: BT-54 girders for both spans

Substructure Type: Rectangular columns supported on piles

Abutment Type: Abutment beam supported on drilled shafts

The designer should input any information that can be used to calculate the dead weight of the bridge, including but not limited to length of bridge, column height(s), deck thickness, bent volume(s), and guard rail volume(s). Also, information about foundations should also be included if the bridge is classified as SDC B.

Note: ***Input*** *all of the below information*.

INPUT

$fc := 4000$ psi

$fye := 60000$ psi

$\rho_{conc} := 0.0868 \ \frac{lb}{in^3}$

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

$A_s := .070$

$S_{D1} := .108$

$S_{DS} := .157$

$SDC := \text{"A"}$

Length of Bridge (ft)	$L := 270$	ft
Skew of Bridge (degrees)	$Skew := 0$	degrees
Span Length (ft)	$Span := 135$	ft
Deck Thickness (in)	$t_{deck} := 6$	in
Deck Width (ft)	$DeckWidth := 80.75$	ft
Superstructure Depth (ft)	$D_s := 5.083$	ft
Number of Bridge Girders	$N := 15$	

Bulb (BT-54) Girder X-Sectional Area (in^2) $BulbGirderArea := 76$ in^2

Guard Rail Area (in^2) $GuardRailArea := 31$ in^2

Bent Volume (ft^3) $BentVolume := 31 \cdot (4 \cdot 4.5) = 558$ ft^3

Column Width (in) $Columnwidth := 42$ in

Number of Columns at Each Bent $N_{col} := 5$

The column height is measured from the bottom of the bent to the top of the pile footing. Other options include measuring from the top of the bent to the ground surface or to a change in diameter
(if possible). If the plastic hinge location (at the bottom of the column) is known, then the column height should be measured from the bottom of the bent to the known hinge point.

Average Column Height for Bent 2 (ft) $ColumnHeight_{Bent2} := 20.05$ ft

Column Area (in^2) $Acolumn := Columnwidth^2 = 1.764 \times 10^3$ in^2

Pile Area (in^2) $Apile := 15.$ in^2

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

$Span := Span \cdot 12 = 1.62 \times 10^3$ in

$L := L \cdot 12 = 3.24 \times 10^3$ in

$DeckWidth := DeckWidth \cdot 12 = 969$ in

$BentVolume := BentVolume \, 12^3 = 9.642 \times 10^5$ in^3

$ColumnHeight_{Bent2} := ColumnHeight_{Bent2} \cdot 12 = 240.708$ in

Find Vertical Reactions at Each Bent:

Live Loads assumed to be present during an earthquake (see ***LRFD Article 3.4.1***)

$$Num\_Lanes := trunc\left(\frac{\frac{DeckWidth - 2 \cdot 1.375}{12}}{12}\right) = 6$$

Number of Lanes On Bridge (Design Lane Width of 10 ft) See ***LRFD 3.6.1.2.4***

$\gamma_{EQ} := 0.5$ ***LRFD Specificaiton C3.4.1 (Extreme Case I)*** **INPUT**

The γ_{EQ} value is to be determined on a project-specific basis. In the standard specification, a value of 0.0 was used, however, the LRFD Specification recommends a value of 0.5. See LRFD Article C3.4.1 under "EXTREME EVENT I"

$LL\_design := 0.64 \ \frac{klf}{lane}$ ***LRFD Specification 3.6.1.2.4***

$Q := LL\_design \cdot \gamma_{EQ} = 0.32 \ \frac{klf}{lane}$

$LL\_foot := Q \cdot Num\_Lanes = 1.92$ klf Live Load per linear foot of deck (includes all lanes)

Note: If the Vertical Reactions at each bent are already known, input them below, otherwise the sheet will calculate vertical reactions based on the given information above.

$DL_{Bent2} :=$ kip $LL_{Bent2} :=$ kip

INPUT

$DL_{Bent3} :=$ kip $LL_{Bent3} :=$ kip

$VR_{Bent2} := DL_{Bent2} + LL_{Bent2} =$ kip

$VR_{Bent3} := DL_{Bent3} + LL_{Bent3} =$ kip

The weight calculation takes into account the entire dead weight of the structure, including the deck, bents, abutments, columns, girders, and railings. Any other expected dead loads should also be included.

$$W := \frac{\rho_{conc} \cdot \left(L \cdot t_{deck} \cdot DeckWidth + BentVolume + 2 \cdot Acolumn \cdot ColumnHeight_{Bent2} \ldots \\ + 2Span \cdot N \cdot BulbGirderArea \ldots \\ + 2 \cdot GuardRailArea \cdot L \right)}{1000}$$

$W = 5186.151$ kips

To determine the vertical reaction at the bent, the bents tributary area will be calculated and multiplied by the total weight. A similar calculation will be done for the live load. This vertical reaction will be used to determine the connection force (below).

$BentTribLength := \frac{Span}{12} = 135$ ft

$BentTribArea := \frac{Span}{L} = 0.5$ Percent of Area Tributary to Bent

$DL_{Bent} := BentTribArea \cdot W = 2593.076$ kip

$LL_{Bent} := BentTribLength \cdot LL\_foot = 259.2$ kip

$VR_{Bent} := DL_{Bent} + LL_{Bent} = 2852.276$ kip

$VR_{Bent2} := VR_{Bent}$

Steps for Seismic Design

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

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

Article 3.4: Determine Deisgn Response Spectrum

Article 3.5: Determine SDC

Article 4.6: Determine Design Forces

Article 4.12: Determine Minimum Support Length

Article 8.2: Column Detailing

Articles 3.4 and 3.5 have already been determined from the "SDC Classification" sheet. Make sure the four values (A_s, S_{DS}, S_{D1}, and SDC) have been input above.

Bent 2 Design

Reinforcement Information ***Guide Article 8.6.5***

The designer should input all information concerning the longitudinal and transverse reinforcement of the column, specifically within the plastic hinge zone. Both circular and rectangular columns are allowed.

Reinforcement Information

$d_{bl} := 1.41$	in	Diameter of Longitudinal Reinforcement
Stirrup := "#4"		Stirrup Type
$A_{sp} := .20$	in^2	Area of Transverse Reinforcement
$D_{sp} := 0.625$	in	Diameter of Transverse Reinforcement
Dprime := 0	in	Diameter of Spiral or Hoop for Circular Columns **INPUT**
s := 4	in	Spacing of Stirrups or Hoops/Ties
sNOhinge := 9	in	Pitch of Spiral or Spacing of Hoops/Ties outside PHL
b := Columnwidth	in	Width of Rectangular Column
Cover := 2	in	Column Concrete Cover

Article 4.6: Determine Design Forces

The Guide Specification requires only a minimum design force for SDC A. This design force is based on the tributary dead load and live load assumed to be present during an earthquake.

$$\text{DesignForce}(A_s) := \begin{array}{|l} A \leftarrow A_s \\ a \leftarrow 0.15 \;\text{ if }\; A < 0.05 \\ a \leftarrow 0.25 \;\text{ if }\; A \geq 0.05 \\ a \end{array}$$

$$\text{VR\_Multiplier} := \text{DesignForce}(A_s) = 0.25$$

$$\text{HorizontalDesignForce} := \frac{VR_{Bent2} \cdot \text{VR\_Multiplier}}{N} = 47.538 \qquad \text{kip}$$

$$\text{HorizontalDesignForce2} := \frac{DL_{Bent} \cdot \text{VR\_Multiplier}}{N} = 43.218 \qquad \text{kip}$$

Article 4.12: Determine Minimum Support Length

$$L := \text{BentTribLength} = 135$$

$$H := \frac{\text{ColumnHeight}_{Bent2}}{12} = 20.059$$

Standard Specifications

$$N_{2Stan} := (8 + 0.02 \cdot L + 0.08 \cdot H) \cdot \left(1 + 0.000125 \cdot \text{Skew}^2\right) = 12.305 \qquad \text{in}$$

ATC-49 Equation (New Design)

$$N_2 := \left[4 + .02 \cdot L + .08 \cdot H + 1.09 \cdot \sqrt{H} \cdot \sqrt{1 + \left(2 \cdot \frac{3}{8}\right)^2}\right] \cdot \left(\frac{1 + 1.25 \cdot S_{D1}}{\cos\left(\frac{\text{Skew} \cdot \pi}{180}\right)}\right) = 16.352 \qquad \text{in}$$

Article 8.2: Column Detailing

Note: If SD1 is greater than or equal to 0.10, the minimum requirements from SDC B must be met. Otherwise, no minimum column detailing is required and the checks below can be ignored.

$$\text{SD1Check}(\text{SD1}) := \begin{array}{|l} a \leftarrow \text{"Minimum SDC B Detailing Required" if } \text{SD1} \geq 0.10 \\ a \leftarrow \text{"No SDC B Detailing Required" otherwise} \\ a \end{array}$$

$$\text{SD1Check}(S_{D1}) = \text{"Minimum SDC B Detailing Required"}$$

The Guide Specifications has a minimum shear reinforcement of 0.003 for spiral or circular hoop reinforced columns and 0.002 for ties in the direction of bending.

Calculate ρ_w:

$$\rho_w := \frac{2A_{sp}}{b \cdot s} = 0.0024$$ ***Guide Equation 8.6.2-10***

$$\text{CheckReinforcement}(\rho_w) := \left| \begin{array}{l} a \leftarrow \text{"PASS"} \quad \text{if} \ \rho_w \geq 0.002 \\ a \leftarrow \text{"FAIL"} \quad \text{otherwise} \\ a \end{array} \right.$$

$$\text{ReinforcementCheck} := \text{CheckReinforcement}(\rho_w) = \text{"PASS"}$$

If the reinforcement check is not satisifed, the spacing between the hoops/ties or pitch of spirals can be reduced or the area of the reinforcement can be increased

Note (***Guide Article 8.8.9***): The Guide Specification has a maximum spacing of six inches within the plastic hinging zone (See below section regarding 'maximum spacing of lateral reinforcement within plastic hinging region' for more information).

All requirements in Artcile 8.8.9 must be 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 long 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

Plastic Hinging Region (4.11.7)

Guide Article C8.8.9

The length over which the transverse reinforcement calculated above is to extend over the plastic hinge length to be calculated. For SDC A, this region can be calulated using Article 5.10.11.4.1e of the LRFD Specifications, as it is here. The LRFD Specifications allows for a shorter plastic hinge.

LRFD Article 5.10.11.4.1e

$$\mathrm{PlasticHingeLength(ColumnDia, ColumnHeight)} := \left| \begin{array}{l} a \leftarrow \mathrm{ColumnDia} \\ b \leftarrow \frac{1}{6} \cdot \mathrm{ColumnHeigh} \\ c \leftarrow 18 \\ \mathrm{PHL} \leftarrow \max(a, b, c) \\ \mathrm{PHL} \end{array} \right.$$

$$\mathrm{PHL} := \mathrm{PlasticHingeLength}\left(\mathrm{Columnwidth}, \mathrm{ColumnHeight}_{\mathrm{Bent2}}\right) = 42 \qquad \mathrm{in}$$

Maximum Spacing of Lateral Reinforcement in Plastic Hinge Region ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

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

$$\mathrm{MaximumSpacing} := \mathrm{Spacingprogram}\left(\mathrm{Columnwidth}, d_{bl}\right) = 6 \qquad \mathrm{in}$$

Check reinforcement spacing vs maximum allowed spacing:

$$\mathrm{SpaceCheck(s, MaximumSpacing)} := \left| \begin{array}{l} a \leftarrow \text{"PASS"} \ \text{ if } s \le \mathrm{MaximumSpacing} \\ a \leftarrow \text{"SPACING GREATER THAN MAXIMUM"} \ \text{otherwise} \\ a \end{array} \right.$$

$$\mathrm{SpacingCheck} := \mathrm{SpaceCheck(s, MaximumSpacing)} = \text{"PASS"}$$

LRFD 5.8.3.3* Nominal Shear Resistance** ***Guide Article 8.6.1

$$V_u := \mathrm{HorizontalDesignForce} \cdot \frac{N}{N_{col}} = 142.614 \qquad \mathrm{kips}$$

(5 Columns)

$$\phi_s := 0.9$$

$$\beta := 2.0$$

LRFD Article 5.8.3.4.1

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

rad

$$Dr := b - Cover - D_{sp} - \frac{d_{bl}}{2} = 38.67 \qquad in$$

$$de := b - Cover = 40 \qquad in$$ ***LRFD Eq. 5.8.2.9-1***

$$dv := 0.9de = 36 \qquad in$$

$$V_c := 0.0316\beta \cdot \sqrt{\frac{fc}{1000}} b \cdot dv = 191.117 \qquad kips$$ ***LRFD Eq. 5.8.3.3-3***

$$V_s := \frac{2A_{sp} \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{sNOhinge} = 96 \qquad kips$$ ***LRFD Eq. 5.8.3.3-4***

$$V_n := \phi_s \cdot .25 fc \cdot b \cdot dv = 1.361 \times 10^6 \qquad kips$$

$$\phi Vn := \min\left[V_n, \left(V_c + V_s\right) \cdot \phi_s\right] = 258.405 \qquad kips$$

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

$$Shearcheck2 := ShearCheck\left(\phi Vn, V_u\right) = \text{"OK"}$$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

LRFD Eq. 5.8.2.5-1

$$Avmin := 0.0316\sqrt{\frac{fc}{1000}} \cdot \frac{b \cdot sNOhinge}{\frac{fye}{1000}} = 0.398 \qquad in^2$$

$$Av := 2A_{sp} = 0.4 \qquad in^2$$

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

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

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.

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

$$vu := \frac{V_u}{\phi_s \cdot b \cdot dv} = 0.105 \qquad \text{ksi}$$

LRFD Eq. 5.8.2.9-1

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

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

$$\text{MaxSpacing} := \text{floor}(\text{spacingProgram}(vu, dv, fc)) = 24 \qquad \text{in}$$

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$\text{Spacecheck}\left(\text{MaxSpacing}, s, V_u, V_c\right) := \left| \begin{array}{l} a \leftarrow \min(\text{MaxSpacing}, 12) \ \text{ if } V_u \le 0.5\,0.9\,V_c \\ a \leftarrow \min(s, \text{MaxSpacing}, 12) \ \text{ otherwise} \\ a \end{array} \right.$$

$$\text{sNOhinge} := \text{Spacecheck}\left(\text{MaxSpacing}, \text{sNOhinge}, V_u, V_c\right) = 9 \qquad \text{in}$$

There are no seismic foundation design requirements for SDC A

Design Summary

Stirrup = "#4"		Stirrup size of transverse reinforcement
s = 4	in	Spacing of stirrups in PHL

$sNOhinge = 9$ in Spacing of stirrups outisde PHL

$PHL = 42$ in Plastic Hinge Length

$N_2 = 16.352$ in Minimum Seat Length

Design Check Summary

ReinforcementCheck = "PASS" Reinforcement ratio

SpacingCheck = "PASS" Max spacing of transverse reinforcement

Shearcheck2 = "OK" Shear capacity outside hinge zone > Vn

MinimumTran= "OK" Minimum shear reinforcement outside hinge zone

Appendix F: I-59 Bridge over Norfolk Southern Railroad SDC A2

Designer: Jordan Law
Project Name: Norfolk Southern RR
Job Number: STMAAF-1059 (342)
Date: 6/4/2012
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.

ORIGIN := 1

Project Known Information

Coordinates: 34.125N, 85.982W

Soil Site Class: D

Superstructure Type: BT-54 girders for both spans

Substructure Type: Rectangular columns supported on piles

Abutment Type: Abutment beam supported on piles

The designer should input any information that can be used to calculate the dead weight of the bridge, including but not limited to length of bridge, column height(s), deck thickness, bent volume(s), and guard rail volume(s). Also, information about foundations should also be included if the bridge is classified as SDC B.
Note: ***Input*** *all of the below information*.

INPUT

$fc := 4000$ psi

$fye := 60000$ psi

$\rho_{conc} := 0.0868 \ \frac{lb}{in^3}$

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

$A_S := .14$

$S_{D1} := .15$

$S_{DS} := .30$

SDC:= "B"

Length of Bridge (ft)	$L := 265$	ft
Skew of Bridge (degrees)	$Skew := 0$	degrees
Span Length 1 (ft)	$Span1 := 125$	ft
Span Length 2 (ft)	$Span2 := 140$	ft
Deck Thickness (in)	$t_{deck} := 6$	in
Superstructure Depth (ft)	$D_s := 5$	ft
Deck Width (ft)	$DeckWidth := 46.75$	ft
Number of Bridge Girders	$N := 9$	

Bulb (BT-54) Girder X-Sectional Area (in^2) $BulbGirderArea := 763$ in^2

Guard Rail Area (in^2) $GuardRailArea := 310$ in^2

Bent Volume (ft^3) $BentVolume := 53 \cdot (4.54) = 954$ ft^3

Column Width (in) $Columnwidth := 42$ in

Number of Columns per Bent $N_{col} := 3$

The column height is measured from the bottom of the bent to the top of the pile footing. Other options include measuring from the top of the bent to the ground surface or to a change in diameter(if possible). If the plastic hinge location (at the bottom of the column) is known, then the column height should be measured from the bottom of the bent to the known hinge point.

Average Column Height for Bent 2 (ft) $ColumnHeight_{Bent2} := 25.25$ ft

Tallest Abutment Height Above Ground (ft) $H_{abutment} :=$ ft

Column Area (in^2) $Acolumn := Columnwidth^2 = 1.764 \times 10^3$ in^2

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

$Span1 := Span1 \cdot 12 = 1.5 \times 10^3$ in

$Span2 := Span2 \cdot 12 = 1.68 \times 10^3$ in

$L := L \cdot 12 = 3.18 \times 10^3$ in

$DeckWidth := DeckWidth \cdot 12 = 561$ in

$BentVolume := BentVolume \cdot 12^3 = 1.649 \times 10^6$ in^3

$ColumnHeight_{Bent2} := ColumnHeight_{Bent2} \cdot 12 = 303$ in

Find Vertical Reactions at Each Bent:

Live Loads assumed to be present during an earthquake (see ***LRFD Article 3.4.1***)

$$Num\_Lanes := trunc\left(\frac{\frac{DeckWidth - 2 \cdot 1.375}{12}}{12}\right) = 3$$

Number of Lanes On Bridge (Design Lane Width of 10 ft) See ***LRFD 3.6.1.2.4***

$\gamma_{EQ} := 0.5$ ***LRFD Specificaiton C3.4.1 (Extreme Case I)*** **INPUT**

The γ_{EQ} value is to be determined on a project-specific basis. In the standard specification, a value of 0.0 was used, however, the LRFD Specification recommends a value of 0.5. See LRFD Article C3.4.1 under "EXTREME EVENT I"

$LL\_design := 0.64 \ \frac{klf}{lane}$ ***LRFD Specification 3.6.1.2.4***

$Q := LL\_design \cdot \gamma_{EQ} = 0.32 \ \frac{klf}{lane}$

$LL\_foot := Q \cdot Num\_Lanes = 0.96$ klf Live Load per linear foot of deck (includes all lanes)

Note: If the Vertical Reactions at each bent are already known, input them below, otherwise the sheet will calculate vertical reactions based on the given information above.

$DL_{Bent2} :=$ ▪ kip $LL_{Bent2} :=$ ▪ kip

INPUT

$DL_{Bent3} :=$ ▪ kip $LL_{Bent3} :=$ ▪ kip

$VR_{Bent2} := DL_{Bent2} + LL_{Bent2} =$ ▪ kip

$VR_{Bent3} := DL_{Bent3} + LL_{Bent3} =$ ▪ kip

The weight calculation takes into account the entire dead weight of the structure, including the deck, bents, abutments, columns, girders, and railings. Any other expected dead loads should also be included.

$$W := \frac{\rho_{conc} \cdot \begin{pmatrix} L \cdot t_{deck} \cdot DeckWidth + BentVolume + 3 \cdot Acolumn \cdot ColumnHeight_{Bent2} \ldots \\ + Span1 \cdot N \cdot BulbGirderArea + Span2 \cdot N \cdot BulbGirderArea \ldots \\ + 2 \cdot GuardRailArea \cdot L \end{pmatrix}}{1000}$$

$W = 3278.339$ kips

To determine the vertical reaction at the bent, the bents tributary area will be calculated and multiplied by the total weight. A similar calculation will be done for the live load. This vertical reaction will be used to determine the connection force (below).

$$BentTribLength := \frac{\frac{Span1 + Span2}{2}}{12} = 132.5 \quad ft$$

$$BentTribArea := \frac{\frac{Span1 + Span2}{2}}{L} = 0.5$$ Percent of Area Tributary to Bent

$DL_{Bent} := BentTribArea \cdot W = 1639.169$ kip

$LL_{Bent} := BentTribLength \cdot LL\_foot = 127.2$ kip

$VR_{Bent} := DL_{Bent} + LL_{Bent} = 1766.369$ kip

$VR_{Bent2} := VR_{Bent}$

Steps for Seismic Design

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

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

Article 3.4: Determine Deisgn Response Spectrum

Article 3.5: Determine SDC

Article 4.6: Determine Design Forces

Article 4.12: Determine Minimum Support Length

Article 8.2: Column Detailing

Articles 3.4 and 3.5 have already been determined from the "SDC Classification" sheet.
Make sure the four values (A_S, S_{DS}, S_{D1}, and SDC) have been input above.

Bent 2 Design

Reinforcement Information ***Guide Article 8.6.5***

The designer should input all information concerning the longitudinal and transverse reinforcement of the column, specifically within the plastic hinge zone. Both circular and rectangular columns are allowed.

Reinforcement Information

$d_{bl} := 1.41$	in	Diameter of Longitudinal Reinforcement
Stirrup := "#4"		Stirrup Type
$A_{sp} := .20$	in^2	Area of Transverse Reinforcement
$D_{sp} := 0.625$	in	Diameter of Transverse Reinforcement
Dprime := 0	in	Diameter of Spiral or Hoop for Circular Columns **INPUT**
s := 4	in	Spacing of Stirrups or Hoops/Ties
sNOhinge := 9	in	Pitch of Spiral or Spacing of Hoops/Ties outside PHL
b := Columnwidth	in	Width of Rectangular Column
Cover := 2	in	Column Concrete Cover

Article 4.6: Determine Design Forces

The Guide Specification requires only a minimum design force for SDC A. This design force is based on the tributary dead load and live load assumed to be present during an earthquake.

$$\text{DesignForce}(A_s) := \begin{array}{|l} A \leftarrow A_s \\ a \leftarrow 0.15 \text{ if } A < 0.05 \\ a \leftarrow 0.25 \text{ if } A \geq 0.05 \\ a \end{array}$$

$$\text{VR\_Multiplier} := \text{DesignForce}(A_s) = 0.25$$

$$\text{HorizontalDesignForce} := \frac{VR_{Bent2} \cdot \text{VR\_Multiplier}}{N} = 49.066 \qquad \text{kip}$$

$$\text{HorizontalDesignForce}_2 := \frac{DL_{Bent} \cdot \text{VR\_Multiplier}}{N} = 45.532 \qquad \text{kip}$$

Article 4.12: Determine Minimum Support Length

$$L := \text{BentTribLength} = 132.5$$

$$H := \frac{\text{ColumnHeight}_{Bent2}}{12} = 25.25$$

Standard Specifications

$$N_{2Stan} := (8 + 0.02 \cdot L + 0.08 H) \cdot \left(1 + 0.000125 \text{Skew}^2\right) = 12.67 \qquad \text{in}$$

ATC-49 Equation (New Design)

$$N_2 := \left[4 + .02L + .08H + 1.09\sqrt{H} \cdot \sqrt{1 + \left(2 \cdot \frac{3}{8}\right)^2}\right] \cdot \left(\frac{1 + 1.25 S_{D1}}{\cos\left(\frac{\text{Skew} \cdot \pi}{180}\right)}\right) = 18.426 \qquad \text{in}$$

Article 8.2: Column Detailing

Note: If SD1 is greater than or equal to 0.10, the minimum requirements from SDC B must be met. Otherwise, no minimum column detailing is required and the checks below can be ignored.

$$\text{SD1Check}(\text{SD1}) := \begin{array}{|l} a \leftarrow \text{"Minimum SDC B Detailing Required" if SD1} \geq 0.10 \\ a \leftarrow \text{"No SDC B Detailing Required" otherwise} \\ a \end{array}$$

$$\text{SD1Check}(S_{D1}) = \text{"Minimum SDC B Detailing Required"}$$

The Guide Specifications has a minimum shear reinforcement of 0.003 for spiral or circular hoop reinforced columns and 0.002 for ties in the direction of bending.

Calculate ρ_w:

$$\rho_w := \frac{2A_{sp}}{b \cdot s} = 0.0024$$

Guide Equation 8.6.2-10

$$\text{CheckReinforcement}(\rho_w) := \begin{array}{|l} a \leftarrow \text{"PASS"} \quad \text{if } \rho_w \geq 0.00: \\ a \leftarrow \text{"FAIL"} \quad \text{otherwise} \\ a \end{array}$$

$$\text{ReinforcementCheck} := \text{CheckReinforcement}(\rho_w) = \text{"PASS"}$$

If the reinforcement check is not satisifed, the spacing between the hoops/ties or pitch of spirals can be reduced or the area of the reinforcement can be increased

Note (***Guide Article 8.8.9***): The Guide Specification has a maximum spacing of six inches within the plastic hinging zone (See below section regarding 'maximum spacing of lateral reinforcement within plastic hinging region' for more information).

All requirements in Artcile 8.8.9 must be 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 long 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

Plastic Hinging Region (4.11.7)

Guide Article C8.8.9

The length over which the transverse reinforcement calculated above is to extend over the plastic hinge length to be calculated. For SDC A, this region can be calulated using Article 5.10.11.4.1e of the LRFD Specifications, as it is here. The LRFD Specifications allows for a shorter plastic hinge.

LRFD Article 5.10.11.4.1e

$$PlasticHingeLength(ColumnDia, ColumnHeight) := \left| \begin{array}{l} a \leftarrow ColumnDia \\ b \leftarrow \frac{1}{6} \cdot ColumnHeigh \\ c \leftarrow 18 \\ PHL \leftarrow max(a, b, c) \\ PHL \end{array} \right.$$

$$PHL := PlasticHingeLength(Columnwidth, ColumnHeight_{Bent2}) = 50.5 \qquad in$$

Maximum Spacing of Lateral Reinforcement in Plastic Hinge Region ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

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

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

Check reinforcement spacing vs maximum allowed spacing:

$$SpaceCheck(s, MaximumSpacing) := \left| \begin{array}{l} a \leftarrow \text{"PASS"} \text{ if } s \le MaximumSpacing \\ a \leftarrow \text{"SPACING GREATER THAN MAXIMUM"} \text{ otherwise} \\ a \end{array} \right.$$

$$SpacingCheck := SpaceCheck(s, MaximumSpacing) = \text{"PASS"}$$

LRFD 5.8.3.3* Nominal Shear Resistance** ***Guide Article 8.6.1

$$V_u := HorizontalDesignForce \cdot \frac{N}{N_{col}} = 147.197 \qquad kips$$

$$\phi_s := 0.9$$

$$\beta := 2.0$$

LRFD Article 5.8.3.4.1

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

$$Dr := b - Cover - D_{sp} - \frac{d_{bl}}{2} = 38.67 \qquad in$$

$$de := b - Cover = 40 \qquad in$$

LRFD Eq. 5.8.2.9-1

$$dv := 0.9 de = 36 \qquad in$$

$$V_c := 0.0316\beta \cdot \sqrt{\frac{fc}{1000}} b \cdot dv = 191.117 \quad \text{kips}$$

LRFD Eq. 5.8.3.3-3

$$V_s := \frac{2A_{sp} \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{sNOhinge} = 96 \quad \text{kips}$$

LRFD Eq. 5.8.3.3-4

$$V_n := \phi_s \cdot .25 fc \cdot b \cdot dv = 1.361 \times 10^6 \quad \text{kips}$$

$$\phi Vn := \min\left[V_n, \left(V_c + V_s\right) \cdot \phi_s\right] = 258.405 \quad \text{kips}$$

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

$$\text{Shearcheck2} := \text{ShearCheck}\left(\phi Vn, V_u\right) = \text{"OK"}$$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

LRFD Eq. 5.8.2.5-1

$$Avmin := 0.0316\sqrt{\frac{fc}{1000}} \cdot \frac{b \cdot sNOhinge}{\frac{fye}{1000}} = 0.398 \quad \text{in}^2$$

$$Av := 2A_{sp} = 0.4 \quad \text{in}^2$$

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

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

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.

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

$$vu := \frac{V_u}{\phi_s \cdot b \cdot dv} = 0.108 \quad \text{ksi}$$

LRFD Eq. 5.8.2.9-1

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

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

$$\text{MaxSpacing} := \text{floor}(\text{spacingProgram}(vu, dv, fc)) = 24 \quad \text{in}$$

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$\text{Spacecheck}\left(\text{MaxSpacing}, s, V_u, V_c\right) := \left| \begin{array}{l} a \leftarrow \min(\text{MaxSpacing}, 12) \ \text{ if } \ V_u \le 0.5 0.9 V_c \\ a \leftarrow \min(s, \text{MaxSpacing}, 12) \ \text{ otherwise} \\ a \end{array} \right.$$

$$\text{sNOhinge} := \text{Spacecheck}\left(\text{MaxSpacing}, \text{sNOhinge}, V_u, V_c\right) = 9 \quad \text{in}$$

There are no seismic foundation design requirements for SDC A

Design Summary

Stirrup = "#4"		Stirrup size of transverse reinforcement
$s = 4$	in	Spacing of stirrups in PHL
sNOhinge = 9	in	Spacing of stirrups outisde PHL
PHL = 50.5	in	Plastic Hinge Length
$N_2 = 18.426$	in	Minimum Seat Length

Design Check Summary

ReinforcementCheck = "PASS"	Reinforcement ratio
SpacingCheck = "PASS"	Max spacing of transverse reinforcement

Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran= "OK"	Minimum shear reinforcement outside hinge zone

Appendix G: Oseligee Creek Bridge SDC A2

Designer: Jordan Law
Project Name: Oseligee Bridge
Job Number: BR-IV20 (515)
Date: 5/24/2012
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.

$ORIGIN := 1$

<u>Project Known Information</u>

Coordinates: 32.902N, 85.196W

Soil Site Class: D

Superstructure Type: AASTHO Type III girders for all spans

Substructure Type: Circular columns supported on drilled shafts

Abutment Type: Abutment beam supported on drilled shafts

Note: ***Input*** *all of the below information*.

The designer should input any information that can be used to calculate the dead weight of the bridge, including but not limited to length of bridge, column height(s), deck thickness, bent volume(s), and guard rail volume(s). Also, information about foundations should also be included if the bridge is classified as SDC B.

$fc := 400($ psi

$fye := 6000($ psi

$\rho_{conc} := 0.0868 \ \frac{lb}{in^3}$

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

$A_S := .074$

$S_{D1} := .11$

$S_{DS} := .17$

$SDC := "B"$

INPUT

Length of Bridge (ft)	$L := 24($	ft
Angle of skew of bridge (degrees)	$Skew := 0$	Degrees
Span (ft)	$Span := 8($	ft
Deck Thickness (in)	$t_{deck} := 7$	in
Deck Width (ft)	$DeckWidth := 32.75$	ft
Depth of Superstructure (ft)	$D_s := 4.187$	ft
Number of Bridge Girders	$N := 4$	

I-Girder X-Sectional Area (in^2) $IGirderArea := 559.5$

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

Number of Columns per Bent $N_{col} := 2$

Drilled Shaft Diameter (in) $DSdia := 42$ in

Drilled Shaft Abutment Diameter (in) $DSabutdia := 42$ in

The column height is measured from the bottom of the bent to the top of the pile footing. Other options include measuring from the top of the bent to the ground surface or to a change in diameter(if possible). If the plastic hinge location (at the bottom of the column) is known, then the column height should be measured from the bottom of the bent to the known hinge point.

Average Column Height of Bent 2 (ft) $ColumnHeight_{Bent2} := 17.93$ ft

Average Column Height of Bent 3 (ft) $ColumnHeight_{Bent3} := 25.83$ ft

Height of tallest abutment above ground (ft) $H_{abutment} :=$ ft

Column Area (in^2) $Acolumn := \frac{Columndia^2 \cdot \pi}{4} = 1.385 \times 10^3$ in^2

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

$ColumnHeight_{Bent2} := ColumnHeight_{Bent2} \cdot 12 = 215.208$ in

$ColumnHeight_{Bent3} := ColumnHeight_{Bent3} \cdot 12 = 310.008$ in

Find Vertical Reactions at Each Bent:

Live Loads assumed to be present during an earthquake (see ***LRFD Article 3.4.1***)

$$Num\_Lanes := trunc\left(\frac{\frac{DeckWidth - 2 \cdot 1.375}{12}}{12}\right) = 2$$

Number of Lanes On Bridge (Design Lane Width of 10 ft) See ***LRFD 3.6.1.2.4***

$\gamma_{EQ} := 0.5$ ***LRFD Specificaiton C3.4.1 (Extreme Case I)*** **INPUT**

The γ_{EQ} value is to be determined on a project-specific basis. In the standard specification, a value of 0.0 was used, however, the LRFD Specification recommends a value of 0.5. See LRFD Article C3.4.1 under "EXTREME EVENT I"

$LL\_design := 0.64 \frac{klf}{lane}$ ***LRFD Specification 3.6.1.2.4***

$Q := LL\_design \cdot \gamma_{EQ} = 0.32 \frac{klf}{lane}$

$LL\_foot := Q \cdot Num\_Lanes = 0.64$ klf Live Load per linear foot of deck (includes all lanes)

Note: If the Vertical Reactions at each bent are already known, input them below, otherwise the sheet will calculate vertical reactions based on the given information above.

$DL_{Bent2} :=$ ▮ kip $LL_{Bent2} :=$ ▮ kip

$DL_{Bent3} :=$ ▮ kip $LL_{Bent3} :=$ ▮ kip **INPUT**

$VR_{Bent2} := DL_{Bent2} + LL_{Bent2} =$ ▮ kip

$VR_{Bent3} := DL_{Bent3} + LL_{Bent3} =$ ▮ kip

The weight calculation takes into account the entire dead weight of the structure, including the deck, bents, abutments, columns, girders, and railings. Any other expected dead loads should also be included.

$$W := \frac{\rho_{conc} \cdot \left(\begin{array}{l} L \cdot t_{deck} \cdot DeckWidth + 2 \cdot BentVolume + 2 \cdot Acolumn \cdot ColumnHeight_{Bent2} \cdots \\ + 2 \cdot Acolumn \cdot ColumnHeight_{Bent3} + L \cdot N \cdot IGirderArea \ldots \\ + 2 \cdot GuardRailArea \cdot L \end{array}\right)}{1000}$$

$W = 1708.667$ kips

To determine the vertical reaction at the bent, the bents tributary area will be calculated and multiplied by the total weight. A similar calculation will be done for the live load. This vertical reaction will be used to determine the connection force (below).

$BentTribLength := \frac{Span}{12} = 80$ ft

$BentTribArea := \frac{Span}{L} = 0.333$ Percent of Area Tributary to Bent

$DL_{Bent} := BentTribArea \cdot W = 569.556$ kip

$LL_{Bent} := BentTribLength \cdot LL\_foot = 51.2$ kip

$VR_{Bent} := DL_{Bent} + LL_{Bent} = 620.756$ kip

$VR_{Bent2} := VR_{Bent}$ $VR_{Bent3} := VR_{Bent}$

Steps for Seismic Design

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

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

Article 3.4: Determine Deisgn Response Spectrum

Article 3.5: Determine SDC

Article 4.6: Determine Design Forces

Article 4.12: Determine Minimum Support Length

Article 8.2: Column Detailing

Articles 3.4 and 3.5 have already been determined from the "SDC Classification" sheet. Make sure the four values (A_S, S_{DS}, S_{D1}, and SDC) have been input above.

Bent 2 Design

Reinforcement Information ***Guide Article 8.6.5***

The designer should input all information concerning the longitudinal and transverse reinforcement of the column, specifically within the plastic hinge zone. Both circular and rectangular columns are allowed.

Reinforcement Information

$d_{bl} := 1.41$	in	Diameter of Longitudinal Reinforcement
$Cover := 2$	in	Column Concrete Cover
Stirrup := "#4"		Stirrup Type
$A_{sp} := .20$	in^2	Area of Transverse Reinforcement
$D_{sp} := 0.625$	in	Diameter of Transverse Reinforcement
Dprime := Columndia − Cover	in	Diameter of Column Core for Circular Columns
$s := 6$	in	Spacing of Stirrups or Hoops/Ties
sNOhinge := 9	in	Pitch of Spiral or Spacing of Hoops/Ties outside PHL
b := Columndia	in	Width of Rectangular Column

INPUT

Article 4.6: Determine Design Forces

The Guide Specification requires only a minimum design force for SDC A. This design force is based on the tributary dead load and live load assumed to be present during an earthquake.

$$\text{DesignForce}(A_s) := \begin{array}{|l} A \leftarrow A_s \\ a \leftarrow 0.15 \text{ if } A < 0.05 \\ a \leftarrow 0.25 \text{ if } A \geq 0.05 \\ a \end{array}$$

$$\text{VR\_Multiplier} := \text{DesignForce}(A_s) = 0.25$$

$$\text{HorizontalDesignForce} := \frac{VR_{Bent2} \cdot \text{VR\_Multiplier}}{N} = 38.797 \quad \text{kip}$$

$$\text{HorizontalDesignForce} := \frac{DL_{Bent} \cdot \text{VR\_Multiplier}}{N} = 35.597 \quad \text{kip}$$

Article 4.12: Determine Minimum Support Length

$$L := \text{BentTribLength} = 80$$

$$H := \frac{\text{ColumnHeight}_{Bent2}}{12} = 17.934$$

Standard Specifications

$$N_{2Stan} := (8 + 0.02 L + 0.08 H)\cdot\left(1 + 0.000125 Skew^2\right) = 11.035 \qquad in$$

ATC-49 Equation (New Design)

$$N_2 := \left[4 + .02 L + .08 H + 1.09\sqrt{H}\cdot\sqrt{1 + \left(2\cdot\frac{3}{8}\right)^2}\right]\cdot\left(\frac{1 + 1.25 S_{D1}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 14.565 \qquad in$$

Article 8.2: Column Detailing

Note: If SD1 is greater than or equal to 0.10, the minimum requirements from SDC B must be met. Otherwise, no minimum column detailing is required and the checks below can be ignored.

$$SD1Check(SD1) := \left| \begin{array}{l} a \leftarrow \text{"Minimum SDC B Detailing Required" if } SD1 \ge 0.1(\\ a \leftarrow \text{"No SDC B Detailing Required" otherwise} \\ a \end{array} \right.$$

$$SD1Check\left(S_{D1}\right) = \text{"Minimum SDC B Detailing Required"}$$

The Guide Specifications has a minimum shear reinforcement of 0.003 for spiral or circular hoop reinforced columns and 0.002 for ties in the direction of bending.

Calculate ρ_s:

$$\rho_s := \frac{4\cdot A_{sp}}{s\cdot Dprime} = 0.0033$$

Guide Equation 8.6.2-10

$$CheckReinforcement\left(\rho_s\right) := \left| \begin{array}{l} a \leftarrow \text{"PASS"} \quad \text{if } \rho_s \ge 0.00 \\ a \leftarrow \text{"FAIL"} \quad \text{otherwise} \\ a \end{array} \right.$$

$$ReinforcementCheck := CheckReinforcement\left(\rho_s\right) = \text{"PASS"}$$

If the reinforcement check is not satisifed, the spacing between the hoops/ties or pitch of spirals can be reduced or the area of the reinforcement can be increased

Note (***Guide Article 8.8.9***): The Guide Specification has a maximum spacing of six inches within the plastic hinging zone (See below section regarding 'maximum spacing of lateral reinforcement within plastic hinging region' for more information).

All requirements in Artcile 8.8.9 must be satisfied:

Cross-tie Requirements:

1) Continuous bar having a hook of not less than 135 Degrees with an extension N**OT** 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

Plastic Hinging Region (4.11.7)

Guide Article C8.8.9

The length over which the transverse reinforcement calculated above is to extend over the plastic hinge length to be calculated. For SDC A, this region can be calulated using Article 5.10.11.4.1e of the LRFD Specifications, as it is here. The LRFD Specifications allows for a shorter plastic hinge.

LRFD Article 5.10.11.4.1e

$$\text{PlasticHingeLength}(\text{ColumnDia}, \text{ColumnHeight}) := \left| \begin{array}{l} a \leftarrow \text{ColumnDia} \\ b \leftarrow \frac{1}{6}\cdot\text{ColumnHeigh} \\ c \leftarrow 18 \\ \text{PHL} \leftarrow \max(a, b, c) \\ \text{PHL} \end{array} \right.$$

$$\text{PHL} := \text{PlasticHingeLength}\left(\text{Columndia}, \text{ColumnHeight}_{\text{Bent2}}\right) = 42 \qquad \text{in}$$

Maximum Spacing of Lateral Reinforcement in Plastic Hinge Region ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

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

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

Check reinforcement spacing vs maximum allowed spacing:

$$\text{SpaceCheck}(s, \text{MaximumSpacing}) := \begin{array}{|l} a \leftarrow \text{"PASS"} \ \text{ if } s \le \text{MaximumSpacing} \\ a \leftarrow \text{"SPACING GREATER THAN MAXIMUM" otherwise} \\ a \end{array}$$

$$\text{SpacingCheck} := \text{SpaceCheck}(s, \text{MaximumSpacing}) = \text{"PASS"}$$

LRFD 5.8.3.3 Nominal Shear Resistance

Guide Article 8.6.1

$$V_u := \text{HorizontalDesignForce} \cdot \frac{N}{N_{col}} = 71.194 \qquad \text{kips}$$

$$\phi_s := 0.9$$

$$\beta := 2.0$$

LRFD Article 5.8.3.4.1

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

$$Dr := b - \text{Cover} - D_{sp} - \frac{d_{bl}}{2} = 38.67 \qquad \text{in}$$

$$de := b - \text{Cover} = 40 \qquad \text{in}$$

LRFD Eq. 5.8.2.9-1

$$dv := 0.9 de = 36 \qquad \text{in}$$

LRFD Eq. 5.8.3.3-3

$$V_c := 0.0316 \beta \cdot \sqrt{\frac{fc}{1000}} b \cdot dv = 191.117 \qquad \text{kips}$$

LRFD Eq. 5.8.3.3-4

$$V_s := \frac{2A_{sp} \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{\text{sNOhinge}} = 96 \qquad \text{kips}$$

$$V_n := \phi_s \cdot .25 fc \cdot b \cdot dv = 1.361 \times 10^6 \qquad \text{kips}$$

$$\phi Vn := \min\left[V_n, (V_c + V_s) \cdot \phi_s\right] = 258.405 \qquad \text{kips}$$

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

$$\text{Shearcheck2} := \text{ShearCheck}(\phi Vn, V_u) = \text{"OK"}$$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

LRFD Eq. 5.8.2.5-1

$$\text{Avmin} := 0.0316\sqrt{\frac{fc}{1000}} \cdot \frac{b \cdot \text{sNOhinge}}{\frac{fye}{1000}} = 0.398 \qquad in^2$$

$$Av := 2A_{sp} = 0.4 \qquad in^2$$

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

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

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.

LRFD 5.8.2.7 Maximum Spacing of Transverse Reinforcement

LRFD Eq. 5.8.2.9-1

$$vu := \frac{V_u}{\phi_s \cdot b \cdot dv} = 0.052 \qquad ksi$$

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

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

$MaxSpacing := floor(spacingProgram(vu, dv, fc)) = 24$ in

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$Spacecheck(MaxSpacing, s, V_u, V_c) := \left| \begin{array}{l} a \leftarrow min(MaxSpacing, 12) \ \text{ if } \ V_u \leq 0.50.9 V_c \\ a \leftarrow min(s, MaxSpacing, 12) \ \text{ otherwise} \\ a \end{array} \right.$$

$sNOhinge := Spacecheck(MaxSpacing, sNOhinge, V_u, V_c) = 12$ in

There are no seismic foundation design requirements for SDC A

Design Summary

Stirrup = "#4"		Stirrup size of transverse reinforcement
s = 6	in	Spacing of stirrups in PHL
sNOhinge = 12	in	Spacing of stirrups outisde PHL
PHL = 42	in	Plastic Hinge Length
$N_2 = 14.565$	in	Minimum Seat Length

Design Check Summary

ReinforcementCheck = "PASS"	Reinforcement ratio
SpacingCheck = "PASS"	Max spacing of transverse reinforcement
Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran = "OK"	Minimum shear reinforcement outside hinge zone

Bent 3 Design

Reinforcement Information ***Guide Article 8.6.5***

The designer should input all information concerning the longitudinal and transverse reinforcement of the column, specifically within the plastic hinge zone. Both circular and rectangular columns are allowed.

Reinforcement Information

$d_{bl} := 1.41$	in	Diameter of Longitudinal Reinforcement
$Cover := 2$	in	Column Concrete Cover
$Stirrup := \text{"\#4"}$		Stirrup Type
$A_{sp} := .20$	in^2	Area of Transverse Reinforcement
$D_{sp} := 0.625$	in	Diameter of Transverse Reinforcement
$Dprime := Columndia - Cover$	in	Diameter of Column Core for Circular Columns
$s := 6$	in	Spacing of Stirrups or Hoops/Ties
$sNOhinge := 9$	in	Pitch of Spiral or Spacing of Hoops/Ties outside PHL
$b := Columndia$	in	Width of Rectangular Column

INPUT

Article 4.6: Determine Design Forces

The Guide Specification requires only a minimum design force for SDC A. This design force is based on the tributary dead load and live load assumed to be present during an earthquake.

$$DesignForce(A_s) := \begin{array}{|l} A \leftarrow A_s \\ a \leftarrow 0.15 \text{ if } A < 0.05 \\ a \leftarrow 0.25 \text{ if } A \geq 0.05 \\ a \end{array}$$

$$VR\_Multiplier := DesignForce(A_s) = 0.25$$

$$HorizontalDesignForce := \frac{VR_{Bent3} \cdot VR\_Multiplier}{N} = 38.797 \qquad kip$$

$$HorizontalDesignForce_2 := \frac{DL_{Bent} \cdot VR\_Multiplier}{N} = 35.597 \qquad kip$$

Article 4.12: Determine Minimum Support Length

$$L := BentTribLength = 80$$

$$H := \frac{ColumnHeight_{Bent3}}{12} = 25.834$$

Standard Specifications

$$N_{3Stan} := (8 + 0.02 \cdot L + 0.08 \cdot H) \cdot \left(1 + 0.000125 \cdot Skew^2\right) = 11.667 \quad in$$

ATC-49 Equation (New Design)

$$N_3 := \left[4 + .02 \cdot L + .08 \cdot H + 1.09 \cdot \sqrt{H} \cdot \sqrt{1 + \left(2 \cdot \frac{3}{8}\right)^2}\right] \cdot \left(\frac{1 + 1.25 \cdot S_{D1}}{\cos\left(\frac{Skew \cdot \pi}{180}\right)}\right) = 16.598 \quad in$$

Article 8.2: Column Detailing

Note: If SD1 is greater than or equal to 0.10, the minimum requirements from SDC B must be met. Otherwise, no minimum column detailing is required and the checks below can be ignored.

SD1Check(SD1) := | a ← "Minimum SDC B Detailing Required" if SD1 ≥ 0.10
| a ← "No SDC B Detailing Required" otherwise
| a

$SD1Check\left(S_{D1}\right)$ = "Minimum SDC B Detailing Required"

The Guide Specifications has a minimum shear reinforcement of 0.003 for spiral or circular hoop reinforced columns and 0.002 for ties in the direction of bending.

Calculate ρ_s:

$$\rho_s := \frac{4 \cdot A_{sp}}{s \cdot Dprime} = 0.0033$$ ***Guide Equation 8.6.2-10***

CheckReinforcement(ρ_s) := | a ← "PASS" if $\rho_s \geq 0.003$
| a ← "FAIL" otherwise
| a

ReinforcementCheck := CheckReinforcement(ρ_s) = "PASS"

If the reinforcement check is not satisifed, the spacing between the hoops/ties or pitch of spirals can be reduced or the area of the reinforcement can be increased

Note (***Guide Article 8.8.9***): The Guide Specification has a maximum spacing of six inches within the plastic hinging zone (See below section regarding 'maximum spacing of lateral reinforcement within plastic hinging region' for more information).

All requirements in Artcile 8.8.9 must be 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 long 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

Plastic Hinging Region (4.11.7)

Guide Article C8.8.9

The length over which the transverse reinforcement calculated above is to extend over the plastic hinge length to be calculated. For SDC A, this region can be calulated using Article 5.10.11.4.1e of the LRFD Specifications, as it is here. The LRFD Specifications allows for a shorter plastic hinge.

LRFD Article 5.10.11.4.1e

$$PlasticHingeLength(ColumnDia, ColumnHeight) := \left| \begin{array}{l} a \leftarrow ColumnDia \\ b \leftarrow \frac{1}{6} \cdot ColumnHeigh \\ c \leftarrow 18 \\ PHL \leftarrow max(a, b, c) \\ PHL \end{array} \right.$$

$$PHL := PlasticHingeLength\left(Columndia, ColumnHeight_{Bent3}\right) = 51.668 \quad in$$

Maximum Spacing of Lateral Reinforcement in Plastic Hinge Region ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

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

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

Check reinforcement spacing vs maximum allowed spacing:

$$\text{SpaceCheck}(s, \text{MaximumSpacing}) := \begin{array}{|l} a \leftarrow \text{"PASS"} \text{ if } s \le \text{MaximumSpacing} \\ a \leftarrow \text{"SPACING GREATER THAN MAXIMUM"} \text{ otherwise} \\ a \end{array}$$

$$\text{SpacingCheck} := \text{SpaceCheck}(s, \text{MaximumSpacing}) = \text{"PASS"}$$

LRFD 5.8.3.3* Nominal Shear Resistance** — ***Guide Article 8.6.1

$$V_u := \text{HorizontalDesignForce} \cdot \frac{N}{N_{col}} = 77.594 \qquad \text{kips}$$

$$\phi_s := 0.9$$

$$\beta := 2.0$$

LRFD Article 5.8.3.4.1

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

$$Dr := b - \text{Cover} - D_{sp} - \frac{d_{bl}}{2} = 38.67 \qquad \text{in}$$

$$de := b - \text{Cover} = 40 \qquad \text{in}$$

LRFD Eq. 5.8.2.9-1

$$dv := 0.9 de = 36 \qquad \text{in}$$

LRFD Eq. 5.8.3.3-3

$$V_c := 0.0316 \beta \cdot \sqrt{\frac{fc}{1000}} b \cdot dv = 191.117 \qquad \text{kips}$$

LRFD Eq. 5.8.3.3-4

$$V_s := \frac{2A_{sp} \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{\text{sNOhinge}} = 96 \qquad \text{kips}$$

$$V_n := \phi_s \cdot .25 fc \cdot b \cdot dv = 1.361 \times 10^6 \qquad \text{kips}$$

$$\phi Vn := \min\left[V_n, \left(V_c + V_s\right)\cdot\phi_s\right] = 258.405 \quad \text{kips}$$

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

$$\text{Shearcheck2} := \text{ShearCheck}\left(\phi Vn, V_u\right) = \text{"OK"}$$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

LRFD Eq. 5.8.2.5-1

$$\text{Avmin} := 0.0316\sqrt{\frac{fc}{1000}}\cdot\frac{b\cdot sNOhinge}{\frac{fye}{1000}} = 0.398 \quad in^2$$

$$Av := 2A_{sp} = 0.4 \quad in^2$$

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

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

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.

LRFD 5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

$$vu := \frac{V_u}{\phi_s\cdot b\cdot dv} = 0.057 \quad \text{ksi}$$

LRFD Eq. 5.8.2.9-1

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

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

$$\text{MaxSpacing} := \text{floor}(\text{spacingProgram}(vu, dv, fc)) = 24 \quad \text{in}$$

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$\text{Spacecheck}\left(\text{MaxSpacing}, s, V_u, V_c\right) := \left| \begin{array}{l} a \leftarrow \min(\text{MaxSpacing}, 12) \quad \text{if } V_u \le 0.5\,0.9\,V_c \\ a \leftarrow \min(s, \text{MaxSpacing}, 12) \quad \text{otherwise} \\ a \end{array} \right.$$

$$\text{sNOhinge} := \text{Spacecheck}\left(\text{MaxSpacing}, \text{sNOhinge}, V_u, V_c\right) = 12 \quad \text{in}$$

There are no seismic foundation design requirements for SDC A

Design Summary

Stirrup = "#4"		Stirrup size of transverse reinforcement
$s = 6$	in	Spacing of stirrups in PHL
sNOhinge = 12	in	Spacing of stirrups outisde PHL
PHL = 51.668	in	Plastic Hinge Length
$N_3 = 16.598$	in	Minimum Seat Length

Design Check Summary

ReinforcementCheck = "PASS"	Reinforcement ratio
SpacingCheck = "PASS"	Max spacing of transverse reinforcement
Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran= "OK"	Minimum shear reinforcement outside hinge zone

Appendix H: Bent Creek Road Bridge SDC B

Designer: Jordan Law
Project Name: Bent Creek Road Bridge
Job Number:STPOA-9032 (600)
Date: 6/4/2012
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.

ORIGIN:= 1

Project Known Information

Coordinates: 32.605N, -85.428W

Soil Site Class: D

Superstructure Type: BT-54 girders for both spans

Substructure Type: Rectangular columns supported on piles

Abutment Type: Abutment beam supported on drilled shafts

The designer should input any information that can be used to calculate the dead weight of the bridge, including but not limited to length of bridge, column height(s), deck thickness, bent volume(s), and guard rail volume(s). Also, information about foundations should also be included if the bridge is classified as SDC B.

Note: ***Input*** *all of the below information*.

$fc := 400($ psi

$fye := 6000($ psi

$\rho_{conc} := 0.0868 \ \frac{lb}{in^3}$

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

$A_s := .104$

$S_{D1} := .156$

$S_{DS} := .243$

SDC:= "B"

INPUT

Length of Bridge (ft)	$L := 27($	ft
Angle of Skew of Bridge (degrees)	$Skew := 0$	Degrees
Span Length (ft)	$Span := 135$	ft
Deck Thickness (in)	$t_{deck} := 6$	in
Deck Width (ft)	$DeckWidth := 80.75$	ft
Superstructure Depth (ft)	$D_s := 5.083$	ft
Number of Bridge Girders	$N := 15$	
Bulb (BT-54) Girder X-Sectional Area (in^2)	$BulbGirderArea := 763$	in^2

Bent Volume (ft^3) $BentVolume := 78\cdot(4\cdot4.5) = 1404$ ft^3

Column Width (in) $Columnwidth := 42$ in

Number of Columns per Bent $N_{col} := 5$

The column height is measured from the bottom of the bent to the top of the pile footing. Other options include measuring from the top of the bent to the ground surface or to a change in diameter(if possible). If the plastic hinge location (at the bottom of the column) is known, then the column height should be measured from the bottom of the bent to the known hinge point.

Average Column Height for Bent 2 (ft) $ColumnHeight_{Bent2} := 20.05$ ft

Height of Tallest Abutment Above Ground (ft) $H_{abutment} :=$ ■ ft

Column Area (in^2) $Acolumn := Columnwidth^2 = 1.764\times 10^3$ in^2

Pile Area (in^2) $Apile := 15.6$ in^2

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

$Span := Span\cdot 12 = 1.62\times 10^3$ in

$L := L\cdot 12 = 3.24\times 10^3$ in

$DeckWidth := DeckWidth\cdot 12 = 969$ in

$BentVolume := BentVolume\,12^3 = 2.426\times 10^6$ in^3

$ColumnHeight_{Bent2} := ColumnHeight_{Bent2}\cdot 12 = 240.708$ in

Find Vertical Reactions at Each Bent:

Live Loads assumed to be present during an earthquake (see ***LRFD Article 3.4.1***)

$$Num\_Lanes := trunc\left(\frac{\frac{DeckWidth - 2\cdot 1.375}{12}}{12}\right) = 6$$

Number of Lanes On Bridge (Design Lane Width of 10 ft) See ***LRFD 3.6.1.2.4***

$\gamma_{EQ} := 0.5$ ***LRFD Specificaiton C3.4.1 (Extreme Case I)*** **INPUT**

The γ_{EQ} value is to be determined on a project-specific basis. In the standard specification, a value of 0.0 was used, however, the LRFD Specification recommends a value of 0.5. See LRFD Article C3.4.1 under "EXTREME EVENT I"

$LL\_design := 0.64\ \frac{klf}{lane}$ ***LRFD Specification 3.6.1.2.4***

$Q := LL\_design \cdot \gamma_{EQ} = 0.32 \qquad \frac{klf}{lane}$

$LL\_foot := Q \cdot Num\_Lanes = 1.92 \qquad klf$ Live Load per linear foot of deck (includes all lanes)

Note: If the Vertical Reactions at each bent are already known, input them below, otherwise the sheet will calculate vertical reactions based on the given information above.

$DL_{Bent2} :=$ kip $\qquad LL_{Bent2} :=$ kip

INPUT

$DL_{Bent3} :=$ kip $\qquad LL_{Bent3} :=$ kip

$VR_{Bent2} := DL_{Bent2} + LL_{Bent2} =$ kip

$VR_{Bent3} := DL_{Bent3} + LL_{Bent3} =$ kip

The weight calculation takes into account the entire dead weight of the structure, including the deck, bents, abutments, columns, girders, and railings. Any other expected dead loads should also be included.

$$W := \frac{\rho_{conc} \cdot \begin{pmatrix} L \cdot t_{deck} \cdot DeckWidth + BentVolume + 2 \cdot Acolumn \cdot ColumnHeight_{Bent2} \cdots \\ + 2Span \cdot N \cdot BulbGirderArea \ldots \\ + 2 \cdot GuardRailArea \cdot L \end{pmatrix}}{1000}$$

$W = 5313.058 \qquad kips$

To determine the vertical reaction at the bent, the bents tributary area will be calculated and multiplied by the total weight. A similar calculation will be done for the live load. This vertical reaction will be used to determine the connection force (below).

$BentTribLength := \frac{Span}{12} = 135 \qquad ft$

$BentTribArea := \frac{Span}{L} = 0.5$ Percent of Area Tributary to Bent

$DL_{Bent} := BentTribArea \cdot W = 2656.529 \qquad kip$

$LL_{Bent} := BentTribLength \cdot LL\_foot = 259.2 \qquad kip$

$VR_{Bent} := DL_{Bent} + LL_{Bent} = 2915.729 \qquad kip$

$VR_{Bent2} := VR_{Bent}$

<u>Steps for Seismic Design</u>

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

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

Article 3.4: Determine Deisgn Response Spectrum

Article 3.5: Determine SDC

Guide Figure 1.3-2: Seismic Design Procedure Flowchart for SDC B

Displacement Demand Analysis (Fig 1.3-2):

- *Article* 4.1: Seismic Design Proportioning
- *Article* 4.2: Determine Analysis Procedure
- *Article* 4.3.1: Determine Horizontal Ground Motion Effects Along Both Axis
- *Article* 4.3.2/4.3.3: Damping and Short Period Considerations
- *Article* 5.4/5.5: Select Analytical Procedure
- *Article* 5.6: Effective Section Properties
- *Article* 5.2: Abutment Modeling
- *Article* 5.3: Foundation Modeling and Liquefaction (if present)
- *Article* 5.1.2/4.4: Conduct Demand Analysis
- *Article* 4.8: Determine Displacement Demands Along Member Local Axes

Displacement Capacity Check ($\Delta_C > \Delta_D$):

- *Article* 4.12: Determine Minimum Support Length
- *Article* 4.14: Shear Key

Guide Figure 1.3-5: Foundation and Detailing Flowcharts

Foundation Design (Fig 1.3-5):

- *Article* 6.8: Liquefaction Consideration
- *Article* 6.3: Spread Footing Design
- *Article* 6.4: Pile Cap Foundation Design
- *Article* 6.5: Drilled Shaft
- *Article* 6.7: Abutment Design

Detailing:

- *Article* 8.3: Determine Flexure and Shear Demands
- *Article* 8.7: Satisfy Requirements for Ductile Member Design
- *Article* 8.6: Shear Demand and Capacity Check for Ductile Elements
- *Article* 8.8: Satisfy Lateral and Longitudinal Reinforcement Requirements

Articles 3.4 and 3.5 have already been determined from the "SDC Classification" sheet.
Make sure the four values (A_s, S_{DS}, S_{D1}, and SDC) have been input above.

Displacement Demand Analysis (Δ_D)

***Article 4.1*: Seismic Design Proportioning**

See Guide Specification

***Article 4.2*: Determine Analysis Procedure**

This is a function of the SDC and the regularity of the bridge.
For a regular bridge in SDC B, Procedure 1 or 2 can be used.
For a non-regular bridge in SDC B, Procedure 2 must be used.

Guide Table 4.2-1

A regular bridge is defined as a bridge having fewer than 7 spans, no abrupt or unusual change in geometry and that saisfy the requirements below (***Guide Table 4.2-3***)

Table 4.2-3: Regular Bridge Requirements

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle (curved bridge)	30°	30°	30°	30°	30°
Maximum span length ratio from span-to-span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span-to-span (excluding abutments)	-	4	4	3	2

***Article 4.3.1*: Determine Horizontal Ground Motion Effects Along Both Axis**

Seismic displacement demands shall be determined independently in two orthogonal directions, typically the longitudinal and transverse axes of the bridge

Article 4.3.3: Displacement Magnification for Short-Period Structures

$u_d := 2$ for SDC B

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

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)**

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.

$$p_o := 1.(\quad \frac{kip}{in}$$

$$\upsilon_{smaxLong} := 0.079 \quad in$$

INPUT

$$\upsilon_{smaxTran} := 4.767 \quad in$$

$$K_{Long} := \frac{p_o \cdot L}{\upsilon_{smaxLong}} = 4.091\times 10^4 \quad \frac{kip}{in}$$ ***Guide Eq. C5.4.2-1***

$$K_{Tran} := \frac{p_o \cdot L}{\upsilon_{smaxTran}} = 679.658 \quad \frac{kip}{in}$$ ***Guide Eq. C5.4.2-2***

The weight of the structure has already been calculated above

Step 4: Calculate the period, T_m.

$$T_{mLong} := 2\pi \cdot \sqrt{\frac{W}{K_{Long} \cdot g}} = 0.115 \quad s$$ ***Guide Eq. C5.4.2-3***

Step 5: Calculate equivalent static earthquake loading p_e.

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

$$Sa_{Long} := acc(S_{DS}, S_{D1}, T_{mLong}, A_s) = 0.229$$

$$p_{eLong} := \frac{Sa_{Long} \cdot W}{L} = 0.375 \qquad \frac{kip}{in}$$ ***Guide Eq. C5.4.2-4***

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.

$$Rd_{Long} := Rdprogram(T_{mLong}, S_{DS}, S_{D1}, u_d) = 3.983$$

$$v_{smaxLong} := Rd_{Long} \cdot \frac{p_{eLong}}{p_o} \cdot \upsilon_{smaxLong} = 0.118 \qquad in$$

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

Step 4: Calculate the period, T_m.

$$T_{mTran} := 2\pi \cdot \sqrt{\frac{W}{K_{Tran} \cdot g}} = 0.894 \qquad s$$ ***Guide Eq. C5.4.2-3***

Step 5: Calculate equivalent static earthquake loading p_e.

$$Sa_{Tran} := acc(S_{DS}, S_{D1}, T_{mTran}, A_s) = 0.175$$

$$p_{eTran} := \frac{Sa_{Tran} \cdot W}{L} = 0.286 \qquad \frac{kip}{in}$$ ***Guide Eq. C5.4.2-4***

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.

$$Rd_{Tran} := Rdprogram\left(T_{mTran}, S_{DS}, S_{D1}, u_d\right) = 1$$

$$v_{smaxTran} := Rd_{Tran} \cdot \frac{p_{eTran}}{p_o} \cdot \upsilon_{smaxTran} = 1.365 \qquad in$$

LRFD Article 4.7.4.3.2: Single-Mode Spectral Method

Single Mode Spectral Analysis

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

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.

$$v_{stran}(x) := -1 \cdot 10^{-8} \cdot x^2 + 0.001x + 1.401$$

INPUT

$$v_{slong}(x) := 7 \cdot 10^{-9} \cdot x^2 + 3 \cdot 10^{-5} \cdot x + 0.074$$

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

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

$$\gamma_{Tran} := \int_0^L \frac{W}{L} \cdot v_{stran}(x)^2\,dx = 5.173 \times 10^4 \qquad \gamma_{Long} := \int_0^L \frac{W}{L} \cdot v_{slong}(x)^2\,dx$$ ***LRFD C4.7.4.3.2b-3***

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

$$T_{mTran1} := 2\pi \cdot \sqrt{\frac{\gamma_{Tran}}{p_o \cdot g \cdot \alpha_{Tran}}} = 0.739 \qquad s$$ ***LRFD Eq. 4.7.4.3.2b-4***

$$T_{mLong1} := 2\pi \cdot \sqrt{\frac{\gamma_{Long}}{p_o \cdot g \cdot \alpha_{Long}}} = 0.166 \qquad s$$ ***LRFD Eq. 4.7.4.3.2b-4***

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smLong} := acc\left(S_{DS}, S_{D1}, T_{mLong1}, A_s\right) = 0.243$$

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.

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

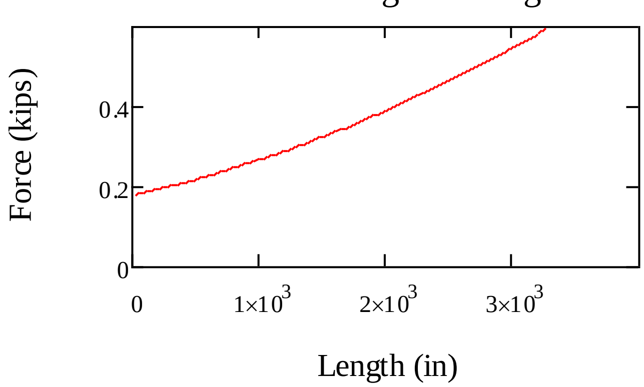
$$PeLong(x) \rightarrow 0.0000727439508146121997927 \cdot x^2 + 1.6973588523409513286e{-8} \cdot x^3 + 0.18088995769233567($$

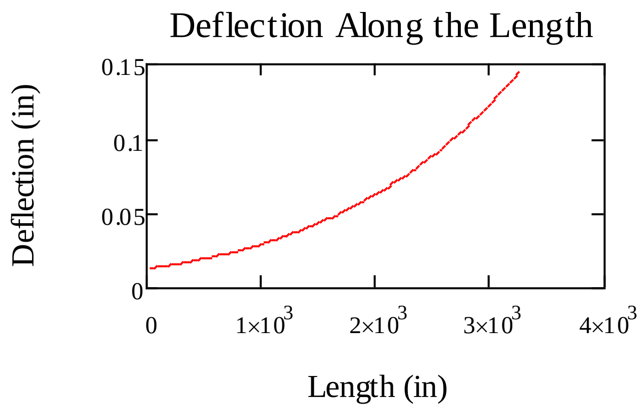
$$dW := \frac{L}{100}$$

$$i := 1..101$$

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

Force Along the Length





Maximum Deflection:

$\max(\Delta long) = 0.146$ in

NOTE: Repeat Steps 5 and 6 for Transverse Direction.

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smTran} := acc\left(S_{DS}, S_{D1}, T_{mTran1}, A_s\right) = 0.211$$

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.

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

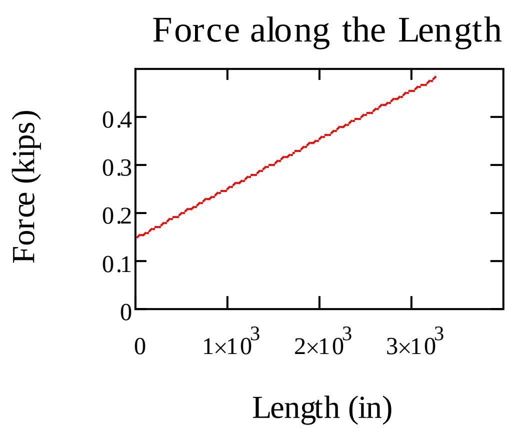
$PeTran(x) \rightarrow 0.00010614966416874472753x^3 + -1.0614966416874472753e-9 x^2 + 0.14874752439966198($

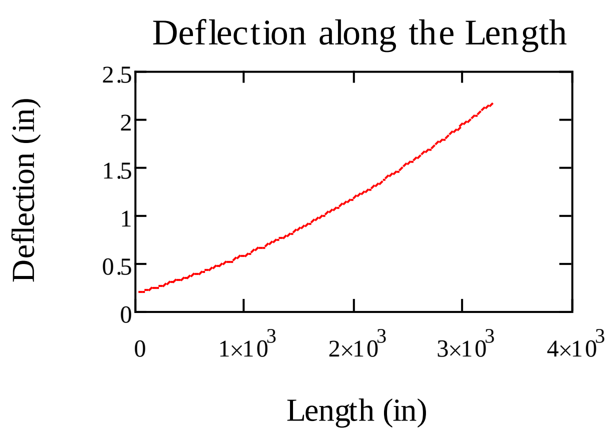
$$dL := \frac{L}{100}$$

$i := 1..101$

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

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





Maximum Deflection:

$\max(\Delta tran) = 2.184$ in

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

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**

This is taken care of in the SAP model.

***Article 5.3*: Foundations Modeling**

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.

Special provisions need to be considered if Liquefaction is present. ***Guide Article 6.8***

Article 4.4: **Combination of Orthogonal Seismic Displacement Demands**

$$\mathrm{LoadCase1} := \sqrt{(1 \cdot v_{smaxLong})^2 + (0.3 v_{smaxTran})^2} = 0.426 \quad \text{in}$$

$$\mathrm{LoadCase2} := \sqrt{(1 \cdot v_{smaxTran})^2 + (0.3 v_{smaxLong})^2} = 1.365 \quad \text{in}$$

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 $\Delta_D < \Delta_C$

BENT 2

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

$\Delta_{Dlong} := 0.052$ in

INPUT

$\Delta_{Dtran} := 3.012$ in

$$\Delta_{DLong} := Rd_{Long} \cdot \Delta_{Dlong} \cdot p_{eLong} = 0.078 \quad \text{in}$$

$$\Delta_{DTran} := Rd_{Tran} \cdot \Delta_{Dtran} \cdot p_{eTran} = 0.862 \quad \text{in}$$

$$\mathrm{LoadCase1} := \sqrt{(1\Delta_{DLong})^2 + (0.3\Delta_{DTran})^2} = 0.27 \quad \text{in}$$

$$\mathrm{LoadCase2} := \sqrt{(1\Delta_{DTran})^2 + (0.3\Delta_{DLong})^2} = 0.863 \quad \text{in}$$

$$\Delta_D := \max(\mathrm{LoadCase1}, \mathrm{LoadCase2}) = 0.863 \quad \text{in}$$

$$H_o := \frac{\mathrm{ColumnHeight}_{Bent2}}{12} = 20.059 \quad \text{ft} \qquad B_o := \frac{\mathrm{Columnwidth}}{12} = 3.5 \quad \text{ft}$$

Transverse Direction

$\Lambda := 2$ Fixed and top and bottom ***Guide Article 4.8.1***

$$x := \frac{\Lambda \cdot B_o}{H_o} = 0.349$$ ***Guide Eq. 4.8.1-3***

$\Delta_{CT} := 0.12 H_o \cdot (-1.27 \cdot \ln(x) - 0.32) = 2.448$ in

Guide Eq. 4.8.1-1

Longitudinal Direction

Guide Article 4.8.1

$\Lambda := 1$ Fixed-Free

Guide Eq. 4.8.1-3

$x := \frac{\Lambda \cdot B_o}{H_o} = 0.174$

$\Delta_{CL} := 0.12 H_o \cdot (-1.27 \cdot \ln(x) - 0.32) = 4.567$ in

Guide Eq. 4.8.1-1

$\Delta_C := \min(\Delta_{CT}, \Delta_{CL}) = 2.448$

$0.12 H_o = 2.407$ in

$$\text{CheckLimit}(\Delta_C, H_o) := \left| \begin{array}{l} a \leftarrow 0.12 H_o \quad \text{if } \Delta_C < 0.12 H_o \\ a \leftarrow \Delta_C \quad \text{otherwise} \end{array} \right.$$

$\Delta_C := \text{CheckLimit}(\Delta_C, H_o)$

$\Delta_C = 2.448$

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

$\text{CheckCapacity}(\Delta_C, \Delta_D) = \text{"OK"}$

If the simplified equations do not work ("FAILURE") for any of the bents, 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.

Pushover Analysis Results (if necessary):

Article 4.12: Minimum Support Length Requirements

Abutment Support Length Requirement *Guide Eq. 4.12.2-1*

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

Bent Support Length Requirement *Guide Eq. 4.12.2-1*

BENT 2

$L := BentTribLength = 135$ $\qquad S_{D1} := 0.3($

$H := \frac{ColumnHeight_{Bent2}}{12} = 20.059$

Standard Specifications

$N_{2Stan} := (8 + 0.02 \cdot L + 0.08 \cdot H) \cdot \left(1 + 0.000125 \cdot Skew^2\right) = 12.305$ in

ATC-49 Equation (New Design)

$$N_2 := \left[4 + .02 \cdot L + .08 \cdot H + 1.09 \cdot \sqrt{H} \cdot \sqrt{1 + \left(2 \cdot \frac{3}{8}\right)^2}\right] \cdot \left(\frac{1 + 1.25 \cdot S_{D1}}{\cos\left(\frac{Skew \cdot \pi}{180}\right)}\right) = 19.81 \quad \text{in}$$

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

$V_{ok} := 1.5 V_n$ *This does not apply to this bridge*

BENT 2 DESIGN

Guide Article 4.11.2: For SDC B, it is acceptable to use the moment capacity based on expected material strengths when the concrete reaches an extreme compressive fiber strain of 0.003.

Force Inputs

INPUT

$M_{neBent2} := 300($	kip – ft	Nominal moment from PCA Column
$V_{pelastic} := 331$	kip	Elastic shear from SAP2000 model
$P_u := 128400($	lb	Axial load from earthquake and dead load combination

Reinforcement Details

$Ag := Acolumn$

$Ae := 0.8 \cdot Ag = 1411$ in^2 ***Guide Eq. 8.6.2-2***

Guide Article 8.6.2

$\mu_D := 2$

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

StirrupSize:= "#4" StirrupSize: Bar size used for stirrups

$s := 4$ in s: Spacing of hoops or pitch of spiral (in)

sNOhinge := 9 in sNOhinge: Spacing of hoops or pitch outside PHL

$Asp := .2($	in^2	Asp: Area of hoop reinforcement in direction of loading (in^2)
$Dsp := 0.62$	in	Dsp: Diameter of spiral or hoop reinforcing (in) **INPUT**
$Cover := 2$	in	Cover: Concrete cover for the Column (in)
$b := Columnwidth$	in	b: Width of rectangular column (in)
$d := b - Cover = 40$	in	d: Effective depth of section in direction of loading (in)
$NumberBars := 12$		Total number of longitudinal bars in column cross-section
$A_{bl} := 1.56$	in^2	Abl: Area of longitudinal bar
$d_{bl} := 1.41$	in	dbl: Diameter of longitudinal bar
$bv := Columnwidth$		bv: Width of column side

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

The design forces shall be the lesser of the forces resulting from the overstrength plastic hinging moment capacity or unreduced elastic seismic forces in columns or pier walls.

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

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.

$\lambda_{mo} := 1.4$ for ASTM A 615 Grade 60 reinforcement ***Guide Article 8.5***

$$M_{pBent2} := \lambda_{mo} \cdot M_{neBent2} \cdot 1000 \cdot 12 = 5.04 \times 10^7 \quad lb - in$$

$$Fixity := ColumnHeight_{Bent2} = 240.708 \quad in$$

$$V_p := \frac{2 \cdot M_{pBent2}}{Fixity \cdot 1000} = 418.765 \quad kips$$

$$V_{pelastic} := V_{pelastic} \cdot max(P_{eTran}, P_{eLong}) = 124.139 \quad kips$$

***Article 8.7*: Requirements for Ductile Member Design**

Each column must satisfy the minimum lateral flexural capacity

$\Lambda := 2$ Fixed and top and bottom ***Guide Article 4.8.1***

$$M_{neminBent} := 0.1 \cdot DL_{Bent} \cdot \left(\frac{\frac{Fixity}{12} + 0.5 D_s}{\Lambda} \right) = 3001.944 \qquad \text{kip ft}$$ ***Guide Eq 8.7.1-1***

$$CheckMoment(M_{ne}, M_e) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } M_{ne} \le M_e \\ a \leftarrow \text{"FAILURE"} \quad \text{otherwise} \end{array} \right.$$

$$CheckMoment(M_{neminBent}, M_{neBent2}) = \text{"FAILURE"}$$

If the moment check comes back "FAILURE" then the column size can be increased or the reinforcement can be increased.

Article 8.6: Shear Demand and Capacity for Ductile Concrete Members

It is recommended to use the plastic hinging forces whenever practical, but in this case the elastic forces will be used.

$$V_u := \min(V_p, V_{pelastic}) = 124.139 \qquad \text{kips} \qquad \qquad \phi_s := 0.9$$

$$V_{pBent2} := V_u$$

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

$$\rho_w := \frac{2 \cdot Asp}{b \cdot s} = 0.0024$$ ***Guide Eq. 8.6.2-10***

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

$$StressCheck(\rho_s, fyh) := \left| \begin{array}{l} fs \leftarrow 2\rho_s \cdot fyh \\ a \leftarrow fs \quad \text{if } fs \le 0.35 \end{array} \right.$$

$$fw := StressCheck(\rho_w, fyh) = 0.286$$ ***Guide Eq. 8.6.2-9***

$$\alpha Prime := \frac{fw}{0.15} + 3.67 - \mu_D = 3.575$$ ***Guide Eq. 8.6.2-8***

If Pu is Compressive:

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

Guide Eq. 8.6.2-3

If Pu is NOT Compressive:

Guide Eq. 8.6.2-4

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

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

$Vc := vc\cdot Ae = 310.464$ kips

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

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

Guide Eq 8.6.3-2 and 8.6.4-1

$Vs := \text{vsprogram}(Asp, fyh, d, s, fc, Ae) = 120$ kips

$\phi Vn := \phi_s\cdot(Vs + Vc) = 387.418$ kips

Guide Eq. 8.6.1-2

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

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

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 Columns:

Guide Eq. 8.6.5-1

$$\text{mintranprogram}(\rho_w) := \begin{array}{|l} a \leftarrow \text{"OK"} \quad \text{if } \rho_w \ge 0.002 \\ a \leftarrow \text{"Increase Shear Reinforcing Ratio"} \quad \text{if } \rho_w < 0.002 \\ a \end{array}$$

$$\text{Transversecheck} := \text{mintranprogram}(\rho_w) = \text{"OK"}$$

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**

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

Guide Eq. 8.8.1-1

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

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

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**

Guide Eq. 8.8.2-1

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

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

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

***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.

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

Guide Eq. 4.11.6-1

$$L_p := \text{PlasticHinge}(\text{Fixity}, \text{fye}, d_{bl}) = 31.947 \quad \text{in}$$

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

'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 ρ_{eTran} to take into account the model loads have not been multiplied by ρ_{eTran}. The location will also need to be **INPUT** into the PlasticHingeRegion program in inches.

$Mp75 := 0.75 M_{pBent2} = 3.78 \times 10^{7}$ lb·in

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

$L_{p1} := PlasticHingeRegion\left(L_p, Columnwidth\right) = 63$ in

The Guide Specifications allows for the use of the plastic hinge length from the LRFD Specification in SDC A and B (***Guide Article C8.8.9***).

$$LRFDPlasticHingeLength(ColumnDia, ColumnHeight) := \begin{array}{|l} a \leftarrow ColumnDia \\ b \leftarrow \frac{1}{6} \cdot ColumnHeight \\ c \leftarrow 18 \\ PHL \leftarrow \max(a, b, c) \\ PHL \end{array}$$

$L_{p2} := LRFDPlasticHingeLength(Columnwidth, Fixity) = 42$ in

Guide Article C8.8.9

The plastic hinge length will be the smaller of the two values, as the Guide Specification allows:

$PHL := \min\left(L_{p1}, L_{p2}\right) = 42$ in

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region: ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

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

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

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

$$\text{FINALSPACING} := \text{SpacingCheck}(\text{MaximumSpacing}, s) = 4 \qquad \text{in}$$

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

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 5.10.11.4.3 (LRFD SPEC.): Column Connections

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.

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

$$\text{Extension} := \text{ExtensionProgram}(\text{Columnwidth}) = 21 \qquad \text{in}$$

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

LRFD 5.8.3.3 Nominal Shear Resistance

$V_u = 124.139 \qquad \text{kips}$

$\phi_s = 0.9$

$\beta := 2.0$

LRFD Article 5.8.3.4.1

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

$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 38.67 \qquad \text{in}$

$de := d = 40 \qquad \text{in}$

LRFD Eq. 5.8.2.9-1

$dv := 0.9de = 36$ in

$V_c := 0.0316\beta\cdot\sqrt{\frac{fc}{1000}}bv\cdot dv = 191.117$ kips *LRFD Eq. 5.8.3.3-3*

$V_s := \frac{2Asp\cdot\frac{fye}{1000}dv\cdot\cot(\theta)}{sNOhinge} = 96$ kips *LRFD Eq. 5.8.3.3-4*

$V_n := \phi_s\cdot.25fc\cdot bv\cdot dv = 1.361\times 10^6$ kips

$\phi Vn := \min\left[V_n,\left(V_c + V_s\right)\cdot\phi_s\right] = 258.405$ kips

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

$Shearcheck2 := ShearCheck\left(\phi Vn, V_u\right) = \text{"OK"}$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement ***LRFD Eq. 5.8.2.5-1***

$Avmin := 0.0316\sqrt{\frac{fc}{1000}}\cdot\frac{bv\cdot sNOhinge}{\frac{fye}{1000}} = 0.398$ in^2

$Av := 2Asp = 0.4$ in^2

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

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

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.

LRFD 5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

$vu := \frac{V_u}{\phi_s\cdot bv\cdot dv} = 0.091$ ksi *LRFD Eq. 5.8.2.9-1*

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

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

$\text{MaxSpacing} := \text{floor}(\text{spacingProgram}(vu, dv, fc)) = 24$ in

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$\text{Spacecheck}(\text{MaxSpacing}, s, V_u, V_c) := \left| \begin{array}{l} a \leftarrow \min(\text{MaxSpacing}, 12) \ \text{ if } V_u \le 0.5 0.9 V_c \\ a \leftarrow \min(s, \text{MaxSpacing}, 12) \ \text{ otherwise} \\ a \end{array} \right.$$

$\text{sNOhinge} := \text{Spacecheck}(\text{MaxSpacing}, \text{sNOhinge}, V_u, V_c) = 9$ in

Design Summary - Bent 2

StirrupSize = "#4"

$s = 4$ in

$\text{sNOhinge} = 9$ in

$\text{PHL} = 42$ in

$\text{Extension} = 21$ in

$N_2 = 19.81$ in

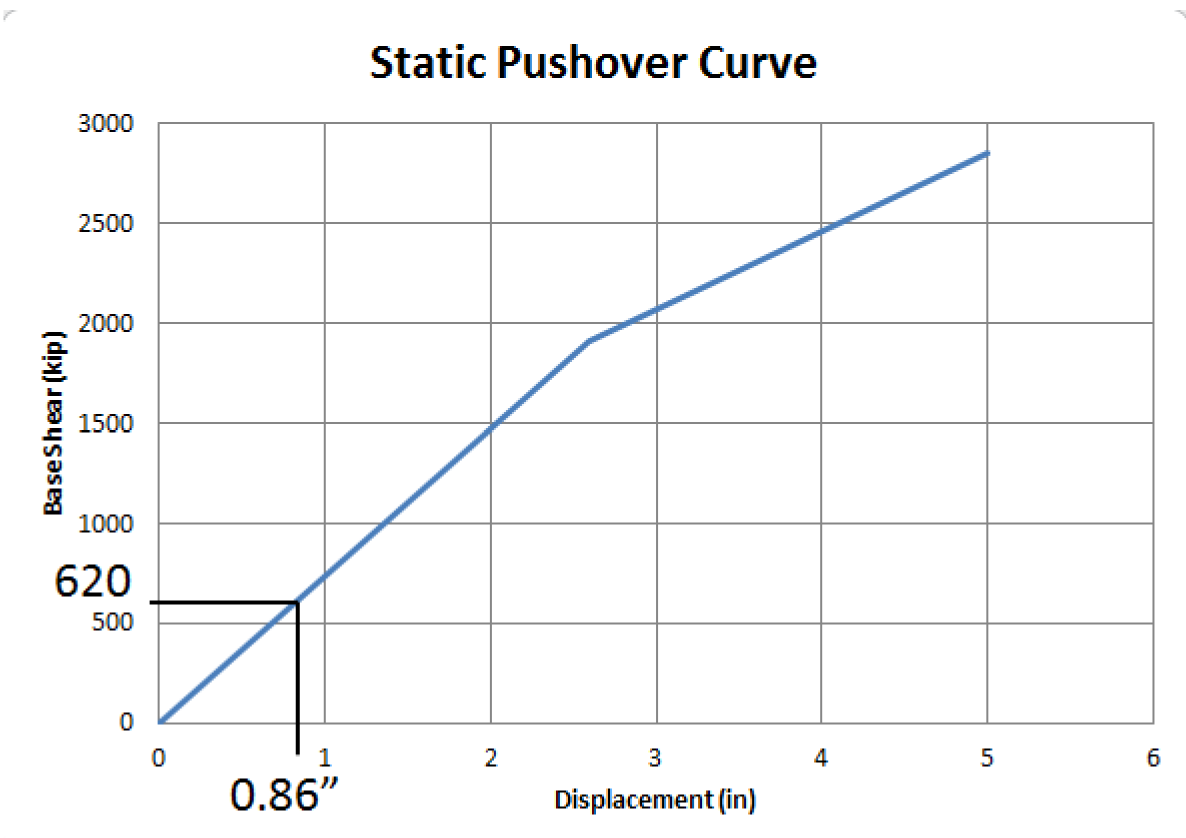
Design Check Summary - Bent 2

Shearcheck = "OK"   Shear capacity > Vn

Transversecheck = "OK"	Minimum shear reinforcement ratio
ReinforcementRatioCheck = "OK"	Maximum longitudinal reinforcement ratio
MinimumA_l = "OK"	Minimum longitudinal reinforcement ratio
scheck = "OK"	Max spacing of transverse reinforcement
Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran= "OK"	Minimum shear reinforcement outside hinge zone

Transverse Connection Design

Pushover Analysis Results



ALDOT Current Connection Steel Angle Design Check

$$Vcolbent := \frac{620}{N} = 41.333 \quad \text{kips}$$

LRFD Article 6.5.4.2: Resistance Factors

$\phi_t := 0.8$ Tension for A307

$\phi_s := 0.75$ Shear for A307

$\phi_{bs} := 0.8($ Block Shear

$\phi_{bb} := 0.8($ Bolts Bearing

$\phi_{sc} := 0.85$ Shear Connectors

$\phi_f := 1.0($ Flexure

$\phi_{sangle} := 1.0($ Shear for the Angle

Bolt Properties

$Fub := 58$	ksi	Strength of Anchor Bolt (It is assumed that ASTM A307 Grade C bolt is used)
$Dia_b := 1.75$	in	Diameter of Anchor Bolt
$Ns := 1$		Number of Shear Planes per Bolt

INPUT

Angle Properties

$Fy := 36$	ksi	Yield Stress of the Angle
$Fu := 58$	ksi	Ultimate Stress of the Angle
$t := 1.0($	in	Thickness of Angle
$h := 6$	in	Height of the Angle
$w := 6$	in	Width of the Angle
$l := 12$	in	Length of the Angle
$k := 1.5$	in	Height of the Bevel
$distanchorhole := 4$	in	Distance from the vertical leg to the center of the hole. This is the location of the holes.
$diahole := Dia_b + \frac{1}{8} = 1.875$	in	Diameter of bolt hole
$BLSHlength := 6$	in	Block Shear Length
$BLSHwidth := 2$	in	Block Shear Width
$Ubs := 1.($		Shear Lag Factor for Block Shear
$a := 2$	in	Distance from the center of the bolt to the edge of plate

INPUT

$b := 3.5$ in — Distance from center of bolt to toe of fillet of connected part

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

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

Clip Angle Check:

AISC J4: Block Shear

$Agv := t \cdot BLSHlength = 6$ in^2

$Anv := t \cdot (BLSHlength - 0.5 diahole) = 5.063$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 diahole) = 1.063$ in^2

AISC Eq. J4-5

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

$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 191.225$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 152.98$ kips

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

AISC D2: Tension Member

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

$Ant := t \cdot [w - (1 \cdot diahole)] = 4.125$ in^2

$Ae := Ant \cdot Ut = 2.475$ in^2 ***AISC Eq. D3-1***

$\phi tPn := \phi_t \cdot Fub \cdot Ae = 114.84$ kips ***AISC Eq. D2-2***

$TensionCheck_{AISC} := ShearCheck(\phi tPn, Vcolbent) = \text{"OK"}$

AISC G: Shear Check

$Cv := 1.0$

$Aw := t \cdot w = 6$ in^2

$\phi sangleVn := \phi_{sangle} \cdot 0.6 Fy \cdot Aw \cdot Cv = 129.6$ kips ***AISC Eq. G2-1***

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

Anchor Bolt Check:

LRFD Article 6.13.2.12: Shear Resistance For Anchor Bolts

$A_b := \dfrac{\pi \cdot Dia_b^{\,2}}{4} = 2.405$ in^2

$\phi sRn := \phi_s \cdot 0.48 A_b \cdot Fub \cdot Ns = 50.222$ kips ***LRFD Eq. 6.13.2.12-1***

$Shear_{Anchorbolts} := ShearCheck(\phi sRn, Vcolbent) = \text{"OK"}$

LRFD Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

$\phi bbRn := 2.4 Dia_b \cdot t \cdot Fub = 243.6$ kips ***LRFD Eq. 6.13.2.9-1***

For Slotted Holes

$\phi bbRns := Lc \cdot t \cdot Fub = 116$ kips ***LRFD Eq. 6.13.2.9-4***

$Bearing_{Boltstandard} := ShearCheck(\phi bbRn, Vcolbent) = \text{"OK"}$

$Bearing_{Boltslotted} := ShearCheck(\phi bbRns, Vcolbent) = \text{"OK"}$

LRFD Article 6.13.2.10: Tensile Resistance

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.

$Tu := \dfrac{Vcolbent \cdot 1}{distanchorhole} = 10.333$ kips

LRFD Eq. 6.13.2.10.2-1

$\phi tTn := \phi_t \cdot 0.76 A_b \cdot Fub = 84.82$ kips

$TensionCheck := ShearCheck(\phi tTn, Tu) = "OK"$

Article 6.13.2.11: Combined Tension and Shear

$Pu := Vcolbent$

LRFD Eq. 6.13.2.11-1
LRFD Eq. 6.13.2.11-2

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

$Tn_{combined} := CombinedProgram(Pu, A_b, Fub, \phi sRn, \phi_s) = 60.225$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 48.18$ kips

$CombinedCheck := ShearCheck(\phi tTn_{combined}, Vcolbent) = "OK"$

Summary

$Dia_b = 1.75$ in

$Shear_{Anchorbolts} = "OK"$

$Bearing_{Boltstandard} = "OK"$

$Bearing_{Boltslotted} = "OK"$

$TensionCheck = "OK"$

$CombinedCheck = "OK"$

$BlockShearCheck = "OK"$

$TensionCheck_{AISC} = "OK"$

$ShearAngleCheck = "OK"$

Appendix I: Bent Creek Road Moment-Interaction Diagrams

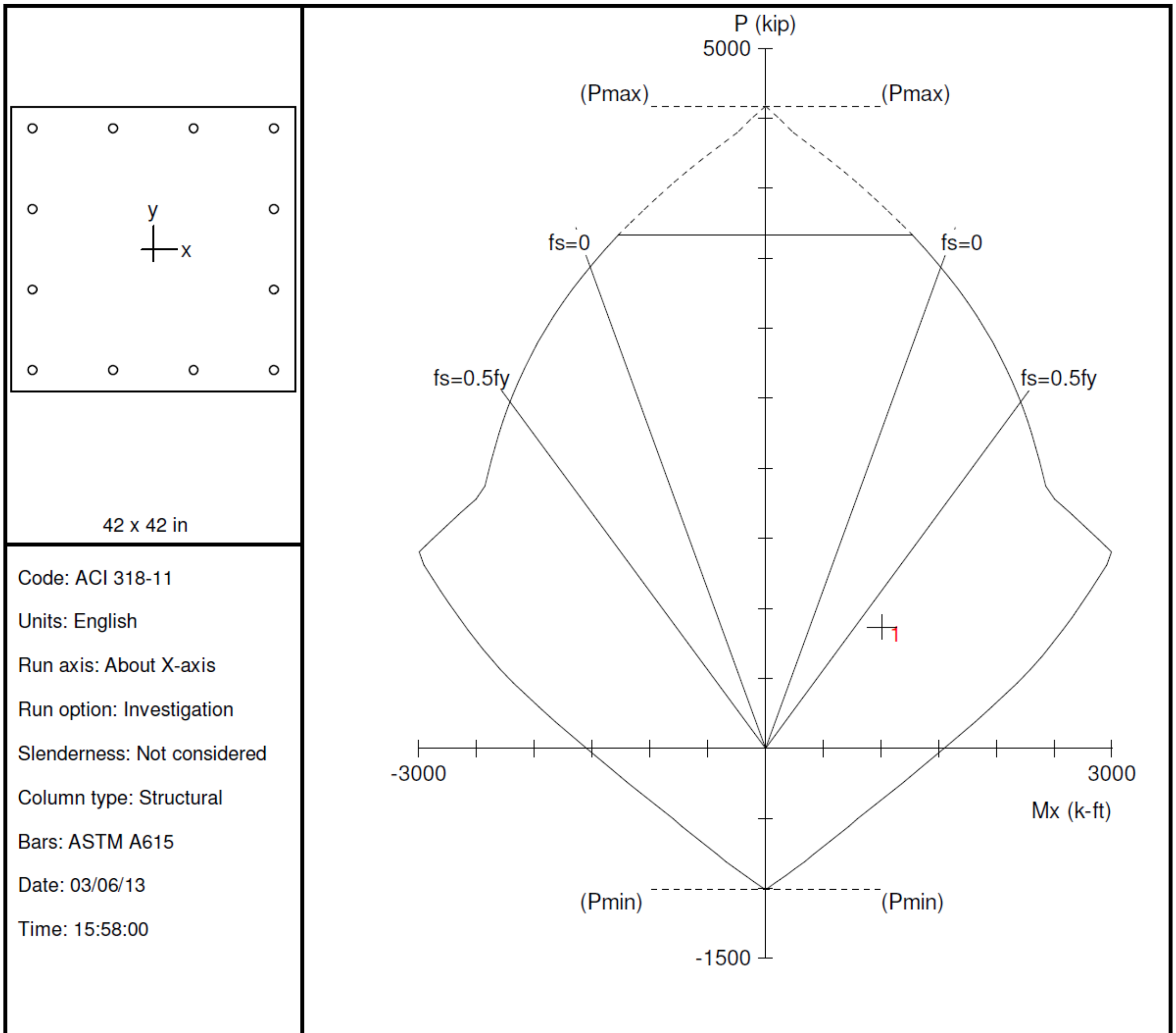
Bent 2

```
STRUCTUREPOINT - spColumn v4.81 (TM)                                                                   Page   1
15 day trial license. Locking Code: 4-1EC20. User: oem, Hewlett-Packard Company                        03/06/13
C:\Users\jd10003\Documents\Research\ALDOT Bridge Design Examples\SDC...\Bent Creek Road Bent 2.col     03:57 PM
```

```
                     oooooo               o
                    oo    oo             oo
 ooooo    oooooo    oo          ooooo    oo      oo   oo   o oooooooooo    o ooooo
oo    o   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
 ooooo    oo   oo   oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
     oo   oooooo    oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
o    oo   oo        oo    oo   oo   oo   oo  o   oo   oo    oo   oo   oo    oo   oo
 ooooo    oo         oooooo     ooooo     ooo     ooooo o   oo   oo   oo    oo   oo (TM)
```

```
=========================================================================================================
                                          spColumn v4.81 (TM)
                    Computer program for the Strength Design of Reinforced Concrete Sections
                                Copyright © 1988-2012, STRUCTUREPOINT, LLC.
                                           All rights reserved
=========================================================================================================
```

```
General Information:
====================
  File Name: C:\Users\jdl0003\Documents\Research\ALDOT Bridge Design Exa...\Bent Creek Road Bent 2.col
  Project:
  Column:                                  Engineer:
  Code:     ACI 318-11                     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:
========
  Rectangular: Width = 42 in               Depth = 42 in

  Gross section area, Ag =  1764 in^2
  Ix =  259308 in^4                        Iy =  259308 in^4
  rx =  12.1244 in                         ry =  12.1244 in
  Xo =  0 in                               Yo =  0 in

Reinforcement:
==============
  Bar Set: 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: Rectangular
  Pattern: All Sides Equal  (Cover to transverse reinforcement)
  Total steel area: As = 18.72 in^2 at rho = 1.06%
  Minimum clear spacing = 10.45 in

  12 #11   Cover = 2 in

Factored Loads and Moments with Corresponding Capacities:
=========================================================
               Pu          Mux       PhiMnx PhiMn/Mu NA depth Dt depth    eps_t    Phi
  No.         kip         k-ft         k-ft               in       in
  --- ------------ ------------ ------------ -------- -------- -------- -------- ------
    1      864.00      1012.50      2592.14    2.560    10.58    38.79  0.00800  0.900

  *** End of output ***
```



Code: ACI 318-11

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 03/06/13

Time: 15:58:00

spColumn v4.81. 15 day trial license. Locking Code: 4-1EC20. User: oem, Hewlett-Packard Company

File: C:\Users\jdl0003\Documents\Research\ALDOT Bridge Design Examples\SDC B\Moment I...\Bent Creek Road Bent 2.col

Project:

Column: Engineer:

f'c = 4 ksi	fy = 60 ksi	Ag = 1764 in^2	12 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 18.72 in^2	rho = 1.06%
fc = 3.4 ksi		Xo = 0.00 in	Ix = 259308 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 259308 in^4
Beta1 = 0.85		Min clear spacing = 10.45 in	Clear cover = 2.50 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Appendix J: I-59 Bridge over Norfolk Southern Railroad SDC B

Designer: Jordan Law

ORIGIN:= 1

Project Name: Norfolk Southern RR
Job Number: STMAAF-1059 (342)
Date: 6/4/2012
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

Coordinates: 34.125N, 85.982W

Soil Site Class: D

Superstructure Type: BT-54 girders for both spans

Substructure Type: Rectangular columns supported on piles

Abutment Type: Abutment beam supported on piles

The designer should input any information that can be used to calculate the dead weight of the bridge, including but not limited to length of bridge, column height(s), deck thickness, bent volume(s), and guard rail volume(s). Also, information about foundations should also be included if the bridge is classified as SDC B.
Note: ***Input** all of the below information*.

$fc := 400($ psi

$A_s := .14$

$fye := 6000($ psi

$S_{D1} := .1$

INPUT

$\rho_{conc} := 0.0868 \frac{lb}{in^3}$

$S_{DS} := .2$

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

SDC:= "B"

Length of Bridge (ft)	$L := 26$	ft
Skew of Bridge (degrees)	$Skew := 0$	degrees
Span Length 1 (ft)	$Span1 := 12$	ft
Span Length 2 (ft)	$Span2 := 14($	ft
Deck Thickness (in)	$t_{deck} := 6$	in
Superstructure Depth (ft)	$D_s := 5$	ft
Deck Width (ft)	$DeckWidth := 46.7$	ft
Number of BridgeGirders	$N := 9$	

Bulb (BT-54) Girder X-Sectional Area (in^2) $BulbGirderArea := 763$ in^2

Guard Rail Area (in^2) $GuardRailArea := 310$ in^2

Bent Volume (ft^3) $BentVolume := 53 \cdot (4.54) = 954$ ft^3

Column Width (in) $Columnwidth := 42$ in

The column height is measured from the bottom of the bent to the top of the pile footing. Other options include measuring from the top of the bent to the ground surface or to a change in diameter(if possible). If the plastic hinge location (at the bottom of the column) is known, then the column height should be measured from the bottom of the bent to the known hinge point.

Average Column Height for Bent 2 (ft) $ColumnHeight_{Bent2} := 25.25$ ft

Tallest Abutment Height Above Ground (ft) $H_{abutment} :=$ ft

Column Area (in^2) $Acolumn := Columnwidth^2 = 1.764 \times 10^3$ in^2

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

$Span1 := Span1 \cdot 12 = 1.5 \times 10^3$ in

$Span2 := Span2 \cdot 12 = 1.68 \times 10^3$ in

$L := L \cdot 12 = 3.18 \times 10^3$ in

$DeckWidth := DeckWidth \cdot 12 = 561$ in

$BentVolume := BentVolume \cdot 12^3 = 1.649 \times 10^6$ in^3

$ColumnHeight_{Bent2} := ColumnHeight_{Bent2} \cdot 12 = 303$ in

Find Vertical Reactions at Each Bent:

Live Loads assumed to be present during an earthquake (see ***LRFD Article 3.4.1***)

$$Num\_Lanes := trunc\left(\frac{\frac{DeckWidth - 2 \cdot 1.375}{12}}{12}\right) = 3$$

Number of Lanes On Bridge (Design Lane Width of 10 ft) See ***LRFD 3.6.1.2.4***

$\gamma_{EQ} := 0.5$ ***LRFD Specificaiton C3.4.1 (Extreme Case I)*** **INPUT**

The γ_{EQ} value is to be determined on a project-specific basis. In the standard specification, a value of 0.0 was used, however, the LRFD Specification recommends a value of 0.5. See LRFD Article C3.4.1 under "EXTREME EVENT I"

$LL\_design := 0.64 \frac{klf}{lane}$ ***LRFD Specification 3.6.1.2.4***

$Q := LL\_design \cdot \gamma_{EQ} = 0.32$ $\frac{klf}{lane}$

$LL\_foot := Q \cdot Num\_Lanes = 0.96$ klf Live Load per linear foot of deck (includes all lanes)

Note: If the Vertical Reactions at each bent are already known, input them below, otherwise the sheet will calculate vertical reactions based on the given information above.

$DL_{Bent2} :=$ ▪ kip $LL_{Bent2} :=$ ▪ kip

INPUT

$DL_{Bent3} :=$ ▪ kip $LL_{Bent3} :=$ ▪ kip

$VR_{Bent2} := DL_{Bent2} + LL_{Bent2} =$ ▪ kip

$VR_{Bent3} := DL_{Bent3} + LL_{Bent3} =$ ▪ kip

The weight calculation takes into account the entire dead weight of the structure, including the deck, bents, abutments, columns, girders, and railings. Any other expected dead loads should also be included.

$$W := \frac{\rho_{conc} \cdot \begin{pmatrix} L \cdot t_{deck} \cdot DeckWidth + BentVolume + 3 \cdot Acolumn \cdot ColumnHeight_{Bent2} \ldots \\ + Span1 \cdot N \cdot BulbGirderArea + Span2 \cdot N \cdot BulbGirderArea \ldots \\ + 2 \cdot GuardRailArea \cdot L \end{pmatrix}}{1000}$$

$W = 3278.339$ kips

To determine the vertical reaction at the bent, the bents tributary area will be calculated and multiplied by the total weight. A similar calculation will be done for the live load. This vertical reaction will be used to determine the connection force (below).

$$BentTribLength := \frac{\frac{Span1 + Span2}{2}}{12} = 132.5 \quad ft$$

$$BentTribArea := \frac{\frac{Span1 + Span2}{2}}{L} = 0.5$$ Percent of Area Tributary to Bent

$DL_{Bent} := BentTribArea \cdot W = 1639.169$ kip

$LL_{Bent} := BentTribLength \cdot LL\_foot = 127.2$ kip

$VR_{Bent} := DL_{Bent} + LL_{Bent} = 1766.369$ kip

$VR_{Bent2} := VR_{Bent}$

Steps for Seismic Design

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

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

Article 3.4: Determine Deisgn Response Spectrum

Article 3.5: Determine SDC

Guide Figure 1.3-2: Seismic Design Procedure Flowchart for SDC B

Displacement Demand Analysis (Fig 1.3-2):

- *Article* 4.1: Seismic Design Proportioning
- *Article* 4.2: Determine Analysis Procedure
- *Article* 4.3.1: Determine Horizontal Ground Motion Effects Along Both Axis
- *Article* 4.3.2/4.3.3: Damping and Short Period Considerations
- *Article* 5.4/5.5: Select Analytical Procedure
- *Article* 5.6: Effective Section Properties
- *Article* 5.2: Abutment Modeling
- *Article* 5.3: Foundation Modeling and Liquefaction (if present)
- *Article* 5.1.2/4.4: Conduct Demand Analysis
- *Article* 4.8: Determine Displacement Demands Along Member Local Axes

Displacement Capacity Check ($\Delta_C > \Delta_D$):

- *Article* 4.12: Determine Minimum Support Length
- *Article* 4.14: Shear Key

Guide Figure 1.3-5: Foundation and Detailing Flowcharts

Foundation Design (Fig 1.3-5):

- *Article* 6.8: Liquefaction Consideration
- *Article* 6.3: Spread Footing Design
- *Article* 6.4: Pile Cap Foundation Design
- *Article* 6.5: Drilled Shaft
- *Article* 6.7: Abutment Design

Detailing:

- *Article* 8.3: Determine Flexure and Shear Demands
- *Article* 8.7: Satisfy Requirements for Ductile Member Design
- *Article* 8.6: Shear Demand and Capacity Check for Ductile Elements
- *Article* 8.8: Satisfy Lateral and Longitudinal Reinforcement Requirements

Articles 3.4 and 3.5 have already been determined from the "SDC Classification" sheet.
Make sure the four values (A_S, S_{DS}, S_{D1}, and SDC) have been input above.

Displacement Demand Analysis (Δ_D)

***Article 4.1*: Seismic Design Proportioning**

See Guide Specification

***Article 4.2*: Determine Analysis Procedure**

This is a function of the SDC and the regularity of the bridge.
For a regular bridge in SDC B, Procedure 1 or 2 can be used.
For a non-regular bridge in SDC B, Procedure 2 must be used.

Guide Table 4.2-1

A regular bridge is defined as a bridge having fewer than 7 spans, no abrupt or unusual change in geometry and that saisfy the requirements below (***Guide Table 4.2-3***)

Table 4.2-3: Regular Bridge Requirements

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle (curved bridge)	30°	30°	30°	30°	30°
Maximum span length ratio from span-to-span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span-to-span (excluding abutments)	-	4	4	3	2

***Article 4.3.1*: Determine Horizontal Ground Motion Effects Along Both Axis**

Seismic displacement demands shall be determined independently in two orthogonal directions, typically the longitudinal and transverse axes of the bridge

Article 4.3.3: Displacement Magnification for Short-Period Structures

$u_d := 2$ for SDC B

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

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)**

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.

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

$$\upsilon_{smaxLong} := 0.067 \quad in$$

INPUT

$$\upsilon_{smaxTran} := 11.478 \quad in$$

$$K_{Long} := \frac{p_o \cdot L}{\upsilon_{smaxLong}} = 4.718 \times 10^4 \quad \frac{kip}{in}$$

Guide Eq. C5.4.2-1

$$K_{Tran} := \frac{p_o \cdot L}{\upsilon_{smaxTran}} = 277.03 \quad \frac{kip}{in}$$

Guide Eq. C5.4.2-2

The weight of the structure has already been calculated above

Step 4: Calculate the period, T_m.

$$T_{mLong} := 2\pi \cdot \sqrt{\frac{W}{K_{Long} \cdot g}} = 0.084 \quad s$$

Guide Eq. C5.4.2-3

Step 5: Calculate equivalent static earthquake loading p_e.

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

$$Sa_{Long} := \mathrm{acc}(S_{DS}, S_{D1}, T_{mLong}, A_s) = 0.262$$

$$p_{eLong} := \frac{Sa_{Long} \cdot W}{L} = 0.27 \qquad \frac{kip}{in}$$ ***Guide Eq. C5.4.2-4***

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.

$$Rd_{Long} := \mathrm{Rdprogram}(T_{mLong}, S_{DS}, S_{D1}, u_d) = 4.337$$

$$v_{smaxLong} := Rd_{Long} \cdot \frac{p_{eLong}}{p_o} \cdot \upsilon_{smaxLong} = 0.079 \qquad in$$

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

Step 4: Calculate the period, T_m.

$$T_{mTran} := 2\pi \cdot \sqrt{\frac{W}{K_{Tran} \cdot g}} = 1.1 \qquad s$$ ***Guide Eq. C5.4.2-3***

Step 5: Calculate equivalent static earthquake loading p_e.

$$Sa_{Tran} := \mathrm{acc}(S_{DS}, S_{D1}, T_{mTran}, A_s) = 0.136$$

$$p_{eTran} := \frac{Sa_{Tran} \cdot W}{L} = 0.141 \qquad \frac{kip}{in}$$ ***Guide Eq. C5.4.2-4***

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.

$$Rd_{Tran} := Rdprogram\left(T_{mTran}, S_{DS}, S_{D1}, u_d\right) = 1$$

$$v_{smaxTran} := Rd_{Tran} \cdot \frac{p_{eTran}}{p_o} \cdot \upsilon_{smaxTran} = 1.614 \qquad \text{in}$$

LRFD Article 4.7.4.3.2: Single-Mode Spectral Method

<u>Single Mode Spectral Analysis</u>

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

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.

$$v_{stran}(x) := 2 \cdot 10^{-7} \cdot x^2 + 0.0025x + 1.503$$

INPUT

$$v_{slong}(x) := 7 \cdot 10^{-9} \cdot x^2 + 2 \cdot 10^{-5} \cdot x + 0.062$$

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

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

$$\gamma_{Tran} := \int_0^L \frac{W}{L} \cdot v_{stran}(x)^2\,dx = 1.513 \times 10^5 \qquad \gamma_{Long} := \int_0^L \frac{W}{L} \cdot v_{slong}(x)^2\,dx \qquad \textbf{\textit{LRFD C4.7.4.3.2b-3}}$$

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

$$T_{mTran1} := 2\pi \cdot \sqrt{\frac{\gamma_{Tran}}{p_o \cdot g \cdot \alpha_{Tran}}} = 0.889 \quad s$$ ***LRFD Eq. 4.7.4.3.2b-4***

$$T_{mLong1} := 2\pi \cdot \sqrt{\frac{\gamma_{Long}}{p_o \cdot g \cdot \alpha_{Long}}} = 0.118 \quad s$$ ***LRFD Eq. 4.7.4.3.2b-4***

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smLong} := acc\left(S_{DS}, S_{D1}, T_{mLong1}, A_s\right) = 0.29$$

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.

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

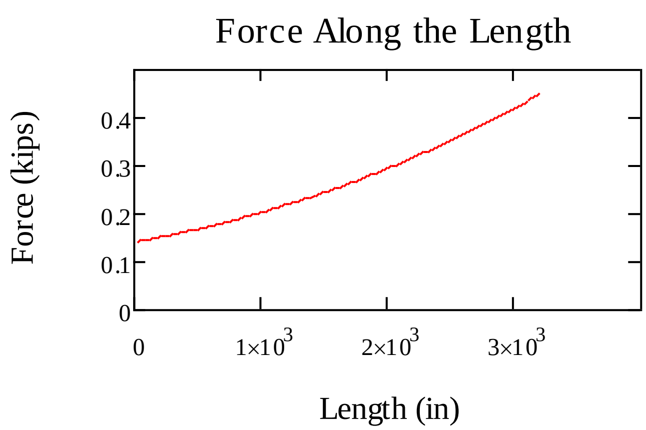
$$PeLong(x) \rightarrow 0.000045555931730057338187 \cdot x + 1.5944576105520068365e-8 \cdot x^2 + 0.14327340529103032$$

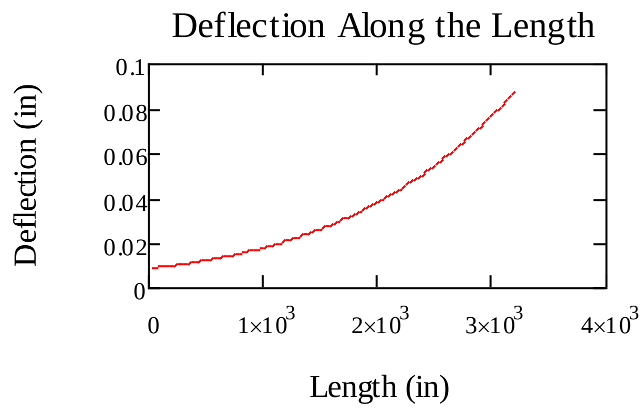
$$dW := \frac{L}{100}$$

$$i := 1..101$$

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

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





Maximum Deflection:

$\max(\Delta long) = 0.089$ in

NOTE: Repeat Steps 5 and 6 for Transverse Direction.

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smTran} := acc\left(S_{DS}, S_{D1}, T_{mTran1}, A_s\right) = 0.169$$

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.

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

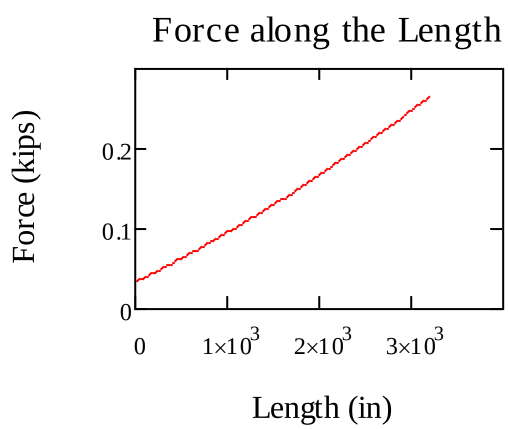
$$PeTran(x) \rightarrow 0.0000579582425916779935e6 \cdot x - 4.6366594073342394805e{-9} \cdot x^2 + 0.0348514504352278110$$

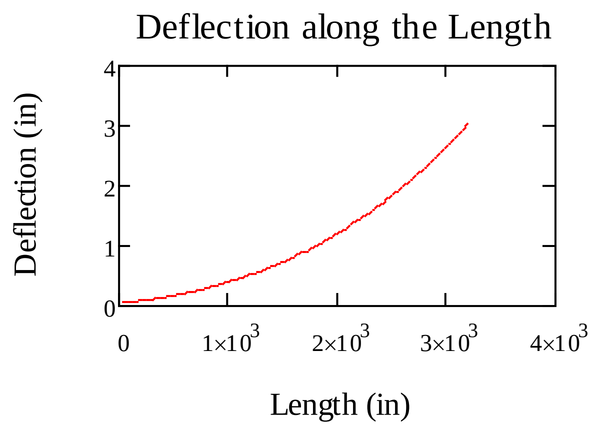
$$dL := \frac{L}{100}$$

$i := 1..101$

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

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





Maximum Deflection:

$\max(\Delta tran) = 3.053$ in

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

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**

This is taken care of in the SAP model.

***Article 5.3*: Foundations Modeling**

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.

Special provisions need to be considered if Liquefaction is present. ***Guide Article 6.8***

Article 4.4: **Combination of Orthogonal Seismic Displacement Demands**

$$LoadCase1 := \sqrt{(1 \cdot v_{smaxLong})^2 + (0.3 v_{smaxTran})^2} = 0.491 \quad in$$

$$LoadCase2 := \sqrt{(1 \cdot v_{smaxTran})^2 + (0.3 v_{smaxLong})^2} = 1.615 \quad in$$

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

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

$\Delta_{Dlong} := 0.041$ in

INPUT

$\Delta_{Dtran} := 5.601$ in

$\Delta_{DLong} := Rd_{Long} \cdot \Delta_{Dlong} \cdot p_{eLong} = 0.049$ in

$\Delta_{DTran} := Rd_{Tran} \cdot \Delta_{Dtran} \cdot p_{eTran} = 0.788$ in

$$LoadCase1 := \sqrt{(1\Delta_{DLong})^2 + (0.3\Delta_{DTran})^2} = 0.241 \quad in$$

$$LoadCase2 := \sqrt{(1\Delta_{DTran})^2 + (0.3\Delta_{DLong})^2} = 0.788 \quad in$$

$\Delta_D := \max(LoadCase1, LoadCase2) = 0.788$ in

$H_o := \dfrac{ColumnHeight_{Bent2}}{12} = 25.25$ ft $\qquad B_o := \dfrac{Columnwidth}{12} = 3.5$ ft

Transverse Direction

$\Lambda := 2$ Fixed and top and bottom ***Guide Article 4.8.1***

$x := \dfrac{\Lambda \cdot B_o}{H_o} = 0.277$ ***Guide Eq. 4.8.1-3***

$\Delta_{CT} := 0.12 H_o \cdot (-1.27 \cdot \ln(x) - 0.32) = 3.967$ in

Guide Eq. 4.8.1-1

Longitudinal Direction

Guide Article 4.8.1

$\Lambda := 1$ Fixed-Free

Guide Eq. 4.8.1-3

$x := \frac{\Lambda \cdot B_o}{H_o} = 0.139$

$\Delta_{CL} := 0.12 H_o \cdot (-1.27 \cdot \ln(x) - 0.32) = 6.634$ in

Guide Eq. 4.8.1-1

$\Delta_C := \min(\Delta_{CT}, \Delta_{CL}) = 3.967$

$0.12 H_o = 3.03$ in

$$\text{CheckLimit}(\Delta_C, H_o) := \left| \begin{array}{l} a \leftarrow 0.12 H_o \text{ if } \Delta_C < 0.12 H_o \\ a \leftarrow \Delta_C \text{ otherwise} \end{array} \right.$$

$\Delta_C := \text{CheckLimit}(\Delta_C, H_o)$

$\Delta_C = 3.967$

$$\text{CheckCapacity}(\Delta_C, \Delta_D) := \left| \begin{array}{l} c \leftarrow \text{"OK" if } \Delta_C \ge \Delta_D \\ c \leftarrow \text{"FAILURE" if } \Delta_C < \Delta_D \end{array} \right.$$

$\text{CheckCapacity}(\Delta_C, \Delta_D) = \text{"OK"}$

If the simplified equations do not work ("FAILURE") for any of the bents, 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.

Pushover Analysis Results (if necessary):

Article 4.12: Minimum Support Length Requirements

Abutment Support Length Requirement *Guide Eq. 4.12.2-1*

$N_{abutment} := 1.5 \left(8 + 0.02 \cdot Span1 + 0.08 \cdot H_{abutment}\right) \cdot \left(1 + 0.000125 \cdot Skew_{abutment}^{\ 2}\right) = \blacksquare$ in

Bent Support Length Requirement *Guide Eq. 4.12.2-1*

$L := \text{BentTribLength} = 132.5$

$S_{D1} := 0.3($

$H := \frac{\text{ColumnHeight}_{Bent2}}{12} = 25.25$

Standard Specifications

$$N_{2Stan} := (8 + 0.02L + 0.08H)\cdot\left(1 + 0.000125Skew^2\right) = 12.67 \qquad \text{in}$$

ATC-49 Equation (New Design)

$$N_2 := \left[4 + .02L + .08H + 1.09\sqrt{H}\cdot\sqrt{1 + \left(2\cdot\frac{3}{8}\right)^2}\right]\cdot\left(\frac{1 + 1.25S_{D1}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 21.335 \qquad \text{in}$$

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

$V_{ok} := 1.5V_n$ *This does not apply to this bridge*

BENT 2 DESIGN

Guide Article 4.11.2: For SDC B, it is acceptable to use the moment capacity based on expected material strengths when the concrete reaches an extreme compressive fiber strain of 0.003.

Force Inputs

$M_{neBent2} := 300($	kip – ft	Nominal moment from PCA Column	
$V_{pelastic} := 60($	kip	Elastic shear from SAP2000 model	**INPUT**
$P_u := 128400($	lb	Axial load from earthquake and dead load combination	

Reinforcement Details

$Ag := Acolumn$

$Ae := 0.8Ag = 1411$	in^2	***Guide Eq. 8.6.2-2***	
$\mu_D := 2$		***Guide Article 8.6.2***	
$n := 2$		n: Number of individual interlocking spiral or hoop core sections	
StirrupSize:= "#4"		StirrupSize: Bar size used for stirrups	
$s := 4$	in	s: Spacing of hoops or pitch of spiral (in)	
sNOhinge := 9	in	sNOhinge: Spacing of hoops or pitch outside PHL	
Asp := 0.4(	in^2	Asp: Area of hoop reinforcement in direction of loading (in^2)	
Dsp := 0.5(	in	Dsp: Diameter of spiral or hoop reinforcing (in)	**INPUT**
Cover := 2	in	Cover: Concrete cover for the Column (in)	

$b := \text{Columnwidth}$ in — b: Width of rectangular column (in)

$d := b - \text{Cover} = 40$ in — d: Effective depth of section in direction of loading (in)

$\text{NumberBars} := 12$ — Total number of longitudinal bars in column cross-section

$A_{bl} := 1.56$ in^2 — Abl: Area of longitudinal bar

$d_{bl} := 1.41$ in — dbl: Diameter of longitudinal bar

$bv := \text{Columnwidth}$ — bv: Width of column side

Article 8.3: Determine Flexure and Shear Demands

The design forces shall be the lesser of the forces resulting from the overstrength plastic hinging moment capacity or unreduced elastic seismic forces in columns or pier walls.

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

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.

$\lambda_{mo} := 1.4$ for ASTM A 615 Grade 60 reinforcement ***Guide Article 8.5***

$M_{pBent2} := \lambda_{mo} \cdot M_{neBent2} \cdot 1000 \cdot 12 = 5.04 \times 10^{7}$ lb − in

$\text{Fixity} := \text{ColumnHeight}_{Bent2} = 303$ in

$$V_p := \frac{2 \cdot M_{pBent2}}{\text{Fixity} \cdot 1000} = 332.673 \text{ kips}$$

$V_{pelastic} := V_{pelastic} \cdot \max(p_{eTran}, p_{eLong}) = 162.979$ kips

Article 8.7: Requirements for Ductile Member Design

Each column must satisfy the minimum lateral flexural capacity

$\Lambda := 2$ Fixed and top and bottom ***Guide Article 4.8.1***

$$M_{neminBent} := 0.1 \cdot DL_{Bent} \cdot \left(\frac{\frac{\text{Fixity}}{12} + 0.5 D_s}{\Lambda} \right) = 2274.347 \text{ kip ft}$$ ***Guide Eq 8.7.1-1***

$$\text{CheckMoment}(M_{ne}, M_e) := \begin{array}{|l} a \leftarrow \text{"OK"} \quad \text{if } M_{ne} \le M_e \\ a \leftarrow \text{"FAILURE"} \quad \text{otherwise} \end{array}$$

$\text{CheckMoment}\left(M_{neminBent}, M_{neBent2}\right) = \text{"OK"}$

If the moment check comes back "FAILURE" then the column size can be increased or the reinforcement can be increased.

Article 8.6: Shear Demand and Capacity for Ductile Concrete Members

It is recommended to use the plastic hinging forces whenever practical, but in this case the elastic forces will be used.

$V_u := \min\left(V_p, V_{pelastic}\right) = 162.979$ kips $\qquad \phi_s := 0.9$

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

$\rho_w := \dfrac{Asp}{b \cdot s} = 0.0024$ ***Guide Eq. 8.6.2-10***

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

$$\text{StressCheck}\left(\rho_s, fyh\right) := \left| \begin{array}{l} fs \leftarrow 2\rho_s \cdot fyh \\ a \leftarrow fs \quad \text{if } fs \le 0.35 \end{array} \right.$$

$fw := \text{StressCheck}\left(\rho_w, fyh\right) = 0.286$ ***Guide Eq. 8.6.2-9***

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

$\alpha Prime := \alpha\text{program}\left(fw, \mu_D\right) = 3$ ***Guide Eq. 8.6.2-8***

If Pu is Compressive:

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

Guide Eq. 8.6.2-3

If Pu is NOT Compressive: ***Guide Eq. 8.6.2-4***

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

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

$Vc := vc \cdot Ae = 310.464$ kips

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

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

Guide Eq 8.6.3-2 and 8.6.4-1

$Vs := \text{vsprogram}(Asp, fyh, d, s, fc, Ae) = 240$ kips

$\phi Vn := \phi_s \cdot (Vs + Vc) = 495.418$ kips ***Guide Eq. 8.6.1-2***

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

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

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 Columns:

$$\text{mintranprogram}(\rho_w) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } \rho_w \geq 0.002 \\ a \leftarrow \text{"Increase Shear Reinforcing Ratio"} \quad \text{if } \rho_w < 0.00? \\ a \end{array} \right.$$

Guide Eq. 8.6.5-1

$$\text{Transversecheck} := \text{mintranprogram}(\rho_w) = \text{"OK"}$$

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**

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

$$\rho\text{program}(A_{long}, Ag) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } A_{long} \leq 0.04Ag \\ a \leftarrow \text{"Section Over Reinforced"} \quad \text{if } A_{long} > 0.04Ag \\ a \end{array} \right.$$

Guide Eq. 8.8.1-1

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

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**

$$\text{minAlprogram}(A_l, Ag) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } A_{long} \geq 0.007Ag \\ a \leftarrow \text{"Increase Longitudinal Reinforcing"} \quad \text{if } A_{long} < 0.007Ag \\ a \end{array} \right.$$

Guide Eq. 8.8.2-1

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

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

***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.

Guide Eq. 4.11.6-1

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

$$L_p := \text{PlasticHinge}\left(\text{Fixity}, \text{fye}, d_{bl}\right) = 36.93 \qquad \text{in}$$

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

'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 ρ_{eTran} to take into account the model loads have not been multiplied by ρ_{eTran}. The location will also need to be **<u>INPUT</u>** into the PlasticHingeRegion program in inches.

$Mp75 := 0.75 M_{pBent2} = 3.78 \times 10^{7}$ lb·in

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

$L_{p1} := PlasticHingeRegion(L_p, Columnwidth) = 63$ in

The Guide Specifications allows for the use of the plastic hinge length from the LRFD Specification in SDC A and B (***Guide Article C8.8.9***).

$$LRFDPlasticHingeLength(ColumnDia, ColumnHeight) := \begin{array}{|l} a \leftarrow ColumnDia \\ b \leftarrow \frac{1}{6} \cdot ColumnHeight \\ c \leftarrow 18 \\ PHL \leftarrow max(a, b, c) \\ PHL \end{array}$$

$L_{p2} := LRFDPlasticHingeLength(Columnwidth, Fixity) = 50.5$ in

Guide Article C8.8.9

The plastic hinge length will be the smaller of the two values, as the Guide Specification allows:

$PHL := min(L_{p1}, L_{p2}) = 50.5$ in

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region: ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

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

$MaximumSpacing := Spacingprogram(Columnwidth, d_{bl}) = 6$ in

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

$FINALSPACING := SpacingCheck(MaximumSpacings) = 4$ in

$scheck := ShearCheck(MaximumSpacings) = \text{"OK"}$

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

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.

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

$Extension := ExtensionProgram(Columnwidth) = 21$ in

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

LRFD 5.8.3.3 Nominal Shear Resistance

$V_u = 162.979$ kips

$\phi_s = 0.9$

$\beta := 2.0$ ***LRFD Article 5.8.3.4.1***

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

$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 38.795$ in

$de := d = 40$ in ***LRFD Eq. 5.8.2.9-1***

$dv := 0.9 de = 36$ in

$V_c := 0.0316 \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 191.117$ kips ***LRFD Eq. 5.8.3.3-3***

$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{sNOhinge} = 192$ kips ***LRFD Eq. 5.8.3.3-4***

$V_n := \phi_s \cdot .25 fc \cdot bv \cdot dv = 1.361 \times 10^6$

kips

$\phi Vn := \min\left[V_n, \left(V_c + V_s\right)\cdot\phi_s\right] = 344.805$ kips

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

$\text{Shearcheck2} := \text{ShearCheck}\left(\phi Vn, V_u\right) = \text{"OK"}$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

$$\text{Avmin} := 0.0316\sqrt{\frac{fc}{1000}}\cdot\frac{bv\cdot sNOhinge}{\frac{fye}{1000}} = 0.398 \qquad in^2$$

LRFD Eq. 5.8.2.5-1

$Av := Asp = 0.4$ in^2

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

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

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.

LRFD 5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

$$vu := \frac{V_u}{\phi_s\cdot bv\cdot dv} = 0.12 \qquad ksi$$

LRFD Eq. 5.8.2.9-1

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

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

$\text{MaxSpacing} := \text{floor}(\text{spacingProgram}(vu, dv, fc)) = 24$ in

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$\text{Spacecheck}(\text{MaxSpacing}, s, V_u, V_c) := \left| \begin{array}{l} a \leftarrow \min(\text{MaxSpacing}, 12) \ \text{ if } V_u \le 0.5 0.9 V_c \\ a \leftarrow \min(s, \text{MaxSpacing}, 12) \ \text{ otherwise} \\ a \end{array} \right.$$

$\text{sNOhinge} := \text{Spacecheck}(\text{MaxSpacing}, \text{sNOhinge}, V_u, V_c) = 9$

Design Summary - Bent 2

StirrupSize = "#4"

$s = 4$ in

$\text{sNOhinge} = 9$ in

$\text{PHL} = 50.5$ in

$\text{Extension} = 21$ in

$N_2 = 21.335$ in

Design Check Summary - Bent 2

Shearcheck = "OK" Shear capacity > Vn

Transversecheck = "OK" Minimum shear reinforcement ratio

ReinforcementRatioCheck = "OK" — Maximum longitudinal reinforcement ratio

MinimumA_{l} = "OK" — Minimum longitudinal reinforcement ratio

scheck = "OK" — Max spacing of transverse reinforcement

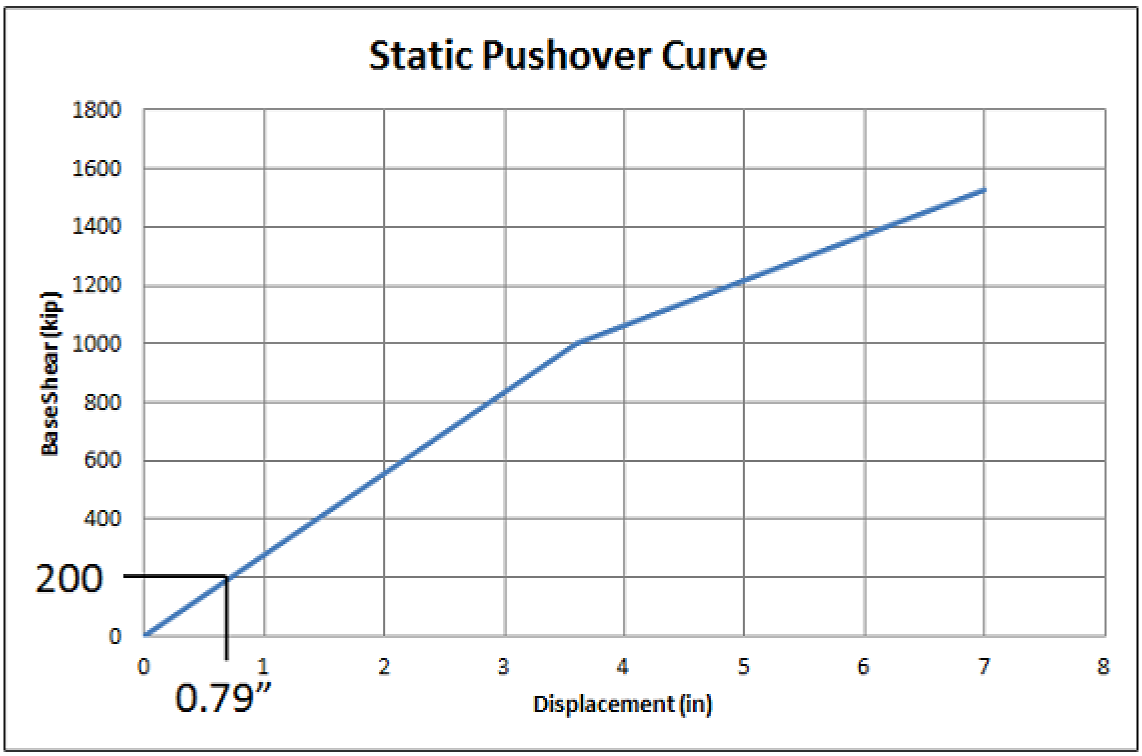
Shearcheck2 = "OK" — Shear capacity outside hinge zone > Vn

MinimumTran= "OK" — Minimum shear reinforcement outside hinge zone

scheck2 = "OK" — Max spacing of transverse reinforcement outside hinge zone

Transverse Connection Design

Pushover Analysis Results



ALDOT Current Connection Steel Angle Design Check

$$\text{Vcolbent} := \frac{200}{N} = 22.222 \quad \text{kips}$$

LRFD 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

Bolt Properties

$Fub := 58$	ksi	Strength of Anchor Bolt (It is assumed that ASTM A307 Grade C bolt is used)	
$Dia_b := 1.375$	in	Diameter of Anchor Bolt	**INPUT**
$Ns := 1$		Number of Shear Planes per Bolt	

Angle Properties

$Fy := 36$	ksi	Yield Stress of the Angle	
$Fu := 58$	ksi	Ultimate Stress of the Angle	
$t := 1.00$	in	Thickness of Angle	
$h := 6$	in	Height of the Angle	
$w := 6$	in	Width of the Angle	
$l := 12$	in	Length of the Angle	
$k := 1.5$	in	Height of the Bevel	**INPUT**
$distanchorhole := 4$	in	Distance from the vertical leg to the center of the hole. This is the location of the holes.	
$diahole := Dia_b + \frac{1}{8} = 1.5$	in	Diameter of bolt hole	
$BLSHlength := 6$	in	Block Shear Length	
$BLSHwidth := 2$	in	Block Shear Width	
$Ubs := 1.0$		Shear Lag Factor for Block Shear	
$a := 2$	in	Distance from the center of the bolt to the edge of plate	

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

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

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

Clip Angle Check:

AISC J4: Block Shear

$Agv := t \cdot BLSHlength = 6$ in^2

$Anv := t \cdot (BLSHlength - 0.5 diahole) = 5.25$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 diahole) = 1.25$ in^2

AISC Eq. J4-5

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

$Rn := \mathrm{BLSHprogram}(Agv, Anv, Ant, Ubs, Fu, Fy) = 202.1$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 161.68$ kips

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

AISC D2: Tension Member

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

$Ant := t \cdot [w - (1 \cdot diahole)] = 4.5$ in^2

$Ae := Ant \cdot Ut = 2.7$ in^2 ***AISC Eq. D3-1***

$\phi tPn := \phi_t \cdot Fub \cdot Ae = 125.28$ kips ***AISC Eq. D2-2***

$TensionCheck_{AISC} := \mathrm{ShearCheck}(\phi tPn, Vcolbent) = \text{"OK"}$

AISC G: Shear Check

$Cv := 1.0$

$Aw := t \cdot w = 6$ in^2

$\phi sangleVn := \phi_{sangle} \cdot 0.6 Fy \cdot Aw \cdot Cv = 129.6$ kips ***AISC Eq. G2-1***

$ShearAngleCheck := ShearCheck(\phi sangleVn, Vcolbent) = "OK"$

Anchor Bolt Check:

LRFD Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 1.485 \quad in^2$$

$\phi sRn := \phi_s \cdot 0.48 A_b \cdot Fub \cdot Ns = 31.005$ kips ***LRFD Eq. 6.13.2.12-1***

$Shear_{Anchorbolts} := ShearCheck(\phi sRn, Vcolbent) = "OK"$

LRFD Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

$\phi bbRn := 2.4 Dia_b \cdot t \cdot Fub = 191.4$ kips ***LRFD Eq. 6.13.2.9-1***

For Slotted Holes

$\phi bbRns := Lc \cdot t \cdot Fub = 116$ kips ***LRFD Eq. 6.13.2.9-4***

$Bearing_{Boltstandard} := ShearCheck(\phi bbRn, Vcolbent) = "OK"$

$Bearing_{Boltslotted} := ShearCheck(\phi bbRns, Vcolbent) = "OK"$

LRFD Article 6.13.2.10: Tensile Resistance

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.

$$Tu := \frac{Vcolbent \cdot 1}{distanchorhole} = 5.556 \quad kips$$

$\phi tTn := \phi_t \cdot 0.76 A_b \cdot Fub = 52.363$ kips

LRFD Eq. 6.13.2.10.2-1

TensionCheck := ShearCheck(ϕtTn , Tu) = "OK"

Article 6.13.2.11: Combined Tension and Shear

Pu := Vcolbent

LRFD Eq. 6.13.2.11-1
LRFD Eq. 6.13.2.11-2

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

$Tn_{combined} := \text{CombinedProgram}(Pu, A_b, Fub, \phi sRn, \phi_s) = 45.644$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 36.515$ kips

CombinedCheck := ShearCheck($\phi tTn_{combined}$, Vcolbent) = "OK"

Summary

$Dia_b = 1.375$ in

$Shear_{Anchorbolts}$ = "OK"

$Bearing_{Boltstandard}$ = "OK"

$Bearing_{Boltslotted}$ = "OK"

TensionCheck = "OK"

CombinedCheck = "OK"

BlockShearCheck = "OK"

$TensionCheck_{AISC}$ = "OK"

ShearAngleCheck = "OK"

Appendix K: I-59 Bridge over Norfolk Southern Railroad Moment-Interaction Diagrams

Bent 2

```
STRUCTUREPOINT - spColumn v4.81 (TM)                                                             Page   1
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untitled.col                                                                                     04:20 PM
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                    oo    oo            oo
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================================================================================================
                                     spColumn v4.81 (TM)
                Computer program for the Strength Design of Reinforced Concrete Sections
                              Copyright © 1988-2012, STRUCTUREPOINT, LLC.
                                         All rights reserved
================================================================================================
```
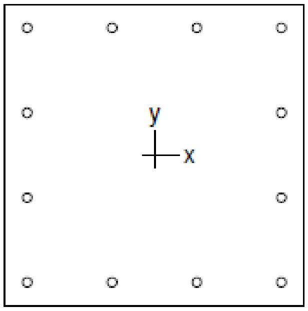

```
General Information:
====================
  File Name: untitled.col
  Project:
  Column:                                  Engineer:
  Code:     ACI 318-11                     Units: English

  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:
========
  Rectangular: Width = 42 in               Depth = 42 in

  Gross section area, Ag =  1764 in^2
  Ix =  259308 in^4                        Iy =  259308 in^4
  rx =  12.1244 in                         ry =  12.1244 in
  Xo =  0 in                               Yo =  0 in

Reinforcement:
==============
  Bar Set: 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: Rectangular
  Pattern: All Sides Equal  (Cover to transverse reinforcement)
  Total steel area: As = 18.72 in^2 at rho = 1.06%
  Minimum clear spacing = 10.45 in

  12 #11   Cover = 2 in

Factored Loads and Moments with Corresponding Capacities:
=========================================================
                Pu          Mux       PhiMnx PhiMn/Mu NA depth Dt depth    eps_t    Phi
  No.          kip         k-ft         k-ft                in       in
  --- ------------ ------------ ------------ -------- -------- -------- -------- ------
    1      1072.50      1415.00      2769.96    1.958    12.09    38.79  0.00663  0.900

  *** End of output ***
```



42 x 42 in

Code: ACI 318-11

Units: English

Run axis: About X-axis

Run option: Investigation

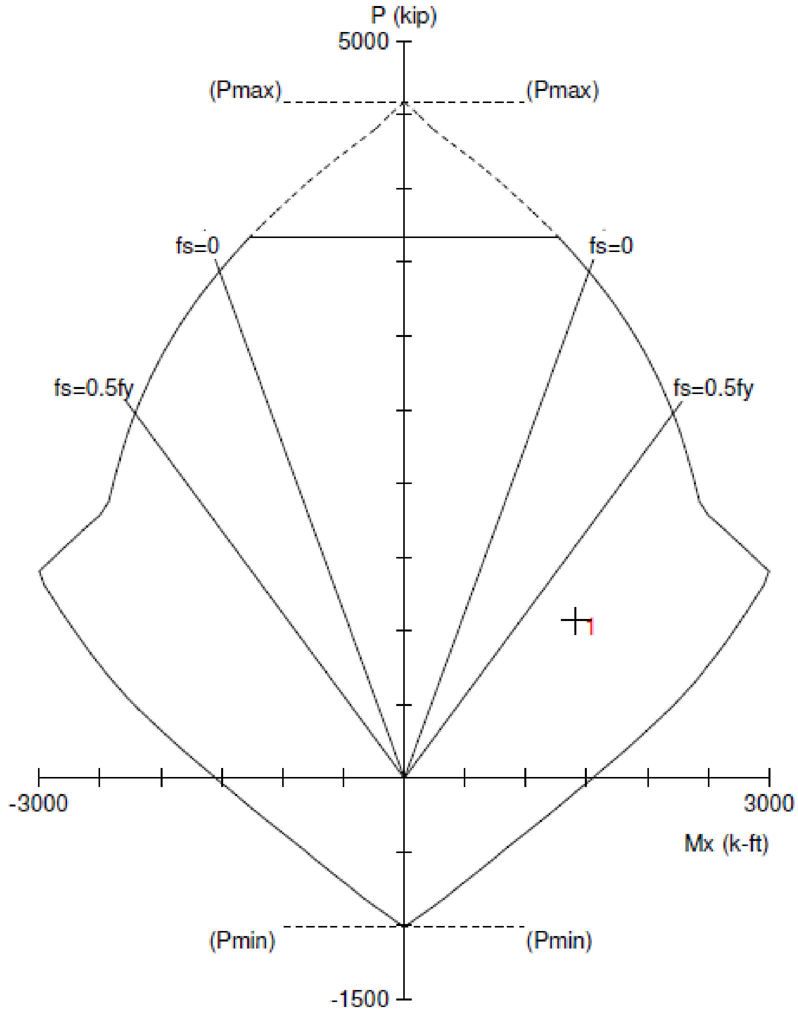
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 03/06/13

Time: 16:22:56



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File: untitled.col

Project:

Column:		Engineer:	
f'c = 4 ksi	fy = 60 ksi	Ag = 1764 in^2	12 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 18.72 in^2	rho = 1.06%
fc = 3.4 ksi		Xo = 0.00 in	Ix = 259308 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 259308 in^4
Beta1 = 0.85		Min clear spacing = 10.45 in	Clear cover = 2.50 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Appendix L: Oseligee Creek Bridge SDC B

ORIGIN:= 1

Designer: Jordan Law
Project Name: Oseligee Bridge
Job Number: BR-IV20 (515)
Date: 5/24/2012
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.

<u>Project Known Information</u>

Coordinates: 32.902N, 85.196W

Soil Site Class: D

Superstructure Type: AASTHO Type III girders for all spans

Substructure Type: Circular columns supported on drilled shafts

Abutment Type: Abutment beam supported on drilled shafts

The designer should input any information that can be used to calculate the dead weight of the bridge, including but not limited to length of bridge, column height(s), deck thickness, bent volume(s), and guard rail volume(s). Also, information about foundations should also be included if the bridge is classified as SDC B.

Note: ***Input*** *all of the below information.*

$fc := 4000$ psi

$fye := 60000$ psi

$\rho_{conc} := 0.0868 \ \frac{lb}{in^3}$

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

$A_s := .12$

$S_{D1} := .16$

$S_{DS} := .27$

SDC:= "B"

INPUT

Length of Bridge (ft)	$L := 240$	ft
Angle of skew of bridge (degrees)	$Skew := 0$	Degrees
Span (ft)	$Span := 80$	ft
Deck Thickness (in)	$t_{deck} := 7$	in
Deck Width (ft)	$DeckWidth := 32.75$	ft
Depth of Superstructure (ft)	$D_s := 4.187$	ft
Number of Bridge Girders	$N := 4$	
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

Number of Columns per Bent $N_{col} := 2$

Drilled Shaft Diameter (in) $DSdia := 42$ in

Drilled Shaft Abutment Diameter (in) $DSabutdia := 42$ in

The column height is measured from the bottom of the bent to the top of the pile footing. Other options include measuring from the top of the bent to the ground surface or to a change in diameter (if possible). If the plastic hinge location (at the bottom of the column) is known, then the column height should be measured from the bottom of the bent to the known hinge point.

Average Column Height of Bent 2 (ft) $ColumnHeight_{Bent2} := 17.93$ ft

Average Column Height of Bent 3 (ft) $ColumnHeight_{Bent3} := 25.83$ ft

Height of tallest abutment above ground (ft) $H_{abutment} :=$ ft

Column Area (in^2) $Acolumn := \dfrac{Columndia^2 \cdot \pi}{4} = 1.385 \times 10^3$ in^2

Drilled Shaft Area (in^2) $Adrilledshaft := \dfrac{DSdia^2 \cdot \pi}{4} = 1.385 \times 10^3$ in^2

Drilled Shaft Abutment Area (in^2) $Adsabut := \dfrac{DSabutdia^2 \cdot \pi}{4} = 1.385 \times 10^3$ 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

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

$ColumnHeight_{Bent2} := ColumnHeight_{Bent2} \cdot 12 = 215.208$ in

$ColumnHeight_{Bent3} := ColumnHeight_{Bent3} \cdot 12 = 310.008$ in

Find Vertical Reactions at Each Bent:

Live Loads assumed to be present during an earthquake (see ***LRFD Article 3.4.1***)

$$Num\_Lanes := trunc\left(\frac{\frac{DeckWidth - 2\cdot 1.375}{12}}{12}\right) = 2$$

Number of Lanes On Bridge (Design Lane Width of 10 ft) See ***LRFD 3.6.1.2.4***

$\gamma_{EQ} := 0.5$ ***LRFD Specificaiton C3.4.1 (Extreme Case I)*** **INPUT**

The γ_{EQ} value is to be determined on a project-specific basis. In the standard specification, a value of 0.0 was used, however, the LRFD Specification recommends a value of 0.5. See LRFD Article C3.4.1 under "EXTREME EVENT I"

$LL\_design := 0.64 \quad \frac{klf}{lane}$ ***LRFD Specification 3.6.1.2.4***

$$Q := LL\_design\cdot\gamma_{EQ} = 0.32 \quad \frac{klf}{lane}$$

$LL\_foot := Q\cdot Num\_Lanes = 0.64 \quad klf$ Live Load per linear foot of deck (includes all lanes)

Note: If the Vertical Reactions at each bent are already known, input them below, otherwise the sheet will calculate vertical reactions based on the given information above.

$DL_{Bent2} := \blacksquare$ kip $\quad LL_{Bent2} := \blacksquare$ kip **INPUT**

$DL_{Bent3} := \blacksquare$ kip $\quad LL_{Bent3} := \blacksquare$ kip

$$VR_{Bent2} := DL_{Bent2} + LL_{Bent2} = \blacksquare \quad kip$$

$$VR_{Bent3} := DL_{Bent3} + LL_{Bent3} = \blacksquare \quad kip$$

The weight calculation takes into account the entire dead weight of the structure, including the deck, bents, abutments, columns, girders, and railings. Any other expected dead loads should also be included.

$$W := \frac{\rho_{conc}\cdot\left(\begin{array}{l} L\cdot t_{deck}\cdot DeckWidth + 2\cdot BentVolume + 2\cdot Acolumn\cdot ColumnHeight_{Bent2} \ldots \\ + 2\cdot Acolumn\cdot ColumnHeight_{Bent3} + L\cdot N\cdot IGirderArea \ldots \\ + 2\cdot GuardRailArea\cdot L \end{array}\right)}{1000}$$

$$W = 1708.667 \quad kips$$

To determine the vertical reaction at the bent, the bents tributary area will be calculated and multiplied by the total weight. A similar calculation will be done for the live load. This vertical reaction will be used to determine the connection force (below).

$$BentTribLength := \frac{Span}{12} = 80 \quad ft$$

$BentTribArea := \frac{Span}{L} = 0.333$ Percent of Area Tributary to Bent

$DL_{Bent} := BentTribArea \cdot W = 569.556$ kip

$LL_{Bent} := BentTribLength \cdot LL\_foot = 51.2$ kip

$VR_{Bent} := DL_{Bent} + LL_{Bent} = 620.756$ kip

$VR_{Bent2} := VR_{Bent}$ $VR_{Bent3} := VR_{Bent}$

Steps for Seismic Design

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

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

Article 3.4: Determine Deisgn Response Spectrum

Article 3.5: Determine SDC

Guide Figure 1.3-2: Seismic Design Procedure Flowchart for SDC B

Displacement Demand Analysis (Fig 1.3-2):

Article 4.1: Seismic Design Proportioning

Article 4.2: Determine Analysis Procedure

Article 4.3.1: Determine Horizontal Ground Motion Effects Along Both Axis

Article 4.3.2/4.3.3: Damping and Short Period Considerations

Article 5.4/5.5: Select Analytical Procedure

Article 5.6: Effective Section Properties

Article 5.2: Abutment Modeling

Article 5.3: Foundation Modeling and Liquefaction (if present)

Article 5.1.2/4.4: Conduct Demand Analysis

Article 4.8: Determine Displacement Demands Along Member Local Axes

Displacement Capacity Check ($\Delta_C > \Delta_D$):

Article 4.12: Determine Minimum Support Length

Article 4.14: Shear Key

Guide Figure 1.3-5: Foundation and Detailing Flowcharts

Foundation Design (Fig 1.3-5):

Article 6.8: Liquefaction Consideration

Article 6.3: Spread Footing Design

Article 6.4: Pile Cap Foundation Design

Article 6.5: Drilled Shaft

Article 6.7: Abutment Design

Detailing:

Article 8.3: Determine Flexure and Shear Demands

Article 8.7: Satisfy Requirements for Ductile Member Design

Article 8.6: Shear Demand and Capacity Check for Ductile Elements

Article 8.8: Satisfy Lateral and Longitudinal Reinforcement Requirements

Articles 3.4 and 3.5 have already been determined from the "SDC Classification" sheet. Make sure the four values (A_S, S_{DS}, S_{D1}, and SDC) have been input above.

Displacement Demand Analysis (Δ_D)

***Article 4.1*: Seismic Design Proportioning**

See Guide Specification

***Article 4.2*: Determine Analysis Procedure**

This is a function of the SDC and the regularity of the bridge.
For a regular bridge in SDC B, Procedure 1 or 2 can be used.
For a non-regular bridge in SDC B, Procedure 2 must be used.

Guide Table 4.2-1

A regular bridge is defined as a bridge having fewer than 7 spans, no abrupt or unusual change in geometry and that saisfy the requirements below (***Guide Table 4.2-3***)

Table 4.2-3: Regular Bridge Requirements

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle (curved bridge)	30°	30°	30°	30°	30°
Maximum span length ratio from span-to-span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span-to-span (excluding abutments)	-	4	4	3	2

***Article 4.3.1*: Determine Horizontal Ground Motion Effects Along Both Axis**

Seismic displacement demands shall be determined independently in two orthogonal directions, typically the longitudinal and transverse axes of the bridge

Article 4.3.3: Displacement Magnification for Short-Period Structures

$u_d := 2$ for SDC B

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

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)**

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 Analysis 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.

$$p_o := 1.(\quad \frac{kip}{in}$$

$$\upsilon_{smaxLong} := 1.67128 \quad in$$

INPUT

$$\upsilon_{smaxTran} := 3.22844 \quad in$$

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

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

The weight of the structure has already been calculated above

Step 4: Calculate the period, T_m.

$$T_{mLong} := 2\pi \cdot \sqrt{\frac{W}{K_{Long} \cdot g}} = 0.318 \qquad s$$

Guide Eq. C5.4.2-3

Step 5: Calculate equivalent static earthquake loading p_e.

$$\begin{aligned} acc\left(SDS, SD1, T_{mLong}, A_s\right) := \; & T_s \leftarrow \frac{SD1}{SDS} \\ & T_o \leftarrow 0.2 T_s \\ & \text{for } a \in T_{mLong} \\ & \quad a \leftarrow \left(SDS - A_s\right) \cdot \frac{T_{mLong}}{T_o} + A_s \ \text{ if } T_{mLong} < T_o \\ & \quad a \leftarrow SDS \ \text{ if } T_{mLong} \ge T_o \wedge T_{mLong} \le T_s \\ & \quad a \leftarrow \frac{SD1}{T_{mLong}} \ \text{ if } T_{mLong} > T_s \\ & Ra \leftarrow a \\ & a \end{aligned}$$

$$Sa_{Long} := acc\left(S_{DS}, S_{D1}, T_{mLong}, A_s\right) = 0.27$$

$$p_{eLong} := \frac{Sa_{Long} \cdot W}{L} = 0.16 \qquad \frac{kip}{in}$$

Guide Eq. C5.4.2-4

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.

$$Rd_{Long} := Rdprogram\left(T_{mLong}, S_{DS}, S_{D1}, u_d\right) = 1.664$$

$$v_{smaxLong} := Rd_{Long} \cdot \frac{p_{eLong}}{p_o} \cdot \upsilon_{smaxLong} = 0.445 \qquad in$$

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

Step 4: Calculate the period, T_m.

$$T_{mTran} := 2\pi \cdot \sqrt{\frac{W}{K_{Tran} \cdot g}} = 0.442 \qquad s$$

Guide Eq. C5.4.2-3

Step 5: Calculate equivalent static earthquake loading p_e.

$$Sa_{Tran} := acc\left(S_{DS}, S_{D1}, T_{mTran}, A_s\right) = 0.27$$

$$p_{eTran} := \frac{Sa_{Tran} \cdot W}{L} = 0.16 \qquad \frac{kip}{in}$$

Guide Eq. C5.4.2-4

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.

$$Rd_{Tran} := Rdprogram\left(T_{mTran}, S_{DS}, S_{D1}, u_d\right) = 1.337$$

$$v_{smaxTran} := Rd_{Tran} \cdot \frac{p_{eTran}}{p_o} \cdot \upsilon_{smaxTran} = 0.692 \qquad in$$

LRFD Article 4.7.4.3.2: Single-Mode Spectral Method

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.

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.

$$v_{stran}(x) := -1 \cdot 10^{-6} \cdot x^2 + 0.0034x - 0.294$$

INPUT

$$v_{slong}(x) := -2 \cdot 10^{-8} \cdot x^2 + 6 \cdot 10^{-5} \cdot x + 1.585$$

$$\alpha_{Tran} := \int_0^L v_{stran}(x)\,dx \qquad \alpha_{Long} := \int_0^L v_{slong}(x)\,dx$$

LRFD C4.7.4.3.2b-1

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

$$\gamma_{Tran} := \int_0^L \frac{W}{L} \cdot v_{stran}(x)^2\,dx = 6.737\times 10^3 \qquad \gamma_{Long} := \int_0^L \frac{W}{L} \cdot v_{slong}(x)^2\,dx$$ ***LRFD C4.7.4.3.2b-3***

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

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

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

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smLong} := acc\left(S_{DS}, S_{D1}, T_{mLong1}, A_s\right) = 0.27$$

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.

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

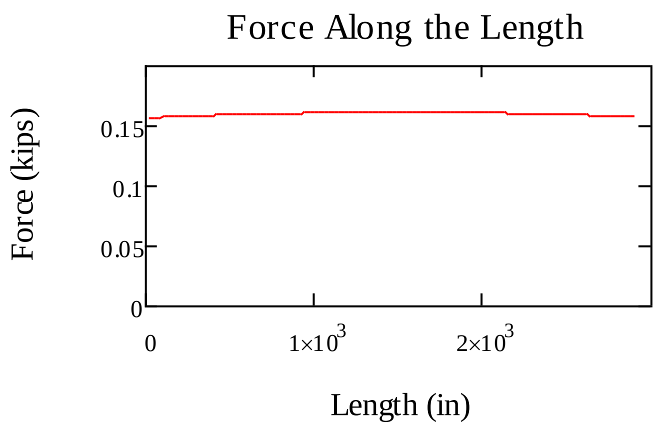
$$PeLong(x) \rightarrow 0.000005944609266572630967x^3 - 1.981536422190876989 3e{-9}x^2 + 0.1570962075512927\ldots$$

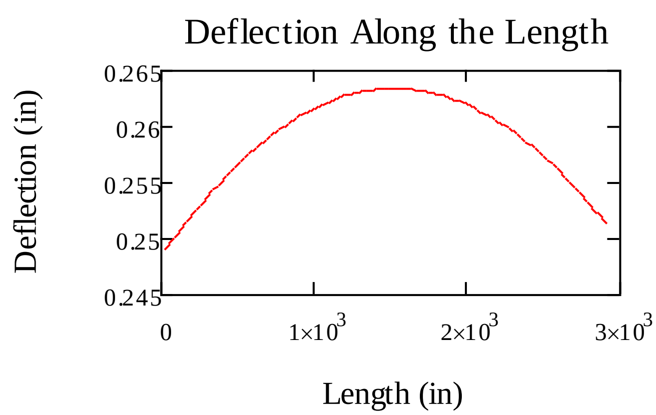
$$dW := \frac{L}{100}$$

$$i := 1..101$$

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

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





Maximum Deflection:

$\max(\Delta long) = 0.263$ in

NOTE: Repeat Steps 5 and 6 for Transverse Direction.

Step 5: Calculate the equivalent Static Earthquake Loading

$C_{smTran} := acc\left(S_{DS}, S_{D1}, T_{mTran1}, A_s\right) = 0.27$

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.

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

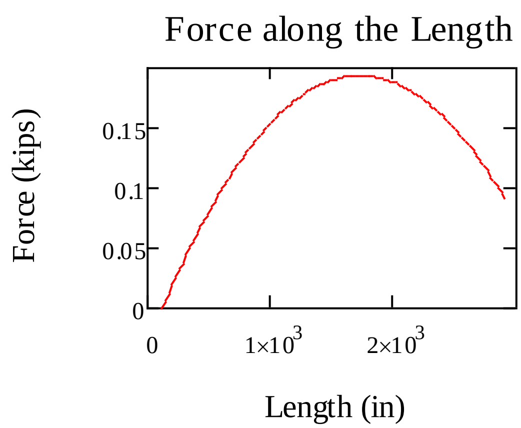
$PeTran(x) \rightarrow 0.00025372660257277175421 x + -7.462547134493286886e{-8} x^2 - 0.0219772013110827298$

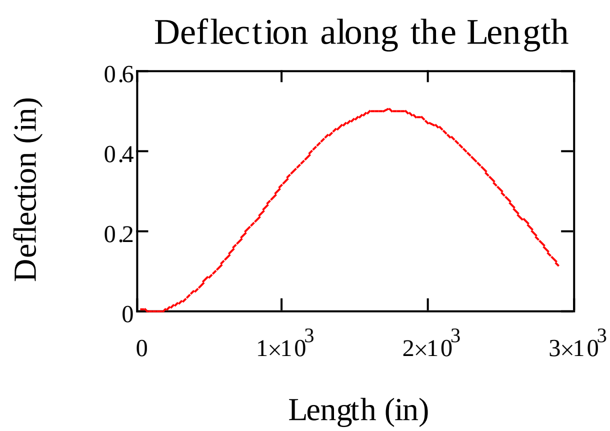
$$dL := \frac{L}{100}$$

$i := 1..101$

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

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





Maximum Deflection:

$\max(\Delta tran) = 0.503$ in

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

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**

This is taken care of in the SAP model.

***Article 5.3*: Foundations Modeling**

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.

Special provisions need to be considered if Liquefaction is present. ***Guide Article 6.8***

Article 4.4: **Combination of Orthogonal Seismic Displacement Demands**

$$LoadCase1 := \sqrt{\left(1 \cdot v_{smaxLong}\right)^2 + \left(0.3 v_{smaxTran}\right)^2} = 0.491 \quad in$$

$$LoadCase2 := \sqrt{\left(1 \cdot v_{smaxTran}\right)^2 + \left(0.3 v_{smaxLong}\right)^2} = 0.704 \quad in$$

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

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

$\Delta_{Dlong} := 1.346$ in

INPUT

$\Delta_{Dtran} := 2.080$ in

$$\Delta_{DLong} := Rd_{Long} \cdot \Delta_{Dlong} \cdot p_{eLong} = 0.359 \quad in$$

$$\Delta_{DTran} := Rd_{Tran} \cdot \Delta_{Dtran} \cdot p_{eTran} = 0.446 \quad in$$

$$LoadCase1 := \sqrt{\left(1\Delta_{DLong}\right)^2 + \left(0.3\Delta_{DTran}\right)^2} = 0.383 \quad in$$

$$LoadCase2 := \sqrt{\left(1\Delta_{DTran}\right)^2 + \left(0.3\Delta_{DLong}\right)^2} = 0.458 \quad in$$

$$\Delta_D := \max(LoadCase1, LoadCase2) = 0.458 \quad in$$

$$H_o := \frac{ColumnHeight_{Bent2}}{12} = 17.934 \quad ft \qquad B_o := \frac{Columndia}{12} = 3.5 \quad ft$$

Transverse Direction

$\Lambda := 2$ Fixed and top and bottom ***Guide Article 4.8.1***

$$x := \frac{\Lambda \cdot B_o}{H_o} = 0.39$$ ***Guide Eq. 4.8.1-3***

$$\Delta_{CT} := 0.12 \cdot H_o \cdot (-1.27 \cdot \ln(x) - 0.32) = 1.883 \quad in$$ ***Guide Eq. 4.8.1-1***

Longitudinal Direction

Guide Article 4.8.1

$\Lambda := 1$ Fixed-Free

Guide Eq. 4.8.1-3

$$x := \frac{\Lambda \cdot B_o}{H_o} = 0.195$$

$$\Delta_{CL} := 0.12 \cdot H_o \cdot (-1.27 \cdot \ln(x) - 0.32) = 3.777 \quad in$$ ***Guide Eq. 4.8.1-1***

$$\Delta_C := \min(\Delta_{CT}, \Delta_{CL}) = 1.883$$

$$0.12 \cdot H_o = 2.152 \quad in$$

$$CheckLimit(\Delta_C, H_o) := \left| \begin{array}{l} a \leftarrow 0.12 \cdot H_o \;\; \text{if } \Delta_C < 0.12 \cdot H_o \\ a \leftarrow \Delta_C \;\; \text{otherwise} \end{array} \right.$$

$$\Delta_C := CheckLimit(\Delta_C, H_o)$$

$$\Delta_C = 2.152$$

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

$$CheckCapacity(\Delta_C, \Delta_D) = \text{"OK"}$$

If the simplified equations do not work ("FAILURE") for any of the bents, 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.

BENT 3

$\Delta_{Dlong} := 1.437$ in

$\Delta_{Dtran} := 2.900$ in

INPUT

$\Delta_{DLong} := Rd_{Long} \cdot \Delta_{Dlong} \cdot p_{eLong} = 0.383$ in

$\Delta_{DTran} := Rd_{Tran} \cdot \Delta_{Dtran} \cdot p_{eTran} = 0.621$ in

$LoadCase1 := \sqrt{(1\Delta_{DLong})^2 + (0.3\Delta_{DTran})^2} = 0.426$ in

$LoadCase2 := \sqrt{(1\Delta_{DTran})^2 + (0.3\Delta_{DLong})^2} = 0.632$ in

$\Delta_D := \max(LoadCase1, LoadCase2) = 0.632$ in

$H_o := \frac{ColumnHeight_{Bent3}}{12} = 25.834$ ft $B_o := \frac{Columndia}{12} = 3.5$ ft

Transverse Direction

$\Lambda := 2$ Fixed and top and bottom ***Guide Article 4.8.1***

$x := \frac{\Lambda \cdot B_o}{H_o} = 0.271$ ***Guide Eq. 4.8.1-3***

$\Delta_{CT} := 0.12 H_o \cdot (-1.27 \cdot \ln(x) - 0.32) = 4.149$ in ***Guide Eq. 4.8.1-1***

Longitudinal Direction

Guide Article 4.8.1

$\Lambda := 1$ Fixed-Free

Guide Eq. 4.8.1-3

$x := \frac{\Lambda \cdot B_o}{H_o} = 0.135$

$\Delta_{CL} := 0.12 H_o \cdot (-1.27 \cdot \ln(x) - 0.32) = 6.878$ in ***Guide Eq. 4.8.1-1***

$\Delta_C := \min(\Delta_{CT}, \Delta_{CL}) = 4.149$

$0.12 H_o = 3.1$ in

$$CheckLimit(\Delta_C, H_o) := \left| \begin{array}{l} a \leftarrow 0.12 H_o \text{ if } \Delta_C < 0.12 H_o \\ a \leftarrow \Delta_C \text{ otherwise} \end{array} \right.$$

$CheckLimit(\Delta_C, H_o) = 4.149$

$$\text{CheckCapacity}(\Delta_C, \Delta_D) := \begin{cases} c \leftarrow \text{"OK"} & \text{if } \Delta_C \geq \Delta_D \\ c \leftarrow \text{"FAILURE"} & \text{if } \Delta_C < \Delta_D \end{cases}$$

$$\text{CheckCapacity}(\Delta_C, \Delta_D) = \text{"OK"}$$

Pushover Analysis Results (if necessary):

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

INPUT
(from SAP2000)

Article 4.12: Minimum Support Length Requirements

Abutment Support Length Requirement *Guide Eq. 4.12.2-1*

$$\text{Nabutment1} := 1.5\left(8 + 0.02\,\text{Span} + 0.08\,H_{abutment}\right)\cdot\left(1 + 0.000125\,\text{Skew}^2\right) = \blacksquare \qquad \text{in}$$

Bent Support Length Requirement *Guide Eq. 4.12.2-1*

BENT 2

$L := \text{BentTribLength} = 80 \qquad S_{D1} := 0.3($

$$H := \frac{\text{ColumnHeight}_{Bent2}}{12} = 17.934$$

Standard Specifications

$$N_{2Stan} := (8 + 0.02\,L + 0.08\,H)\cdot\left(1 + 0.000125\,\text{Skew}^2\right) = 11.035 \qquad \text{in}$$

ATC-49 Equation (New Design)

$$N_2 := \left[4 + .02L + .08H + 1.09\sqrt{H}\cdot\sqrt{1 + \left(2\cdot\frac{3}{8}\right)^2}\right]\cdot\left(\frac{1 + 1.25\,S_{D1}}{\cos\left(\frac{\text{Skew}\cdot\pi}{180}\right)}\right) = 17.606 \qquad \text{in}$$

BENT 3

$L := \text{BentTribLength} = 80 \qquad S_{D1} := 0.3($

$$H := \frac{ColumnHeight_{Bent3}}{12} = 25.834$$

Standard Specifications

$$N_{3Stan} := (8 + 0.02 L + 0.08 H)\cdot\left(1 + 0.000125 Skew^2\right) = 11.667 \quad in$$

ATC-49 Equation (New Design)

$$N_3 := \left[4 + .02L + .08H + 1.09\sqrt{H}\cdot\sqrt{1 + \left(2\cdot\frac{3}{8}\right)^2}\right]\cdot\left(\frac{1 + 1.25 S_{D1}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 20.064 \quad in$$

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

$V_{ok} := 1.5 V_n$ *This does not apply to this bridge*

BENT 2 DESIGN

Guide Article 4.11.2: For SDC B, it is acceptable to use the moment capacity based on expected material strengths when the concrete reaches an extreme compressive fiber strain of 0.003.

Force Inputs

$M_{neBent2} := 1538$	kip – ft	Nominal moment from PCA Column	**INPUT**
$V_{pelastic} := 515$	kip	Elastic shear from SAP2000 model	
$P_u := 128400$	lb	Axial load from earthquake and dead load combination	

Reinforcement Details

$Ag := Acolumn$

$Ae := 0.8 Ag = 1108$ in^2 ***Guide Eq. 8.6.2-2***

$\mu_D := 2$ ***Guide Article 8.6.2***

$n := 2$		n: Number of individual interlocking spiral or hoop core sections	
StirrupSize := "#4"		StirrupSize: Bar size used for stirrups	
$s := 6$	in	s: Spacing of hoops or pitch of spiral (in)	
$sNOhinge := 9$	in	sNOhinge: Spacing of hoops or pitch outside PHL	
$Asp := 0.2$	in^2	Asp: Area of hoop reinforcement in direction of loading (in^2)	
$Dsp := 0.625$	in	Dsp: Diameter of spiral or hoop reinforcing (in)	**INPUT**

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

$b := Columndia$ in b: Diameter of column (in)

$d := b - Cover = 36$ in d: Effective depth of section in direction of loading (in)

$Dprime := b - 2 \cdot Cover$ in Dprime: Diameter (in column) of hoop reinforcing (in)

$NumberBars := 12$ Total number of longitudinal bars in column cross-section

$A_{bl} := 1.56$ in^2 Abl: Area of longitudinal bar

$d_{bl} := 1.41$ in dbl: Diameter of longitudinal bar

$bv := Columndia$ bv: Diameter of column

Article 8.3: Determine Flexure and Shear Demands

The design forces shall be the lesser of the forces resulting from the overstrength plastic hinging moment capacity or unreduced elastic seismic forces in columns or pier walls.

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

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.

$\lambda_{mo} := 1.4$ for ASTM A 615 Grade 60 reinforcement ***Guide Article 8.5***

$M_{pBent2} := \lambda_{mo} \cdot M_{neBent2} \cdot 1000 \cdot 12 = 2.584 \times 10^{7}$ lb – in

$Fixity := ColumnHeight_{Bent2} = 215.208$ in

$$V_p := \frac{2 \cdot M_{pBent2}}{Fixity \cdot 1000} = 240.125 \quad kips$$

$V_{pelastic} := V_{pelastic} \cdot max(p_{eTran}, p_{eLong}) = 82.497$ kips

Article 8.7: Requirements for Ductile Member Design

Each column must satisfy the minimum lateral flexural capacity

$\Lambda := 2$ Fixed and top and bottom ***Guide Article 4.8.1***

$$M_{neminBent} := 0.1 \cdot DL_{Bent} \cdot \left(\frac{\frac{Fixity}{12} + 0.5 D_s}{\Lambda} \right) = 570.346 \quad kip\ ft$$ ***Guide Eq 8.7.1-1***

$$\text{CheckMoment}(M_{ne}, M_e) := \begin{array}{|l} a \leftarrow \text{"OK"} \quad \text{if } M_{ne} \le M_e \\ a \leftarrow \text{"FAILURE"} \quad \text{otherwise} \end{array}$$

$$\text{CheckMoment}(M_{neminBent}, M_{neBent2}) = \text{"OK"}$$

If the moment check comes back "FAILURE" then the column size can be increased or the reinforcement can be increased.

Article 8.6: Shear Demand and Capacity for Ductile Concrete Members

It is recommended to use the plastic hinging forces whenever practical, but in this case the elastic forces will be used.

$$V_u := \min(V_p, V_{pelastic}) = 82.497 \quad \text{kips} \qquad \phi_s := 0.9$$

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

$$\rho_s := \frac{4 \cdot Asp}{s \cdot Dprime} = 0.0044$$ ***Guide Eq. 8.6.2-7***

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

$$\text{StressCheck}(\rho_s, fyh) := \begin{array}{|l} fs \leftarrow \rho_s \cdot fyh \\ a \leftarrow fs \quad \text{if } fs \le 0.35 \end{array}$$

$$fs := \text{StressCheck}(\rho_s, fyh) = 0.267$$ ***Guide Eq. 8.6.2-6***

$$\alpha\text{program}(fs, \mu_D) := \begin{array}{|l} \alpha\text{prime} \leftarrow \frac{fs}{0.15} + 3.67 - \mu_D \\ a \leftarrow 0.3 \ \text{ if } \alpha\text{prime} \le 0.3 \\ a \leftarrow \alpha\text{prime} \ \text{ if } \alpha\text{prime} > 0.3 \wedge \alpha\text{prime} < 3 \\ a \leftarrow 3 \ \text{ if } \alpha\text{prime} \ge 3 \\ a \end{array}$$ ***Guide Eq. 8.6.2-5***

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

If Pu is Compressive:

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

Guide Eq. 8.6.2-3

If Pu is NOT Compressive:

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

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

Guide Eq. 8.6.2-4

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

Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity** ***Guide Eq 8.6.3-2 and 8.6.4-1

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

$Vs := \mathrm{vsprogram}(n, Asp, fyh, Dprime, s, fc, Ae) = 188.496$ kips

$\phi Vn := \phi_s\cdot(Vs + Vc) = 389.1$ kips

Guide Eq. 8.6.1-2

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

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

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:

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

Guide Eq. 8.6.5-1

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

If the minimum shear reinforcement program responses "Increase Shear Reinforcing Ratio", it is recommended to <u>decrease the spacing (s)</u> or i<u>ncrease 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**

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

$$\rho\text{program}(A_{long}, Ag) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } A_{long} \leq 0.04Ag \\ a \leftarrow \text{"Section Over Reinforced"} \quad \text{if } A_{long} > 0.04Ag \\ a \end{array} \right.$$

Guide Eq. 8.8.1-1

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

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**

$$\text{minAlprogram}(A_l, Ag) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } A_{long} \geq 0.007Ag \\ a \leftarrow \text{"Increase Longitudinal Reinforcing"} \quad \text{if } A_{long} < 0.007Ag \\ a \end{array} \right.$$

Guide Eq. 8.8.2-1

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

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

***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.

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

Guide Eq. 4.11.6-1

$L_p := \text{PlasticHinge}(\text{Fixity}, \text{fye}, d_{bl}) = 29.907$ in

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

'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 ρ_{eTran} to take into account the model loads have not been multiplied by ρ_{eTran}. The location will also need to be **INPUT** into the PlasticHingeRegion program in inches.

$Mp75 := 0.75 M_{pBent2} = 1.938\times 10^{7}$ lb·in

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

$$L_{p1} := \text{PlasticHingeRegion}\left(L_p, \text{Columndia}\right) = 63 \qquad \text{in}$$

The Guide Specifications allows for the use of the plastic hinge length from the LRFD Specification in SDC A and B (***Guide Article C8.8.9***).

$$\text{LRFDPlasticHingeLength}(\text{ColumnDia}, \text{ColumnHeight}) := \left| \begin{array}{l} a \leftarrow \text{ColumnDia} \\ b \leftarrow \frac{1}{6}\cdot\text{ColumnHeight} \\ c \leftarrow 18 \\ \text{PHL} \leftarrow \max(a, b, c) \\ \text{PHL} \end{array} \right.$$

$$L_{p2} := \text{LRFDPlasticHingeLength}(\text{Columndia}, \text{Fixity}) = 42 \qquad \text{in}$$

Guide Article C8.8.9

The plastic hinge length will be the smaller of the two values, as the Guide Specification allows:

$$\text{PHL} := \min\left(L_{p1}, L_{p2}\right) = 42 \qquad \text{in}$$

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region: ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

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

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

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

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

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

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

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.

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

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

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

LRFD 5.8.3.3 **Nominal Shear Resistance**

$V_u = 82.497$ kips

$\phi_s = 0.9$

$\beta := 2.0$ ***LRFD Article 5.8.3.4.1***

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

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

$de := d = 36$ in ***LRFD Eq. 5.8.2.9-1***

$dv := 0.9de = 32.4$ in

$V_c := 0.0316\beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 172.005$ kips ***LRFD Eq. 5.8.3.3-3***

$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{sNOhinge} = 86.4$ kips ***LRFD Eq. 5.8.3.3-4***

$V_n := \phi_s \cdot .25 fc \cdot bv \cdot dv = 1.225 \times 10^6$ kips

$\phi Vn := \min\left[V_n, \left(V_c + V_s\right) \cdot \phi_s\right] = 232.565$ kips

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

$$\text{Shearcheck2} := \text{ShearCheck}(\phi Vn, V_u) = \text{"OK"}$$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

LRFD Eq. 5.8.2.5-1

$$\text{Avmin} := 0.0316\sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot sNOhinge}{\frac{fye}{1000}} = 0.398 \qquad in^2$$

$$Av := 2Asp = 0.4 \qquad in^2$$

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

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

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.

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

$$vu := \frac{V_u}{\phi_s \cdot bv \cdot dv} = 0.067 \qquad ksi$$

LRFD Eq. 5.8.2.9-1

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

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

$MaxSpacing := floor(spacingProgram(vu, dv, fc)) = 24$ in

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$Spacecheck(MaxSpacing, s, V_u, V_c) := \left| \begin{array}{l} a \leftarrow min(MaxSpacing, 12) \ \text{ if } \ V_u \le 0.5 0.9 V_c \\ a \leftarrow min(s, MaxSpacing, 12) \ \text{ otherwise} \\ a \end{array} \right.$$

$sNOhinge := Spacecheck(MaxSpacing, sNOhinge, V_u, V_c) = 9$ in

Design Summary - Bent 2

$StirrupSize = \text{"\#4"}$

$s = 6$ in

$sNOhinge = 9$ in

$PHL = 42$ in

$Extension = 21$ in

$N_2 = 17.606$ in

Design Check Summary - Bent 2

$Shearcheck = \text{"OK"}$	Shear capacity > Vn
$Transversecheck = \text{"OK"}$	Minimum shear reinforcement ratio
$ReinforcementRatioCheck = \text{"OK"}$	Maximum longitudinal reinforcement ratio
$MinimumA_l = \text{"OK"}$	Minimum longitudinal reinforcement ratio
$scheck = \text{"OK"}$	Max spacing of transverse reinforcement
$Shearcheck2 = \text{"OK"}$	Shear capacity outside hinge zone > Vn
$MinimumTran = \text{"OK"}$	Minimum shear reinforcement outside hinge zone

BENT 3 DESIGN

Guide Article 4.11.2: For SDC B, it is acceptable to use the moment capacity based on expected material strengths when the concrete reaches an extreme compressive fiber strain of 0.003.

Force Inputs

$M_{neBent3} := 1538$	kip – ft	Nominal moment from PCA Column	**INPUT**
$V_{pelastic} := 515$	kip	Elastic shear from SAP2000 model	
$P_u := 128400$	lb	Axial load from earthquake and dead load combination	

Reinforcement Details

$Ag := Acolumn$

$Ae := 0.8 Ag = 1108$ in^2 ***Guide Eq. 8.6.2-2***

$\mu_D := 2$ ***Guide Article 8.6.2***

$n := 2$		n: Number of individual interlocking spiral or hoop core sections	
StirrupSize := "#4"		StirrupSize: Bar size used for stirrups	
$s := 6$	in	s: Spacing of hoops or pitch of spiral (in)	
$sNOhinge := 9$	in	sNOhinge: Spacing of hoops or pitch outside PHL	
$Asp := 0.20$	in^2	Asp: Area of hoop reinforcement in direction of loading (in^2)	
$Dsp := 0.625$	in	Dsp: Diameter of spiral or hoop reinforcing (in)	**INPUT**
$Cover := 3$	in	Cover: Concrete cover for the Column (in)	
$b := Columndia$	in	b: Diameter of column (in)	
$d := b - Cover = 39$	in	d: Effective depth of section in direction of loading (in)	
$Dprime := b - 2 \cdot Cover$	in	Dprime: Diameter (in column) of hoop reinforcing (in)	
$NumberBars := 12$		Total number of longitudinal bars in column cross-section	
$A_{bl} := 1.56$	in^2	Abl: Area of longitudinal bar	
$d_{bl} := 1.41$	in	dbl: Diameter of longitudinal bar	
$bv := Columndia$		bv: Diameter of column	

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

The design forces shall be the lesser of the forces resulting from the overstrength plastic hinging moment capacity or unreduced elastic seismic forces in columns or pier walls.

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

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.

$\lambda_{mo} := 1.4$ for ASTM A 615 Grade 60 reinforcement ***Guide Article 8.5***

$M_{pBent3} := \lambda_{mo} \cdot M_{neBent3} \cdot 1000 \cdot 12 = 2.584 \times 10^7$ lb – in

$Fixity := ColumnHeight_{Bent3} = 310.008$ in

$V_p := \frac{2 \cdot M_{pBent3}}{Fixity \cdot 1000} = 166.695$ kips

$V_{pelastic} := V_{pelastic} \cdot \max(p_{eTran}, p_{eLong}) = 82.497$ kips

***Article 8.7*: Requirements for Ductile Member Design**

Each column must satisfy the minimum lateral flexural capacity

$\Lambda := 2$ Fixed and top and bottom ***Guide Article 4.8.1***

$$M_{neminBent} := 0.1 \cdot DL_{Bent} \cdot \left(\frac{\frac{Fixity}{12} + 0.5 D_s}{\Lambda} \right) = 795.32 \text{ kip ft}$$ ***Guide Eq 8.7.1-1***

$$CheckMoment(M_{ne}, M_e) := \begin{vmatrix} a \leftarrow \text{"OK"} \quad \text{if } M_{ne} \le M_e \\ a \leftarrow \text{"FAILURE"} \quad \text{otherwise} \end{vmatrix}$$

$CheckMoment(M_{neminBent}, M_{neBent2}) = \text{"OK"}$

If the moment check comes back "FAILURE" then the column size can be increased or the reinforcement can be increased.

***Article 8.6*: Shear Demand and Capacity for Ductile Concrete Members**

It is recommended to use the plastic hinging forces whenever practical, but in this case the elastic forces will be used.

$V_u := \min(V_p, V_{pelastic}) = 82.497$ kips $\phi_s := 0.9$

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

$\rho_s := \frac{4 \cdot Asp}{s \cdot Dprime} = 0.0037$ ***Guide Eq. 8.6.2-7***

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

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

$$fs := StressCheck(\rho_s, fyh) = 0.222$$ ***Guide Eq. 8.6.2-6***

$$\alpha program(fs, \mu_D) := \left| \begin{array}{l} \alpha prime \leftarrow \frac{fs}{0.15} + 3.67 - \mu_D \\ a \leftarrow 0.3 \ \text{ if } \ \alpha prime \le 0.3 \\ a \leftarrow \alpha prime \ \text{ if } \ \alpha prime > 0.3 \wedge \alpha prime < 3 \\ a \leftarrow 3 \ \text{ if } \ \alpha prime \ge 3 \\ a \end{array} \right.$$ ***Guide Eq. 8.6.2-5***

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

If Pu is Compressive:

$$vcprogram(\alpha Prime, fc, P_u, Ag) := \left| \begin{array}{l} vc \leftarrow 0.032\alpha Prime\left(1 + \frac{P_u}{2Ag \cdot 1000}\right) \cdot \sqrt{\frac{fc}{1000}} \\ min1 \leftarrow 0.11\sqrt{\frac{fc}{1000}} \\ min2 \leftarrow 0.047\alpha Prime\sqrt{\frac{fc}{1000}} \\ minimum \leftarrow min(min1, min2) \\ a \leftarrow vc \ \text{ if } \ vc \le minimum \\ a \leftarrow minimum \ \text{if } \ vc > minimum \\ a \end{array} \right.$$ ***Guide Eq. 8.6.2-3***

If Pu is NOT Compressive:

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

Guide Eq. 8.6.2-4

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

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

Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity** ***Guide Eq 8.6.3-2 and 8.6.4-1

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

$Vs := \text{vsprogram}(n, Asp, fyh, Dprime, s, fc, Ae) = 226.195$ kips

$\phi Vn := \phi_s \cdot (Vs + Vc) = 423.029$ kips

Guide Eq. 8.6.1-2

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

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

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:

Guide Eq. 8.6.5-1

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

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

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**

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

Guide Eq. 8.8.1-1

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

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

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**

$$minAlprogram(A_l, Ag) := \begin{array}{|l} a \leftarrow \text{"OK"} \quad \text{if } A_{long} \geq 0.007Ag \\ a \leftarrow \text{"Increase Longitudinal Reinforcing"} \quad \text{if } A_{long} < 0.007Ag \\ a \end{array}$$

Guide Eq. 8.8.2-1

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

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

***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.

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

Guide Eq. 4.11.6-1

$L_p := PlasticHinge(Fixity, fye, d_{bl}) = 37.491$ in

Article 4.11.7: Reinforced Concrete Column Plastic Hinge Region

'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 ρ_{eTran} to take into account the model loads have not been multiplied by ρ_{eTran}. The location will also need to be **INPUT** into the PlasticHingeRegion program in inches.

$Mp75 := 0.75\,M_{pBent2} = 1.938\times 10^{7}$ lb·in

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

$L_{p1} := PlasticHingeRegion(L_p, Columndia) = 63$ in

The Guide Specifications allows for the use of the plastic hinge length from the LRFD Specification in SDC A and B (***Guide Article C8.8.9***).

$$LRFDPlasticHingeLength(ColumnDia, ColumnHeight) := \left| \begin{array}{l} a \leftarrow ColumnDia \\ b \leftarrow \frac{1}{6}\cdot ColumnHeight \\ c \leftarrow 18 \\ PHL \leftarrow max(a, b, c) \\ PHL \end{array} \right.$$

$L_{p2} := LRFDPlasticHingeLength(Columndia, Fixity) = 51.668$ in

Guide Article C8.8.9

The plastic hinge length will be the smaller of the two values, as the Guide Specification allows:

$PHL := \min(L_{p1}, L_{p2}) = 51.668$ in

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region: ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

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

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

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

$FINALSPACING := SpacingCheck(MaximumSpacing, s) = 6$ in

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

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

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.

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

$Extension := ExtensionProgram(Columndia) = 21$ in

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

LRFD 5.8.3.3 **Nominal Shear Resistance**

$V_u = 82.497$ kips

$\phi_s = 0.9$

$\beta := 2.0$ ***LRFD Article 5.8.3.4.1***

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

$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 37.67$ in

$de := d = 39$ in ***LRFD Eq. 5.8.2.9-1***

$dv := 0.9de = 35.1$ in

$V_c := 0.0316\beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 186.339$ kips ***LRFD Eq. 5.8.3.3-3***

$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{sNOhinge} = 93.6$ kips ***LRFD Eq. 5.8.3.3-4***

$V_n := \phi_s \cdot .25 fc \cdot bv \cdot dv = 1.327 \times 10^6$ kips

$\phi Vn := \min\left[V_n, \left(V_c + V_s\right) \cdot \phi_s\right] = 251.945$ kips

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

$Shearcheck2 := ShearCheck\left(\phi Vn, V_u\right) = \text{"OK"}$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

$Avmin := 0.0316\sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot sNOhinge}{\frac{fye}{1000}} = 0.398$ in^2 ***LRFD Eq. 5.8.2.5-1***

$Av := 2 \cdot Asp = 0.4$ in^2

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

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

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.

LRFD 5.8.2.7 Maximum Spacing of Transverse Reinforcement

$$vu := \frac{V_u}{\phi_s \cdot bv \cdot dv} = 0.062 \quad \text{ksi}$$

LRFD Eq. 5.8.2.9-1

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

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

$$MaxSpacing := floor(spacingProgram(vu, dv, fc)) = 24 \quad \text{in}$$

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$Spacecheck\left(MaxSpacing, s, V_u, V_c\right) := \left| \begin{array}{l} a \leftarrow \min(MaxSpacing, 12) \ \text{ if } V_u \le 0.50.9V_c \\ a \leftarrow \min(s, MaxSpacing, 12) \ \text{ otherwise} \\ a \end{array} \right.$$

$$sNOhinge := Spacecheck\left(MaxSpacing, sNOhinge, V_u, V_c\right) = 12 \quad \text{in}$$

Design Summary - Bent 3

$StirrupSize = "\#4"$

$s = 6$ in

$sNOhinge = 12$ in

$PHL = 51.668$ in

$Extension = 21$ in

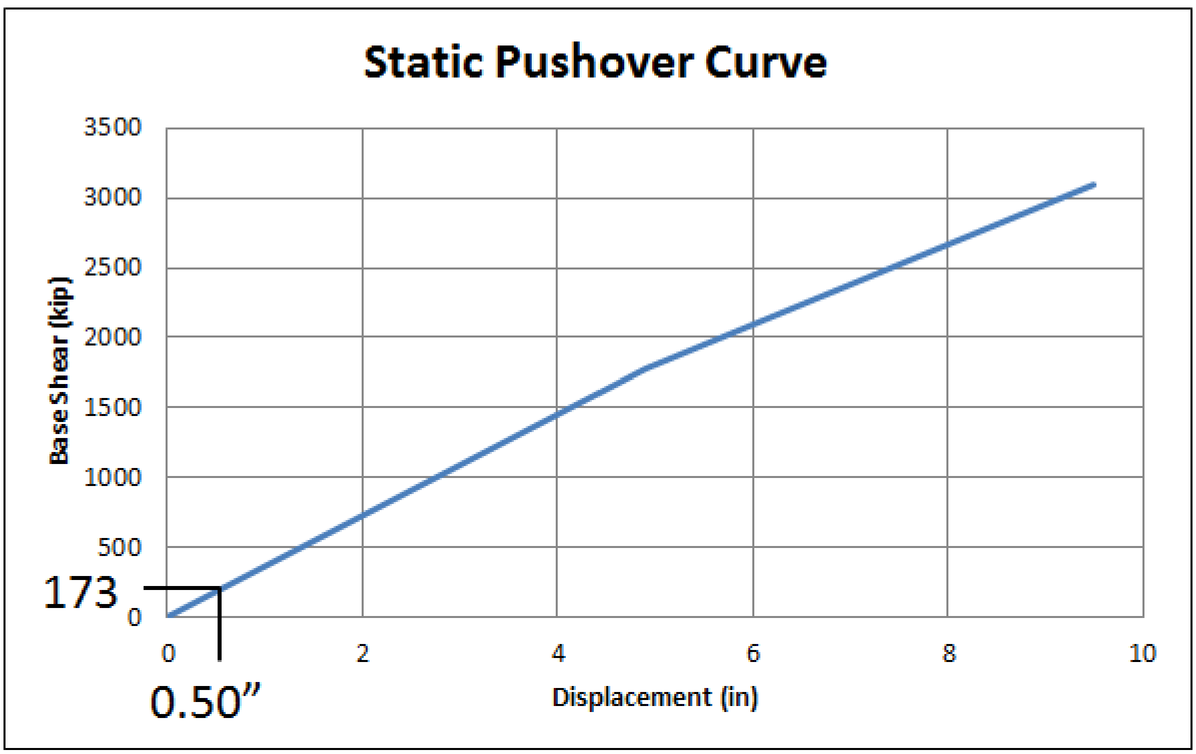
$N_3 = 20.064$ in

Design Check Summary - Bent 3

Shearcheck = "OK"	Shear capacity > Vn
Transversecheck = "OK"	Minimum shear reinforcement ratio
ReinforcementRatioCheck = "OK"	Maximum longitudinal reinforcement ratio
$MinimumA_l$ = "OK"	Minimum longitudinal reinforcement ratio
scheck = "OK"	Max spacing of transverse reinforcement
Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran= "OK"	Minimum shear reinforcement outside hinge zone

Transverse Connection Design

Pushover Analysis Results



ALDOT Current Connection Steel Angle Design Check

$Vcolbent := \frac{173}{N} = 43.25$ kips

LRFD 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

Bolt Properties

$Fub := 58$	ksi	Strength of Anchor Bolt (It is assumed that ASTM A307 Grade C bolt is used)
$Dia_b := 1.75$	in	Diameter of Anchor Bolt **INPUT**
$Ns := 1$		Number of Shear Planes per Bolt

Angle Properties

$Fy := 36$	ksi	Yield Stress of the Angle
$Fu := 58$	ksi	Ultimate Stress of the Angle
$t := 1.00$	in	Thickness of Angle
$h := 6$	in	Height of the Angle
$w := 6$	in	Width of the Angle
$l := 12$	in	Length of the Angle
$k := 1.5$	in	Height of the Bevel **INPUT**
$distanchorhole := 4$	in	Distance from the vertical leg to the center of the hole. This is the location of the holes.
$diahole := Dia_b + \frac{1}{8} = 1.875$	in	Diameter of bolt hole
$BLSHlength := 6$	in	Block Shear Length

$BLSHwidth := 2$ in — Block Shear Width

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

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

$b := 3.5$ in — Distance from center of bolt to toe of fillet of connected part

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

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

Clip Angle Check:

AISC J4: Block Shear

$Agv := t \cdot BLSHlength = 6$ in^2

$Anv := t \cdot (BLSHlength - 0.5 diahole) = 5.063$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 diahole) = 1.063$ in^2

AISC Eq. J4-5

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

$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 191.225$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 152.98$ kips

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

AISC D2: Tension Member

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

$Ant := t \cdot [w - (1 \cdot diahole)] = 4.125$ in^2

$Ae := Ant \cdot Ut = 2.475$ in^2 ***AISC Eq. D3-1***

$\phi tPn := \phi_t \cdot Fub \cdot Ae = 114.84$ kips **AISC Eq. D2-2**

$TensionCheck_{AISC} := ShearCheck(\phi tPn, Vcolbent) = \text{"OK"}$

AISC G: Shear Check

$Cv := 1.0$

$Aw := t \cdot w = 6$ in^2

$\phi sangleVn := \phi_{sangle} \cdot 0.6 Fy \cdot Aw \cdot Cv = 129.6$ kips **AISC Eq. G2-1**

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

Anchor Bolt Check:

LRFD Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 2.405 \quad in^2$$

$\phi sRn := \phi_s \cdot 0.48 A_b \cdot Fub \cdot Ns = 50.222$ kips **LRFD Eq. 6.13.2.12-1**

$Shear_{Anchorbolts} := ShearCheck(\phi sRn, Vcolbent) = \text{"OK"}$

LRFD Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

$\phi bbRn := 2.4 Dia_b \cdot t \cdot Fub = 243.6$ kips **LRFD Eq. 6.13.2.9-1**

For Slotted Holes

$\phi bbRns := Lc \cdot t \cdot Fub = 116$ kips **LRFD Eq. 6.13.2.9-4**

$Bearing_{Boltstandard} := ShearCheck(\phi bbRn, Vcolbent) = \text{"OK"}$

$Bearing_{Boltslotted} := ShearCheck(\phi bbRns, Vcolbent) = \text{"OK"}$

LRFD Article 6.13.2.10: Tensile Resistance

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.

$$Tu := \frac{Vcolbent \cdot 1}{distanchorhole} = 10.813 \quad kips$$

LRFD Eq. 6.13.2.10.2-1

$$\phi tTn := \phi_t \cdot 0.76 A_b \cdot Fub = 84.82 \quad kips$$

TensionCheck := ShearCheck(ϕtTn , Tu) = "OK"

Article 6.13.2.11: Combined Tension and Shear

Pu := Vcolbent

LRFD Eq. 6.13.2.11-1
LRFD Eq. 6.13.2.11-2

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

$$Tn_{combined} := CombinedProgram\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 53.894 \quad kips$$

$$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 43.115 \quad kips$$

CombinedCheck := ShearCheck(ϕtTn combined , Vcolbent) = "FAILURE"

Summary

$Dia_b = 1.75$ in

$Shear_{Anchorbolts}$ = "OK"

$Bearing_{Boltstandard}$ = "OK"

$Bearing_{Boltslotted}$ = "OK"

TensionCheck = "OK"

CombinedCheck = "FAILURE"

$TensionCheck_{AISC}$ = "OK"

BlockShearCheck = "OK"

ShearAngleCheck = "OK"

Appendix M: Oseligee Creek Bridge Moment-Interaction Diagrams

Bents 2 and 3

```
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=========================================================================================================
                                      spColumn v4.81 (TM)
                Computer program for the Strength Design of Reinforced Concrete Sections
                           Copyright © 1988-2012, STRUCTUREPOINT, LLC.
                                      All rights reserved
=========================================================================================================
```
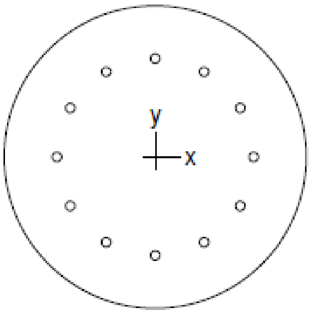

```
General Information:
====================
  File Name: untitled.col
  Project:
  Column:                                    Engineer:
  Code:     ACI 318-11                       Units: English

  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
  rx =  10.5 in                              ry =  10.5 in
  Xo =  0 in                                 Yo =  0 in

Reinforcement:
==============
  Bar Set: 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: Spiral; #3 ties with #10 bars,  #5 with larger bars.
  phi(a) = 0.85,  phi(b) = 0.9,  phi(c) = 0.75

  Layout: Circular
  Pattern: All Sides Equal  (Cover to transverse reinforcement)
  Total steel area: As = 18.72 in^2 at rho = 1.35%
  Minimum clear spacing = 5.67 in

  12 #11   Cover = 6 in

Factored Loads and Moments with Corresponding Capacities:
=========================================================
                  Pu          Mux       PhiMnx PhiMn/Mu NA depth Dt depth    eps_t    Phi
  No.            kip         k-ft         k-ft                in       in
  --- ------------ ------------ ------------ -------- -------- -------- -------- ------
    1      1294.00       810.00      1707.86    2.108    20.15    34.67  0.00216  0.755

  *** End of output ***
```



42 in diam.

Code: ACI 318-11

Units: English

Run axis: About X-axis

Run option: Investigation

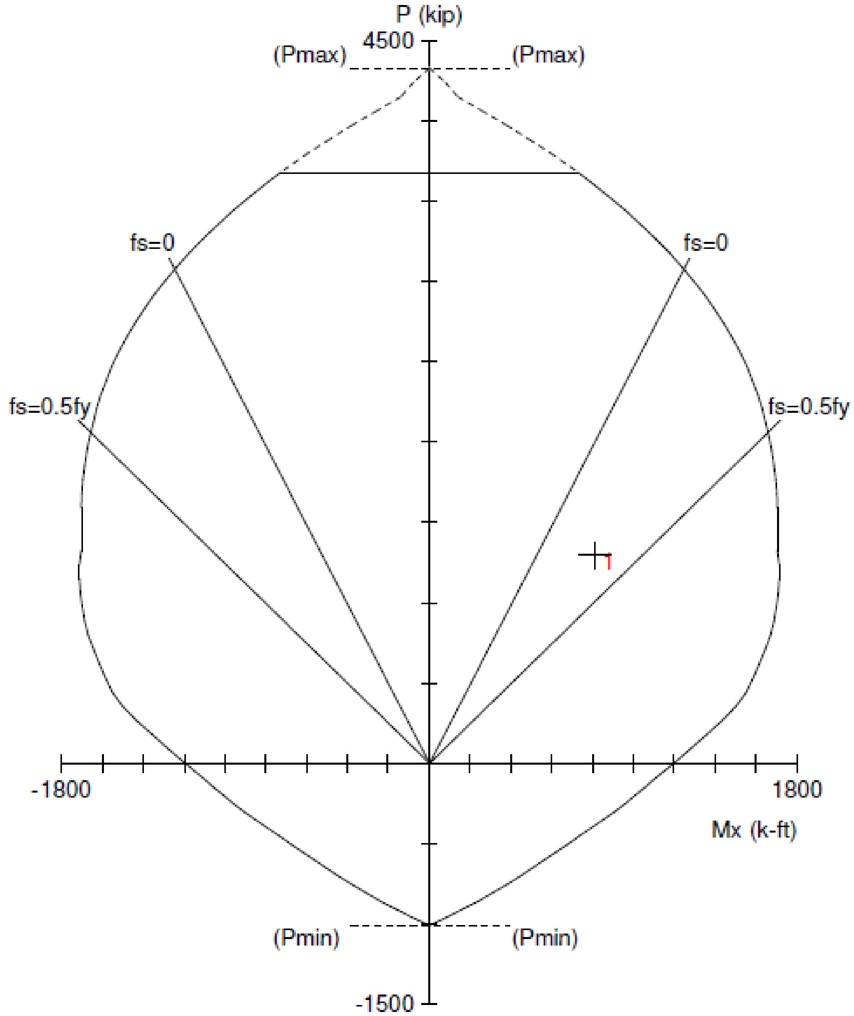
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 03/06/13

Time: 16:34:07



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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		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		Min clear spacing = 5.67 in	Clear cover = 6.63 in
Confinement: Spiral			
phi(a) = 0.85, phi(b) = 0.9, phi(c) = 0.75			

Appendix N: Little Bear Creek Bridge SDC B

Designer: Jordan Law

$ORIGIN := 1$

Project Name: Little Bear Creek Bridge
Job Number: APD-355 (501)
Date: 5/24/2012
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

Coordinates: 34.461N, 88.003W

Soil Site Class: C

Superstructure Type: AASTHO Type III girders for end spans
BT-72 girders in middle span

Substructure Type: Circular columns supported on drilled shafts

Abutment Type: Abutment beam supported on drilled shafts

Note: ***Input** all of the below information.*

The designer should input any information that can be used to calculate the dead weight of the bridge, including but not limited to length of bridge, column height(s), deck thickness, bent volume(s), and guard rail volume(s). Also, information about foundations should also be included if the bridge is classified as SDC B.

$A_s := .14$

$fc := 4000$ psi

$S_{D1} := .18$

$fye := 60000$ psi

$\rho_{conc} := 0.0868 \ \frac{lb}{in^3}$

$S_{DS} := .33$

INPUT

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

$SDC := "B"$

Length of Bridge (ft)	$L := 300$	ft
Skew of Bridge (degrees)	$Skew := 0$	degrees
End Spans (ft)	$EndSpan := 85$	ft
Middle Span (ft)	$MidSpan := 130$	ft
Deck Thickness (in)	$t_{deck} := 7$	in

Deck Width (ft)	DeckWidth := 42.7	ft
Superstructure Depth (ft)	D_s := 6.58	ft
Number of Bridge Girdes	N := 6	
I-Girder (AASHTO Type III) X-Sectional Area (in^2)	IGirderArea := 559.	in^2
Bulb (BT-72) Girder X-Sectional Area (in^2)	BulbGirderArea := 76	in^2
Guard Rail Area (in^2)	GuardRailArea := 31	in^2
Bent Volume (ft^3)	BentVolume := 40·(7·5 + 2.4·2.5) = 1.64× 10^3	ft^3
Column Diameter (in)	Columndia := 5	in
Number of Columns per Bent	N_{col} := 2	
Drilled Shaft Diameter (in)	DSdia := 6	in
Drilled Shaft Abutment Diameter (in)	DSabutdia := 42	in

The column height is measured from the bottom of the bent to the top of the pile footing. Other options include measuring from the top of the bent to the ground surface or to a change in diameter (if possible). If the plastic hinge location (at the bottom of the column) is known, then the column height should be measured from the bottom of the bent to the known hinge point.

Average Column Height for Bent 2 (ft)	$ColumnHeight_{Bent2}$:= 12.06	ft
Average Column Height for Bent 3 (ft)	$ColumnHeight_{Bent3}$:= 16.88	ft
Tallest Abutment Height Above Ground (ft)	$H_{abutment}$:=	ft
Column Area (in^2)	$Acolumn := \frac{Columndia^2 \cdot \pi}{4} = 2.29 \times 10^3$	in^2
Drilled Shaft Area (in^2)	$Adrilledshaft := \frac{DSdia^2 \cdot \pi}{4} = 2.827 \times 10^3$	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.

$EndSpan := EndSpan \cdot 12 = 1.02 \times 10^3$ in

$MidSpan := MidSpan \cdot 12 = 1.56 \times 10^3$ 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

$ColumnHeight_{Bent2} := ColumnHeight_{Bent2} \cdot 12 = 144.756$ in

$ColumnHeight_{Bent3} := ColumnHeight_{Bent3} \cdot 12 = 202.572$ in

Find Vertical Reactions at Each Bent:

Live Loads assumed to be present during an earthquake (see ***LRFD Article 3.4.1***)

$$Num\_Lanes := trunc\left(\frac{\frac{DeckWidth - 2 \cdot 1.375}{12}}{12}\right) = 3$$

Number of Lanes On Bridge (Design Lane Width of 10 ft) See ***LRFD 3.6.1.2.4***

$\gamma_{EQ} := 0.5$ ***LRFD Specificaiton C3.4.1 (Extreme Case I)*** **INPUT**

The γ_{EQ} value is to be determined on a project-specific basis. In the standard specification, a value of 0.0 was used, however, the LRFD Specification recommends a value of 0.5. See LRFD Article C3.4.1 under "EXTREME EVENT I"

$LL\_design := 0.64$ $\frac{klf}{lane}$ ***LRFD Specification 3.6.1.2.4***

$Q := LL\_design \cdot \gamma_{EQ} = 0.32$ $\frac{klf}{lane}$

$LL\_foot := Q \cdot Num\_Lanes = 0.96$ klf Live Load per linear foot of deck (includes all lanes)

Note: If the Vertical Reactions at each bent are already known, input them below, otherwise the sheet will calculate vertical reactions based on the given information above.

$DL_{Bent2} :=$ ▮ kip $LL_{Bent2} :=$ ▮ kip

$DL_{Bent3} :=$ ▮ kip $LL_{Bent3} :=$ ▮ kip **INPUT**

$VR_{Bent2} := DL_{Bent2} + LL_{Bent2} =$ ▮ kip

$VR_{Bent3} := DL_{Bent3} + LL_{Bent3} =$ ▮ kip

The weight calculation takes into account the entire dead weight of the structure, including the deck, bents, abutments, columns, girders, and railings. Any other expected dead loads should also be included.

$$W := \frac{\rho_{conc} \cdot \begin{pmatrix} L \cdot t_{deck} \cdot DeckWidth + 2 \cdot BentVolume + 2 \cdot Acolumn \cdot ColumnHeight_{Bent2} \ldots \\ + 2 \cdot Acolumn \cdot ColumnHeight_{Bent3} + 2EndSpan \cdot N \cdot IGirderArea \ldots \\ + N \cdot MidSpan \cdot BulbGirderArea + 2 \cdot GuardRailArea \cdot L \end{pmatrix}}{1000}$$

$W = 3163.856$ kips

To determine the vertical reaction at the bent, the bents tributary area will be calculated and multiplied by the total weight. A similar calculation will be done for the live load. This vertical reaction will be used to determine the connection force (below).

$$BentTribLength := \frac{\frac{EndSpan + MidSpan}{2}}{12} = 107.5 \quad ft$$

$$BentTribArea := \frac{\frac{EndSpan + MidSpan}{2}}{L} = 0.358$$ Percent of Area Tributary to Bent

$$DL_{Bent} := BentTribArea \cdot W = 1133.715 \quad kip$$

$$LL_{Bent} := BentTribLength \cdot LL\_foot = 103.2 \quad kip$$

$$VR_{Bent} := DL_{Bent} + LL_{Bent} = 1236.915 \quad kip$$

$VR_{Bent2} := VR_{Bent}$ $\qquad$ $VR_{Bent3} := VR_{Bent}$

<u>Steps for Seismic Design</u>

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

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

Article 3.4: Determine Deisgn Response Spectrum

Article 3.5: Determine SDC

Guide Figure 1.3-2: Seismic Design Procedure Flowchart for SDC B

Displacement Demand Analysis (Fig 1.3-2):

- *Article* 4.1: Seismic Design Proportioning
- *Article* 4.2: Determine Analysis Procedure
- *Article* 4.3.1: Determine Horizontal Ground Motion Effects Along Both Axis
- *Article* 4.3.2/4.3.3: Damping and Short Period Considerations
- *Article* 5.4/5.5: Select Analytical Procedure
- *Article* 5.6: Effective Section Properties
- *Article* 5.2: Abutment Modeling
- *Article* 5.3: Foundation Modeling and Liquefaction (if present)
- *Article* 5.1.2/4.4: Conduct Demand Analysis
- *Article* 4.8: Determine Displacement Demands Along Member Local Axes

Displacement Capacity Check ($\Delta_C > \Delta_D$):

Article 4.12: Determine Minimum Support Length

Article 4.14: Shear Key

Guide Figure 1.3-5: Foundation and Detailing Flowcharts

Foundation Design (Fig 1.3-5):

Article 6.8: Liquefaction Consideration

Article 6.3: Spread Footing Design

Article 6.4: Pile Cap Foundation Design

Article 6.5: Drilled Shaft

Article 6.7: Abutment Design

Detailing:

Article 8.3: Determine Flexure and Shear Demands

Article 8.7: Satisfy Requirements for Ductile Member Design

Article 8.6: Shear Demand and Capacity Check for Ductile Elements

Article 8.8: Satisfy Lateral and Longitudinal Reinforcement Requirements

Articles 3.4 and 3.5 have already been determined from the "SDC Classification" sheet.
Make sure the four values (A_S, S_{DS}, S_{D1}, and SDC) have been input above.

Displacement Demand Analysis (Δ_D)

***Article 4.1*: Seismic Design Proportioning**

See Guide Specification

***Article 4.2*: Determine Analysis Procedure**

This is a function of the SDC and the regularity of the bridge.
For a regular bridge in SDC B, Procedure 1 or 2 can be used.
For a non-regular bridge in SDC B, Procedure 2 must be used.

Guide Table 4.2-1

A regular bridge is defined as a bridge having fewer than 7 spans, no abrupt or unusual change in geometry and that saisfy the requirements below (***Guide Table 4.2-3***)

Table 4.2-3: Regular Bridge Requirements

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle (curved bridge)	30°	30°	30°	30°	30°
Maximum span length ratio from span-to-span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span-to-span (excluding abutments)	-	4	4	3	2

***Article 4.3.1*: Determine Horizontal Ground Motion Effects Along Both Axis**

Seismic displacement demands shall be determined independently in two orthogonal directions, typically the longitudinal and transverse axes of the bridge

Article 4.3.3: Displacement Magnification for Short-Period Structures

$u_d := 2$ for SDC B

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

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

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

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.

$p_o := 1.0 \ \dfrac{kip}{in}$

$\upsilon_{smaxLong} := 0.64720$ in

INPUT

$\upsilon_{smaxTran} := 5.26305$ in

$$K_{Long} := \frac{p_o \cdot L}{\upsilon_{smaxLong}} = 5562.388 \qquad \frac{kip}{in}$$ ***Guide Eq. C5.4.2-1***

$$K_{Tran} := \frac{p_o \cdot L}{\upsilon_{smaxTran}} = 684.014 \qquad \frac{kip}{in}$$ ***Guide Eq. C5.4.2-2***

The weight of the structure has already been calculated above

Step 4: Calculate the period, T_m.

$$T_{mLong} := 2\pi \cdot \sqrt{\frac{W}{K_{Long} \cdot g}} = 0.241 \qquad s$$ ***Guide Eq. C5.4.2-3***

Step 5: Calculate equivalent static earthquake loading p_e.

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

$$Sa_{Long} := acc(S_{DS}, S_{D1}, T_{mLong}, A_s) = 0.33$$

$$p_{eLong} := \frac{Sa_{Long} \cdot W}{L} = 0.29 \qquad \frac{kip}{in}$$ ***Guide Eq. C5.4.2-4***

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.

$Rd_{Long} := Rdprogram(T_{mLong}, S_{DS}, S_{D1}, u_d) = 1.914$

$v_{smaxLong} := Rd_{Long} \cdot \frac{p_{eLong}}{p_o} \cdot \upsilon_{smaxLong} = 0.359$ in

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

Step 4: Calculate the period, T_m.

$T_{mTran} := 2\pi \cdot \sqrt{\frac{W}{K_{Tran} \cdot g}} = 0.687$ s

Guide Eq. C5.4.2-3

Step 5: Calculate equivalent static earthquake loading p_e.

$Sa_{Tran} := acc(S_{DS}, S_{D1}, T_{mTran}, A_s) = 0.262$

$p_{eTran} := \frac{Sa_{Tran} \cdot W}{L} = 0.23$ $\frac{kip}{in}$

Guide Eq. C5.4.2-4

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.

$Rd_{Tran} := Rdprogram(T_{mTran}, S_{DS}, S_{D1}, u_d) = 1$

$v_{smaxTran} := Rd_{Tran} \cdot \frac{p_{eTran}}{p_o} \cdot \upsilon_{smaxTran} = 1.211$ in

LRFD Article 4.7.4.3.2: 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.

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.

$$v_{stran}(x) := 1\cdot10^{-7}\cdot x^2 + 0.0017x + 0.341$$

INPUT

$$v_{slong}(x) := -2\cdot10^{-9}\cdot x^2 + 0.0001x + 0.222$$

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

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

$$\gamma_{Tran} := \int_0^L \frac{W}{L}\cdot v_{stran}(x)^2\,dx = 6.102\times10^4 \qquad \gamma_{Long} := \int_0^L \frac{W}{L}\cdot v_{slong}(x)^2\,dx$$ ***LRFD C4.7.4.3.2b-3***

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

$$T_{mTran1} := 2\pi\cdot\sqrt{\frac{\gamma_{Tran}}{p_o\cdot g\cdot\alpha_{Tran}}} = 0.672 \quad s$$ ***LRFD Eq. 4.7.4.3.2b-4***

$$T_{mLong1} := 2\pi\cdot\sqrt{\frac{\gamma_{Long}}{p_o\cdot g\cdot\alpha_{Long}}} = 0.194 \quad s$$ ***LRFD Eq. 4.7.4.3.2b-4***

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smLong} := acc\left(S_{DS}, S_{D1}, T_{mLong1}, A_s\right) = 0.33$$

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.

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

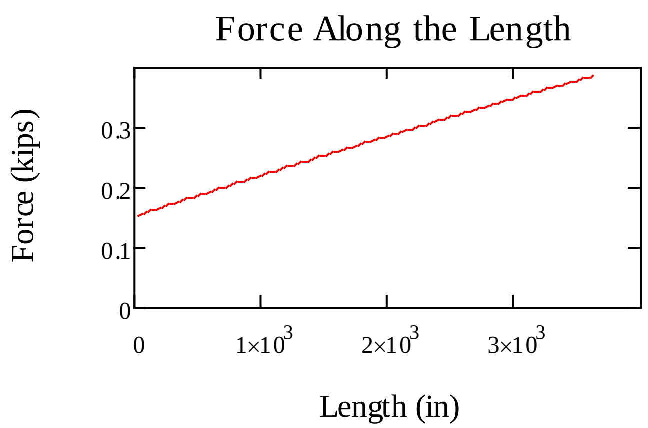
$PeLong(x) \rightarrow 0.0000694998726129264473066 \cdot x - 1.3899974522585289461e\text{-}9 \cdot x^2 + 0.1544982168185354\ldots$

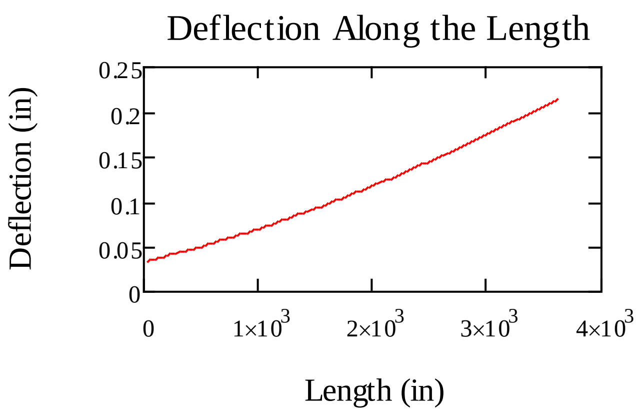
$dW := \frac{L}{100}$

$i := 1..101$

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

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





Maximum Deflection:

$\max(\Delta long) = 0.215$ in

NOTE: Repeat Steps 5 and 6 for Transverse Direction.

Step 5: Calculate the equivalent Static Earthquake Loading

$C_{smTran} := acc\left(S_{DS}, S_{D1}, T_{mTran1}, A_s\right) = 0.268$

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.

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

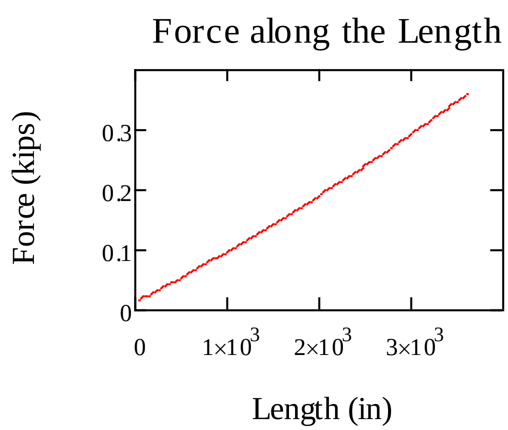
$PeTran(x) \rightarrow 0.000079524611576029477214 + 4.6779183280017339553e{-9} \cdot x^2 + 0.015961057335141916\ldots$

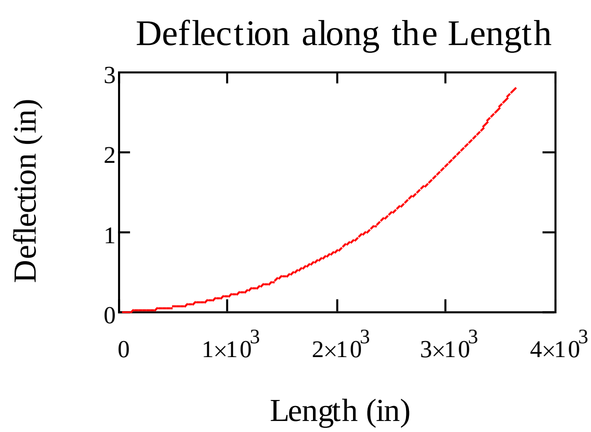
$dL := \frac{L}{100}$

$i := 1..101$

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

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





Maximum Deflection:

$\max(\Delta tran) = 2.815$ in

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

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**

This is taken care of in the SAP model.

***Article 5.3*: Foundations Modeling**

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.

Special provisions need to be considered if Liquefaction is present. ***Guide Article 6.8***

***Article 4.4*: Combination of Orthogonal Seismic Displacement Demands**

$$LoadCase1 := \sqrt{\left(1 \cdot v_{smaxLong}\right)^2 + \left(0.3 v_{smaxTran}\right)^2} = 0.511 \quad in$$

$$LoadCase2 := \sqrt{\left(1 \cdot v_{smaxTran}\right)^2 + \left(0.3 v_{smaxLong}\right)^2} = 1.216 \quad in$$

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

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

$\Delta_{Dlong} := 0.256$ in **INPUT**

$\Delta_{Dtran} := 0.795$ in

$\Delta_{DLong} := Rd_{Long} \cdot \Delta_{Dlong} \cdot p_{eLong} = 0.142$ in

$\Delta_{DTran} := Rd_{Tran} \cdot \Delta_{Dtran} \cdot p_{eTran} = 0.183$ in

$$LoadCase1 := \sqrt{(1\Delta_{DLong})^2 + (0.3\Delta_{DTran})^2} = 0.153 \quad in$$

$$LoadCase2 := \sqrt{(1\Delta_{DTran})^2 + (0.3\Delta_{DLong})^2} = 0.188 \quad in$$

$$\Delta_D := \max(LoadCase1, LoadCase2) = 0.188 \quad in$$

$$H_o := \frac{ColumnHeight_{Bent2}}{12} = 12.063 \quad ft \qquad B_o := \frac{Columndia}{12} = 4.5 \quad ft$$

Transverse Direction

$\Lambda := 2$ Fixed and top and bottom ***Guide Article 4.8.1***

$$x := \frac{\Lambda \cdot B_o}{H_o} = 0.746$$ ***Guide Eq. 4.8.1-3***

$$\Delta_{CT} := 0.12 H_o \cdot (-1.27 \cdot \ln(x) - 0.32) = 0.075 \quad in$$ ***Guide Eq. 4.8.1-1***

Longitudinal Direction

Guide Article 4.8.1

$\Lambda := 1$ Fixed-Free

Guide Eq. 4.8.1-3

$$x := \frac{\Lambda \cdot B_o}{H_o} = 0.373$$

$$\Delta_{CL} := 0.12 H_o \cdot (-1.27 \cdot \ln(x) - 0.32) = 1.35 \quad in$$ ***Guide Eq. 4.8.1-1***

$$\Delta_C := \min(\Delta_{CT}, \Delta_{CL}) = 0.075$$

$$0.12 H_o = 1.448 \quad in$$

$$CheckLimit(\Delta_C, H_o) := \left| \begin{array}{l} a \leftarrow 0.12 H_o \ \text{ if } \ \Delta_C < 0.12 H_o \\ a \leftarrow \Delta_C \ \text{ otherwise} \end{array} \right.$$

$$\Delta_C := CheckLimit(\Delta_C, H_o)$$

$$\Delta_C = 1.448$$

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

$$CheckCapacity(\Delta_C, \Delta_D) = \text{"OK"}$$

If the simplified equations do not work ("FAILURE") for any of the bents, 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.

BENT 3

INPUT

$\Delta_{Dlong} := 0.370$ in

$\Delta_{Dtran} := 2.240$ in

$\Delta_{DLong} := Rd_{Long} \cdot \Delta_{Dlong} \cdot p_{eLong} = 0.206$ in

$\Delta_{DTran} := Rd_{Tran} \cdot \Delta_{Dtran} \cdot p_{eTran} = 0.516$ in

$LoadCase1 := \sqrt{\left(1\Delta_{DLong}\right)^2 + \left(0.3\Delta_{DTran}\right)^2} = 0.257$ in

$LoadCase2 := \sqrt{\left(1\Delta_{DTran}\right)^2 + \left(0.3\Delta_{DLong}\right)^2} = 0.519$ in

$\Delta_D := \max(LoadCase1, LoadCase2) = 0.519$ in

$H_o := \dfrac{ColumnHeight_{Bent3}}{12} = 16.881$ ft $\qquad B_o := \dfrac{Columndia}{12} = 4.5$ ft

Transverse Direction

$\Lambda := 2$ Fixed and top and bottom ***Guide Article 4.8.1***

$x := \dfrac{\Lambda \cdot B_o}{H_o} = 0.533$ ***Guide Eq. 4.8.1-3***

$\Delta_{CT} := 0.12 H_o \cdot (-1.27 \ln(x) - 0.32) = 0.97$ in ***Guide Eq. 4.8.1-1***

Longitudinal Direction

Guide Article 4.8.1

$\Lambda := 1$ Fixed-Free

Guide Eq. 4.8.1-3

$x := \dfrac{\Lambda \cdot B_o}{H_o} = 0.267$

$\Delta_{CL} := 0.12 H_o \cdot (-1.27 \ln(x) - 0.32) = 2.753$ in ***Guide Eq. 4.8.1-1***

$\Delta_C := \min\left(\Delta_{CT}, \Delta_{CL}\right) = 0.97$

$0.12 H_o = 2.026$ in

$$\mathrm{CheckLimit}(\Delta_C, H_o) := \left| \begin{array}{l} a \leftarrow 0.12 H_o \quad \text{if } \Delta_C < 0.12 H_o \\ a \leftarrow \Delta_C \quad \text{otherwise} \end{array} \right.$$

$$\mathrm{CheckLimit}(\Delta_C, H_o) = 2.026$$

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

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

Pushover Analysis Results (if necessary):

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

INPUT

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

Abutment Support Length Requirement ***Guide Eq. 4.12.2-1***

$$\mathrm{Nabutment} := 1.5(8 + 0.02 \mathrm{EndSpan} + 0.08 H_{abutment}) \cdot (1 + 0.000125 \mathrm{Skew}_{abutment}{}^2) = \blacksquare \quad \text{in}$$

Bent Support Length Requirement ***Guide Eq. 4.12.2-1***

BENT 2

$$L := \mathrm{BentTribLength} = 107.5 \qquad S_{D1} := 0.3($$

$$H := \frac{\mathrm{ColumnHeight}_{Bent2}}{12} = 12.063$$

Standard Specifications

$$N_{2Stan} := (8 + 0.02 \cdot L + 0.08 H) \cdot (1 + 0.000125 \mathrm{Skew}^2) = 11.115 \quad \text{in}$$

ATC-49 Equation (New Design)

$$N_2 := \left[4 + .02 L + .08 H + 1.09 \sqrt{H} \cdot \sqrt{1 + \left(2 \cdot \frac{3}{8}\right)^2} \right] \cdot \left(\frac{1 + 1.25 S_{D1}}{\cos\left(\frac{\mathrm{Skew} \cdot \pi}{180}\right)} \right) = 16.29 \quad \text{in}$$

BENT 3

$L := BentTribLength = 107.5$ $\qquad$ $S_{D1} := 0.3($

$$H := \frac{ColumnHeight_{Bent3}}{12} = 16.881$$

Standard Specifications

$$N_{3Stan} := (8 + 0.02 \cdot L + 0.08 H) \cdot \left(1 + 0.000125 Skew^2\right) = 11.5 \qquad in$$

ATC-49 Equation (New Design)

$$N_3 := \left[4 + .02 L + .08 H + 1.09 \sqrt{H} \cdot \sqrt{1 + \left(2 \cdot \frac{3}{8}\right)^2}\right] \cdot \left(\frac{1 + 1.25 S_{D1}}{\cos\left(\frac{Skew \cdot \pi}{180}\right)}\right) = 18.01 \qquad in$$

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

$V_{ok} := 1.5 V_n$ *This does not apply to this bridge*

BENT 2 DESIGN

Guide Article 4.11.2: For SDC B, it is acceptable to use the moment capacity based on expected material strengths when the concrete reaches an extreme compressive fiber strain of 0.003.

Force Inputs

$M_{neBent2} := 453($	kip – ft	Nominal moment from PCA Column
$V_{pelastic} := 102($	kip	Elastic shear from SAP2000 model
$P_u := 128400($	lb	Axial load from earthquake and dead load combination

INPUT

Reinforcement Details

$Ag := Acolumn$

$Ae := 0.8 Ag = 1832$ $\quad in^2$ $\qquad$ ***Guide Eq. 8.6.2-2***

$\mu_D := 2$ $\qquad$ ***Guide Article 8.6.2***

$n := 2$ $\qquad$ n: Number of individual interlocking spiral or hoop core sections

StirrupSize:= "#5" $\qquad$ Tiesize: Bar size used for ties

$s := 6$	in	s: Spacing of hoops or pitch of spiral (in)
$sNOhinge := 10$	in	sNOhinge: Spacing of hoops or pitch outside PHL
$Asp := 0.31$	in^2	Asp: Area of spiral or hoop reinforcing bar (in^2)
$Dsp := 0.625$	in	Dsp: Diameter of spiral or hoop reinforcing (in) **INPUT**
$Cover := 3$	in	Cover: Concrete cover for the Column (in)
$b := Columndia$	in	b: Diameter of column (in)
$d := b - Cover = 51$	in	d: Effective depth of section in direction of loading (in)
$Dprime := b - 2 \cdot Cover$	in	Dprime: Diameter (in column) of hoop reinforcing (in)
$NumberBars := 16$		Total number of longitudinal bars in column cross-section
$A_{bl} := 1.56$	in^2	Abl: Area of longitudinal bar
$d_{bl} := 1.41$	in	dbl: Diameter of longitudinal bar
$bv := Columndia$		bv: Diameter of column

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

The design forces shall be the lesser of the forces resulting from the overstrength plastic hinging moment capacity or unreduced elastic seismic forces in columns or pier walls.

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

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.

$\lambda_{mo} := 1.4$ for ASTM A 615 Grade 60 reinforcement ***Guide Article 8.5***

$M_{pBent2} := \lambda_{mo} \cdot M_{neBent2} \cdot 1000 \cdot 12 = 7.615 \times 10^{7}$ lb − in

$Fixity := ColumnHeight_{Bent2} = 144.756$ in

$$V_p := \frac{2 \cdot M_{pBent2}}{Fixity \cdot 1000} = 1.052 \times 10^{3} \text{ kips}$$

$V_{pelastic} := V_{pelastic} \cdot \max(p_{eTran}, p_{eLong}) = 295.821$ kips

***Article 8.7*: Requirements for Ductile Member Design**

Each column must satisfy the minimum lateral flexural capacity

$\Lambda := 2$ Fixed and top and bottom ***Guide Article 4.8.1***

$$M_{neminBent} := 0.1 \cdot DL_{Bent} \cdot \left(\frac{\frac{Fixity}{12} + 0.5 D_s}{\Lambda} \right) = 870.381$$ kip ft ***Guide Eq 8.7.1-1***

$$CheckMoment(M_{ne}, M_e) := \begin{array}{|l} a \leftarrow \text{"OK"} \quad \text{if } M_{ne} \le M_e \\ a \leftarrow \text{"FAILURE"} \quad \text{otherwise} \end{array}$$

$$CheckMoment(M_{neminBent}, M_{neBent2}) = \text{"OK"}$$

If the moment check comes back "FAILURE" then the column size can be increased or the reinforcement can be increased.

Article 8.6: Shear Demand and Capacity for Ductile Concrete Members

It is recommended to use the plastic hinging forces whenever practical, but in this case the elastic forces will be used.

$V_u := \min(V_p, V_{pelastic}) = 295.821$ kips $\phi_s := 0.9$

$V_{pBent2} := V_u$

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

$$\rho_s := \frac{4 \cdot Asp}{s \cdot Dprime} = 0.0043$$ ***Guide Eq. 8.6.2-7***

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

$$StressCheck(\rho_s, fyh) := \begin{array}{|l} fs \leftarrow \rho_s \cdot fyh \\ a \leftarrow fs \quad \text{if } fs \le 0.35 \end{array}$$

$$fs := StressCheck(\rho_s, fyh) = 0.258$$ ***Guide Eq. 8.6.2-6***

$$\alpha program(fs, \mu_D) := \begin{array}{|l} \alpha prime \leftarrow \frac{fs}{0.15} + 3.67 - \mu_D \\ a \leftarrow 0.3 \quad \text{if } \alpha prime \le 0.3 \\ a \leftarrow \alpha prime \quad \text{if } \alpha prime > 0.3 \wedge \alpha prime < 3 \\ a \leftarrow 3 \quad \text{if } \alpha prime \ge 3 \\ a \end{array}$$ ***Guide Eq. 8.6.2-5***

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

If Pu is Compressive:

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

Guide Eq. 8.6.2-3

If Pu is NOT Compressive:

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

Guide Eq. 8.6.2-4

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

$Vc := vc \cdot Ae = 403.079$ kips

Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity** ***Guide Eq 8.6.3-2 and 8.6.4-1

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

$Vs := vsprogram(n, Asp, fyh, Dprime, s, fc, Ae) = 467.469$ kips

$\phi Vn := \phi_s \cdot (Vs + Vc) = 783.493$ kips

Guide Eq. 8.6.1-2

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

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

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:

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

Guide Eq. 8.6.5-1

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

If the minimum shear reinforcement program responses "Increase Shear Reinforcing Ratio", it is recommended to <u>decrease the spacing (s)</u> or i<u>ncrease 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**

$A_{long} := NumberBars \cdot A_{bl} = 24.96$ $\quad in^2$

$$\rho program(A_{long}, Ag) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } A_{long} \leq 0.04 Ag \\ a \leftarrow \text{"Section Over Reinforced"} \quad \text{if } A_{long} > 0.04 Ag \\ a \end{array} \right.$$

Guide Eq. 8.8.1-1

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

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**

$$minAlprogram(A_l, Ag) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } A_{long} \geq 0.007 Ag \\ a \leftarrow \text{"Increase Longitudinal Reinforcing"} \quad \text{if } A_{long} < 0.007 Ag \\ a \end{array} \right.$$

Guide Eq. 8.8.2-1

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

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

***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.

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

Guide Eq. 4.11.6-1

$$L_p := PlasticHinge(Fixity, fye, d_{bl}) = 24.27 \quad in$$

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

'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 ρ_{eTran} to take into account the model loads have not been multiplied by ρ_{eTran}. The location will also need to be **INPUT** into the PlasticHingeRegion program in inches.

$Mp75 := 0.75 M_{pBent2} = 5.712\times 10^{7}$ lb·in

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

$L_{p1} := PlasticHingeRegion(L_p, Columndia) = 81$ in

The Guide Specifications allows for the use of the plastic hinge length from the LRFD Specification in SDC A and B (***Guide Article C8.8.9***).

$$LRFDPlasticHingeLength(ColumnDia, ColumnHeight) := \left| \begin{array}{l} a \leftarrow ColumnDia \\ b \leftarrow \frac{1}{6} \cdot ColumnHeigh \\ c \leftarrow 18 \\ PHL \leftarrow max(a, b, c) \\ PHL \end{array} \right.$$

$L_{p2} := LRFDPlasticHingeLength(Columndia, Fixity) = 54$ in

Guide Article C8.8.9:

The plastic hinge length will be the smaller of the two values, as the Guide Specification allows:

$PHL := min(L_{p1}, L_{p2}) = 54$ in

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region: ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

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

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

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

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

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

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

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.

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

$\text{Extension} := \text{ExtensionProgram}(\text{Columndia}) = 27$ in

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

LRFD 5.8.3.3 **Nominal Shear Resistance**

$V_u = 295.821$ kips

$\phi_s = 0.9$

$\beta := 2.0$ ***LRFD Article 5.8.3.4.1***

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

$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 49.67$ in

$de := d = 51$ in ***LRFD Eq. 5.8.2.9-1***

$dv := 0.9 de = 45.9$ in

$V_c := 0.0316 \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 313.295$ kips ***LRFD Eq. 5.8.3.3-3***

$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{sNOhinge} = 170.748 \qquad \text{kips}$$

LRFD Eq. 5.8.3.3-4

$$V_n := \phi_s \cdot .25 fc \cdot bv \cdot dv = 2.231 \times 10^6 \qquad \text{kips}$$

$$\phi Vn := \min\left[V_n, \left(V_c + V_s\right) \cdot \phi_s\right] = 435.639 \qquad \text{kips}$$

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

$$\text{Shearcheck2} := \text{ShearCheck}\left(\phi Vn, V_u\right) = \text{"OK"}$$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

$$Avmin := 0.0316 \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot sNOhinge}{\frac{fye}{1000}} = 0.569 \qquad \text{in}^2$$

LRFD Eq. 5.8.2.5-1

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

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

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

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.

LRFD 5.8.2.7 Maximum Spacing of Transverse Reinforcement

$$vu := \frac{V_u}{\phi_s \cdot bv \cdot dv} = 0.133 \qquad \text{ksi}$$

LRFD Eq. 5.8.2.9-1

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

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

$$\text{MaxSpacing} := \text{floor}(\text{spacingProgram}(vu, dv, fc)) = 24 \quad \text{in}$$

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$\text{Spacecheck}(\text{MaxSpacing}, s, V_u, V_c) := \left| \begin{array}{l} a \leftarrow \min(\text{MaxSpacing}, 12) \text{ if } V_u \le 0.5 0.9 V_c \\ a \leftarrow \min(s, \text{MaxSpacing}, 12) \text{ otherwise} \\ a \end{array} \right.$$

$$\text{sNOhinge} := \text{Spacecheck}(\text{MaxSpacing}, \text{sNOhinge}, V_u, V_c) = 10 \quad \text{in}$$

Design Summary - Bent 2

StirrupSize = "#5"

$s = 6$ in

$\text{sNOhinge} = 10$ in

$\text{PHL} = 54$ in

$\text{Extension} = 27$ in

$N_2 = 16.29$ in

Design Check Summary - Bent 2

Shearcheck = "OK" — Shear capacity > Vn

Transversecheck = "OK" — Minimum shear reinforcement ratio

ReinforcementRatioCheck = "OK"	Maximum longitudinal reinforcement ratio
$\text{Minimum}A_l$ = "OK"	Minimum longitudinal reinforcement ratio
scheck = "OK"	Max spacing of transverse reinforcement
Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran= "OK"	Minimum shear reinforcement outside hinge zone

BENT 3 DESIGN

Guide Article 4.11.2: For SDC B, it is acceptable to use the moment capacity based on expected material strengths when the concrete reaches an extreme compressive fiber strain of 0.003.

Force Inputs

$M_{neBent3}$:= 453	kip – ft	Nominal moment from PCA Column	
$V_{pelastic}$:= 102	kip	Elastic shear from SAP2000 model	**INPUT**
P_u := 128400	lb	Axial load from earthquake and dead load combination	

Reinforcement Details

Ag := Acolumn

Ae := 0.8Ag = 1832	in^2	***Guide Eq. 8.6.2-2***
μ_D := 2		***Guide Article 8.6.2***

n := 2		n: Number of individual interlocking spiral or hoop core sections	
StirrupSize:= "#5"		Tiesize: Bar size used for ties	
s := 6	in	s: Spacing of hoops or pitch of spiral (in)	
sNOhinge := 10	in	sNOhinge: Spacing of hoops or pitch outside PHL	
Asp := 0.3	in^2	Asp: Area of sprial or hoop reinforcing bar (in^2)	
Dsp := 0.62	in	Dsp: Diameter of spiral or hoop reinforcing (in)	**INPUT**
Cover := 3	in	Cover: Concrete cover for the Column (in)	
b := Columndi	in	b: Diameter of column (in)	

$d := b - Cover = 51$ in d: Effective depth of section in direction of loading (in)

$Dprime := b - 2 \cdot Cover$ in Dprime: Diameter (in column) of hoop reinforcing (in)

$NumberBars := 16$ Total number of longitudinal bars in column cross-section

$A_{bl} := 1.56$ in^2 Abl: Area of longitudinal bar

$d_{bl} := 1.41$ in dbl: Diameter of longitudinal bar

$bv := Columndia$ bv: Diameter of column

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

The design forces shall be the lesser of the forces resulting from the overstrength plastic hinging moment capacity or unreduced elastic seismic forces in columns or pier walls.

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

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.

$\lambda_{mo} := 1.4$ for ASTM A 615 Grade 60 reinforcement ***Guide Article 8.5***

$M_{pBent3} := \lambda_{mo} \cdot M_{neBent3} \cdot 1000 \cdot 12 = 7.615 \times 10^7$ lb − in

$Fixity := ColumnHeight_{Bent3} = 202.572$ in

$$V_p := \frac{2 \cdot M_{pBent3}}{Fixity \cdot 1000} = 751.875 \text{ kips}$$

$V_{pelastic} := V_{pelastic} \cdot \max(p_{eTran}, p_{eLong}) = 295.821$ kips

***Article 8.7*: Requirements for Ductile Member Design**

Each column must satisfy the minimum lateral flexural capacity

$\Lambda := 2$ Fixed and top and bottom ***Guide Article 4.8.1***

$$M_{neminBent} := 0.1 \cdot DL_{Bent} \cdot \left(\frac{\frac{Fixity}{12} + 0.5 D_s}{\Lambda} \right) = 1143.493 \text{ kip ft}$$ ***Guide Eq 8.7.1-1***

$$CheckMoment(M_{ne}, M_e) := \begin{array}{|l} a \leftarrow \text{"OK"} \quad \text{if } M_{ne} \le M_e \\ a \leftarrow \text{"FAILURE"} \quad \text{otherwise} \end{array}$$

$CheckMoment(M_{neminBent}, M_{neBent2}) = \text{"OK"}$

If the moment check comes back "FAILURE" then the column size can be increased or the reinforcement can be increased.

Article 8.6: Shear Demand and Capacity for Ductile Concrete Members

It is recommended to use the plastic hinging forces whenever practical, but in this case the elastic forces will be used.

$V_u := \min(V_p, V_{pelastic}) = 295.821$ kips $\qquad \phi_s := 0.9$

$V_{pBent3} := V_u$

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

$$\rho_s := \frac{4 \cdot Asp}{s \cdot Dprime} = 0.0043$$ ***Guide Eq. 8.6.2-7***

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

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

$$fs := StressCheck(\rho_s, fyh) = 0.258$$ ***Guide Eq. 8.6.2-6***

$$\alpha program(fs, \mu_D) := \left| \begin{array}{l} \alpha prime \leftarrow \frac{fs}{0.15} + 3.67 - \mu_D \\ a \leftarrow 0.3 \;\; \text{if } \alpha prime \le 0.3 \\ a \leftarrow \alpha prime \;\; \text{if } \alpha prime > 0.3 \wedge \alpha prime < 3 \\ a \leftarrow 3 \;\; \text{if } \alpha prime \ge 3 \\ a \end{array} \right.$$ ***Guide Eq. 8.6.2-5***

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

If Pu is Compressive:

$$vcprogram(\alpha Prime, fc, P_u, Ag) := \left| \begin{array}{l} vc \leftarrow 0.032\alpha Prime \left(1 + \frac{P_u}{2Ag \cdot 1000}\right) \cdot \sqrt{\frac{fc}{1000}} \\ min1 \leftarrow 0.11\sqrt{\frac{fc}{1000}} \\ min2 \leftarrow 0.047\alpha Prime\sqrt{\frac{fc}{1000}} \\ minimum \leftarrow \min(min1, min2) \\ a \leftarrow vc \;\; \text{if } vc \le minimum \\ a \leftarrow minimum \;\; \text{if } vc > minimum \\ a \end{array} \right.$$ ***Guide Eq. 8.6.2-3***

If Pu is NOT Compressive:

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

Guide Eq. 8.6.2-4

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

$Vc := vc \cdot Ae = 403.079$ kips

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

Guide Eq 8.6.3-2 and 8.6.4-1

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

$Vs := vsprogram(n, Asp, fyh, Dprime, s, fc, Ae) = 467.469$ kips

$\phi Vn := \phi_s \cdot (Vs + Vc) = 783.493$ kips

Guide Eq. 8.6.1-2

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

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

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:

Guide Eq. 8.6.5-1

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

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

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**

$A_{long} := NumberBars \cdot A_{bl} = 24.96 \quad in^2$

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

Guide Eq. 8.8.1-1

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

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**

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

Guide Eq. 8.8.2-1

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

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

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.

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

Guide Eq. 4.11.6-1

$L_p := PlasticHinge(Fixity, fye, d_{bl}) = 28.896$ in

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

'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 ρ_{eTran} to take into account the model loads have not been multiplied by ρ_{eTran}. The location will also need to be **<u>INPUT</u>** into the PlasticHingeRegion program in inches.

$Mp75 := 0.75 M_{pBent2} = 5.712 \times 10^7$ lb·in

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

$L_{p1} := \text{PlasticHingeRegion}(L_p, \text{Columndia}) = 81$ in

The Guide Specifications allows for the use of the plastic hinge length from the LRFD Specification in SDC A and B (***Guide Article C8.8.9***).

$$\text{LRFDPlasticHingeLength}(\text{ColumnDia}, \text{ColumnHeight}) := \begin{array}{|l} a \leftarrow \text{ColumnDia} \\ b \leftarrow \frac{1}{6} \cdot \text{ColumnHeigh} \\ c \leftarrow 18 \\ PHL \leftarrow \max(a, b, c) \\ PHL \end{array}$$

$L_{p2} := \text{LRFDPlasticHingeLength}(\text{Columndia}, \text{Fixity}) = 54$ in

Guide Article C8.8.9

The plastic hinge length will be the smaller of the two values, as the Guide Specification allows:

$PHL := \min(L_{p1}, L_{p2}) = 54$ in

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region: ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

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

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

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

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

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

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 5.10.11.4.1e (LRFD SPEC.): Extension Length

The spacing in the hinge region shall continue into the drilled shaft or cap beam the Extension length.

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

$\text{Extension} := \text{ExtensionProgram}(\text{Columndia}) = 27$ in

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

LRFD 5.8.3.3 Nominal Shear Resistance

$V_u = 295.821$ kips

$\phi_s = 0.9$

$\beta := 2.0$ ***LRFD Article 5.8.3.4.1***

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

$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 49.67$ in

$de := d = 51$ in ***LRFD Eq. 5.8.2.9-1***

$dv := 0.9 de = 45.9$ in

$V_c := 0.0316\beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 313.295$ kips ***LRFD Eq. 5.8.3.3-3***

$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{sNOhinge} = 170.748$ kips ***LRFD Eq. 5.8.3.3-4***

$V_n := \phi_s \cdot .25 fc \cdot bv \cdot dv = 2.231 \times 10^6$ kips

$\phi Vn := \min\left[V_n, \left(V_c + V_s\right) \cdot \phi_s\right] = 435.639$ kips

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

$\text{Shearcheck2} := \text{ShearCheck}\left(\phi Vn, V_u\right) = \text{"OK"}$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

LRFD Eq. 5.8.2.5-1

$$\text{Avmin} := 0.0316\sqrt{\frac{fc}{1000}}\cdot\frac{bv\cdot sNOhinge}{\frac{fye}{1000}} = 0.569 \qquad in^2$$

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

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

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

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.

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

$$vu := \frac{V_u}{\phi_s\cdot bv\cdot dv} = 0.133 \qquad ksi$$

LRFD Eq. 5.8.2.9-1

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

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

$$\text{MaxSpacing} := \text{floor}(\text{spacingProgram}(vu, dv, fc)) = 24 \qquad in$$

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$\text{Spacecheck}\left(\text{MaxSpacing}, s, V_u, V_c\right) := \begin{array}{|l} a \leftarrow \min(\text{MaxSpacing}, 12) \quad \text{if } V_u \le 0.5 0.9 V_c \\ a \leftarrow \min(s, \text{MaxSpacing}, 12) \quad \text{otherwise} \\ a \end{array}$$

$$\text{sNOhinge} := \text{Spacecheck}\left(\text{MaxSpacing}, \text{sNOhinge}, V_u, V_c\right) = 10 \ \text{in}$$

Design Summary - Bent 3

StirrupSize = "#5"

$s = 6$ in

$\text{sNOhinge} = 10$ in

$\text{PHL} = 54$ in

$\text{Extension} = 27$ in

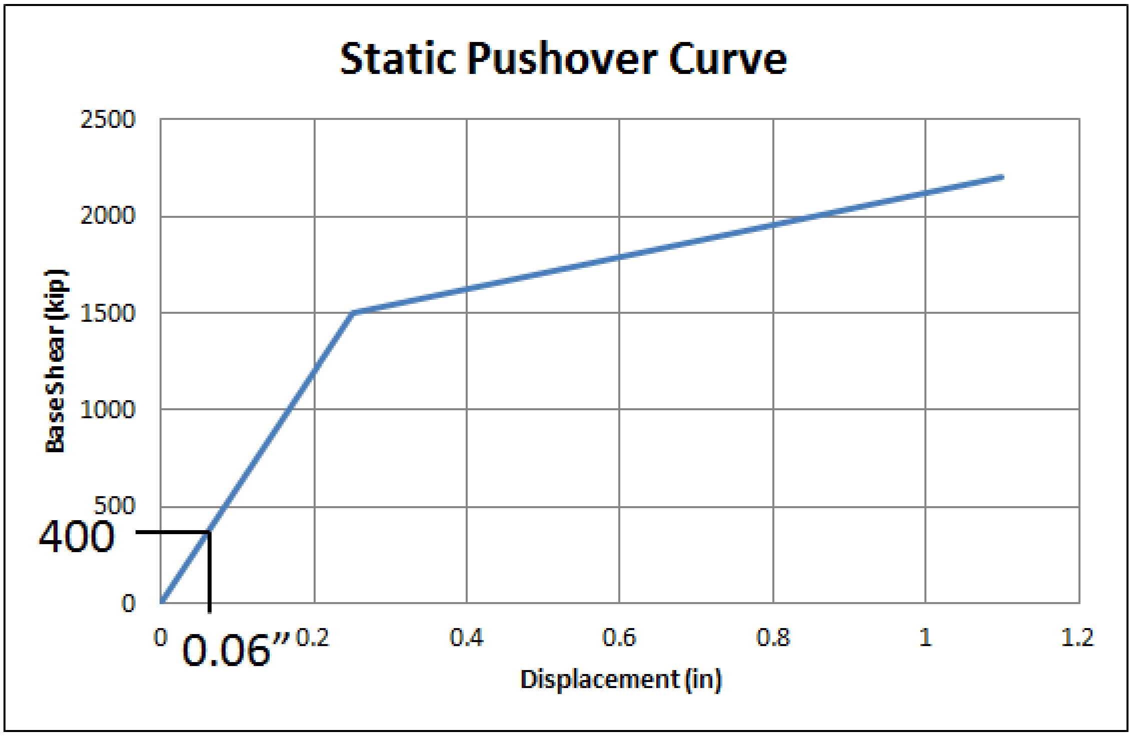
$N_3 = 18.01$ in

Design Check Summary - Bent 3

Shearcheck = "OK"	Shear capacity > Vn
Transversecheck = "OK"	Minimum shear reinforcement ratio
ReinforcementRatioCheck = "OK"	Maximum longitudinal reinforcement ratio
MinimumA_l = "OK"	Minimum longitudinal reinforcement ratio
scheck = "OK"	Max spacing of transverse reinforcement
Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran = "OK"	Minimum shear reinforcement outside hinge zone

Transverse Connection Design

Pushover Analysis Results



ALDOT Current Connection Steel Angle Design Check

$Vcolbent := \frac{400}{N} = 66.667$ kips

LRFD 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

Bolt Properties

$Fub := 58$	ksi	Strength of Anchor Bolt (It is assumed that ASTM A307 Grade C bolt is used)
$Dia_b := 2.25$	in	Diameter of Anchor Bolt

INPUT

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

Angle Properties

$Fy := 36$	ksi	Yield Stress of the Angle
$Fu := 58$	ksi	Ultimate Stress of the Angle
$t := 1.00$	in	Thickness of Angle
$h := 6$	in	Height of the Angle
$w := 6$	in	Width of the Angle
$l := 12$	in	Length of the Angle
$k := 1.5$	in	Height of the Bevel
$distanchorhole := 4$	in	Distance from the vertical leg to the center of the hole. This is the location of the holes.
$diahole := Dia_b + \frac{1}{8} = 2.375$	in	Diameter of bolt hole
$BLSHlength := 6$	in	Block Shear Length
$BLSHwidth := 2$	in	Block Shear Width
$Ubs := 1.0$		Shear Lag Factor for Block Shear
$a := 2$	in	Distance from the center of the bolt to the edge of plate
$b := 3.5$	in	Distance from center of bolt to toe of fillet of connected part
$Lc := 2$	in	Clear dist. between the hole and the end of the member

INPUT

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

Clip Angle Check:

AISC J4: Block Shear

$Agv := t \cdot BLSHlength = 6$ in^2

$Anv := t \cdot (BLSHlength - 0.5diahole) = 4.813$ in^2

$Ant := t \cdot (BLSHwidth - 0.5diahole) = 0.813$ in^2

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

AISC Eq. J4-5

$Rn := \mathrm{BLSHprogram}(Agv, Anv, Ant, Ubs, Fu, Fy) = 176.725$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 141.38$ kips

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

AISC D2: Tension Member

$Ut := 0.6$

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

$Ant := t \cdot [w - (1 \cdot diahole)] = 3.625$ in^2

$Ae := Ant \cdot Ut = 2.175$ in^2 ***AISC Eq. D3-1***

$\phi tPn := \phi_t \cdot Fub \cdot Ae = 100.92$ kips ***AISC Eq. D2-2***

$TensionCheck_{AISC} := ShearCheck(\phi tPn, Vcolbent) = \text{"OK"}$

AISC G: Shear Check

$Cv := 1.0$

$Aw := t \cdot w = 6$ in^2

$\phi sangleVn := \phi_{sangle} \cdot 0.6 Fy \cdot Aw \cdot Cv = 129.6$ kips ***AISC Eq. G2-1***

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

Anchor Bolt Check:

LRFD Article 6.13.2.12: Shear Resistance For Anchor Bolts

$$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 3.976 \quad in^2$$

$\phi sRn := \phi_s \cdot 0.48 A_b \cdot Fub \cdot Ns = 83.021$ kips ***LRFD Eq. 6.13.2.12-1***

$Shear_{Anchorbolts} := ShearCheck(\phi sRn, Vcolbent) = "OK"$

LRFD Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

$\phi bbRn := 2.4 Dia_b \cdot t \cdot Fub = 313.2$ kips **_LRFD Eq. 6.13.2.9-1_**

For Slotted Holes

$\phi bbRns := Lc \cdot t \cdot Fub = 116$ kips **_LRFD Eq. 6.13.2.9-4_**

$Bearing_{Boltstandard} := ShearCheck(\phi bbRn, Vcolbent) = "OK"$

$Bearing_{Boltslotted} := ShearCheck(\phi bbRns, Vcolbent) = "OK"$

LRFD Article 6.13.2.10: Tensile Resistance

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.

$$Tu := \frac{Vcolbent \cdot 1}{distanchorhole} = 16.667 \quad \text{kips}$$

LRFD Eq. 6.13.2.10.2-1

$\phi tTn := \phi_t \cdot 0.76 A_b \cdot Fub = 140.212$ kips

$TensionCheck := ShearCheck(\phi tTn, Tu) = "OK"$

Article 6.13.2.11: Combined Tension and Shear

LRFD Eq. 6.13.2.11-1
LRFD Eq. 6.13.2.11-2

$Pu := Vcolbent$

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

$Tn_{combined} := CombinedProgram(Pu, A_b, Fub, \phi sRn, \phi_s) = 104.451$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 83.561$ kips

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

Summary

$Dia_b = 2.25$ in

$Shear_{Anchorbolts} = \text{"OK"}$

$Bearing_{Boltstandard} = \text{"OK"}$

$Bearing_{Boltslotted} = \text{"OK"}$

$TensionCheck = \text{"OK"}$

$CombinedCheck = \text{"OK"}$

$BlockShearCheck = \text{"OK"}$

$TensionCheck_{AISC} = \text{"OK"}$

$ShearAngleCheck = \text{"OK"}$

Appendix O: Little Bear Creek Bridge Moment-Interaction Diagrams

Bents 2 and 3

```
STRUCTUREPOINT - spColumn v4.81 (TM)                                                              Page   1
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=====================================================================================================
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                     Computer program for the Strength Design of Reinforced Concrete Sections
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```
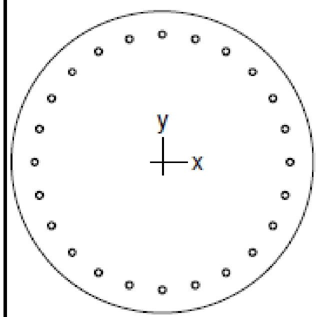

```
General Information:
====================
  File Name: untitled.col
  Project:
  Column:                                 Engineer:
  Code:     ACI 318-11                    Units: English

  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
  rx =  10.5 in                           ry =  10.5 in
  Xo =  0 in                              Yo =  0 in

Reinforcement:
==============
  Bar Set: 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: Spiral; #3 ties with #10 bars,  #5 with larger bars.
  phi(a) = 0.85,  phi(b) = 0.9,  phi(c) = 0.75

  Layout: Circular
  Pattern: All Sides Equal  (Cover to transverse reinforcement)
  Total steel area: As = 18.72 in^2 at rho = 1.35%
  Minimum clear spacing = 5.67 in

  12 #11   Cover = 6 in

Factored Loads and Moments with Corresponding Capacities:
=========================================================
              Pu          Mux        PhiMnx PhiMn/Mu NA depth Dt depth    eps_t    Phi
  No.        kip         k-ft          k-ft                in       in
  --- ------------ ------------ ------------ -------- -------- -------- -------- ------
    1      1294.00       810.00      1707.86    2.108    20.15    34.67  0.00216  0.755

  *** End of output ***
```



54 in diam.

Code: ACI 318-11

Units: English

Run axis: About X-axis

Run option: Investigation

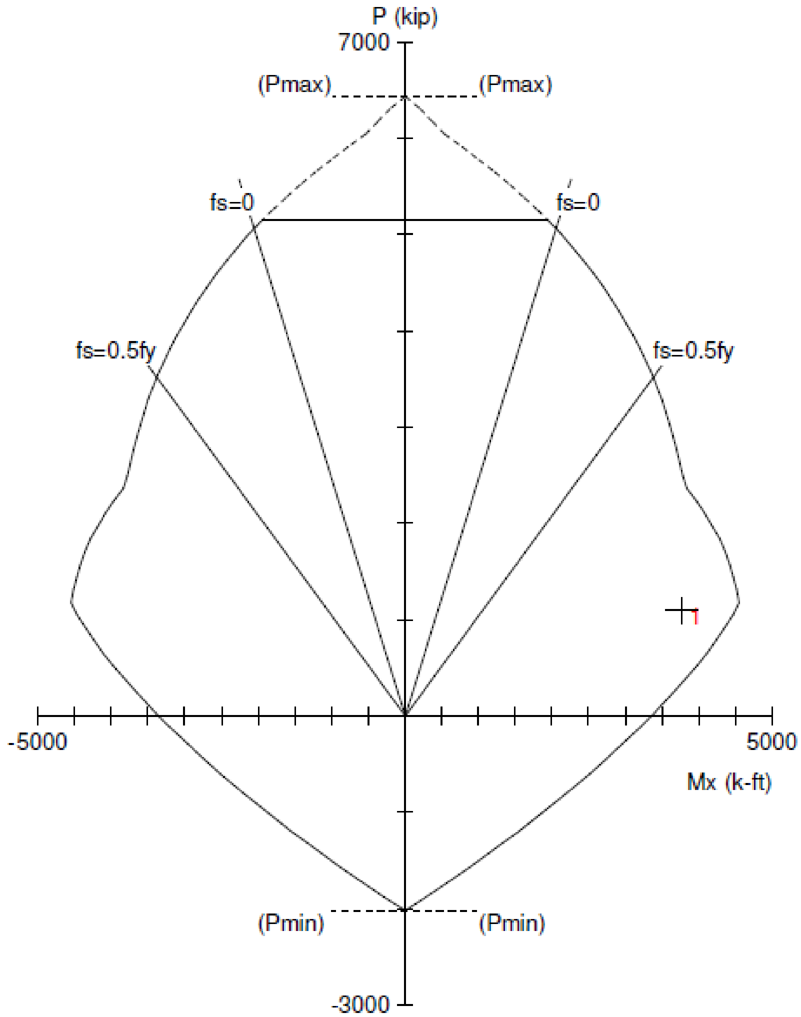
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 03/06/13

Time: 16:39:35



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File: C:\Users\jdl0003\Documents\Research\ALDOT Bridge Design Examples\SDC B\M...\Little Bear Creek Bents 2 and 3.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		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		Min clear spacing = 4.54 in	Clear cover = 3.50 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Appendix P: Scarham Creek Bridge SDC B

Designer: Jordan Law
Project Name: Scarham Bridge
Job Number: STPAA-0075 (502)
Date: 5/24/2012
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.

ORIGIN:= 1

Project Known Information

Coordinates: 34.294N, 86.164W

Soil Site Class: C

Superstructure Type: AASTHO BT-72 girders for all spans

Substructure Type: Circular columns supported on drilled shafts

Abutment Type: Abutment beam supported on drilled shafts

The designer should input any information that can be used to calculate the dead weight of the bridge, including but not limited to length of bridge, column height(s), deck thickness, bent volume(s), and guard rail volume(s). Also, information about foundations should also be included if the bridge is classified as SDC B.

Note: ***Input** all of the below information*.

$A_s := .14$

$fc := 400($ psi

$S_{D1} := .15$

$fye := 6000($ psi

$\rho_{conc} := 0.0868 \ \frac{lb}{in^3}$

$S_{DS} := .30$

INPUT

$g := 386.4$

SDC:= "B"

Length of Bridge (ft)	L := 520	ft
Skew of Bridge (degrees)	Skew := 0	degrees
Span Length (ft)	Span := 130	ft
Deck Thickness (in)	$t_{deck} := 7$	in
Deck Width (ft)	DeckWidth := 40	ft
Depth of Superstructure (ft)	$D_s := 6.583$	ft
Number of Bridge Girders	N := 6	
Girder X-Sectional Area (in^2)	GirderArea := 767	in^2

Bent Volume (ft^3) $BentVolume := 7.5 \cdot 5.5 \cdot 40 = 1.65 \times 10^3$

Guard Rail Area (in^2) $GuardRailArea := 31($ in^2

Bents 2 and 4 Column Diameter (in) $Columndia_{Bent24} := 6($ in

Bent 3 Column Diameter (in) $Columndia_{Bent3} := 72$ in

Number of Columns per Bent $N_{col} := 2$

The column height is measured from the bottom of the bent to the top of the pile footing. Other options include measuring from the top of the bent to the ground surface or to a change in diameter
(if possible). If the plastic hinge location (at the bottom of the column) is known, then the column height should be measured from the bottom of the bent to the known hinge point.

Tallest Above Ground Column Height Bent 2 (ft) $ColumnHeight_{Bent2} := 34.02$ ft

Talllest Above Ground Column Height Bent 3 (ft) $ColumnHeight_{Bent3} := 59.13$ ft

Tallest Above Ground Column Height Bent 4 (ft) $ColumnHeight_{Bent4} := 32.15$ ft

Tallest Above Ground Abutment Height (ft) $H_{abutment} :=$ ft

Length of Strut 2 & 4 (ft) $LStrut24 := 1($ ft

Length of Strut 3 (ft) $LStrut3 := 1($ ft

Strut 2 & 4 Depth (in) $Strut24Depth := 72$ in

Strut 2 & 4 Width (in) $Strut24Width := 42$ in

Strut 3 Depth (in) $Strut3Depth := 12($ in

Strut 3 Width (in) $Strut3Width := 42$ in

Bents 2 and 4 Column Area (in^2) $Acolumn_{Bent24} := \frac{Columndia_{Bent24}^2 \cdot \pi}{4} = 2.827 \times 10^3$ in^2

Bent 3 Column Area (in^2) $Acolumn_{Bent3} := \frac{Columndia_{Bent3}^2 \cdot \pi}{4} = 4.072 \times 10^3$ 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

Drill Shaft 2 Diameter (in) $DSdia2 := 6($ in

Drill Shaft 3 Diameter (in) $DSdia3 := 7($ in

Drill Shaft 4 (Abutment) Diameter (in) $DSdia4 := 6($ in

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

$BentVolume := BentVolume 12^3 = 2.851 \times 10^6$ in^3

$ColumnHeight_{Bent2} := ColumnHeight_{Bent2} \cdot 12 = 408.264$ in

$ColumnHeight_{Bent3} := ColumnHeight_{Bent3} \cdot 12 = 709.632$ in

$ColumnHeight_{Bent4} := ColumnHeight_{Bent4} \cdot 12 = 385.872$ in

$Strut1 := Strut1 \cdot 12 = 4.788 \times 10^3$ in^3

$Strut2 := Strut2 \cdot 12 = 7.56 \times 10^3$ in^3

Find Vertical Reactions at Each Bent:

Live Loads assumed to be present during an earthquake (see ***LRFD Article 3.4.1***)

$$Num\_Lanes := trunc\left(\frac{\frac{DeckWidth - 2 \cdot 1.375}{12}}{12}\right) = 3$$

Number of Lanes On Bridge (Design Lane Width of 10 ft) See ***LRFD 3.6.1.2.4***

$\gamma_{EQ} := 0.5$ ***LRFD Specificaiton C3.4.1 (Extreme Case I)*** **INPUT**

The γ_{EQ} value is to be determined on a project-specific basis. In the standard specification, a value of 0.0 was used, however, the LRFD Specification recommends a value of 0.5. See LRFD Article C3.4.1 under "EXTREME EVENT I"

$LL\_design := 0.64$ $\frac{klf}{lane}$ ***LRFD Specification 3.6.1.2.4***

$Q := LL\_design \cdot \gamma_{EQ} = 0.32$ $\frac{klf}{lane}$

$LL\_foot := Q \cdot Num\_Lanes = 0.96$ klf Live Load per linear foot of deck (includes all lanes)

Note: If the Vertical Reactions at each bent are already known, input them below, otherwise the sheet will calculate vertical reactions based on the given information above.

$DL_{Bent2} :=$ ▪ kip $LL_{Bent2} :=$ ▪ kip

$DL_{Bent3} :=$ ▪ kip $LL_{Bent3} :=$ ▪ kip **INPUT**

$DL_{Bent4} :=$ ▪ kip $LL_{Bent4} :=$ ▪ kip

$VR_{Bent2} := DL_{Bent2} + LL_{Bent2} =$ ▪ kip

$VR_{Bent3} := DL_{Bent3} + LL_{Bent3} =$ ▪ kip

$VR_{Bent4} := DL_{Bent4} + LL_{Bent4} = \blacksquare$ kip

The weight calculation takes into account the entire dead weight of the structure, including the deck, bents, abutments, columns, girders, and railings. Any other expected dead loads should also be included.

$$W := \frac{\rho_{conc} \cdot \left(\begin{array}{l} L \cdot t_{deck} \cdot DeckWidth + 2 \cdot BentVolume + 2 \cdot Acolumn_{Bent24} \cdot ColumnHeight_{Bent2} \cdots \\ + Acolumn_{Bent3} \cdot ColumnHeight_{Bent3} + Acolumn_{Bent24} \cdot ColumnHeight_{Bent4} \cdots \\ + L \cdot N \cdot GirderArea + 2 \cdot GuardRailArea \cdot L \end{array} \right)}{1000}$$

$W = 5689.793$ kips

Note: An elevation view of the bridge shows that the tributary area for Bents 2, 3, and 4 are identical, and therefore the tributary weights will be equal. The information below should be adjusted for different bridges.

$BentTribLength := \frac{Span}{12} = 130$ ft

$BentTribArea := \frac{Span}{L} = 0.25$ Percent of Area Tributary to Bent

$DL_{Bent} := BentTribArea \cdot W = 1422.448$ kip

$LL_{Bent} := BentTribLength \cdot LL\_foot = 124.8$ kip

$VR_{Bent} := DL_{Bent} + LL_{Bent} = 1547.248$ kip

$VR_{Bent2} := VR_{Bent}$ $VR_{Bent3} := VR_{Bent}$ $VR_{Bent4} := VR_{Bent}$

Steps for Seismic Design

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

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

Article 3.4: Determine Deisgn Response Spectrum

Article 3.5: Determine SDC

Guide Figure 1.3-2: Seismic Design Procedure Flowchart for SDC B

Displacement Demand Analysis (Fig 1.3-2):

Article 4.1: Seismic Design Proportioning

Article 4.2: Determine Analysis Procedure

Article 4.3.1: Determine Horizontal Ground Motion Effects Along Both Axis

Article 4.3.2/4.3.3: Damping and Short Period Considerations

Article 5.4/5.5: Select Analytical Procedure

Article 5.6: Effective Section Properties

Article 5.2: Abutment Modeling

Article 5.3: Foundation Modeling and Liquefaction (if present)

Article 5.1.2/4.4: Conduct Demand Analysis

Article 4.8: Determine Displacement Demands Along Member Local Axes

Displacement Capacity Check ($\Delta_C > \Delta_D$):

Article 4.12: Determine Minimum Support Length

Article 4.14: Shear Key

Guide Figure 1.3-5: Foundation and Detailing Flowcharts

Foundation Design (Fig 1.3-5):

Article 6.8: Liquefaction Consideration

Article 6.3: Spread Footing Design

Article 6.4: Pile Cap Foundation Design

Article 6.5: Drilled Shaft

Article 6.7: Abutment Design

Detailing:

Article 8.3: Determine Flexure and Shear Demands

Article 8.7: Satisfy Requirements for Ductile Member Design

Article 8.6: Shear Demand and Capacity Check for Ductile Elements

Article 8.8: Satisfy Lateral and Longitudinal Reinforcement Requirements

Articles 3.4 and 3.5 have already been determined from the "SDC Classification" sheet.
Make sure the four values (A_s, S_{DS}, S_{D1}, and SDC) have been input above.

Displacement Demand Analysis (Δ_D)

***Article 4.1*: Seismic Design Proportioning**

See Guide Specification

***Article 4.2*: Determine Analysis Procedure**

This is a function of the SDC and the regularity of the bridge.
For a regular bridge in SDC B, Procedure 1 or 2 can be used.
For a non-regular bridge in SDC B, Procedure 2 must be used.

Guide Table 4.2-1

A regular bridge is defined as a bridge having fewer than 7 spans, no abrupt or unusual change in geometry and that saisfy the requirements below (***Guide Table 4.2-3***)

Table 4.2-3: Regular Bridge Requirements

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle (curved bridge)	30°	30°	30°	30°	30°
Maximum span length ratio from span-to-span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span-to-span (excluding abutments)	-	4	4	3	2

***Article 4.3.1*: Determine Horizontal Ground Motion Effects Along Both Axis**

Seismic displacement demands shall be determined independently in two orthogonal directions, typically the longitudinal and transverse axes of the bridge

Article 4.3.3: Displacement Magnification for Short-Period Structures

$u_d := 2$ for SDC B

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

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)**

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.

$p_o := 1.($ $\frac{kip}{in}$

$\upsilon_{smaxLong} := 0.38207$ in

$\upsilon_{smaxTran} := 4.33004$ in

INPUT

$$K_{Long} := \frac{p_o \cdot L}{\upsilon_{smaxLong}} = 1.633 \times 10^4 \quad \frac{kip}{in}$$ ***Guide Eq. C5.4.2-1***

$$K_{Tran} := \frac{p_o \cdot L}{\upsilon_{smaxTran}} = 1.441 \times 10^3 \quad \frac{kip}{in}$$ ***Guide Eq. C5.4.2-2***

The weight of the structure has already been calculated above

Step 4: Calculate the period, T_m.

$$T_{mLong} := 2\pi \cdot \sqrt{\frac{W}{K_{Long} \cdot g}} = 0.189 \quad s$$ ***Guide Eq. C5.4.2-3***

Step 5: Calculate equivalent static earthquake loading p_e.

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

$$Sa_{Long} := acc(S_{DS}, S_{D1}, T_{mLong}, A_s) = 0.3$$

$$p_{eLong} := \frac{Sa_{Long} \cdot W}{L} = 0.274 \qquad \frac{kip}{in}$$ ***Guide Eq. C5.4.2-4***

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.

$$Rd_{Long} := Rdprogram(T_{mLong}, S_{DS}, S_{D1}, u_d) = 2.156$$

$$v_{smaxLong} := Rd_{Long} \cdot \frac{p_{eLong}}{p_o} \cdot \upsilon_{smaxLong} = 0.225 \qquad in$$

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

Step 4: Calculate the period, T_m.

$$T_{mTran} := 2\pi \cdot \sqrt{\frac{W}{K_{Tran} \cdot g}} = 0.635 \qquad s$$ ***Guide Eq. C5.4.2-3***

Step 5: Calculate equivalent static earthquake loading p_e.

$$Sa_{Tran} := acc(S_{DS}, S_{D1}, T_{mTran}, A_s) = 0.236$$

$$p_{eTran} := \frac{Sa_{Tran} \cdot W}{L} = 0.215 \qquad \frac{kip}{in}$$ ***Guide Eq. C5.4.2-4***

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.

$$Rd_{Tran} := Rdprogram(T_{mTran}, S_{DS}, S_{D1}, u_d) = 1$$

$$v_{smaxTran} := Rd_{Tran} \cdot \frac{p_{eTran}}{p_o} \cdot \upsilon_{smaxTran} = 0.932 \qquad in$$

LRFD Article 4.7.4.3.2: Single-Mode Spectral Method

<u>Single Mode Spectrum Analysis</u>

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

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.

$$v_{stran}(x) := -3 \cdot 10^{-7} \cdot x^2 + 0.0016x + 1.409$$

INPUT

$$v_{slong}(x) := -1 \cdot 10^{-8} \cdot x^2 + 0.0001x + 0.156$$

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

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

$$\gamma_{Tran} := \int_0^L \frac{W}{L} \cdot v_{stran}(x)^2\,dx = 4.146\times 10^4 \qquad \gamma_{Long} := \int_0^L \frac{W}{L} \cdot v_{slong}(x)^2\,dx$$ ***LRFD C4.7.4.3.2b-3***

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

$$T_{mTran1} := 2\pi \cdot \sqrt{\frac{\gamma_{Tran}}{p_o \cdot g \cdot \alpha_{Tran}}} = 0.52 \quad s$$ ***LRFD Eq. 4.7.4.3.2b-4***

$$T_{mLong1} := 2\pi \cdot \sqrt{\frac{\gamma_{Long}}{p_o \cdot g \cdot \alpha_{Long}}} = 0.182 \quad s$$ ***LRFD Eq. 4.7.4.3.2b-4***

Step 5: Calculate the equivalent Static Earthquake Loading

$C_{smLong} := acc(S_{DS}, S_{D1}, T_{mLong1}, A_s) = 0.3$

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.

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

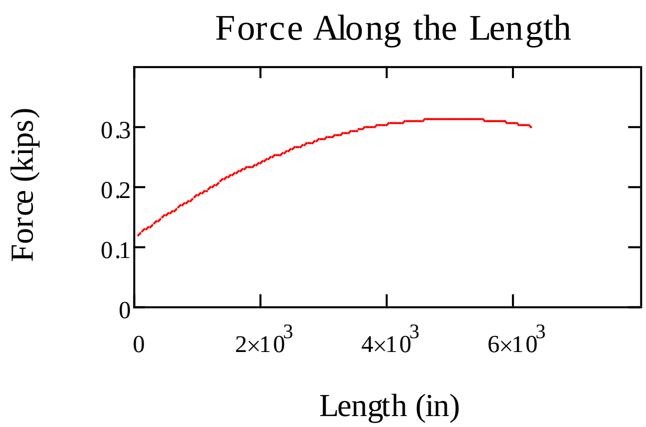
$PeLong(x) \rightarrow 0.000077153933263394182453 \cdot x + -7.715393326339418245\text{3e-9} \cdot x^2 + 0.1205915976906851\text{07}$

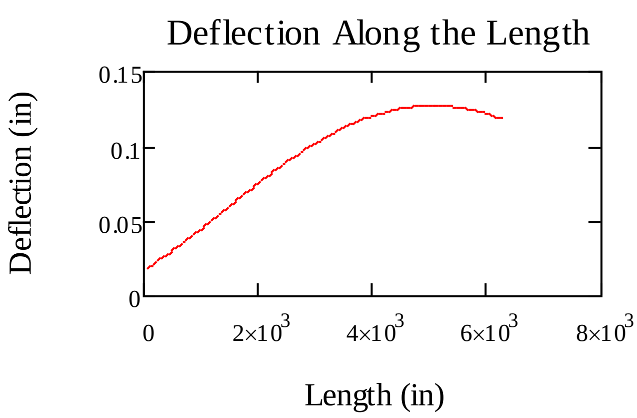
$dW := \frac{L}{100}$

$i := 1..101$

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

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





Maximum Deflection:

$max(\Delta long) = 0.127$ in

NOTE: Repeat Steps 5 and 6 for Transverse Direction.

Step 5: Calculate the equivalent Static Earthquake Loading

$$C_{smTran} := acc\left(S_{DS}, S_{D1}, T_{mTran1}, A_s\right) = 0.288$$

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.

$$PeTran(x) := \frac{\beta_{Tran} \cdot C_{smTran}}{\gamma_{Tran}} \cdot \frac{W}{L} \cdot v_{stran}(x)$$

LRFD Eq. C4.7.4.3.2b-5

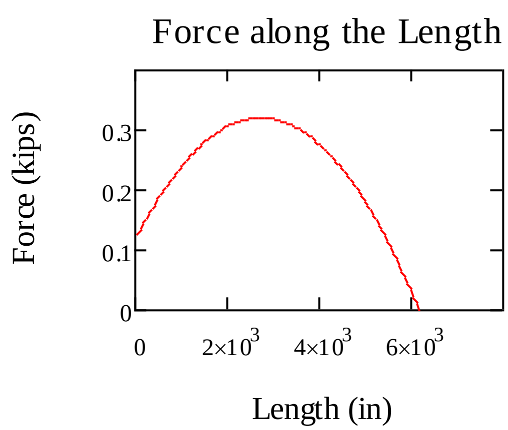
$$PeTran(x) \rightarrow 0.00014476176919215373563 \cdot x + -2.7142831723528825431e-8 \cdot x^2 + 0.127507975826563912$$

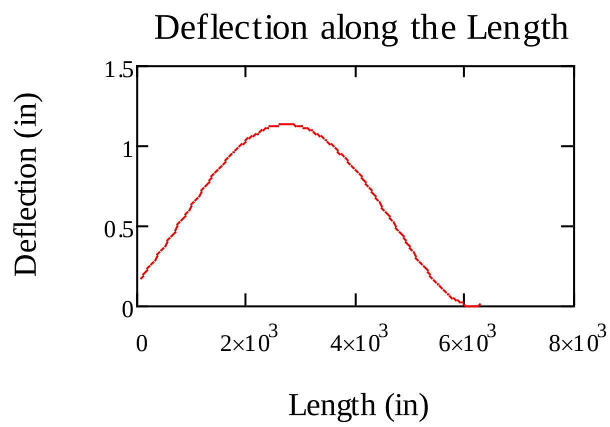
$$dL := \frac{L}{100}$$

$$i := 1..101$$

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

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





Maximum Deflection:

$\max(\Delta tran) = 1.135$ in

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

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**

This is taken care of in the SAP model.

***Article 5.3*: Foundations Modeling**

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.

Special provisions need to be considered if Liquefaction is present. ***Guide Article 6.8***

***Article 4.4*: Combination of Orthogonal Seismic Displacement Demands**

$$LoadCase1 := \sqrt{\left(1 \cdot v_{smaxLong}\right)^2 + \left(0.3 v_{smaxTran}\right)^2} = 0.359 \quad in$$

$$LoadCase2 := \sqrt{\left(1 \cdot v_{smaxTran}\right)^2 + \left(0.3 v_{smaxLong}\right)^2} = 0.935 \quad in$$

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 $\Delta_D < \Delta_C$

Guide Article 4.8

Since the bridge has frame bents, the simplified equations cannot be used; therefore a pushover analysis must be done.

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.

Pushover Analysis Results (if necessary):

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

INPUT

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

Abutment Support Length Requirement ***Guide Eq. 4.12.2-1***

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

Bent Support Length Requirement ***Guide Eq. 4.12.2-1***

BENT 2

$L := BentTribLength = 130 \qquad S_{D1} := 0.3($

$$H := \frac{ColumnHeight_{Bent2}}{12} = 34.022$$

Standard Specifications

$$N_{2Stan} := (8 + 0.02 L + 0.08 H)\cdot\left(1 + 0.000125 Skew^{2}\right) = 13.322 \quad in$$

ATC-49 Equation (New Design)

$$N_2 := \left[4 + .02 L + .08 H + 1.09\sqrt{H}\cdot\sqrt{1 + \left(2\cdot\frac{3}{8}\right)^{2}}\right]\cdot\left(\frac{1 + 1.25 S_{D1}}{\cos\left(\frac{Skew\cdot\pi}{180}\right)}\right) = 23.745 \quad in$$

BENT 3

$L := BentTribLength = 130$ $S_{D1} := 0.30$

$$H := \frac{ColumnHeight_{Bent3}}{12} = 59.136$$

Standard Specifications

$$N_{3Stan} := (8 + 0.02 L + 0.08 H) \cdot \left(1 + 0.000125 Skew^2\right) = 15.331 \quad in$$

ATC-49 Equation (New Design)

$$N_3 := \left[4 + .02 L + .08 H + 1.09 \sqrt{H} \cdot \sqrt{1 + \left(2 \cdot \frac{3}{8}\right)^2}\right] \cdot \left(\frac{1 + 1.25 S_{D1}}{\cos\left(\frac{Skew \cdot \pi}{180}\right)}\right) = 29.987 \quad in$$

BENT 4

$L := BentTribLength = 130$ $S_{D1} := 0.30$

$$H := \frac{ColumnHeight_{Bent4}}{12} = 32.156$$

Standard Specifications

$$N_{4Stan} := (8 + 0.02 L + 0.08 H) \cdot \left(1 + 0.000125 Skew^2\right) = 13.172 \quad in$$

ATC-49 Equation (New Design)

$$N_4 := \left[4 + .02 L + .08 H + 1.09 \sqrt{H} \cdot \sqrt{1 + \left(2 \cdot \frac{3}{8}\right)^2}\right] \cdot \left(\frac{1 + 1.25 S_{D1}}{\cos\left(\frac{Skew \cdot \pi}{180}\right)}\right) = 23.236 \quad in$$

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

$V_{ok} := 1.5 V_n$ *This does not apply to this bridge*

BENT 2 DESIGN

Guide Article 4.11.2: For SDC B, it is acceptable to use the moment capacity based on expected material strengths when the concrete reaches an extreme compressive fiber strain of 0.003.

Force Inputs

$M_{neBent2} := 5765$ kip – ft Nominal moment from PCA Column

$V_{pelastic} := 1360$ kip Elastic shear from SAP2000 model **INPUT**

$P_u := 128400$ lb Axial load from earthquake and dead load combination

Reinforcement Details

$Ag := Acolumn_{Bent24}$

$Ae := 0.8Ag = 2262$ in^2 ***Guide Eq. 8.6.2-2***

Guide Article 8.6.2

$\mu_D := 2$

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

StirrupSize:= "#6" Tiesize: Bar size used for ties

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

$sNOhinge := 12$ in sNOhinge: Spacing of hoops or pitch outside PHL

$Asp := .44$ in^2 Asp: Area of hoop reinforcement in direction of loading (in^2)

$Dsp := 0.75$ in Dsp: Diameter of spiral or hoop reinforcing (in) **INPUT**

$Cover := 3$ in Cover: Concrete cover for the Column (in)

$b := Columndia_{Bent24}$ in b: Diameter of column (in)

$d := b - Cover = 57$ in d: Effective depth of section in direction of loading (in)

$Dprime := b - 2 \cdot Cover$ in Dprime: Diameter (in column) of hoop reinforcing (in)

$NumberBars := 24$ Total number of longitudinal bars in column cross-section

$A_{bl} := 1.56$ in^2 Abl: Area of longitudinal bar

$d_{bl} := 1.41$ in dbl: Diameter of longitudinal bar

$bv := Columndia_{Bent24}$ bv: Diameter of column

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

The design forces shall be the lesser of the forces resulting from the overstrength plastic hinging moment capacity or unreduced elastic seismic forces in columns or pier walls.

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

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.

$\lambda_{mo} := 1.4$ for ASTM A 615 Grade 60 reinforcement **_Guide Article 8.5_**

$M_{pBent2} := \lambda_{mo} \cdot M_{neBent2} \cdot 1000 \cdot 12 = 9.685 \times 10^{7}$ lb − in

$Fixity := ColumnHeight_{Bent2} = 408.264$ in

$V_p := \dfrac{2 \cdot M_{pBent2}}{Fixity \cdot 1000} = 474.458$ kips

$V_{pelastic} := V_{pelastic} \cdot \rho_{eTran} = 292.872$ kips

***Article 8.7*: Requirements for Ductile Member Design**

Each column must satisfy the minimum lateral flexural capacity

$\Lambda := 2$ Fixed and top and bottom **_Guide Article 4.8.1_**

$M_{neminBent} := 0.1 \cdot DL_{Bent} \cdot \left(\dfrac{\dfrac{Fixity}{12} + 0.5 D_s}{\Lambda} \right) = 2653.826$ kip ft **_Guide Eq 8.7.1-1_**

$$CheckMoment(M_{ne}, M_e) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } M_{ne} \le M_e \\ a \leftarrow \text{"FAILURE"} \quad \text{otherwise} \end{array} \right.$$

$CheckMoment(M_{neminBent}, M_{neBent2}) = \text{"OK"}$

If the moment check comes back "FAILURE" then the column size can be increased or the reinforcement can be increased.

Article 8.6: Shear Demand and Capacity for Ductile Concrete Members

It is recommended to use the plastic hinging forces whenever practical, but in this case the elastic forces will be used.

$V_u := \min(V_p, V_{pelastic}) = 292.872$ kips $\phi_s := 0.9$

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

$\rho_s := \dfrac{4 \cdot Asp}{s \cdot Dprime} = 0.0054$ **_Guide Eq. 8.6.2-7_**

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

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

$fs := \text{StressCheck}(\rho_s, fyh) = 0.326$ ***Guide Eq. 8.6.2-6***

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

Guide Eq. 8.6.2-5

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

If Pu is Compressive:

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

Guide Eq. 8.6.2-3

If Pu is NOT Compressive:

Guide Eq. 8.6.2-4

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

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

$Vc := vc \cdot Ae = 497.628$ kips

Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity** ***Guide Eq 8.6.3-2 and 8.6.4-1

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

$Vs := vsprogram(n, Asp, fyh, Dprime, s, fc, Ae) = 746.442$ kips

$\phi Vn := \phi_s \cdot (Vs + Vc) = 1.12 \times 10^3$ kips

Guide Eq. 8.6.1-2

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

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

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:

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

Guide Eq. 8.6.5-1

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

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**

$A_{long} := NumberBars \cdot A_{bl} = 37.44$ in^2

$$\rho program(A_{long}, Ag) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } A_{long} \leq 0.04 Ag \\ a \leftarrow \text{"Section Over Reinforced"} \quad \text{if } A_{long} > 0.04 Ag \\ a \end{array} \right.$$

Guide Eq. 8.8.1-1

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

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**

$$\text{minAlprogram}(A_l, Ag) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \quad \text{if } A_{long} \geq 0.007Ag \\ a \leftarrow \text{"Increase Longitudinal Reinforcing"} \quad \text{if } A_{long} < 0.007Ag \\ a \end{array} \right.$$

Guide Eq. 8.8.2-1

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

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

***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.

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

Guide Eq. 4.11.6-1

$L_p := \text{PlasticHinge}(\text{Fixity}, \text{fye}, d_{bl}) = 45.351$ in

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

'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.

$Mp75 := 0.75 M_{pBent2} = 7.264 \times 10^{7}$ lb·in

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

$L_{p1} := \text{PlasticHingeRegion}(L_p, \text{Columndia}_{Bent24}) = 90$ in

The Guide Specifications allows for the use of the plastic hinge length from the LRFD Specification in SDC A and B (***Guide Article C8.8.9***).

$$\text{LRFDPlasticHingeLength}(\text{ColumnDia}, \text{ColumnHeight}) := \left| \begin{array}{l} a \leftarrow \text{ColumnDia} \\ b \leftarrow \frac{1}{6} \cdot \text{ColumnHeigh} \\ c \leftarrow 18 \\ PHL \leftarrow \max(a, b, c) \\ PHL \end{array} \right.$$

$L_{p2} := \text{LRFDPlasticHingeLength}(\text{Columndia}_{Bent24}, \text{Fixity}) = 68.044$ in

Guide Article C8.8.9

The plastic hinge length will be the smaller of the two values, as the Guide Specification allows:

$PHL := \min(L_{p1}, L_{p2}) = 68.044$ in

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region: ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

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

$$\text{MaximumSpacing} := \text{Spacingprogram}(\text{Columndia}_{Bent24}, d_{bl}) = 6 \quad \text{in}$$

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

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

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

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

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.

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

$$\text{Extension} := \text{ExtensionProgram}(\text{Columndia}_{Bent24}) = 30 \quad \text{in}$$

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

LRFD 5.8.3.3 **Nominal Shear Resistance**

$V_u = 292.872$ kips

$\phi_s = 0.9$

$\beta := 2.0$ ***LRFD Article 5.8.3.4.1***

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

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 55.545 \quad \text{in}$$

$$de := d = 57 \quad \text{in}$$ ***LRFD Eq. 5.8.2.9-1***

$$dv := 0.9de = 51.3 \quad \text{in}$$

LRFD Eq. 5.8.3.3-3

$$V_c := 0.0316\beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 389.059 \quad \text{kips}$$

LRFD Eq. 5.8.3.3-4

$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{sNOhinge} = 225.72 \quad \text{kips}$$

$$V_n := \phi_s \cdot .25 fc \cdot bv \cdot dv = 2.77 \times 10^6 \quad \text{kips}$$

$$\phi Vn := \min\left[V_n, \left(V_c + V_s\right) \cdot \phi_s\right] = 553.301 \quad \text{kips}$$

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

$$Shearcheck2 := ShearCheck\left(\phi Vn, V_u\right) = \text{"OK"}$$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

LRFD Eq. 5.8.2.5-1

$$Avmin := 0.0316 \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot sNOhinge}{\frac{fye}{1000}} = 0.758 \quad \text{in}^2$$

$$Av := 2 \cdot Asp = 0.88 \quad \text{in}^2$$

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

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

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.

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

$$vu := \frac{V_u}{\phi_s \cdot bv \cdot dv} = 0.106 \quad \text{ksi}$$

LRFD Eq. 5.8.2.9-1

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

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

$$\text{MaxSpacing} := \text{floor}(\text{spacingProgram}(vu, dv, fc)) = 24 \quad \text{in}$$

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$\text{Spacecheck}(\text{MaxSpacing}, s, V_u, V_c) := \left| \begin{array}{l} a \leftarrow \min(\text{MaxSpacing}, 12) \;\; \text{if } V_u \le 0.5 0.9 V_c \\ a \leftarrow \min(s, \text{MaxSpacing}, 12) \;\; \text{otherwise} \\ a \end{array} \right.$$

$$\text{sNOhinge} := \text{Spacecheck}(\text{MaxSpacing}, \text{sNOhinge}, V_u, V_c) = 12 \quad \text{in}$$

Design Summary - Bent 2

StirrupSize = "#6"

$s = 6$ in

$\text{sNOhinge} = 12$ in

$\text{PHL} = 68.044$ in

$\text{Extension} = 30$ in

$N_2 = 23.745$ in

Design Check Summary - Bent 2

Shearcheck = "OK"	Shear capacity > Vn
Transversecheck = "OK"	Minimum shear reinforcement ratio
ReinforcementRatioCheck = "OK"	Maximum longitudinal reinforcement ratio
MinimumA_l = "OK"	Minimum longitudinal reinforcement ratio
scheck = "OK"	Max spacing of transverse reinforcement
Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran = "OK"	Minimum shear reinforcement outside hinge zone

Bent 2 Strut Design

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.

Force Inputs

$V_{pStrut2} := 901 \cdot p_{eTran} = 194.028$ kips

$M_{pStrut2} := 11920 \cdot p_{eTran} = 2566.941$ kip·ft

$P_{uStrut2} := 1529 \cdot p_{eTran} = 329.266$ kips

Reinforcement Details

$Ag := Strut24Depth \cdot Strut24Width$

$Ae := 0.8 Ag = 2.419 \times 10^3$ in^2 ***Guide Eq. 8.6.2-2***

$\mu_D := 2$ ***Guide Article 8.6.2***

StirrupSize := "#5"

s := 4 in — s: Spacing of hoops or pitch of spiral (in)

sNOhinge := 14 in — sNOhinge: Spacing of transverser reinforcement outside PHL

Asp := 0.31 in^2 — Asp: Area of spiral or hoop reinforcing (in^2)

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

Cover := 2 in — Cover: Concrete cover for the Column (in)

$b := Strut24Depth = 72$ in b: Depth of the Strut (in)

$d := b - Cover = 70$ in d: Effective Depth (in)

$bv := Strut24Widtl$ bv: Effective width of strut (in)

$NumberBars_1 := 1($ Total number of longitudinal bars along strut width (top and bottom)

$NumberBars_2 := 2($ Total number of longitudinal bars along strut depth (side)

$A_{bl1} := 1.5($ in^2 Abl1: Area of longitudinal bar 1

$A_{bl2} := .3$ in^2 Abl2: Area of longitudinal bar 2

$d_{bl1} := 1.4$ in dbl1: Diameter of longitudinal bar 1

$d_{bl2} := .62$ in dbl2: Diameter of longitudinal bar 2

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

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

$$\rho_w := \frac{Av}{s \cdot b} = 0.0022$$ ***Guide Eq. 8.6.2-10***

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

$$StressCheckRect(\rho_w, fyh) := \left| \begin{array}{l} fs \leftarrow 2 \cdot \rho_w \cdot fyh \\ a \leftarrow fs \quad \text{if } fs \le 0.35 \end{array} \right.$$ ***Guide Eq. 8.6.2-9***

$$fw := StressCheckRect(\rho_w, fyh) = 0.258$$

$$\alpha Prime := \alpha program(fw, \mu_D) = 3$$ ***Guide Eq. 8.6.2-8***

If Pu is not compressive then vc=0 ***Guide Eq. 8.6.2-4***

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.

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

$$Vc := vc \cdot Ag = 665.28 \quad kips$$

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

$$\text{vsprogramRect}(Av, fyh, d, s, fc, Ae) := \left| \begin{array}{l} vs \leftarrow \dfrac{Av \cdot fyh \cdot d}{s} \\ maxvs \leftarrow 0.25\sqrt{\dfrac{fc}{1000}} \cdot Ae \\ a \leftarrow vs \ \text{ if } \ vs \le maxvs \\ a \leftarrow maxvs \ \text{ if } \ vs > maxvs \\ a \end{array} \right.$$

Guide Eq. 8.6.3-2 and 8.6.4-1

$Vs := \text{vsprogramRect}(Av, fyh, d, s, fc, Ae) = 651$ kips

Guide Eq. 8.6.1-2

$\phi Vn := \phi_s \cdot (Vs + Vc) = 1.185 \times 10^3$ kips

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

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

Guide Eq. 8.6.5-2

$$\text{mintranprogramRect}(\rho_s) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \ \text{ if } \ \rho_s \ge 0.002 \\ a \leftarrow \text{"Increase Shear Reinforcing Ratio"} \ \text{ if } \ \rho_s < 0.002 \\ a \end{array} \right.$$

$\text{Transversecheck} := \text{mintranprogramRect}(\rho_w) = \text{"OK"}$

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**

$A_{long} := A_{bl1} \cdot \text{NumberBars}_1 + A_{bl2} \cdot \text{NumberBars}_2 = 31.16$ in^2

Guide Eq. 8.8.1-1

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

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: ***Guide Eq. 8.8.2-1***

$\text{MinimumA}_l := \text{minAlprogram}\left(A_{long}, Ag\right) = \text{"OK"}$

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

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

'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.

The Guide Specifications allows for the use of the plastic hinge length from the LRFD Specification in SDC A and B (***Guide Article C8.8.9***).

$$\text{LRFDPlasticHingeLength}(\text{ColumnDia}, \text{ColumnHeight}) := \left| \begin{array}{l} a \leftarrow \text{ColumnDia} \\ b \leftarrow \frac{1}{6} \cdot \text{ColumnHeigh} \\ c \leftarrow 18 \\ PHL \leftarrow \max(a, b, c) \\ PHL \end{array} \right.$$

$L_{p2} := LRFDPlasticHingeLength(max(Strut24Depth, Strut24Width), LStrut24) = 72$ in

Guide Article C8.8.9

The plastic hinge length will be the smaller of the two values, as the Guide Specification allows:

$PHL := L_{p2} = 72$ in

$Extension := ExtensionProgram(max(Strut24Depth, Strut24Width)) = 36$

Guide Article 8.8.9

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region

Shall Not Exceed the Smallest of:

$MaximumSpacing := Spacingprogram\left(max(Strut24Depth, Strut24Width), d_{bl}\right) = 6$ in

$FINALSPACING := SpacingCheck(MaximumSpacings) = 4$ in

$scheck := ShearCheck(MaximumSpacings) = "OK"$

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****.*

LRFD 5.8.3.3 **Nominal Shear Resistance**

$\phi_s = 0.9$

$\beta := 2.0$

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

LRFD Article 5.8.3.4.1

$de := 69.4$ 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

$$\text{dvprogram}(de, dv, h) := \left| \begin{array}{l} x \leftarrow 0.9\,de \\ y \leftarrow 0.75\,h \\ z \leftarrow \max(x, y) \\ a \leftarrow dv \;\; \text{if } dv \geq z \\ a \leftarrow z \;\; \text{if } dv < z \\ a \end{array} \right.$$

LRFD Eq. 5.8.2.9-2

$$dv := \text{dvprogram}(de, \text{dvpreliminary}, \text{Strut24Depth}) = 66.75 \quad \text{in}$$

$$V_c := 0.0316\beta \cdot \sqrt{\frac{fc}{1000}}\, bv \cdot dv = 354.362 \quad \text{kips}$$

LRFD Eq. 5.8.3.3-3

$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{\text{sNOhinge}} = 177.364 \quad \text{kips}$$

LRFD Eq. 5.8.3.3-4

$$\phi V_n := \left(V_c + V_s\right) \cdot \phi_s = 478.554 \quad \text{kips}$$

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

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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

$$Avmin := 0.0316\sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot \text{sNOhinge}}{\frac{fye}{1000}} = 0.619 \quad \text{in}^2$$

LRFD Eq. 5.8.2.5-1

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

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

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.

LRFD 5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

$vu := \frac{V_p}{\phi_s \cdot bv \cdot dv} = 0.188$ ksi *LRFD Eq. 5.8.2.9-1*

$MaxSpacing := spacingProgram(vu, dv, fc) = 24$ in *LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2*

$$Spacecheck(MaxSpacing, s, V_u, V_c) := \begin{array}{|l} a \leftarrow min(MaxSpacing, 12) \ \text{ if } V_u \le 0.50.9 \\ a \leftarrow min(s, MaxSpacing, 12) \ \text{ otherwise} \\ a \end{array}$$

$sNOhinge := Spacecheck(MaxSpacing, sNOhinge, V_u, V_c) = 12$ in

Design Summary - Strut 2

StirrupSize = "#5"

s = 4 in

sNOhinge = 12 in

PHL = 72 in

Extension = 36 in

Design Check Summary - Strut 2

Shearcheck = "OK"	Shear capacity > Vn
Transversecheck = "OK"	Minimum shear reinforcement ratio
ReinforcementRatioCheck = "OK"	Maximum longitudinal reinforcement ratio
$MinimumA_l$ = "OK"	Minimum longitudinal reinforcement ratio
scheck = "OK"	Max spacing of transverse reinforcement
Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran = "OK"	Minimum shear reinforcement outside hinge zone

BENT 3 DESIGN

Guide Article 4.11.2: For SDC B, it is acceptable to use the moment capacity based on expected material strengths when the concrete reaches an extreme compressive fiber strain of 0.003.

Force Inputs

$M_{neBent3} := 991($ kip – ft Nominal moment from PCA Column **INPUT**

$V_{pelastic} := 136($ kip Elastic shear from SAP2000 model

$P_u := 128400$ lb Axial load from earthquake and dead load combination

Reinforcement Details

$Ag := Acolumn_{Bent3}$

$Ae := 0.8Ag = 3257$ in^2 ***Guide Eq. 8.6.2-2***

$\mu_D := 2$ ***Guide Article 8.6.2***

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

$StirrupSize := "\#6"$ Tiesize: Bar size used for ties

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

$sNOhinge := 10$ in sNOhinge: Spacing of hoops or pitch outside PHL **INPUT**

$Asp := 0.44$ in^2 Asp: Area of hoop reinforcement in direction of loading (in^2)

$Dsp := 0.75$ in Dsp: Diameter of spiral or hoop reinforcing (in)

$Cover := 3$ in Cover: Concrete cover for the Column (in)

$b := Columndia_{Bent3}$ in b: Diameter of column (in)

$d := b - Cover = 69$ in d: Effective depth of section in direction of loading (in)

$Dprime := b - 2 \cdot Cover$ in Dprime: Diameter (in column) of hoop reinforcing (in)

$NumberBars := 32$ Total number of longitudinal bars in column cross-section

$A_{bl} := 1.56$ in^2 Abl: Area of longitudinal bar

$d_{bl} := 1.41$ in dbl: Diameter of longitudinal bar

$bv := Columndia_{Bent3}$ bv: Diameter of column

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

The design forces shall be the lesser of the forces resulting from the overstrength plastic hinging moment capacity or unreduced elastic seismic forces in columns or pier walls.

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

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.

$\lambda_{mo} := 1.4$ for ASTM A 615 Grade 60 reinforcement **_Guide Article 8.5_**

$M_{pBent3} := \lambda_{mo} \cdot M_{neBent3} \cdot 1000 \, 12 = 1.665 \times 10^{8}$ lb − in

$Fixity := ColumnHeight_{Bent3} = 709.632$ in

$V_p := \dfrac{2 \cdot M_{pBent3}}{Fixity \, 1000} = 469.223$ kips

$V_{pelastic} := V_{pelastic} \cdot p_{eTran} = 292.872$ kips

Article 8.7: Requirements for Ductile Member Design

Each column must satisfy the minimum lateral flexural capacity

$\Lambda := 2$ Fixed and top and bottom **_Guide Article 4.8.1_**

$M_{neminBent} := 0.1 \cdot DL_{Bent} \cdot \left(\dfrac{\dfrac{Fixity}{12} + 0.5 D_s}{\Lambda} \right) = 4439.994$ kip ft **_Guide Eq 8.7.1-1_**

$$CheckMoment(M_{ne}, M_e) := \begin{vmatrix} a \leftarrow \text{"OK"} & \text{if } M_{ne} \le M_e \\ a \leftarrow \text{"FAILURE"} & \text{otherwise} \end{vmatrix}$$

$CheckMoment(M_{neminBent}, M_{neBent2}) = \text{"OK"}$

If the moment check comes back "FAILURE" then the column size can be increased or the reinforcement can be increased.

Article 8.6: Shear Demand and Capacity for Ductile Concrete Members

It is recommended to use the plastic hinging forces whenever practical, but in this case the elastic forces will be used.

$V_u := min(V_p, V_{pelastic}) = 292.872$ kips $\phi_s := 0.9$

Article 8.6.2: Concrete Shear Capacity

$\rho_s := \dfrac{4 \cdot Asp}{s \cdot Dprime} = 0.0044$ **_Guide Eq. 8.6.2-7_**

$fyh := \dfrac{fye}{1000} = 60$

ksi

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

$$fs := \text{StressCheck}(\rho_s, fyh) = 0.267$$

Guide Eq. 8.6.2-6

Guide Eq. 8.6.2-5

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

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

If Pu is Compressive:

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

Guide Eq. 8.6.2-3

If Pu is NOT Compressive:

Guide Eq. 8.6.2-4

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

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

$$Vc := vc \cdot Ae = 716.585 \qquad \text{kips}$$

Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity** ***Guide Eq 8.6.3-2 and 8.6.4-1

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

$$Vs := vsprogram(n, Asp, fyh, Dprime, s, fc, Ae) = 912.319 \qquad \text{kips}$$

$$\phi Vn := \phi_s \cdot (Vs + Vc) = 1.466 \times 10^3 \qquad \text{kips}$$

Guide Eq. 8.6.1-2

$$ShearCheck(\phi Vn, V_u) := \begin{vmatrix} a \leftarrow \text{"OK"} \quad \text{if } \phi Vn \ge V_u \\ a \leftarrow \text{"FAILURE"} \quad \text{if } \phi Vn < V_u \\ a \end{vmatrix}$$

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

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:

$$mintranprogram(\rho_s) := \begin{vmatrix} a \leftarrow \text{"OK"} \quad \text{if } \rho_s \ge 0.003 \\ a \leftarrow \text{"Increase Shear Reinforcing Ratio"} \quad \text{if } \rho_s < 0.003 \\ a \end{vmatrix}$$

Guide Eq. 8.6.5-1

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

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**

$$A_{long} := NumberBars \cdot A_{bl} = 49.92 \qquad \text{in}^2$$

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

Guide Eq. 8.8.1-1

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

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**

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

Guide Eq. 8.8.2-1

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

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

***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.

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

Guide Eq. 4.11.6-1

$$L_p := \text{PlasticHinge}(\text{Fixity}, fye, d_{bl}) = 69.461 \quad in$$

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

'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.

$$Mp75 := 0.75\,M_{pBent2} = 7.264\times 10^{7} \quad lb\cdot in$$

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

$$L_{p1} := \text{PlasticHingeRegion}(L_p, \text{Columndia}_{Bent3}) = 108 \quad in$$

The Guide Specifications allows for the use of the plastic hinge length from the LRFD Specification in SDC A and B (***Guide Article C8.8.9***).

$$\text{LRFDPlasticHingeLength}(\text{ColumnDia}, \text{ColumnHeight}) := \left| \begin{array}{l} a \leftarrow \text{ColumnDia} \\ b \leftarrow \dfrac{1}{6}\cdot \text{ColumnHeight} \\ c \leftarrow 18 \\ PHL \leftarrow \max(a, b, c) \\ PHL \end{array} \right.$$

$L_{p2} := LRFDPlasticHingeLength(Columndia_{Bent3}, Fixity) = 118.272$ in

Guide Article C8.8.9

The plastic hinge length will be the smaller of the two values, as the Guide Specification allows:

$PHL := min(L_{p1}, L_{p2}) = 108$ in

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region: ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

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

$MaximumSpacing := Spacingprogram(Columndia_{Bent3}, d_{bl}) = 6$ in

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

$FINALSPACING := SpacingCheck(MaximumSpacing, s) = 6$ in

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

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

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.

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

$Extension := ExtensionProgram(Columndia_{Bent3}) = 36$ in

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

LRFD 5.8.3.3 **Nominal Shear Resistance**

$V_u = 292.872$ kips

$\phi_s = 0.9$

$\beta := 2.0$ ***LRFD Article 5.8.3.4.1***

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

$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 67.545$ in

$de := d = 69$ in ***LRFD Eq. 5.8.2.9-1***

$dv := 0.9 de = 62.1$ in

$V_c := 0.0316 \beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 565.16$ kips ***LRFD Eq. 5.8.3.3-3***

$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{sNOhinge} = 327.888$ kips ***LRFD Eq. 5.8.3.3-4***

$V_n := \phi_s \cdot .25 fc \cdot bv \cdot dv = 4.024 \times 10^6$ kips

$\phi Vn := \min\left[V_n, \left(V_c + V_s\right) \cdot \phi_s\right] = 803.743$ kips

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

$Shearcheck2 := ShearCheck\left(\phi Vn, V_u\right) = \text{"OK"}$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

LRFD Eq. 5.8.2.5-1

$Avmin := 0.0316 \sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot sNOhinge}{\frac{fye}{1000}} = 0.758$ in^2

$Av := 2Asp = 0.88$ in^2

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

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

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.

LRFD 5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

$$vu := \frac{V_u}{\phi_s \cdot bv \cdot dv} = 0.073 \quad ksi$$ ***LRFD Eq. 5.8.2.9-1***

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

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

$MaxSpacing := floor(spacingProgram(vu, dv, fc)) = 24$ in

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$Spacecheck(MaxSpacing, s, V_u, V_c) := \begin{array}{|l} a \leftarrow \min(MaxSpacing, 12) \quad \text{if } V_u \le 0.5 0.9 V_c \\ a \leftarrow \min(s, MaxSpacing, 12) \quad \text{otherwise} \\ a \end{array}$$

$sNOhinge := Spacecheck(MaxSpacing, sNOhinge, V_u, V_c) = 10$ in

Design Summary - Bent 3

StirrupSize = "#6"

$s = 6$ in

$sNOhinge = 10$ in

$PHL = 108$ in

$Extension = 36$ in

$N_3 = 29.987$ in

Design Check Summary - Bent 3

Shearcheck = "OK"	Shear capacity > Vn
Transversecheck = "OK"	Minimum shear reinforcement ratio
ReinforcementRatioCheck = "OK"	Maximum longitudinal reinforcement ratio
$MinimumA_l$ = "OK"	Minimum longitudinal reinforcement ratio
scheck = "OK"	Max spacing of transverse reinforcement
Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran = "OK"	Minimum shear reinforcement outside hinge zone

Bent 3 Strut Design

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.

Force Inputs

INPUT

$V_{pStrut2} := 1517 \cdot p_{eTran} = 326.682$ kips

$M_{pStrut2} := 19051 \cdot p_{eTran} = 4102.583$ kip·ft

$P_{uStrut2} := 125 \cdot p_{eTran} = 26.918$ kips

Reinforcement Details

$Ag := Strut3Depth \cdot Strut3Width$

$Ae := 0.8 Ag = 4.032 \times 10^3$ in^2 ***Guide Eq. 8.6.2-2***

$\mu_D := 2$ ***Guide Article 8.6.2***

StirrupSize := "#6"

$s := 3.5$ in s: Spacing of hoops or pitch of spiral (in)

$sNOhinge := 18$ in sNOhinge: Spacing of transverser reinforcement outside PHL

$Asp := .44$ in^2 Asp: Area of spiral or hoop reinforcing (in^2)

$Dsp := .75$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT

$Cover := 2$ in Cover: Concrete cover for the Column (in)

$b := Strut3Depth = 120$ in b: Depth of the Strut (in)

$d := b - Cover = 118$ in d: Effective Depth (in)

$bv := Strut3Width$ bv: Effective width of strut (in)

$NumberBars_1 := 16$ Total number of longitudinal bars along strut width (top and bottom)

$NumberBars_2 := 36$ Total number of longitudinal bars along strut depth (side)

$A_{bl1} := 1.56$ in^2 Abl1: Area of longitudinal bar 1

$A_{bl2} := .31$ in^2 Abl2: Area of longitudinal bar 2

$d_{bl1} := 1.41$ in dbl1: Diameter of longitudinal bar 1

$d_{bl2} := .625$ in dbl2: Diameter of longitudinal bar 2

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

$Av := 2 \cdot Asp = 0.88$ in^2

Guide Eq. 8.6.2-10

$$\rho_w := \frac{Av}{s \cdot b} = 0.0021$$

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

Guide Eq. 8.6.2-9

$$StressCheckRect(\rho_w, fyh) := \left| \begin{array}{l} fs \leftarrow 2 \cdot \rho_w \cdot fyh \\ a \leftarrow fs \quad \text{if } fs \le 0.35 \end{array} \right.$$

$$fw := StressCheckRect(\rho_w, fyh) = 0.251$$

$\alpha Prime := \alpha program(fw, \mu_D) = 3$ ***Guide Eq. 8.6.2-8***

If Pu is not compressive then vc=0 ***Guide Eq. 8.6.2-4***

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.

$vc := vcprogram(\alpha Prime, fc, P_u, Ag) = 0.216$ ksi

$Vc := vc \cdot Ag = 1.091 \times 10^3$ kips

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

$$vsprogramRect(Av, fyh, d, s, fc, Ae) := \left| \begin{array}{l} vs \leftarrow \dfrac{Av \cdot fyh \cdot d}{s} \\ maxvs \leftarrow 0.25 \sqrt{\dfrac{fc}{1000}} \cdot Ae \\ a \leftarrow vs \ \text{ if } \ vs \le maxvs \\ a \leftarrow maxvs \ \text{ if } \ vs > maxvs \\ a \end{array} \right.$$

Guide Eq. 8.6.3-2 and 8.6.4-1

$Vs := vsprogramRect(Av, fyh, d, s, fc, Ae) = 1.78 \times 10^3$ kips

$\phi Vn := \phi_s \cdot (Vs + Vc) = 2.584 \times 10^3$ kips ***Guide Eq. 8.6.1-2***

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

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 ***Guide Eq. 8.6.5-2***

$$mintranprogramRect(\rho_s) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \ \text{ if } \ \rho_s \ge 0.002 \\ a \leftarrow \text{"Increase Shear Reinforcing Ratio"} \ \text{ if } \ \rho_s < 0.002 \\ a \end{array} \right.$$

$Transversecheck := mintranprogramRect(\rho_w) = \text{"OK"}$

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**

$A_{long} := A_{bl1} \cdot NumberBars_1 + A_{bl2} \cdot NumberBars_2 = 36.12 \quad in^2$

Guide Eq. 8.8.1-1

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

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:

Guide Eq. 8.8.2-1

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

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

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

'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 ρ_{eTran} to take into account the model loads have not been multiplied by ρ_{eTran}. The location will also need to be **INPUT** into the PlasticHingeRegion program in inches.

The Guide Specifications allows for the use of the plastic hinge length from the LRFD Specification in SDC A and B (***Guide Article C8.8.9***).

$$\text{LRFDPlasticHingeLength}(\text{ColumnDia}, \text{ColumnHeight}) := \left| \begin{array}{l} a \leftarrow \text{ColumnDia} \\ b \leftarrow \frac{1}{6} \cdot \text{ColumnHeigh} \\ c \leftarrow 18 \\ \text{PHL} \leftarrow \max(a, b, c) \\ \text{PHL} \end{array} \right.$$

$L_{p2} := \text{LRFDPlasticHingeLength}(\max(\text{Strut3Depth}, \text{Strut3Width}), \text{LStrut3}) = 120$ in

Guide Article C8.8.9

The plastic hinge length will be the smaller of the two values, as the Guide Specification allows:

$\text{PHL} := L_{p2} = 120$ in

$\text{Extension} := \text{ExtensionProgram}(\max(\text{Strut3Depth}, \text{Strut3Width})) = 60$

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

$\text{MaximumSpacing} := \text{Spacingprogram}\left(\max(\text{Strut3Depth}, \text{Strut3Width}), d_{bl}\right) = 6$ in

$\text{FINALSPACING} := \text{SpacingCheck}(\text{MaximumSpacing}, s) = 3.5$ in

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

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**.*

LRFD 5.8.3.3 **Nominal Shear Resistance**

$\phi_s = 0.9$

$\beta := 2.0$

$\theta := \frac{\pi}{180} \cdot 45 = 0.785$ rad ***LRFD Article 5.8.3.4.1***

$de := 69.4$ 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

$$dvprogram(de, dv, h) := \left| \begin{array}{l} x \leftarrow 0.9de \\ y \leftarrow 0.75h \\ z \leftarrow max(x, y) \\ a \leftarrow dv \ \text{ if } dv \geq z \\ a \leftarrow z \ \text{ if } dv < z \\ a \end{array} \right.$$

LRFD Eq. 5.8.2.9-2

$dv := dvprogram(de, dvpreliminary, Strut3Depth) = 90$ in

$$V_c := 0.0316\beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 477.792 \text{ kips}$$

LRFD Eq. 5.8.3.3-3

$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot cot(\theta)}{sNOhinge} = 264 \text{ kips}$$

LRFD Eq. 5.8.3.3-4

$$\phi V_n := (V_c + V_s) \cdot \phi_s = 667.613 \text{ kips}$$

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

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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

$$Avmin := 0.0316\sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot sNOhinge}{\frac{fye}{1000}} = 0.796 \text{ in}^2$$

LRFD Eq. 5.8.2.5-1

$Av := 2 \cdot Asp = 0.88$ in^2

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

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.

LRFD 5.8.2.7 Maximum Spacing of Transverse Reinforcement

$vu := \frac{V_p}{\phi_s \cdot bv \cdot dv} = 0.138$ ksi

LRFD Eq. 5.8.2.9-1

MaxSpacing := spacingProgram(vu, dv, fc) = 24 in

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

MAXSPACING := Spacecheck (MaxSpacing, sNOhinge) = 18 in

$$\text{Spacecheck}\left(\text{MaxSpacing}, s, V_u, V_c\right) := \left| \begin{array}{l} a \leftarrow \min(\text{MaxSpacing}, 12) \ \text{ if } \ V_u \le 0.50. \\ a \leftarrow \min(s, \text{MaxSpacing}, 12) \ \ \text{otherwise} \\ a \end{array} \right.$$

$\text{sNOhinge} := \text{Spacecheck}\left(\text{MaxSpacing}, \text{sNOhinge}, V_u, V_c\right) = 12$ in

Design Summary - Strut 3

StirrupSize = "#6"

s = 3.5 in

sNOhinge = 12 in

PHL = 120 in

Extension = 60 in

Design Check Summary - Strut 3

Shearcheck = "OK"	Shear capacity > Vn
Transversecheck = "OK"	Minimum shear reinforcement ratio
ReinforcementRatioCheck = "OK"	Maximum longitudinal reinforcement ratio
MinimumA_l = "OK"	Minimum longitudinal reinforcement ratio
scheck = "OK"	Max spacing of transverse reinforcement
Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran = "OK"	Minimum shear reinforcement outside hinge zone

BENT 4 DESIGN

Guide Article 4.11.2: For SDC B, it is acceptable to use the moment capacity based on expected material strengths when the concrete reaches an extreme compressive fiber strain of 0.003.

Force Inputs

$M_{neBent4} := 5765$ kip – ft Nominal moment from PCA Column

INPUT

$V_{pelastic} := 1360$ kip Elastic shear from SAP2000 model

$P_u := 128400$ lb Axial load from earthquake and dead load combination

Reinforcement Details

$Ag := Acolumn_{Bent24}$

$Ae := 0.8 Ag = 2262$ in^2 ***Guide Eq. 8.6.2-2***

Guide Article 8.6.2

$\mu_D := 2$

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

StirrupSize := "#6" Tiesize: Bar size used for ties

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

$sNOhinge := 12$ in sNOhinge: Spacing of hoops or pitch outside PHL

$Asp := 0.44$ in^2 Asp: Area of hoop reinforcement in direction of loading (in^2)

$Dsp := 0.75$ in Dsp: Diameter of spiral or hoop reinforcing (in)

INPUT

$Cover := 3$ in Cover: Concrete cover for the Column (in)

$b := Columndia_{Bent24}$ in b: Diameter of column (in)

$d := b - Cover = 57$ in d: Effective depth of section in direction of loading (in)

$Dprime := b - 2 \cdot Cover$ in Dprime: Diameter (in column) of hoop reinforcing (in)

$NumberBars := 32$ Total number of longitudinal bars in column cross-section

$A_{bl} := 1.56$ in^2 Abl: Area of longitudinal bar

$d_{bl} := 1.41$ in dbl: Diameter of longitudinal bar

$bv := Columndia_{Bent24}$ bv: Diameter of column

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

The design forces shall be the lesser of the forces resulting from the overstrength plastic hinging moment capacity or unreduced elastic seismic forces in columns or pier walls.

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

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.

$\lambda_{mo} := 1.4$ for ASTM A 615 Grade 60 reinforcement ***Guide Article 8.5***

$M_{pBent4} := \lambda_{mo} \cdot M_{neBent4} \cdot 1000 \cdot 12 = 9.685 \times 10^{7}$ lb – in

$Fixity := ColumnHeight_{Bent4} = 385.872$ in

$V_p := \dfrac{2 \cdot M_{pBent4}}{Fixity \cdot 1000} = 501.99$ kips

$V_{pelastic} := V_{pelastic} \cdot p_{eTran} = 292.872$ kips

***Article 8.7*: Requirements for Ductile Member Design**

Each column must satisfy the minimum lateral flexural capacity

$\Lambda := 2$ Fixed and top and bottom ***Guide Article 4.8.1***

$M_{neminBent} := 0.1 \cdot DL_{Bent} \cdot \left(\dfrac{\dfrac{Fixity}{12} + 0.5 D_s}{\Lambda} \right) = 2521.112$ kip ft ***Guide Eq 8.7.1-1***

$$CheckMoment(M_{ne}, M_e) := \begin{vmatrix} a \leftarrow \text{"OK"} \ \ \text{if } M_{ne} \le M_e \\ a \leftarrow \text{"FAILURE"} \ \ \text{otherwise} \end{vmatrix}$$

$CheckMoment(M_{neminBent}, M_{neBent2}) = \text{"OK"}$

If the moment check comes back "FAILURE" then the column size can be increased or the reinforcement can be increased.

Article 8.6: Shear Demand and Capacity for Ductile Concrete Members

It is recommended to use the plastic hinging forces whenever practical, but in this case the elastic forces will be used.

$V_u := \min(V_p, V_{pelastic}) = 292.872$ kips $\qquad \phi_s := 0.9$

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

$$\rho_s := \frac{4 \cdot Asp}{s \cdot Dprime} = 0.0054$$ ***Guide Eq. 8.6.2-7***

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

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

$$fs := StressCheck(\rho_s, fyh) = 0.326$$ ***Guide Eq. 8.6.2-6***

$$\alpha program(fs, \mu_D) := \left| \begin{array}{l} \alpha prime \leftarrow \frac{fs}{0.15} + 3.67 - \mu_D \\ a \leftarrow 0.3 \ \text{ if } \ \alpha prime \le 0.3 \\ a \leftarrow \alpha prime \ \text{ if } \ \alpha prime > 0.3 \wedge \alpha prime < 3 \\ a \leftarrow 3 \ \text{ if } \ \alpha prime \ge 3 \\ a \end{array} \right.$$ ***Guide Eq. 8.6.2-5***

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

If Pu is Compressive:

$$vcprogram(\alpha Prime, fc, P_u, Ag) := \left| \begin{array}{l} vc \leftarrow 0.032 \alpha Prime \left(1 + \frac{P_u}{2Ag \cdot 1000}\right) \cdot \sqrt{\frac{fc}{1000}} \\ min1 \leftarrow 0.11 \sqrt{\frac{fc}{1000}} \\ min2 \leftarrow 0.047 \alpha Prime \sqrt{\frac{fc}{1000}} \\ minimum \leftarrow \min(min1, min2) \\ a \leftarrow vc \ \text{ if } \ vc \le minimum \\ a \leftarrow minimum \ \text{ if } \ vc > minimum \\ a \end{array} \right.$$ ***Guide Eq. 8.6.2-3***

If Pu is NOT Compressive: ***Guide Eq. 8.6.2-4***

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

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

$Vc := vc \cdot Ae = 497.628$ kips

Article 8.6.3 & 8.6.4*: Shear Reinforcement Capacity** ***Guide Eq 8.6.3-2 and 8.6.4-1

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

$Vs := \text{vsprogram}(n, Asp, fyh, Dprime, s, fc, Ae) = 746.442$ kips

$\phi Vn := \phi_s \cdot (Vs + Vc) = 1.12\times 10^3$ kips ***Guide Eq. 8.6.1-2***

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

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

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:

$$\text{mintranprogram}(\rho_s) := \left| \begin{array}{l} a \leftarrow \text{"OK"} \ \text{ if } \ \rho_s \ge 0.003 \\ a \leftarrow \text{"Increase Shear Reinforcing Ratio"} \ \text{ if } \ \rho_s < 0.003 \\ a \end{array} \right.$$ ***Guide Eq. 8.6.5-1***

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

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

$A_{long} := NumberBars \cdot A_{bl} = 49.92 \quad in^2$

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

Guide Eq. 8.8.1-1

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

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

Guide Eq. 8.8.2-1

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

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

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

***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.

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

Guide Eq. 4.11.6-1

$L_p := \mathrm{PlasticHinge}(\mathrm{Fixity}, \mathrm{fye}, d_{bl}) = 43.56$ in

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

'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 ρ_{eTran} to take into account the model loads have not been multiplied by ρ_{eTran}. The location will also need to be **<u>INPUT</u>** into the PlasticHingeRegion program in inches.

$Mp75 := 0.75\,M_{pBent2} = 7.264\times 10^{7}$ lb·in

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

$L_{p1} := \text{PlasticHingeRegion}\left(L_p, \text{Columndia}_{\text{Bent24}}\right) = 90$ in

The Guide Specifications allows for the use of the plastic hinge length from the LRFD Specification in SDC A and B (***Guide Article C8.8.9***).

$$\text{LRFDPlasticHingeLength}(\text{ColumnDia}, \text{ColumnHeight}) := \left| \begin{array}{l} a \leftarrow \text{ColumnDia} \\ b \leftarrow \frac{1}{6}\cdot\text{ColumnHeigh} \\ c \leftarrow 18 \\ \text{PHL} \leftarrow \max(a, b, c) \\ \text{PHL} \end{array} \right.$$

$L_{p2} := \text{LRFDPlasticHingeLength}\left(\text{Columndia}_{\text{Bent24}}, \text{Fixity}\right) = 64.312$ in

Guide Article C8.8.9

The plastic hinge length will be the smaller of the two values, as the Guide Specification allows:

$\text{PHL} := \min\left(L_{p1}, L_{p2}\right) = 64.312$ in

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region: ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

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

$\text{MaximumSpacing} := \text{Spacingprogram}\left(\text{Columndia}_{\text{Bent24}}, d_{bl}\right) = 6$ in

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

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

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

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

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.

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

$$\text{Extension} := \text{ExtensionProgram}\left(\text{Columndia}_{\text{Bent24}}\right) = 30 \qquad \text{in}$$

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

LRFD 5.8.3.3 Nominal Shear Resistance

$$V_u = 292.872 \qquad \text{kips}$$

$$\phi_s = 0.9$$

$$\beta := 2.0$$

LRFD Article 5.8.3.4.1

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

$$Dr := bv - Cover - Dsp - \frac{d_{bl}}{2} = 55.545 \qquad \text{in}$$

$$de := d = 57 \qquad \text{in}$$

LRFD Eq. 5.8.2.9-1

$$dv := 0.9de = 51.3 \qquad \text{in}$$

$$V_c := 0.0316\beta\cdot\sqrt{\frac{fc}{1000}}bv\cdot dv = 389.059 \qquad \text{kips}$$

LRFD Eq. 5.8.3.3-3

$$V_s := \frac{2Asp\cdot\frac{fye}{1000}dv\cdot\cot(\theta)}{sNOhinge} = 225.72 \qquad \text{kips}$$

LRFD Eq. 5.8.3.3-4

$$V_n := \phi_s\cdot .25fc\cdot bv\cdot dv = 2.77\times 10^6 \qquad \text{kips}$$

$$\phi Vn := \min\left[V_n, \left(V_c + V_s\right)\cdot\phi_s\right] = 553.301 \qquad \text{kips}$$

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

$Shearcheck2 := ShearCheck(\phi Vn, V_u) = \text{"OK"}$

If ShearCheck2 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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

LRFD Eq. 5.8.2.5-1

$$Avmin := 0.0316\sqrt{\frac{fc}{1000}} \cdot \frac{bv \cdot sNOhinge}{\frac{fye}{1000}} = 0.758 \qquad in^2$$

$$Av := 2 \cdot Asp = 0.88 \qquad in^2$$

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

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

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.

LRFD 5.8.2.7 **Maximum Spacing of Transverse Reinforcement**

$$vu := \frac{V_u}{\phi_s \cdot bv \cdot dv} = 0.106 \qquad ksi$$

LRFD Eq. 5.8.2.9-1

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

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

$MaxSpacing := floor(spacingProgram(vu, dv, fc)) = 24$ in

The following check determines the maximum spacing of the hoops or ties outside of the PHL. If the minimum area of transverse reinforcement from LRFD 5.8.2.5 is required, then it is included in the check along with LRFD 5.8.2.7 and an assumed 12" ALDOT standard maximum spacing. Otherwise, the check only considers 5.8.2.7 and the 12" ALDOT standard

$$Spacecheck(MaxSpacing, s, V_u, V_c) := \left| \begin{array}{l} a \leftarrow min(MaxSpacing, 12) \text{ if } V_u \le 0.5 0.9 V_c \\ a \leftarrow min(s, MaxSpacing, 12) \text{ otherwise} \\ a \end{array} \right.$$

$sNOhinge := Spacecheck(MaxSpacing, sNOhinge, V_u, V_c) = 12$ in

Design Summary - Bent 4

$StirrupSize = "\#6"$

$s = 6$ in

$sNOhinge = 12$ in

$PHL = 64.312$ in

$Extension = 30$ in

$N_4 = 23.236$ in

Design Check Summary - Bent 4

Shearcheck = "OK"	Shear capacity > Vn
Transversecheck = "OK"	Minimum shear reinforcement ratio
ReinforcementRatioCheck = "OK"	Maximum longitudinal reinforcement ratio
$MinimumA_l$ = "OK"	Minimum longitudinal reinforcement ratio
scheck = "OK"	Max spacing of transverse reinforcement
Shearcheck2 = "OK"	Shear capacity outside hinge zone > Vn
MinimumTran = "OK"	Minimum shear reinforcement outside hinge zone
scheck2 = "OK"	Max spacing of transverse reinforcement outside hinge zone

Bent 4 Strut Design

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.

Force Inputs

$V_{pStrut2} := 968 \cdot p_{eTran} = 208.456$ kips

$M_{pStrut2} := 12527 p_{eTran} = 2697.657$ kip·ft

$P_{uStrut2} := 605 \cdot p_{eTran} = 130.285$ kips

Reinforcement Details

$Ag := Strut24Depth \cdot Strut24Width$

$Ae := 0.8 Ag = 2.419 \times 10^{3}$ in^{2} ***Guide Eq. 8.6.2-2***

$\mu_D := 2$ ***Guide Article 8.6.2***

StirrupSize := "#5"

$s := 4$ in s: Spacing of hoops or pitch of spiral (in)

$sNOhinge := 14$ in sNOhinge: Spacing of transverser reinforcement outside PHL

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

$Dsp := .625$ in Dsp: Diameter of spiral or hoop reinforcing (in) **INPUT**

$Cover := 2$ in Cover: Concrete cover for the Column (in)

$b := Strut24Depth = 72$ in b: Depth of the Strut (in)

$d := b - Cover = 70$ in d: Effective Depth (in)

$bv := Strut24Width$ bv: Effective width of strut (in)

$NumberBars_1 := 16$ Total number of longitudinal bars along strut width (top and bottom)

$NumberBars_2 := 20$ Total number of longitudinal bars along strut depth (side)

$A_{bl1} := 1.56$ in^{2} Abl1: Area of longitudinal bar 1

$A_{bl2} := .31$ in^{2} Abl2: Area of longitudinal bar 2

$d_{bl1} := 1.41$ in dbl1: Diameter of longitudinal bar 1

$d_{bl2} := .625$ in dbl2: Diameter of longitudinal bar 2

Article 8.6.2: Concrete Shear Capacity

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

$\rho_w := \frac{Av}{s \cdot b} = 0.0022$ **_Guide Eq. 8.6.2-10_**

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

$StressCheckRect(\rho_w, fyh) := \left| \begin{array}{l} fs \leftarrow 2 \cdot \rho_w \cdot fyh \\ a \leftarrow fs \ \text{ if } \ fs \le 0.35 \end{array} \right.$ **_Guide Eq. 8.6.2-9_**

$fw := StressCheckRect(\rho_w, fyh) = 0.258$

$\alpha Prime := \alpha program(fw, \mu_D) = 3$ **_Guide Eq. 8.6.2-8_**

If Pu is not compressive then vc=0 **_Guide Eq. 8.6.2-4_**

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

$Vc := vc \cdot Ag = 665.28$ kips

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

Guide Eq. 8.6.3-2 and 8.6.4-1

$vsprogramRect(Av, fyh, d, s, fc, Ae) := \left| \begin{array}{l} vs \leftarrow \frac{Av \cdot fyh \cdot d}{s} \\ maxvs \leftarrow 0.25\sqrt{\frac{fc}{1000}} \cdot Ae \\ a \leftarrow vs \ \text{ if } \ vs \le maxvs \\ a \leftarrow maxvs \ \text{ if } \ vs > maxvs \\ a \end{array} \right.$

$Vs := vsprogramRect(Av, fyh, d, s, fc, Ae) = 651$ kips

$\phi Vn := \phi_s \cdot (Vs + Vc) = 1.185 \times 10^3$ kips **_Guide Eq. 8.6.1-2_**

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

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

$$\text{mintranprogramRect}(\rho_s) := \begin{array}{|l} a \leftarrow \text{"OK"} \quad \text{if } \rho_s \geq 0.002 \\ a \leftarrow \text{"Increase Shear Reinforcing Ratio"} \quad \text{if } \rho_s < 0.002 \\ a \end{array}$$ ***Guide Eq. 8.6.5-2***

$$\text{CheckTransverse} := \text{mintranprogramRect}(\rho_w) = \text{"OK"}$$

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**

$$A_{long} := A_{bl1} \cdot \text{NumberBars}_1 + A_{bl2} \cdot \text{NumberBars}_2 = 31.16 \quad in^2$$ ***Guide Eq. 8.8.1-1***

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

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: ***Guide Eq. 8.8.2-1***

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

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

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

'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 ρ_{eTran} to take into account the model loads have not been multiplied by ρ_{eTran}. The location will also need to be <u>**INPUT**</u> into the PlasticHingeRegion program in inches.

The Guide Specifications allows for the use of the plastic hinge length from the LRFD Specification in SDC A and B (***Guide Article C8.8.9***).

$$\text{LRFDPlasticHingeLength}(\text{ColumnDia}, \text{ColumnHeight}) := \left| \begin{array}{l} a \leftarrow \text{ColumnDia} \\ b \leftarrow \frac{1}{6}\cdot\text{ColumnHeigh} \\ c \leftarrow 18 \\ \text{PHL} \leftarrow \max(a, b, c) \\ \text{PHL} \end{array} \right.$$

$L_{p2} := \text{LRFDPlasticHingeLength}(\max(\text{Strut24Depth}, \text{Strut24Width}), \text{LStrut24}) = 72$ in

Guide Article C8.8.9

The plastic hinge length will be the smaller of the two values, as the Guide Specification allows:

$\text{PHL} := L_{p2} = 72$ in

Maximum Spacing of Lateral Reinforcing in Plastic Hinge Region ***Guide Article 8.8.9***

Shall Not Exceed the Smallest of:

$\text{MaximumSpacing} := \text{Spacingprogram}\left(\max(\text{Strut24Depth}, \text{Strut24Width}), d_{bl}\right) = 6$ in

$\text{FINALSPACING} := \text{SpacingCheck}(\text{MaximumSpacings}) = 4$ in

$scheck := ShearCheck(MaximumSpacings) = "OK"$

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****.*

LRFD 5.8.3.3 **Nominal Shear Resistance**

$\phi_s = 0.9$

$\beta := 2.($

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

LRFD Article 5.8.3.4.1

$de := 69.4$ 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

$$dvprogram(de, dv, h) := \left| \begin{array}{l} x \leftarrow 0.9de \\ y \leftarrow 0.75h \\ z \leftarrow max(x, y) \\ a \leftarrow dv \ \ \text{if } dv \geq z \\ a \leftarrow z \ \ \text{if } dv < z \\ a \end{array} \right.$$

LRFD Eq. 5.8.2.9-2

$dv := dvprogram(de, dvpreliminary, Strut24Depth) = 66.75$ in

$$V_c := 0.0316\beta \cdot \sqrt{\frac{fc}{1000}} bv \cdot dv = 354.362$$ kips

LRFD Eq. 5.8.3.3-3

$$V_s := \frac{2Asp \cdot \frac{fye}{1000} dv \cdot \cot(\theta)}{sNOhinge} = 177.364$$ kips

LRFD Eq. 5.8.3.3-4

$\phi V_n := (V_c + V_s) \cdot \phi_s = 478.554$ kips

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

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.

LRFD 5.8.2.5 Minimum Transverse Reinforcement

$$Avmin := 0.0316\sqrt{\frac{fc}{1000}}\cdot\frac{bv\cdot sNOhinge}{\frac{fye}{1000}} = 0.619 \quad in^2$$

LRFD Eq. 5.8.2.5-1

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

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

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.

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

$Mp := 428($

$$vu := \frac{V_p}{\phi_s\cdot bv\cdot dv} = 0.199 \quad ksi$$

LRFD Eq. 5.8.2.9-1

$MaxSpacing := spacingProgram(vu, dv, fc) = 24 \quad in$

LRFD Eq. 5.8.2.7-1 and 5.8.2.7-2

$$Spacecheck(MaxSpacing, s, V_u, V_c) := \begin{array}{|l} a \leftarrow min(MaxSpacing, 12) \ \text{ if } V_u \le 0.5\,0.9 \\ a \leftarrow min(s, MaxSpacing, 12) \ \text{ otherwise} \\ a \end{array}$$

$sNOhinge := Spacecheck(MaxSpacing, sNOhinge, V_u, V_c) = 12 \quad in$

Design Summary - Strut 4

$StirrupSize = "\#5"$

$s = 4$ in

$sNOhinge = 12$ in

$PHL = 72$ in

Design Check Summary - Strut 4

$Shearcheck = "OK"$ Shear capacity > Vn

Transversecheck = "OK" — Minimum shear reinforcement ratio

ReinforcementRatioCheck = "OK" — Maximum longitudinal reinforcement ratio

MinimumA_l = "OK" — Minimum longitudinal reinforcement ratio

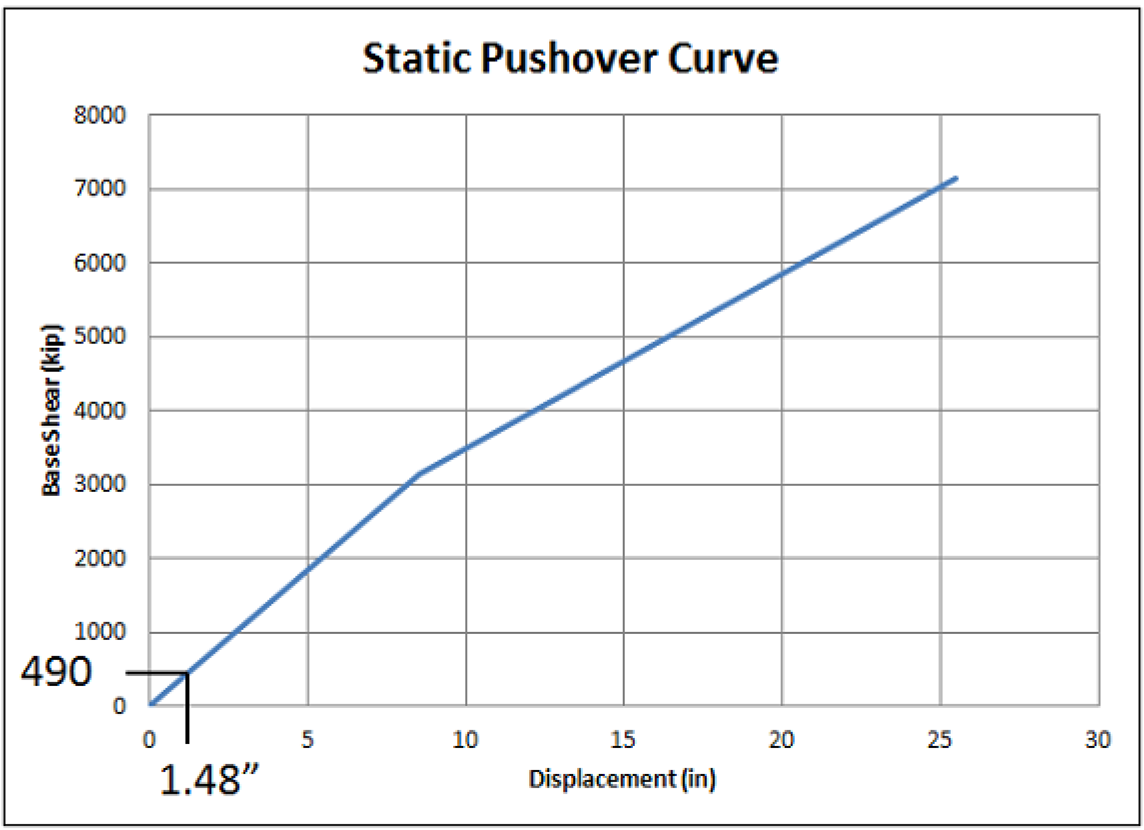
scheck = "OK" — Max spacing of transverse reinforcement

Shearcheck2 = "OK" — Shear capacity outside hinge zone > Vn

MinimumTran= "OK" — Minimum shear reinforcement outside hinge zone

Transverse Connection Design

Pushover Analysis Results



ALDOT Current Connection Steel Angle Design Check

$$\text{Vcolbent} := \frac{490}{N} = 81.667 \quad \text{kips}$$

LRFD 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

Bolt Properties

$Fub := 58$	ksi	Strength of Anchor Bolt (It is assumed that ASTM A307 Grade C bolt is used)
$Dia_b := 2.5$	in	Diameter of Anchor Bolt
$Ns := 1$		Number of Shear Planes per Bolt

INPUT

Angle Properties

$Fy := 36$	ksi	Yield Stress of the Angle
$Fu := 58$	ksi	Ultimate Stress of the Angle
$t := 1.00$	in	Thickness of Angle
$h := 6$	in	Height of the Angle
$w := 6$	in	Width of the Angle
$l := 12$	in	Length of the Angle
$k := 1.5$	in	Height of the Bevel
$distanchorhole := 4$	in	Distance from the vertical leg to the center of the hole. This is the location of the holes.
$diahole := Dia_b + \frac{1}{8} = 2.625$	in	Diameter of bolt hole
$BLSHlength := 6$	in	Block Shear Length
$BLSHwidth := 2$	in	Block Shear Width

INPUT

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

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

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

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

$$ShearCheck(\phi Vn, V_u) := \begin{vmatrix} a \leftarrow \text{"OK"} \quad \text{if } \phi Vn \geq V_u \\ a \leftarrow \text{"FAILURE"} \quad \text{if } \phi Vn < V_u \\ a \end{vmatrix}$$

Clip Angle Check:

AISC J4: Block Shear

$Agv := t \cdot BLSHlength = 6$ in^2

$Anv := t \cdot (BLSHlength - 0.5 diahole) = 4.688$ in^2

$Ant := t \cdot (BLSHwidth - 0.5 diahole) = 0.688$ in^2

$$BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) := \begin{vmatrix} b \leftarrow 0.6 Fu \cdot Anv + Ubs \cdot Fu \cdot Ant \\ c \leftarrow 0.6 Fy \cdot Agv + Ubs \cdot Fu \cdot Ant \\ a \leftarrow b \quad \text{if } b \leq c \\ a \leftarrow c \quad \text{if } b > c \\ a \end{vmatrix}$$ ***AISC Eq. J4-5***

$Rn := BLSHprogram(Agv, Anv, Ant, Ubs, Fu, Fy) = 169.475$ kips

$\phi bsRn := \phi_{bs} \cdot Rn = 135.58$ kips

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

AISC D2: Tension Member

$Ut := 0.6$ 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

$Ae := Ant \cdot Ut = 2.025$ in^2 ***AISC Eq. D3-1***

$\phi tPn := \phi_t \cdot Fub \cdot Ae = 93.96$ kips ***AISC Eq. D2-2***

$TensionCheck_{AISC} := ShearCheck(\phi tPn, Vcolbent) = "OK"$

AISC G: Shear Check

$Cv := 1.0$

$Aw := t \cdot w = 6$ in^2

$\phi sangleVn := \phi_{sangle} \cdot 0.6 Fy \cdot Aw \cdot Cv = 129.6$ kips ***AISC Eq. G2-1***

$ShearAngleCheck := ShearCheck(\phi sangleVn, Vcolbent) = "OK"$

Anchor Bolt Check:

LRFD Article 6.13.2.12: Shear Resistance For Anchor Bolts

$A_b := \frac{\pi \cdot Dia_b^{\,2}}{4} = 4.909$ in^2

$\phi sRn := \phi_s \cdot 0.48 A_b \cdot Fub \cdot Ns = 102.494$ kips ***LRFD Eq. 6.13.2.12-1***

$Shear_{Anchorbolts} := ShearCheck(\phi sRn, Vcolbent) = "OK"$

LRFD Article 6.13.2.9: Bearing Resistance at Bolt Holes

For Standard Holes

$\phi bbRn := 2.4 Dia_b \cdot t \cdot Fub = 348$ kips ***LRFD Eq. 6.13.2.9-1***

For Slotted Holes

$\phi bbRns := Lc \cdot t \cdot Fub = 116$ kips ***LRFD Eq. 6.13.2.9-4***

$Bearing_{Boltstandard} := ShearCheck(\phi bbRn, Vcolbent) = "OK"$

$Bearing_{Boltslotted} := ShearCheck(\phi bbRns, Vcolbent) = "OK"$

LRFD Article 6.13.2.10: Tensile Resistance

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.

$Tu := \frac{Vcolbent \cdot 1}{distanchorhole} = 20.417$ kips

LRFD Eq. 6.13.2.10.2-1

$\phi tTn := \phi_t \cdot 0.76 A_b \cdot Fub = 173.102$ kips

TensionCheck := ShearCheck(ϕtTn , Tu) = "OK"

Article 6.13.2.11: Combined Tension and Shear

LRFD Eq. 6.13.2.11-1
LRFD Eq. 6.13.2.11-2

Pu := Vcolbent

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

$Tn_{combined} := \text{CombinedProgram}\left(Pu, A_b, Fub, \phi sRn, \phi_s\right) = 130.747$ kips

$\phi tTn_{combined} := \phi_t \cdot Tn_{combined} = 104.598$ kips

CombinedCheck := ShearCheck($\phi tTn_{combined}$, Vcolbent) = "OK"

Summary

$Dia_b = 2.5$ in

$Shear_{Anchorbolts}$ = "OK"

$Bearing_{Boltstandard}$ = "OK"

$Bearing_{Boltslotted}$ = "OK"

TensionCheck = "OK"

CombinedCheck = "OK"

BlockShearCheck = "OK"

$TensionCheck_{AISC}$ = "OK"

ShearAngleCheck = "OK"

Appendix Q: Scarham Creek Bridge Moment-Interaction Diagrams

Bents 2 and 4

```
STRUCTUREPOINT - spColumn v4.81 (TM)                                                                        Page   1
15 day trial license. Locking Code: 4-1EC20. User: oem, Hewlett-Packard Company                            03/06/13
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  ooooo    oo       oooooo      ooooo         ooo      ooooo o   oo   oo   oo    oo   oo (TM)
```

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=============================================================================================
                                  spColumn v4.81 (TM)
                Computer program for the Strength Design of Reinforced Concrete Sections
                            Copyright © 1988-2012, STRUCTUREPOINT, LLC.
                                      All rights reserved
=============================================================================================
```

```
General Information:
====================
  File Name: untitled.col
  Project:
  Column:                                    Engineer:
  Code:     ACI 318-11                       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
  rx =  10.5 in                              ry =  10.5 in
  Xo =  0 in                                 Yo =  0 in

Reinforcement:
==============
  Bar Set: 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: Spiral; #3 ties with #10 bars,  #5 with larger bars.
  phi(a) = 0.85,  phi(b) = 0.9,  phi(c) = 0.75

  Layout: Circular
  Pattern: All Sides Equal  (Cover to transverse reinforcement)
  Total steel area: As = 18.72 in^2 at rho = 1.35%
  Minimum clear spacing = 5.67 in

  12 #11   Cover = 6 in

Factored Loads and Moments with Corresponding Capacities:
=========================================================
                Pu           Mux       PhiMnx PhiMn/Mu NA depth Dt depth    eps_t    Phi
  No.          kip          k-ft         k-ft                in       in
  --- ------------ ------------ ------------ -------- -------- -------- -------- ------
    1      1294.00       810.00      1707.86    2.108    20.15    34.67  0.00216  0.755

  *** End of output ***
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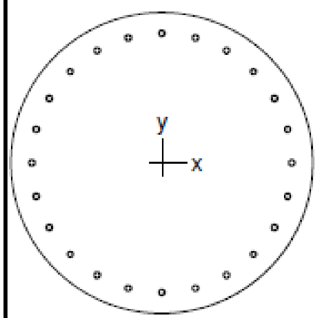


60 in diam.

Code: ACI 318-11

Units: English

Run axis: About X-axis

Run option: Investigation

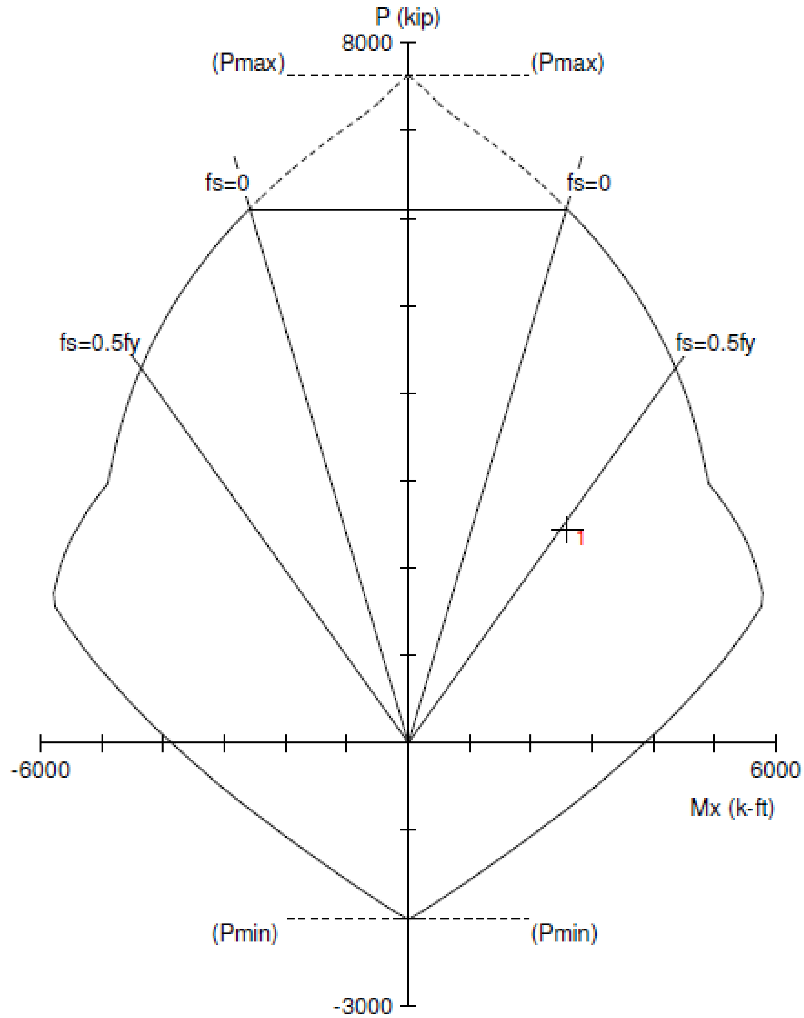
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 03/06/13

Time: 16:45:58



spColumn v4.81. 15 day trial license. Locking Code: 4-1EC20. User: oem, Hewlett-Packard Company

File: C:\Users\jdl0003\Documents\Research\ALDOT Bridge Design Examples\SDC B\Mo...\Scarham Creek Bents 2 and 4.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		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		Min clear spacing = 5.32 in	Clear cover = 3.50 in
Confinement: Tied			
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65			

Bent 3

```
STRUCTUREPOINT - spColumn v4.81 (TM)                                                        Page   1
15 day trial license. Locking Code: 4-1EC20. User: oem, Hewlett-Packard Company             03/06/13
untitled.col                                                                                04:31 PM
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                    oo    oo             oo
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     oo   oooooo    oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
o    oo   oo        oo    oo   oo   oo   oo  o   oo   oo    oo   oo   oo    oo   oo
 ooooo    oo         oooooo     ooooo     ooo     ooooo o   oo   oo   oo    oo   oo (TM)
```

```
========================================================================================================
                                        spColumn v4.81 (TM)
                     Computer program for the Strength Design of Reinforced Concrete Sections
                                Copyright © 1988-2012, STRUCTUREPOINT, LLC.
                                          All rights reserved
========================================================================================================
```

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General Information:
====================
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  Project:
  Column:                                  Engineer:
  Code:     ACI 318-11                     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
  rx =  10.5 in                            ry =  10.5 in
  Xo =  0 in                               Yo =  0 in

Reinforcement:
==============
  Bar Set: 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: Spiral; #3 ties with #10 bars,  #5 with larger bars.
  phi(a) = 0.85,  phi(b) = 0.9,  phi(c) = 0.75

  Layout: Circular
  Pattern: All Sides Equal  (Cover to transverse reinforcement)
  Total steel area: As = 18.72 in^2 at rho = 1.35%
  Minimum clear spacing = 5.67 in

  12 #11   Cover = 6 in

Factored Loads and Moments with Corresponding Capacities:
=========================================================
                  Pu           Mux        PhiMnx PhiMn/Mu NA depth Dt depth    eps_t    Phi
  No.            kip          k-ft          k-ft                in       in
  --- ------------ ------------ ------------ -------- -------- -------- -------- ------
    1      1294.00       810.00      1707.86    2.108    20.15    34.67  0.00216  0.755

  *** End of output ***
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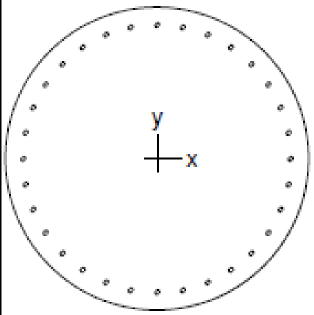
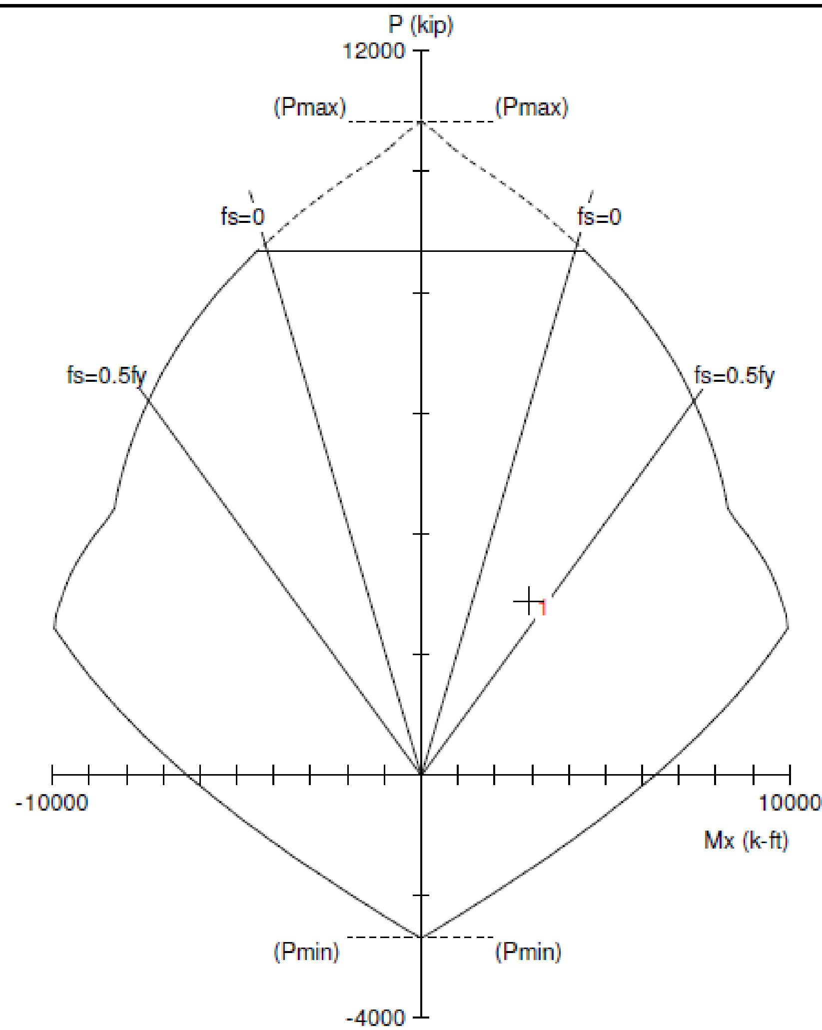


72 in diam.



Code: ACI 318-11

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 03/06/13

Time: 16:52:45

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File: C:\Users\jdl0003\Documents\Research\ALDOT Bridge Design Examples\SDC B\Moment In...\Scarham Creek Bent 3.col

Project:

Column:

Engineer:

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		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		Min clear spacing = 4.82 in	Clear cover = 3.50 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65