FINAL REPORT

Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

Project IA-H2, FY 02

Report No. ITRC FR 02-3

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June 2004

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This research project was sponsored by the State of Illinois, acting by and through its Department of Transportation, according to the terms of the Memorandum of Understanding established with the Illinois Transportation Research Center. The Illinois Transportation Research Center is a joint Public-Private-University cooperative transportation research unit underwritten by the Illinois Department of Transportation. The purpose of the Center is the conduct of research in all modes of transportation to provide the knowledge and technology base to improve the capacity to meet the present and future mobility needs of individuals, industry and commerce of the State of Illinois.

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### Abstract

This report provides the Illinois Department of Transportation (IDOT) with data to assess the impact of the *Recommended LRFD Guidelines for Seismic Design of Highway Bridges* developed by a joint venture between the Applied Technical Council (ATC) and the Multidisciplinary Center for Earthquake Engineering Research (MCCER) based on a study initiated by the National Cooperative Highway Research Program in 1998 (NCHRP Project 12-49). Substructures of four southern Illinois bridges with Seismic Performance Categories (SPC) of “B” and “C” are designed for earthquake loads using the proposed NCHRP Specification and the *AASHTO Standard Specifications for Highway Bridges* (1996). It is also noted that for comparison purposes in all Illinois bridges investigated, similar earthquake resisting systems were used in the NCHRP design as in the AASHTO design, as requested by the project Technical Review Panel. Also, Operational performance objective was specified for the NCHRP design. The results of analysis and design of the first three Illinois bridges using both the AASHTO and the proposed NCHRP specifications and the corresponding cost impact analysis are presented. It is observed that the total construction cost of the interior bents designed using the proposed NCHRP Specifications were 1.96 to 5.43 times higher than the cost obtained using the AASHTO Specifications.

To evaluate the realism of the NCHRP design spectrum for the southern Illinois region, a ground motion study is conducted at all four bridge sites where the spectral accelerations are determined and are compared with the corresponding USGS map values (1996). Comparison of the ground motion accelerations obtained in this study with the ones obtained from the USGS maps (1996) indicates that they are comparable when a 2500-year return period is considered.

### Key Words

- Highway bridges, Seismic design, LRFD, NCHRP 12-49, AASHTO
EVALUATION OF COMPREHENSIVE SEISMIC DESIGN OF BRIDGES (LRFD) IN ILLINOIS

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ACKNOWLEDGEMENTS

The work was conducted through a contact between Illinois Transportation Research Center (ITRC) and Southern Illinois University Edwardsville (SIUE), ITRC Project IA-H2-02. The research team acknowledges the valuable assistance it obtained from the Illinois Department of Transportation (IDOT) Technical Review Panel (TRP): Mr. Tom Domagalski (chair), Mr. Chad Hodel, Mr. Ruben Boehler, Mr. Salah Khayyat, Mr. Bill Kramer, Mr. Justin Mann, and Mr. Mike Trello; and the ITRC administrators Dr. Steven Hanna and Prof. Dianne Kay. The TRP members’ openness, insights, and enthusiasm and the ITRC administrators’ willingness to work under adverse budgetary conditions are praiseworthy.

The research team also acknowledges the following graduate students for their contributions in verifying the structural and geotechnical computer models at various stages of the project: Mr. Vincent Nganga (SIUE), Ms. Rachel Mertz (SIUE), and Ms. Robin Cisco (SIUC). The research team is also grateful to Ms. Joy Tedford, SIUE Civil Engineering Secretary, for her assistance in processing numerous contacts and invoices involved in this project in a professional and timely manner.
EXECUTIVE SUMMARY

This report presents the results of a research project, funded by the Illinois Transportation Research Center (contract # IA-H2-02), in order to provide the Illinois Department of Transportation with information to assess the impact of the Recommended LRFD Guidelines for the Seismic Design of Highway Bridges developed by a joint venture of the Applied Technology Council and the Multidisciplinary Center for Earthquake Engineering Research (ATC/MCEER 2002) on the seismic design of bridges in southern Illinois.

The technical basis of the seismic provisions within both the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges (AASHTO 1996) and the AASHTO LRFD Bridge Design Specifications (AASHTO 1998) are essentially the same as that of the seismic design guidelines published over two decades ago by the Applied Technology Council (ATC 1981). The primary objective of the Recommended LRFD Guidelines is to provide seismic design provisions that reflect the latest research findings, design philosophies, and design approaches. It is noted that the technical content of the Recommended LRFD Guidelines is based on a study initiated by the National Cooperative Highway Research Program in 1998 to develop a new set of seismic design provisions for highway bridges (NCHRP Project 12-49) for possible incorporation into the future AASHTO LRFD bridge design specifications.

In comparison with the AASHTO Standard Specifications, the proposed NCHRP Specification has adopted a dual-criteria strategy of two-level design earthquakes. One based on an expected or Frequent Earthquake (FE) with a 50% probability of exceedance in the 75-year design life of a bridge (with a return period of 108 years), and the second based on a rare or Maximum Considered Earthquake (MCE) with a 3% probability of exceedance in 75 years (with an approximate return period of 2500 years). Current AASHTO Specifications consider a single-level earthquake with a 10% probability of exceedance in 50 years (with a return period of 475 years). Also, the proposed NCHRP Specification defines two seismic performance levels in terms of the anticipated performance of the bridge in the rare earthquake event: Life Safety – which means the bridge should not collapse (partial or complete replacement may be required)
and serious personal injury or loss of life should be avoided, and Operational – which means the bridge will be functional immediately after a rare earthquake. New 1996 US Geological Survey (USGS) maps are used in the proposed NCHRP Specification instead of the 1988 USGS maps used in the current AASHTO Standard Specifications (AASHTO 1996 and 1998).

The primary objectives of the research project are:

1) Seismic design of substructure of typical Illinois bridges located in southern Illinois using the AASHTO Specifications and the proposed NCHRP Specification (IDOT provided the bridge specifications and bridge site soil boring data).

2) Identify and summarize the differences in earthquake loads, their forces on each substructure unit, their effect on substructure design (e.g., footing size, piling design, rebar detailing, etc.), and construction cost differences for the southern Illinois bridges designed above using the AASHTO Standard Specifications and the proposed NCHRP Specification.

3) Evaluate the realism of the new USGS accelerations for the bridge sites given for the specified bridges by comparing them with accelerations obtained using an independent procedure.

To achieve the first two objectives of this study, the substructure of the four typical existing Illinois bridges located in Johnson Country, St. Clair County, Pulaski County, and in Madison County are redesigned according to the seismic provisions given in the proposed NCHRP Specification where the “Operational” performance objective is specified. The Madison County bridge has been relocated by the project Technical Review Panel (TRP) to another site in Pulaski County. It is noted that the design of the fifth bridge located in a site with potential soil liquefaction consideration is not investigated due to the termination of the project effective June 30, 2004. This is the result of both the appropriation and re-appropriation for ITRC not being included in the Illinois Department of Transportation (IDOT) budget for FY 2005.
The superstructure of these bridges consists of continuous (two to four span) steel girders while the earthquake loads are mainly resisted by the solid wall bents or multi-column interior bents, with pile supported footings. IDOT provided soil boring data at the bridge sites. For appropriate comparison, the substructures of all bridges are also redesigned according to Division 1A of the AASHTO Specifications with exception of the St. Clair County bridge which was designed in 2000, where the adequacy of the substructure bents is checked according to the AASHTO Specifications. It is also noted that for comparison purposes in all Illinois bridges investigated, similar earthquake resisting systems are used in the NCHRP design as in the AASHTO design, as requested by the project TRP.

The results of analysis and design of the first three Illinois bridges using the seismic provisions of both the AASHTO and the proposed NCHRP specifications and the corresponding cost impact analysis are presented in this report. The results of a similar study for the fourth Illinois bridge (currently in progress) will be submitted to the project Technical Review Panel as an addendum report due to the time limitation imposed on the project as a result of elimination of state funding for the ITRC in FY 2005.

To obtain the third objective of this study, the acceleration coefficients at four sites in the State of Illinois for both bedrock and ground surface levels are regenerated using existing source paths and site models, and attenuation relationships supplemented by new developments to produce synthetic time histories, response spectra values at frequencies of interest, uniform hazard spectra, and site amplification factors. The results are compared with the 1996 USGS acceleration coefficients used in the proposed NCHRP Specification.

Comparison of the design earthquake response spectrum for the MCE, which governs the NCHRP design forces and moments in substructure members in cases investigated, indicates that the peak values of the spectrum are significantly higher (2.9 to 5.7 times) than the corresponding values obtained from the AASHTO design earthquake response spectrum at the bridge sites given in three southern Illinois counties. Also, the AASHTO response modification factors for all bridges investigated were significantly larger (2.0 to 3.3 times) than the corresponding values.
used in the NCHRP designs for the MCE since the “Operational” performance objective was specified for the NCHRP design.

Due to the above reasons, the NCHRP design forces and moments for the substructure elements and foundations were significantly higher than the corresponding values used in AASHTO Specifications. Consequently, larger amounts of reinforcing steel and/or concrete were used in the NCHRP wall or multi-column bents where an extensive number of reinforced concrete or steel H piles were needed in the foundations in comparison to the AASHTO design.

More specifically, the concrete volume and reinforcing steel weight required in the NCHRP design are 3.16 and 2.39 times the corresponding values obtained using AASHTO Specifications in the Johnson County bridge while it is estimated that the total cost of the interior bents designed using the NCHRP Specification is 1.96 times the value obtained using the AASHTO Specifications. For the St. Clair County Bridge, the concrete volume and reinforcing steel weight needed for the substructures designed according to the proposed NCHRP Specification are 3.65 and 4.30 times the corresponding values obtained using AASHTO Specifications while it is observed that the NCHRP substructure bent design cost is 5.23 times the corresponding value obtained in the AASHTO design. In the Pulaski County bridge, the concrete volume and reinforcing steel weight needed for the substructures designed according to the proposed NCHRP Specification are 4.88 and 5.61 times the corresponding values obtained using AASHTO Specifications while it is observed that the NCHRP substructure bent design cost is 5.43 times the corresponding value obtained in the AASHTO design.

Comparison of the ground motion accelerations obtained in this study with the ones obtained from the USGS maps (1996) indicates that they are comparable when a 2500-year return period is considered. Slightly lower acceleration coefficients were found in three bridge sites, and a slightly higher acceleration coefficient was found in one bridge site investigated.
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1. INTRODUCTION

1.1 Statement of the Problem

The technical basis of the current seismic provisions within both the AASHTO Standard Specifications for Highway Bridges (AASHTO, 1996) and the AASHTO LRFD Bridge Design Specifications (AASHTO, 1998) adopted by the American Association of State Highway and Transportation Officials are essentially the same as that of the ATC-6 provisions, published by Applied Technology Council (ATC) in 1981, which were initially adopted in the AASHTO Standard Specification in 1991.

Recent seismic events, such as the 1989 Loma Prieta and 1994 Northridge, California, earthquakes, the 1995 Kobe, Japan, earthquake, the 1999 Taiwan Chi-Chi earthquake, and recent technological advances have been stimuli for improving the seismic performance of transportation structures (Penzien, 2000).

The Applied Technology Council (ATC), in a joint venture with the Multidisciplinary Center for Earthquake Engineering Research (MCEER), has recently completed a project to develop comprehensive specifications for the seismic design of highway bridges (National Cooperative Highway Research Program, NCHRP, Project 12-49). The primary objective of the NCHRP Project 12-49 was to develop seismic design provisions that reflect the latest research findings, design philosophies, and design approaches. Henceforth, implementations of the newly developed provisions will ensure enhanced seismic performance of highway bridges. The Federal Highway Administration has funded the development of a stand-alone guide specification through MCEER based on the results of the NCHRP Project 12-49 that can be more readily used for seismic design (ATC/MCEER, 2002). In 2002, The American Association of State Highway and Transportation Officials (AASHTO) were considering this recommended specification for possible incorporation into the future AASHTO LFRD Bridge Specifications (Capron et al., 2001). Hereafter this document will be referred to as the proposed NCHRP Specification while the AASHTO Standard Specifications for Highway Bridges (AASHTO, 1996) will be referred to as the AASHTO Specifications.
The proposed NCHRP Specification defines two seismic performance levels in terms of the anticipated performance of the bridge in the rare earthquake event: Life Safety – which means the bridge should not collapse (partial or complete replacement may be required) and serious personal injury or loss of life should be avoided, and Operational – which means the bridge will be functional immediately after a rare earthquake. Operational is a higher level of seismic performance and it typically applies to bridges in priority routes. Life Safety is the minimum acceptable level of seismic performance allowed by the proposed NCHRP Specification.

Also, the proposed NCHRP Specification has adopted a dual-criteria strategy of two-level design earthquakes. One based on a frequent or expected earthquake with a 50% probability of exceedance in the 75-year design life of a bridge (with an approximate return period of 100 years) and the second based on a rare or Maximum Considered Earthquake (MCE) with a 3% probability of exceedance in 75 years (with an approximate return period of 2500 years). The AASHTO Specifications consider a single-level earthquake with a 10% probability of exceedance in the 50 years (with an approximate return period of 500 years).

The proposed NCHRP Specification constitutes a significant advance over the existing AASHTO Specification for seismic design of bridges; however, limited information is available regarding its material and construction cost impacts in the Midwest region of the United States. The result of this investigation will provide the Illinois Department of Transportation (IDOT) with data to adequately assess the impact of the proposed NCHRP Specification on the seismic design of bridges in Illinois, which in turn will permit IDOT to analyze the impact on bridge funding for future bridge projects.

### 1.2 Research Objectives and Scope

The primary objectives of the research project are:

4) Seismic design of substructure of typical Illinois bridges located in southern Illinois using the AASHTO Specifications and the proposed NCHRP Specification (IDOT provided the bridge specifications and bridge site soil boring data).
5) Identify and summarize the differences in earthquake loads, their forces on each substructure unit, their effect on substructure design (e.g., footing size, piling design, rebar detailing, etc.), and construction cost differences for the southern Illinois bridges designed above using the AASHTO Specifications and the proposed NCHRP Specification.

6) Evaluate the realism of the new USGS accelerations for the bridge sites given for the specified bridges by comparing them with accelerations obtained using an independent procedure.

To achieve the first two objectives of this study, the substructure of the four typical existing Illinois bridges located in Johnson Country (SN 044-0041, designed in 1970), St. Clair County (SN 082-0344 designed in 2000), Pulaski County (SN 077-0033, designed in 1965), and in Madison County (SN 060-0244, designed in 2001) are redesigned according to the seismic provisions given in the proposed NCHRP Specification where the “Operational” performance objective is specified. The Madison County bridge has been relocated by the project Technical Review Panel (TRP) to another site in Pulaski County with a latitude of 37°-17’ and a longitude of 89°-9’ (on I-57 over the Cache River). Hereafter this bridge will be referred to as the relocated Madison County Bridge. Locations of these bridges are shown in Figure 1. It is noted that the design of the fifth bridge located in a site with potential soil liquefaction consideration is not investigated due to the termination of the project effective June 30, 2004. This is the result of both the appropriation and re-appropriation for ITRC not being included in the Illinois Department of Transportation (IDOT) budget for FY 2005.

The superstructure these bridges consists of continuous (two to four span) steel girders while the earthquake loads are mainly resisted by the solid wall bents or multi-column interior bents, with pile supported footings. IDOT provided soil boring data at the bridge sites. For appropriate comparison, the substructures of all bridges are also redesigned according to the Division 1A of the AASHTO Specifications with exception of the St. Clair County bridge which was designed in 2000, where the adequacy of the substructure bents is checked according to the AASHTO
Specifications. It is also noted that for comparison purposes in all Illinois bridges investigated, similar earthquake resisting systems are used in the NCHRP design as in the AASHTO design, as requested by the project TRP.

To obtain the third objective of this study, the acceleration coefficients at four sites in the State of Illinois for both bedrock and ground surface levels are regenerated using existing source paths and site models, and attenuation relationships supplemented by new developments to produce synthetic time histories, response spectra values at frequencies of interest, uniform hazard spectra, and site amplification factors.

1.3 Overview of the Report

This written report includes seven chapters and seven appendices. In the following chapter (Chapter 2), a brief overview of the proposed NCHRP Specification for the seismic design of highway bridges and the outline of the procedure used to independently obtain ground motion accelerations comparable to the USGS values are presented. The results of analysis and design of the first three Illinois bridges using the seismic provisions of both the AASHTO and the proposed NCHRP specifications and the corresponding cost impact analysis are presented in the following three chapters (Chapters 3-5). The results of a similar study for the fourth Illinois bridge (currently in progress) will be submitted to the project Technical Review Panel as an addendum report due to the time limitation imposed on the project as a result of elimination of state funding for the ITRC in FY 2005. The seismic ground motion study conducted at four Illinois bridge sites is presented in Chapter 6, where the results are compared with the USGS acceleration used in obtaining the MCE in the proposed NCHRP Specification. The conclusions of the research project are summarized in Chapter 7. The detailed engineering computations used in Chapters 3 - 5 are presented in Appendices A - F, and the corresponding soil profiles constructed based on the provided soil boring data are given in Appendix G.
Figure 1.1 – Location of the Southern Illinois Bridge Sites Investigated
2. METHODOLOGY

2.1 Introduction

Highway bridges in different countries have had less than satisfactory performance in recent earthquakes, e.g., in 1989 Loma Prieta, 1994 Northridge, and 1995 Kobe earthquakes as reported by the Earthquake Engineering Research Institute reconnaissance reports (EERI 1990, 1995a, 1995b). Significant structural damage in such bridges have resulted in full or partial failures with considerable economic losses. The current bridge seismic design specifications in the AASHTO LRFD Bridge Design Specifications (AASHTO 1998) are based on Division I-A of the AASHTO Standard Specifications for Highway Bridges (AASHTO 1996) which in turn were based on essentially the seismic design guidelines published over two decades ago by the Applied Technology Council (ATC 1981). Thus, the current LRFD Specifications use at least 20 years of out-of-date seismic design criteria and detailing provisions. To address this weakness, in 1998, The National Cooperative Highway Research Program (NCHRP) initiated a study entitled “Comprehensive Specifications for the Seismic Design of Bridges (NCHRP Project 12-49)” to develop new seismic design provisions for highway bridges that reflect the latest research findings, design philosophies, and design approaches for possible incorporation into the future AASHTO LRFD bridge design specifications. As a result of this project, a standalone recommended LRFD guideline for seismic design of highway bridges was completed by a joint venture of the ATC and the Multidisciplinary Center for Earthquake Engineering Research in 2002 (ATC/MCEER 2002).

In the following section, a brief overview of the proposed NCHRP Specification (ATC/MCEER 2002) used to design the Illinois bridges in this study is presented. In Section 2.3, the methodology used in the ground motion study at the bridge sites to independently obtain seismic spectral accelerations comparable to the ones used the proposed NCHRP Specification.
2.2 The Proposed NCHRP Specification for Seismic Design of Highway Bridges

The philosophy of the proposed NCHRP Specification (ATC/MCEER 2002) can be summarized as the following:

- Minimized loss of life and serious injuries due to unacceptable performance.
- Low probabilities of collapse due to earthquake motions.
- Essential bridges should stay functional even after major earthquake.
- Upper level event ground motions used in design should have a low probability of being exceeded during the approximate 75-year design life of the bridge.
- The provision should be applicable to all regions of the United States.
- The designer should not be restricted from considering and employing new and innovative design approaches and details.

New concepts and major modifications of the proposed NCHRP Specification are listed below:

- Performance objectives design earthquake
- Fault distance zone effect (vertical acceleration effect)
- Nonlinear static displacement capacity verification (pushover) analysis method
- Increasing the reduction factor by pushover analysis
- No analysis design concept for regular bridges in the lower seismic hazard area
- Earthquake Resisting Systems and Elements (ERS and ERE)
- New soil factors
- New spectral shapes
- New concepts of spring constant for foundations
- Liquefaction effects
- Steel, concrete, and bearing design requirements
- Seismic isolation provisions
- New “40% combination rule” ($Q_x+0.4Q_y$ and $0.4Q_x+1.0Q_y$)
• Modeling of soil-structure interaction and structural discontinuities at expansion joints

The following levels of design forces and the corresponding expected structural damages have been introduced in the proposed NCHRP Specification:

1- **Frequent Earthquake** (50% probability of exceedance in 75 years): This is the design level under which the bridge should remain essentially elastic. This will result in a performance that is equivalent to an elastic (no damage) design for an expected earthquake with an approximate 100-year return period.

2- **Rare Earthquake** (3% probability of exceedance in 75 years/1.5 median deterministic): This design level is recommended in order to assure that a no-collapse performance for the Maximum Considered Earthquake (MCE) is satisfied where two performance objective levels are considered in rare earthquakes with a 2500-year return period – Life Safety (significant damage) and Operational (minimal damage), as given in Table 2.1. There are a number of design approaches that can be used to achieve the desired performance objectives as shown in Figure 2.1

Design response spectra for the rare earthquake (MCE) and expected earthquake (FE) are constructed using the 0.2 and 1.0 seconds spectral accelerations obtained from the USGS maps. Figures 2.2 and 2.3 show the corresponding values in the Central U.S. region for the MCE. The base curve constructed with these values is then modified according to the short ($F_a$) and long period ($F_v$) site coefficients determined based on the Site Classification and mapped spectral acceleration (see Figure 2.4). Based on Performance Objective Level of the bridge (i.e., Life Safety or Operational as given in Table 2.1) and the Seismic Hazard Level (as given in Table 2.2), Seismic Design and Analysis Procedure (SDAP) and Seismic Detailing Requirement (SDR) are specified (see Table 2.3).

In Capacity Spectrum Analysis Approach the bridge is considered as a single degree of freedom system. Elastic Response Spectrum Analysis (ERSA) is used for bridges with a regular
configuration. Displacement Capacity Verification (DCV) requires nonlinear static pushover analysis. ERSA is combined with DCV only in bridges with SDAP of “E”. Nonlinear Dynamic Analysis is required for structures with a seismic isolation system with an effective vibration period greater than 3 seconds or effective damping greater than 30 percent. The result of the analysis is divided by the response modification factors as given in Table 2.5 for various substructure elements to account for inelastic behavior of such elements. Based on the type of foundation used and the SDAP of the bridge, NCHRP provisions allow the foundation to be modeled as a rigid support, or a flexible spring, or be fixed at an estimated depth. A summary flowchart of the seismic design process of the proposed NCHRP Specifications is shown in Figure 2.5.

2.3 Ground Motion Study

To evaluate the realism of the USGS map accelerations used in the computation of design spectrum in the proposed NCHRP Specification, the following steps are identified to independently obtain similar accelerations for the bridge sites studied in Illinois:

Step 1 - Develop procedures to generate horizontal bedrock motions at the bridge sites in Illinois, from a seismologically based model, due mainly to shear waves generated from seismic sources affecting the sites of interest. The seismologically-based model will include effects of attenuation (Atkinson and Boore 1995; Toro et al. 1997; Frankel, et al. 1996; Pezeshk, et al., 1998; Somerville, et al., 2001; and Campbell and Bozorgnia, 2003), characteristics of the source zone, recurrence interval (1000 and 2500 years), and seismotectonic setting of the New Madrid seismic zone, Wabash zone, and other potential seismic sources in the region. Recurrence intervals of 1000 and 2500 years were considered based on the information from the “Federal Highway Administration (FHWA) Mid-America Ground Motion Workshop in Collinsville, Illinois” that suggested 1000 year return periods be considered as options in replacing the current 2500 year return period.

Step 2 - Generate peak ground accelerations, 1-second response spectrum accelerations, and 0.2-second response spectrum accelerations for the selected four sites by transmitting the seismic waves at the bedrock through soil.

Each step includes several tasks as described in more detail in Chapter 6.
Figure 2.1 – NCHRP Design Approaches (ATC/MCEER 2002)
Figure 2.2 – MCE 0.2 Second Spectral Acceleration Map in Central U.S. (USGS 1996)
Figure 2.3 – MCE 1.0 Second Spectral Acceleration Map in Central U.S. (USGS 1996)
Figure 2.4 – NCHRP Design Spectrum Construction (ATC/MCEER 2002)
Seismic Design Start

- Identify Site, Structure Variables, and Determine Seismic Hazard Level

- Multi-span Bridge
  - No
  - Distance to fault > 50 km
    - No
    - Vertical effect
      - No seismic analysis required
    - Yes
      - Permissible ERS
        - Yes
          - Permissible with owner approval
            - No
              - Not recommended for new bridges
            - Yes
              - Connections Details
        - No
          - Abutments are parts of ERS
            - Yes
              - Operational
                - No
                  - SDAP A2/C/D/E SDR 2/3/5/6
                - Yes
                  - SDAP A1/A2/B/C/D/E SDR 1/2/3/4
          - No
            - Operational
              - No
                - SDAP D/E SDR 2/3/5/6
              - Yes
                - SDAP D/E SDR 1/2/3/4
          - No
            - SDAP E SDR 2/3/5/6
          - SDAP E SDR 1/2/3/4

- Apply SDAP
  - Design components based on F/R

- SDR < 3
  - No
    - Check Liquefaction
  - Yes
    - Apply SDR

SDAP=Seismic Design and Analysis Procedure
SDR=Seismic Design Requirements
ERS=Earthquake Resisting System

Figure 2.5 – NCHRP Bridge Seismic Design Process
Table 2.1 – NCHRP Performance Objectives (ATC/MCEER 2002)

<table>
<thead>
<tr>
<th>Probability of Exceedance for Design Earthquake Ground Motions</th>
<th>Performance Objective Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rare Earthquake 3% in 75 years/1.5 Median Deterministic</td>
<td>Life Safety</td>
</tr>
<tr>
<td>Expected Earthquake 50% in 75 Years</td>
<td>Service</td>
</tr>
<tr>
<td></td>
<td>Damage</td>
</tr>
<tr>
<td></td>
<td>Service</td>
</tr>
<tr>
<td></td>
<td>Damage</td>
</tr>
</tbody>
</table>

Table 2.2 – NCHRP Seismic hazard Level (ATC/MCEER 2002)

<table>
<thead>
<tr>
<th>Seismic Hazard Level</th>
<th>Value of $F_s$</th>
<th>Value of $F_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>$F_s \leq 0.15$</td>
<td>$F_s \leq 0.15$</td>
</tr>
<tr>
<td>II</td>
<td>$0.15 &lt; F_s \leq 0.25$</td>
<td>$0.15 &lt; F_s \leq 0.35$</td>
</tr>
<tr>
<td>III</td>
<td>$0.25 &lt; F_s \leq 0.40$</td>
<td>$0.35 &lt; F_s \leq 0.60$</td>
</tr>
<tr>
<td>IV</td>
<td>$0.40 &lt; F_s$</td>
<td>$0.60 &lt; F_s$</td>
</tr>
</tbody>
</table>

Table 2.3 – NCHRP Seismic Design and Analysis Procedure (SDAP) and Seismic Detailing Requirement (SDR) (ATC/MCEER 2002)

<table>
<thead>
<tr>
<th>Seismic Hazard Level</th>
<th>Life Safety</th>
<th>Operational</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SDAP</td>
<td>SDR</td>
</tr>
<tr>
<td>I</td>
<td>A1</td>
<td>1</td>
</tr>
<tr>
<td>II</td>
<td>A2</td>
<td>2</td>
</tr>
<tr>
<td>III</td>
<td>B/C/D/E</td>
<td>3</td>
</tr>
<tr>
<td>IV</td>
<td>C/D/E</td>
<td>4</td>
</tr>
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</table>

Table 2.4 – NCHRP Minimum Analysis Requirements

<table>
<thead>
<tr>
<th>SDAP</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Analysis Requirement</td>
<td>Not Required</td>
<td>Not Required</td>
<td>CSDA</td>
<td>ERSA</td>
<td>ERSA with DCV</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Substructure Element</th>
<th>Performance Objective</th>
<th>Life Safety</th>
<th>Operational</th>
</tr>
</thead>
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<tr>
<td></td>
<td></td>
<td>SDAP D</td>
<td>SDAP E</td>
</tr>
<tr>
<td>Wall Piers-larger dimension</td>
<td>2</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>Columns – Single and Multiple</td>
<td>4</td>
<td>6</td>
<td>1.5</td>
</tr>
<tr>
<td>Pile Bents and Drilled Shafts – Vertical Piles – above ground</td>
<td>4</td>
<td>6</td>
<td>1.5</td>
</tr>
<tr>
<td>Pile Bents and Drilled Shafts – Vertical Piles – 2 diameters below ground</td>
<td>1</td>
<td>1.5</td>
<td>1</td>
</tr>
<tr>
<td>Pile Bents and Drilled Shafts – Vertical Piles – in ground</td>
<td>N/A</td>
<td>2.5</td>
<td>N/A</td>
</tr>
<tr>
<td>Pile Bents with Batter Piles</td>
<td>N/A</td>
<td>2</td>
<td>N/A</td>
</tr>
<tr>
<td>Seismically Isolated Structures</td>
<td>1.5</td>
<td>1.5</td>
<td>1</td>
</tr>
<tr>
<td>Steel Braced Frame – Ductile Components</td>
<td>3</td>
<td>4.5</td>
<td>1</td>
</tr>
<tr>
<td>Steel Brace Frame – Nominally Ductile Components</td>
<td>1.5</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>All Elements for Expected Earthquake</td>
<td>1.3</td>
<td>1.3</td>
<td>0.9</td>
</tr>
<tr>
<td>Connections (superstructure to abutment; joints within superstructure; column to cap beam; column to foundation)</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
</tr>
</tbody>
</table>
3. SEISMIC ANALYSIS AND DESIGN INVESTIGATION OF JOHNSON COUNTY BRIDGE

3.1 Introduction

The main purpose of this investigation is to provide the Illinois Department of Transportation (IDOT) with adequate information to assess the impact of the proposed NCHRP Specification on the seismic design of the Johnson County Bridge in Southern Illinois. The bridge substructure piers are redesigned for earthquake loads using both the seismic provisions given in Division 1A of the AASHTO Specifications (AASHTO 1996) and the proposed NCHRP Specification (MCEER 2001) where the “Operational” performance objective is specified. Detailed computations for design and analysis of this bridge according to the proposed NCHRP Specification and the AASHTO Specifications are given in Appendices A and B, respectively.

3.2 Existing Structure

The existing three span bridge with span lengths of 30’-11.5”, 46’-6”, and 30’-11.5” was designed in 1970. The superstructure consists of an 8” thick (42’ wide) reinforced concrete slab supported on six hot rolled W 27x94 steel girders spaced at 7’-2”. The superstructure is fixed in both longitudinal and transverse directions at the first pier by bolsters, and it is attached by rockers to the second pier and the non-integral abutments allowing it to move freely in longitudinal direction while it is restrained in the transverse direction. Both piers consist of 25’-8” wide 2’-2” thick reinforced concrete walls supported on 2’-3” thick footing and two rows of battered H piles. Existing bridge geometry is shown in Figure 3.1.

3.3 SAP Modeling and Basic Seismic Parameters

The bridge is modeled using SAP 2000 beam elements (CSI 1998). The model is a “stick figure” model, where the stiffnesses and masses of the members of the superstructure and substructure, are converted into equivalent beam elements. The foundation of the bridge is modeled using springs, whose stiffnesses are calculated based on the pile stiffnesses. Note that the foundation
model requires iteration, since the piles are not designed initially and the spring stiffnesses, and hence the finite element analysis, has to be updated throughout the design phase.

According to the proposed NCHRP Specification and with the provided geotechnical information at the bridge site (See Appendix G), the Johnson Country Bridge site is classified as Site Class “D” with Seismic Hazard Level of IV where Seismic Design and Analysis Procedure (SDAP) “D” and Seismic Detailing Requirements (SDR) of 6 are used to obtain the “Operational” performance objective. No potential for liquefaction exists at the given site. The Elastic Response Spectrum Method is required. Thus, the SAP model is analyzed using modal analysis, and then the response spectrum procedure is used to distribute the required seismic forces and displacements throughout the structure. The structure is analyzed independently in both the longitudinal and transverse directions where an adequate number of modes is included in each direction to obtain at least 90% mass participation.

According to the AASHTO Specifications, Division 1A, the Seismic Performance Category (SPC) of “B” is selected where the Soil Profile Type I with Site Coefficient of $S = 1.0$ and the Acceleration Coefficient of $A = 0.15g$ are used.

For the Johnson County bridge, design earthquake response spectra for both NCHRP 100-year and 2500-year events (Frequent Earthquake, FE, and Maximum Considered Earthquake, MCE, respectively) are compared with the AASHTO design earthquake response spectrum (500-year event) in Figure 3.2.

For NCHRP and AASHTO bridge models, the results of free vibration analysis are summarized in Tables 3.3 and 3.4, respectively, where the main mode shapes are shown.

### 3.4 Substructure Design Forces

To obtain the design member forces according to the proposed NCHRP Specification, 100% earthquake forces in one direction are combined with 40% earthquake forces in the other direction (instead of 30% used in the AASHTO Specifications), and then they are reduced by the
appropriate Response Modification factors (R-factors) which depend on not only the type of substructure element (as in the AASHTO Specification) but also on the performance level, SDAP, and period of the structure (e.g., R of 1 and 0.9 are used in the wall piers for the MCE and FE, respectively, in comparison to an R of 2 given in the AASHTO Specifications).

The summary of the design forces and moments in the fixed and expansion piers are given in Figures 3.1 and 3.2 for the proposed NCHRP Specification and the AASHTO Specifications, respectively. It is noted that the results for the MCE are considerably larger than the corresponding values for the FE, thus, the MCE design forces control the final design.

3.5 Wall Design

The walls are designed considering moment and axial force interaction effects, using the PCA column design program (PCA 1999). The superstructure model is considered pinned at the top of one pier, and free to move longitudinally at the top of the expansion pier. This results in moments on the fixed pier being larger than those on the expansion pier.

When designed according to the AASHTO Specifications, both piers are designed to have the same wall thickness (26 inches). The vertical reinforcing for the fixed pier is 2.27 times heavier (46 # 9) than the expansion pier (46 #6) due to the higher moments. Transverse reinforcement is similar ( #6 @ 12”) for both piers.

When designed according to the NCHRP provisions, the fixed pier wall thickness is 10” thicker than the wall in the expansion pier (36” vs. 26”), and the vertical reinforcing for the fixed pier is almost 2.5 times more than of the expansion pier (2.5% vs 1.01%) due to the higher moments. For the fixed pier the transverse reinforcement in the strong direction is 1.55 times than that of the expansion piers (#4 and #5 at 5” spacing, respectively). The transverse reinforcing steel used in the weak direction at the plastic hinge regions (about 6’ from the bottom of the wall) is extensive. For the fixed bent, the first layer consists of 82 legs #5 rebars (anti-buckling reinforcement) while the second layer consists of 41 legs #4 rebars with 4” spacing used between the layers. For the expansion bent, the first layer consists of 29 legs #4 rebars (anti-buckling
reinforcement) while the second layer consists of 19 legs #3 rebars with 4” spacing used between
the layers.

Figures 3.5-6 and Figures 3.7-8 provide design detail summaries for the walls at the fixed and
extension piers designed according to the AASHTO Specifications and the proposed NCHRP
Specification, respectively.

3.6 Foundation Design

Circular cast-in-place drilled shafts are used as piles. Individual pile stiffness is calculated based
on results from the computer program LPILE. Pile load-deformation plots are produced and pile
properties are based on a secant stiffness calculation. Group effects are only considered in the
design of the foundation according to the proposed NCHRP Specification where pile centers
have to be as close as 3D apart using the P-multiplier method specified by the IDOT Technical
Review Panel (Walsh et al. 2000). The group effects are not used in this design according to the
AASHTO Specifications since pile centers could be reasonably placed at 5D apart. Piles were
design based on the forces obtained from the finite element analysis. PCA Column software was
used to verify the design adequacy of the pile vertical rebars selected.

When designed according to the AASHTO Specifications, eight 18” piles are used for both piers.
However, twelve and six 36” diameter piles are used at the fixed bent and the expansion bent,
respectively, when designed according to the proposed NCHRP Specification.

Figures 3.9, and Figures 3.10-11 provide foundations design detail summaries for the piles, pile
caps, and connections at the fixed and expansion piers designed according to the AASHTO
Specifications and the proposed NCHRP Specification, respectively.
3.7 Material and Cost Comparisons

Figures 3.12 and 3.13 show a detailed comparison of the amount of concrete and steel used in the design of substructures of the Johnson County bridge using the AASHTO Specifications and the proposed NCHRP Specification.

Concrete volume and reinforcing steel weight needed for the substructures designed according to proposed NCHRP provision are 3.16 and 2.39 times the corresponding values obtained using AASHTO Standard Specifications. It is noted that the main contributors to the significant increase in concrete use is the increased size of the foundations used (number of piles, diameter of the piles, and the size of the pile caps) while the primary contributors to the significant increase in using steel are the increased amount of reinforcement in the piles and the transverse reinforcement in the walls when designed according to the NCHRP provisions.

The cost comparisons of the substructure (interior bents) designed according to the AASHTO and the proposed NCHRP Specifications based on estimated quantities and the unit costs obtained from the 2002 Means Heavy Construction Data and IDOT bid tabulations are given below.

<table>
<thead>
<tr>
<th></th>
<th>AASHTO</th>
<th>NCHRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete:</td>
<td>$81,450</td>
<td>$141,480</td>
</tr>
<tr>
<td>Epoxy Coated Rebars:</td>
<td>$28,500</td>
<td>$68,310</td>
</tr>
<tr>
<td>Drilled CIP Piles:</td>
<td>$13,980</td>
<td>$33,860</td>
</tr>
<tr>
<td><strong>Total Cost:</strong></td>
<td><strong>$123,930</strong></td>
<td><strong>$243,650</strong></td>
</tr>
</tbody>
</table>

It is observed that the estimated total construction cost of using the proposed NCHRP Specification is 1.97 times the value obtained using the AASHTO Specifications.
Figure 3.1 – Johnson County Existing Bridge Geometry -- Side View, Pier Elevation, and Superstructure
Figure 3.2 – Design Earthquake Response Spectrum for the Johnson County Bridge
Figure 3.3 – Free Vibration Results (NCHRP MCE Model)
Mode Shape 1 (UX: 69%)  

Mode Shape 2 (UY: 23%)  

Mode Shape 3 (UY: 77%)  

Mode Shape 4 (UX: 18%)  

**Figure 3.4 – Free Vibration Results (AASHTO Model)**

<table>
<thead>
<tr>
<th>MODE</th>
<th>PERIOD</th>
<th>UX</th>
<th>UY</th>
<th>UZ</th>
</tr>
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<tbody>
<tr>
<td>1</td>
<td>0.4035</td>
<td>68.93</td>
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<td>2</td>
<td>0.2898</td>
<td>0.00</td>
<td>22.88</td>
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<tr>
<td>3</td>
<td>0.2067</td>
<td>0.00</td>
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<td>4</td>
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<tr>
<td>5</td>
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<td>100.00</td>
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</table>

With Spring for the MCE
Figure 3.5 – AASHTO Wall Design Details for Fixed Bent
Figure 3.6 – AASHTO Wall Design Details for Expansion Bent
Figure 3.7 – NCHRP Wall Design Details for Fixed Bent
Figure 3.8 – NCHRP Wall Design Details for Expansion Bent
Figure 3.9 – AASHTO Foundation Design Details for Fixed and Expansion Bents
Figure 3.10 – NCHRP Foundation Design Details for Fixed Bent
Figure 3.11 – NCHRP Foundation Design Details for Expansion Bent
Comparison of Concrete Volume

<table>
<thead>
<tr>
<th>Cubic Yards of Concrete</th>
<th>Piles(CY)</th>
<th>Pilecap(CY)</th>
<th>Wall(CY)</th>
<th>Total(CY)</th>
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<tr>
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<td>64</td>
<td>84</td>
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<td>NCHRP</td>
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Figure 3.12 – Concrete Volume Comparison (AASHTO vs. NCHRP)
Figure 3.13 – Reinforcing Steel Weight Comparison (AASHTO vs. NCHRP)
Table 3.1 – NCHRP Seismic Design Force and Displacement Result Summary

<table>
<thead>
<tr>
<th>Displacement</th>
<th>Longitudinal Direction (in)</th>
<th>Transverse Direction (in)</th>
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<td>MCE</td>
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<td>Bottom Fix</td>
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<td>Bottom Exp</td>
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</tr>
<tr>
<td>FE</td>
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<td></td>
</tr>
<tr>
<td>Top</td>
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<td>Bottom Exp</td>
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<table>
<thead>
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<th>MCE</th>
<th>Final Design Forces &amp; Moments (EQ + DL)</th>
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<tr>
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<td>Fix Pier</td>
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<td>Exp Pier</td>
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<table>
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<tr>
<td>Support / Location</td>
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<tr>
<td></td>
<td>Exp Pier</td>
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Table 3.2 – AASHTO Seismic Design Force and Displacement Result Summary

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<th>Max (in)</th>
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<tr>
<td>Top of Fixed Pier</td>
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<td>Bottom of Fixed Pier Pile Cap</td>
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<tr>
<td>Bearing of Expansion Pier</td>
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<td>Bottom of Expansion Pier Pile Cap</td>
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**DIVISION 1-A**

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<table>
<thead>
<tr>
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<td>543</td>
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<td>8</td>
<td>0</td>
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**DIVISION 1-A**

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<thead>
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<th>Axial kips</th>
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</thead>
<tbody>
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<td></td>
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<tr>
<td>Top</td>
<td>105</td>
<td>17</td>
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<td>1810</td>
<td>129</td>
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<td>Exp Pier</td>
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<td></td>
</tr>
<tr>
<td>Top</td>
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<tr>
<td>Bottom</td>
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<td>8</td>
<td>375</td>
<td>43</td>
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4. SEISMIC ANALYSIS AND DESIGN INVESTIGATION OF ST. CLAIR COUNTY BRIDGE

4.1 Introduction

The main purpose of this investigation is to provide the Illinois Department of Transportation (IDOT) with data to assess the impact of the proposed NCHRP Specification on the seismic design of the St. Clair County bridge in southern Illinois. The bridge substructure pier is redesigned for earthquake loads using the proposed NCHRP Specification where the “Operational” performance objective is selected. Since the existing bridge was designed in 1999 according to the AASHTO Specifications, only the strength adequacy of the existing bridge is checked, and substructure cost comparison between the two designs is presented. Detailed computations for design and analysis of this bridge according to the proposed NCHRP Specification and the analysis and design check according to the AASHTO Specifications are given in Appendices C and D, respectively.

4.2 Existing Structure

The existing two-span bridge with equal span lengths of 41.50 m (136 ft). The superstructure consists of a 195 mm (7.7 in.) thick and 20.8 m (64 ft) wide reinforced concrete slab supported on eleven 1.372 m (48 in.) deep plate girders spaced at 1.905 m (6.25 ft). The superstructure is fixed in both longitudinal and transverse directions at the interior bent. However, the superstructure is only fixed in the transverse direction at the abutments (free to move in the longitudinal direction by Type I elastometric expansion bearings). In addition, the bridge has an overall skew of 26.53° at the abutments and interior bent. The abutments are non-integral and are supported by two rows of piles arranged in nine alternate spaces of 2.5 m (8.2 ft). The front row piles are battered. The 4.7 m (15.4 ft) long wing walls are laid along the skew.

The interior bent consists of five tapered reinforced concrete columns with cross-sectional dimensions of 1.75 m (5.74 ft) wide at the top and 1.125 m (3.69 ft) wide at the bottom, and 0.9m (2.95 ft) thick along the column height supported on 2.9 m (9.5 ft) wide, 10.195 m (33.5 ft)
long, and 0.9 m (2.95 ft.) thick footing with three rows of H plies (36 HP 12x53 piles) of which two rows are battered. Existing bridge geometry is shown in Figures 4.1 and 4.2.

4.3 SAP Modeling and Basic Seismic Parameters

The bridge is modeled using SAP 2000 beam elements. The model is a “stick figure” model, where the stiffnesses and masses of the members of the superstructure and substructure, are converted into equivalent beam elements. The foundation of the bridge is modeled using springs, whose stiffnesses are calculated based on the pile stiffnesses. The existing foundation configuration is used in the AASHTO model. However, the foundation model used in the NCHRP model requires iterations since the piles are not designed initially and the spring stiffnesses, and hence the finite element analysis, has to be updated throughout the design phase. Steel piles are used for the analysis. Concrete with a compressive strength of 24 MPa (3500 psi) and reinforcing steel with yield strength of 400 MPa (58,000 psi) are used.

According to the proposed NCHRP Specifications and with the provided geotechnical information at the bridge site (the soil profile used at the bridge site was produced using the available boring logs is given in Appendix G), the St. Clair Country Bridge site is classified as Site Class “D” with Seismic Hazard Level of IV where Seismic Design and Analysis Procedure (SDAP) “D” and Seismic Detailing Requirements (SDR) of 6 are used to obtain the “Operational” performance objective. No potential for liquefaction exists at the given site. The Elastic Response Spectrum Method is required. Thus, the SAP model is analyzed using modal analysis, and then the response spectrum procedure is used to distribute the required seismic forces and displacements throughout the structure. The structure is analyzed independently in both the longitudinal and transverse directions where an adequate number of modes is included in each direction to obtain at least 90% mass participation.

According to the AASHTO Specifications, Division 1A, Seismic Performance Category (SPC) of “B” is used with the Soil Profile Type III with Site Coefficient of S = 1.5 and the Acceleration Coefficient of A = 0.1125 g.

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For the St. Clair County Bridge, design earthquake response spectra for both NCHRP 100-year and 2500-year events (Frequent Earthquake and Maximum Considered Earthquake, respectively) are compared with the AASHTO design earthquake response spectrum (500-year event) in Figure 4.3. For NCHRP and AASHTO bridge models, the results of free vibration analysis are summarized in Figures 4.4 and 4.5, respectively, where the main mode shapes are shown.

### 4.4 Substructure Design Forces

To obtain the design member forces according to the proposed NCHRP Specification, 100% earthquake forces in one direction are combined with 40% earthquake forces in the other direction (instead of 30% used in Division 1-A of the AASHTO Specifications), and then they are reduced by the appropriate Response Modification factors (R-factors) which depend not only the type of substructure element (as in the AASHTO Specifications) but also on the performance level, SDAP, and period of the structure (e.g., R of 1.5 and 0.849 are used in the multi column bent for the MCE and FE, respectively, in comparison to an R of 5 given in the AASHTO Specification).

The summary of the design forces and moments in the interior bent are given in Tables 4.1 and 4.2 for the proposed NCHRP Specification and the AASHTO Specifications, respectively. It is noted that the results for the MCE are considerably larger than the corresponding values for the FE, thus, the MCE design forces control the final design.

### 4.5 Reinforced Concrete Column Design and Connections

The reinforced concrete columns in the interior bent are designed considering moment and axial force interaction effects, using the PCA column design program. The reinforcing steel ratios used in the existing columns are 0.92% and 1.44% at the top and bottom of the columns, respectively, are found adequate to resist the AASHTO design moments and axial forces. The column shear capacities are found to be adequate with use of minimum shear reinforcement, since the factored shear forces are less than twice the provided design capacities.
When designed according to the proposed NCHRP Specification, the column dimensions need to be increased considerably at the bottom to 56”x 35” and slightly at the top to 70”x 35” and the reinforcing steel ratios increase to 2.64% at the top and 3.31% at the top. Anti-buckling steel (No. 6 at 4 in. spacing) governs the design of transverse steel in plastic hinge regions.

Column to cap beam and column to crash wall connections at the interior fixed bent are detailed according to the NCHRP provisions. No special connection detailing is required for this design under AASHTO Division I-A. Standard splices, hooks, tension development, etc. are detailed as required by normal design practice. Figures 4.6a-b show the overall geometry and the design details for the interior bent designed according to the proposed NCHRP Specification.

4.6 Foundation Design

In the existing structure, 36 HP 12 x 53 are used in three rows (24 of which are battered). Approximate length of each pile in place is 78’. The piles are analyzed using the combined moment and axial force obtained from a combination of the LPILE results and the response spectrum analysis. The worst case pile is determined to be loaded approximately 17% over the design capacity from the AASHTO interaction equation. Since this number is not unreasonably high, the existing pile configuration is used in the cost comparisons. The abutments are restrained in the transverse direction and free in the longitudinal direction.

The substructure foundation design was investigated for several abutment lateral-resisting options (roller, pin, springs with soil resistance, springs without soil resistance in transverse direction) for the MCE per request from the TRP. It was found that it was more economical not to rely on the abutments to resist the earthquake load in the transverse direction. Thus, the abutments were idealized as rollers in the SAP model (the results were presented to TRP in the June 2003 meeting).

For the NCHRP design, five rows of driven vertical steel piles (HP 14 x 117) spaced at 6.25 ft. are found necessary to resist the design forces (105 piles total) as shown in Figure 4.7. The footing length and width are increased to 131.25 ft and 31.25 ft. Individual pile stiffness is
calculated based on results from the computer program LPILE. Pile load-deformation plots are produced and pile properties are based on a secant stiffness calculation. The group effects are not used in either AASHTO or NCHRP designs since pile spacing is greater than 5d.

4.7 Material and Cost Comparisons

Figures 4.8, 4.9, and 4.10 show a detailed comparison of the amount of concrete, steel and the total length of driven steel piles used in the design of substructures of the St. Clair County Bridge using the AASHTO Specifications and the proposed NCHRP Specification.

Concrete volume and reinforcing steel weight needed for the substructures designed according to the proposed NCHRP Specification are 3.65 and 4.3 times the corresponding values obtained using the AASHTO Specifications. It is noted that the main contributors to the significant increase in concrete use is the increased size of the footing while the primary contributors for using more reinforcing steel are the increased amount of reinforcement in the footing and the columns.

The cost comparisons based on estimated quantities and the unit costs obtained from the 2002 Means Heavy Construction Data and IDOT bid tabulations of the substructure (interior bent) designed according to the AASHTO and the proposed NCHRP Specifications are given below.

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<th>NCHRP</th>
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</thead>
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<td>Concrete:</td>
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<td>Epoxy Coated Rebars:</td>
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<td>HP Piles:</td>
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<td><strong>Total Cost:</strong></td>
<td><strong>$168,020</strong></td>
<td><strong>$879,100</strong></td>
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It is observed that the estimated NCHRP substructure interior bent construction cost is 5.23 times more than the value obtained using the AASHTO Specifications.
Figure 4.1 – St. Clair County Existing Bridge Geometry -- Side View and Superstructure
Figure 4.2 – St. Clair County Existing Bridge Geometry – Interior Bent
Figure 4.3 – Design Earthquake Response Spectrum for the St. Clair County Bridge
<table>
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100.00 100.00 99.99

Figure 4.4 – Free Vibration Results (NCHRP MCE Model)
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<td><strong>100.00</strong></td>
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</table>
Figure 4.6a – NCHRP Design Details for Interior Bent

9 # 5 at each side
9 # 5 Clamp

16 # 5 @ 3 in
Ah= 6 # 9

8 # 5 at each side
11 # 5 @ 3 in

Half of the Maximum Cross section Dimension=35 in

Ah= 6 # 9

Plastic hinge zone
36 #6 @ 4 in

SECTION A-A

42 # 11

104 # 11

103 # 9

HP 14X117
Figure 4.6b – NCHRP Design Details for Interior Bent (cont.)
Figure 4.7 – NCHRP Interior Bent Footing Geometry (Piles Used: 105 HP 14x117)
Figure 4.8 – Concrete Volume Comparison (AASHTO vs. NCHRP)
Figure 4.9 – Reinforcing Steel Weight Comparison (AASHTO vs. NCHRP)
Comparison of Piles Length

Figure 4.10 – Pile Length Comparison (AASHTO vs. NCHRP)
Table 4.1 – NCHRP Seismic Design Force Result Summary

<table>
<thead>
<tr>
<th>NCHRP - MCE</th>
<th>Final Design Forces and Moments (EQ+DL) By R=1.5</th>
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<td>Shear 2  Shear 3  Moment 2  Moment 3  Axial</td>
<td>Shear 2  Shear 3  Moment 2  Moment 3  Axial</td>
</tr>
<tr>
<td></td>
<td>kips  kips  kip-ft  kip-ft  kips</td>
<td>kips  kips  kip-ft  kip-ft  kips</td>
</tr>
<tr>
<td>Location</td>
<td>top</td>
<td>bottom</td>
</tr>
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<td>231  406  3369  996  1072</td>
<td>304  307  2554  1311  936</td>
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<tr>
<td></td>
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<td>307  317  1299  5083  964</td>
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<tr>
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<td>304  399  3709  1314  726</td>
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<tr>
<td></td>
<td>233  540  1681  3860  822</td>
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<tr>
<td></td>
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<table>
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<tr>
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<td>Shear 2  Shear 3  Moment 2  Moment 3  Axial</td>
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<td>18  51  159  309  570</td>
<td>36  58  184  608  574</td>
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<td>18  53  158  309  545</td>
<td>36  60  183  608  545</td>
</tr>
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</table>

**Longitudinal Combination** - 100% load in X direction + 40% load in Y direction + Dead load

**Transverse Combination** - 100% load in Y direction + 40% load in X direction + Dead load
Table 4.2 – AASHTO Seismic Design Force Result Summary

<table>
<thead>
<tr>
<th>AASHTO Location</th>
<th>Load Combination 1 - (1.0<em>EQL + 0.3</em>EQT)+DL</th>
<th>Load Combination 2 - (0.3<em>EQL + 1.0</em>EQT)+DL</th>
</tr>
</thead>
<tbody>
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<td>Longitudinal (R = 3)</td>
<td>Transverse (R = 5)</td>
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<td>54</td>
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</tr>
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<td>55</td>
</tr>
<tr>
<td></td>
<td>bottom</td>
<td>55</td>
</tr>
</tbody>
</table>

**R = 3.0 (Single Columns); (Substructure)**

**R = 5.0 (Multiple column bent); (Substructure)**

**COMD1** - 100% load in X (longitudinal) direction + 30% load in Y (transverse) direction + Dead load

**COMD2** - 100% load in Y (transverse) direction + 30% load in X (longitudinal) direction + Dead load
5. SEISMIC ANALYSIS AND DESIGN INVESTIGATION OF
PULASKI COUNTY BRIDGE

5.1 Introduction

The main purpose of this investigation is to provide the Illinois Department of Transportation (IDOT) with adequate information to assess the impact of the proposed NCHRP Specification on the seismic design of the Pulaski County bridge in Southern Illinois. The bridge substructure piers are redesigned for earthquake loads using both the seismic provisions given in Division 1A of the AASHTO Specifications and the proposed NCHRP Specification where the “Operational” performance objective is specified. Detailed computations for design and analysis of this bridge according to the proposed NCHRP Specifications and the AASHTO Specification are given in Appendices E and F, respectively.

5.2 Existing Structure

The existing four-span bridge with span lengths of 40’- 8”, 73’- 3”, 73’ -3”, and 40’- 8” was designed in 1965. Its superstructure consists of a 6.5” thick (30’ wide) reinforced concrete slab supported on five W 33x130 steel girders spaced at 6’- 6” (o.c.). The superstructure is fixed in both longitudinal and transverse directions at the second pier by bolsters, and it is attached by rockers to the other piers and the non-integral abutments allowing it to move freely in the longitudinal direction while it is restrained in the transverse direction. All interior bents consist of hammerhead piers (15’- 6” wide at the bottom, 12’- 00” wide at the narrow section at the top, 2’-6” thick reinforced concrete walls) supported on 2’-3’’ thick footings and two rows of battered H piles. Existing bridge geometry is shown in Figures 5.1-3.

5.3 SAP Modeling and Basic Seismic Parameters

The bridge is modeled using SAP 2000 beam elements. The model is a “stick figure” model, where the stiffnesses and masses of the members of the superstructure and substructure are converted into equivalent beam elements. The foundation of the bridge is modeled using
springs, whose stiffnesses are calculated based on the pile stiffnesses. Note that the foundation model requires iteration, since the piles are not designed initially and the spring stiffnesses, and hence the finite element analysis, has to be updated throughout the design phase. Steel piles are used for analysis.

Several individual models of the hammerhead piers were considered to verify that the stick model is capturing the dynamic behavior of piers. After some finite element studies of varying complexity, it was determined that the stick model would properly model the piers as long as the rotational inertia of the deck was considered in the model.

According to the proposed NCHRP Specification and with the provided geotechnical information at the bridge site (see Appendix G), the Pulaski Country Bridge site is classified as Site Class “D” with Seismic Hazard Level of IV where Seismic Design and Analysis Procedure (SDAP) “D” and Seismic Detailing Requirements (SDR) of 6 are used to obtain the “Operational” performance objective. No potential for liquefaction exists at the given site. The Elastic Response Spectrum Method is required. Thus, the SAP model is analyzed using modal analysis, and then the response spectrum procedure is used to distribute the required seismic forces and displacements throughout the structure. The structure is analyzed independently in both the longitudinal and transverse directions where an adequate number of modes is included in each direction to obtain at least 90% mass participation.

According to the AASHTO Specifications, Division 1A, Seismic Performance Category (SPC) of “C” is selected where the Soil Profile Type III with Site Coefficient of \( S = 1.5 \) and the Acceleration Coefficient of \( A = 0.22g \) are used.

For the Pulaski County Bridge, design earthquake response spectra for both NCHRP 100-year and 2500-year events (Frequent Earthquake and Maximum Considered Earthquake, respectively) are compared with the AASHTO design earthquake response spectrum (500-year event) in Figure 5.4.
For NCHRP and AASHTO bridge models, the results of free vibration analysis are summarized in Figures 5.5 and 5.6, respectively, where the significant mode shapes are shown.

### 5.4 Substructure Design Forces

To obtain the design member forces according to the proposed NCHRP Specification, 100% earthquake forces in one direction are combined with 40% earthquake forces in the other direction (instead of 30% used in Division 1-A of the AASHTO Specifications), and then they are reduced by the appropriate Response Modification factors (R-factors) which depend not only the type of substructure element (as in the AASHTO Specifications) but also on the performance level, SDAP, and period of the structure. For example, R of 1 and 1.5 are used in the piers acting as a wall pier in the strong direction and a single column in the week direction, respectively, for the MCE, and R of 0.76 is used for the FE in comparison to R of 2 and 1.0 used for flexure and shear, respectively, in the AASHTO Specifications.

After several trials due to the high intensity of the seismic design forces in fixed bent in case of NCHRP design, it was decided to redesign the other two expansion bents as fixed bents. The summary of the design forces and moments in the three piers are shown in Table 5.1 and 5.2 for the proposed NCHRP Specification and the AASHTO Specifications, respectively. It was found that the design forces obtained for the MCE were considerably larger than the corresponding values for the FE, thus, the substructure design is governed by the MCE design forces (see Appendix E).

### 5.5 Wall Design in Hammerhead Piers

The walls are designed considering moment and axial force interaction effects using the PCA column design program. The superstructure model is considered pinned at the top of fixed pier, and free to move longitudinally at the top of the expansion piers.

When designed according to the AASHTO specification, fixed and expansion piers are designed to have the same wall thickness (30” and 48”, respectively), however, the bottom dimension of
the hammerhead pier at the fixed bent is increased by 6” (from 15’ – 6” to 16’). For ease of comparison, the same pier dimensions are used in the NCHRP design for the interior bents as used in the AASHTO design.

The vertical reinforcing steel in the middle pier (bent 2) designed according to NCHRP provisions (196 #11) is 1.44 times the steel used in other piers (136 #11 in bents 1 or 3) where the transverse reinforcement is similar for both all interior bents. The transverse steel in the weak direction at the plastic hinge regions at the bottom of the piers (70 legs #5 rebars, used as anti-buckling reinforcement) is extensive compared to the ones used at the top of the piers (24 legs of #4 rebars).

When designed according to the AASHTO specifications, the vertical reinforcing used in the fixed pier is almost similar to the one used in the expansion piers (44 #11 vs 42 #11, respectively) where the transverse reinforcement is similar for both types of bents.

Figures 5.7 a-b and Figures 5.8 a-b provide design detail summaries for the hammerhead walls at the fixed and expansion piers designed according to the NCHRP provisions and the AASHTO specifications, respectively, where special detailing required for the wall to footing connections by the NCHRP provision are also shown.

### 5.6 Foundation Design

For the NCHRP design, 40 driven vertical steel HP 14 x 117 piles (in four rows spaced at 70 in. o.c.) are found necessary to resist the design forces at each fixed pier (bents 1, 2, and 3 --120 piles total). For the AASHTO design, 15 HP 14 x 89 piles (in three rows of 5) and 10 HP 14 x 89 piles (in two rows of 5) are used in the fixed pier (bent 2) and expansion piers (bents 1 and 3), respectively (35 plies total), where all piles are spaced at 6 ft. (o.c.).

Overall dimensions of the footings for NCHRP and AASHTO designs are given in Figures 5.7c and Figures 5.8 c-d, respectively. In both designs, the footing length and width are increased to accommodate the piles. The group effects are not used in either AASHTO or NCHRP designs.
since pile spacing is greater than 5d (Walsh et al. 2000). Individual pile stiffness is calculated based on results from the computer program LPILE. Pile load-deformation plots are produced and pile properties are based on a secant stiffness calculation.

5.7 Material and Cost Comparisons

Figures 5.9, 5.10 and 5.11 show a detailed comparison of the amount of concrete, steel and the total length of driven steel piles used in the design of substructures of the Pulaski County Bridge using the AASHTO Specifications and the proposed NCHRP Specification.

Concrete volume and reinforcing steel weight needed for the substructures designed according to the proposed NCHRP Specification are 4.88 and 5.61 times the corresponding values obtained using AASHTO Specifications. It is noted that the main contributors to the significant increase in concrete use is the increased size of the footing while the primary contributors for using more reinforcing steel are the increased amount of reinforcement in the walls.

The cost comparisons of the substructure (interior bents) designed according to the AASHTO and the proposed NCHRP Specifications based on estimated quantities and the unit costs obtained from the 2002 Means Heavy Construction Data and IDOT bid tabulations are given below.

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<thead>
<tr>
<th>Item</th>
<th>AASHTO</th>
<th>NCHRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete:</td>
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<td>$359,450</td>
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<tr>
<td>Epoxy Coated Rebars:</td>
<td>$21,700</td>
<td>$121,730</td>
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<tr>
<td>HP Piles:</td>
<td>$69,850</td>
<td>$417,240</td>
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<tr>
<td><strong>Total Cost:</strong></td>
<td><strong>$165,340</strong></td>
<td><strong>$898,420</strong></td>
</tr>
</tbody>
</table>

It is noted that the estimated total construction cost of using the proposed NCHRP Specification is 5.43 times the value obtained using the AASHTO Specifications.
Figure 5.1 – Existing Bridge Geometry – Side View

Figure 5.2 – Existing Bridge Geometry – Superstructure
Figure 5.3 – Existing Bridge Geometry – Interior Bent
Figure 5.4 – Design Earthquake Response Spectrum for Pulaski County Bridge
### Modal Participating Mass Ratios

With Spring for MCE

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<th>Mode</th>
<th>Period</th>
<th>INDIVIDUAL MODE(%)</th>
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</thead>
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Figure 5.5 – Free Vibration Results (NCHRP MCE Model)
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<td>Total</td>
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<td>89.74</td>
<td>96.87</td>
<td>97.31</td>
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</table>

Mode Shape 1 (UX - 68%)

Mode Shape 2 (UY - 17%)

Mode Shape 8 (UY - 13%)

Mode Shape 9 (UY - 43%)
Figure 5.6 – Free Vibration Results (AASHTO Model)

Figure 5.7a – NCHRP Wall Design Details (Bent 2)
Figure 5.7b – NCHRP Wall Design Details (Bents 1 & 3)
PILE DATA
Type: HP 14X117
Pile Length: 60'
Embedment Length into pile cap: 12"

Figure 5.7c – NCHRP Footing Geometry (All Bents)
Figure 5.8a – AASHTO Wall Design Details (Fixed Bent)
Figure 5.8b – AASHTO Wall Design Details (Expansion Bent)
PILE DATA
Type: HP 14X89
Pile Length: 45'
Embendment Length into pile cap: 12"

Figure 5.8c – AASHTO Footing Geometry (Fixed Bent)
**PILE DATA**

Type: HP 14X89  
Pile Length: 45'  
Embedment Length into pile cap: 12"

Figure 5.8d – AASHTO Footing Geometry (Expansion Bent)
Figure 5.9 – Concrete Volume Comparison (AASHTO vs. NCHRP)
Comparison of Steel Weight

Figure 5.10 – Reinforcing Steel Weight Comparison (AASHTO vs. NCHRP)
Figure 5.11 – Pile Length Comparison (AASHTO vs. NCHRP)
Table 5.1 – NCHRP Seismic Design Force Result Summary

<table>
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<th>MCE</th>
<th>Final Design Forces &amp; Moments [(EQ/ R)+ DL]</th>
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</thead>
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<tr>
<td></td>
<td>Transverse</td>
</tr>
<tr>
<td></td>
<td>Longitudinal</td>
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<td>Top</td>
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**Longitudinal Combination** - 100% load in X direction + 40% load in Y direction + Dead load

**Transverse Combination** - 100% load in Y direction + 40% load in X direction + Dead load
<table>
<thead>
<tr>
<th>Pier</th>
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<td></td>
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<td>Kip</td>
<td>Kip-f</td>
<td>Kip-f</td>
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</tr>
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<td>-514.8</td>
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<td>1601.5</td>
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</table>

COMD1 - 100% load in X (longitudinal) direction + 30% load in Y (transverse) direction + Dead load
COMD2 - 100% load in Y (transverse) direction + 30% load in X (longitudinal) direction + Dead load
6. GROUND MOTION INVESTIGATION AT SOUTHERN ILLINOIS BRIDGE SITES

6.1 Introduction

Adoption of the proposed NCHRP Specification may result in a significant increase in the level of earthquake forces for bridges located in the Central United States as opposed to the current AASHTO Specifications. Comparison of the design earthquake response spectrum for the 2500-year event, 3% probability of exceedance in 75 years (the Maximum Considered Earthquake which controlled the NCHRP design forces and moments in substructure members in all three bridges investigated), indicates that the peak values of the spectrum are significantly higher (2.9 to 5.7 times) than the corresponding values obtained from the AASHTO design earthquake response spectrum corresponding to a 475 year event (15% probability of exceedance in 75 years) at the bridge sites given in three southern Illinois counties (Johnson County, St. Clair County, and Pulaski County).

The purpose of this investigation is to obtain acceleration coefficients at the four sites in southern Illinois for both bedrock and ground surface levels using existing source paths and site models, and attenuation relationships supplemented by new developments to produce synthetic time histories, response spectra values at frequencies of interest, uniform hazard spectra, and site amplification factors.

To accomplish the objectives of this study, as stated in Section 2.3, two major tasks are identified: 1) development of horizontal bedrock motion, and 2) site response analysis. Detailed descriptions of these tasks are given in the Sections 6.2 and 6.3, and the summary of the results is presented in Section 6.4.

6.2 Task 1 - Development of Horizontal Bedrock Motions

Task 1 consists of development of procedures to generate horizontal bedrock motions for four bridge sites in southern Illinois based on the latest available information. This task includes three sub-tasks as briefly described below.
**Task 1A - Identification of Seismic Source Zones:** A detailed literature search was performed to identify seismic sources that will be used to characterize seismicity in the New Madrid region and any other potential seismic sources in the region including the Wabash region. The research publication by Van Arsdale and Johnston (1999) was used to define seismic sources. In addition, the report by Toro and Silva (2001) was used to identify seismic sources and to quantify the rates of occurrence and maximum magnitude for various sources. Recent and ongoing research under the auspices of the Mid-America Earthquake Center (MAEC) was considered with specific attention to seismic sources and parameters that are relevant to Illinois.

    We believe that the parameters used by Toro and Silva (2001) and Van Arsdale and Johnston (1999) are more suitable for the study region (see Figure 6.1). The focus of this project was the State of Illinois; therefore, attentions were focused on seismic sources and parameters that are prevalent to Illinois. Figure 6.1 shows the New Madrid seismic zone and other seismic sources used by Toro Silva (2001) and also used in this study. Figure 6.2 shows the background seismic sources used by Toro and Silva (2001) which were also included in this study. Figure 6.3 shows the three zones used in this study to represent the Wabash seismic zone (adopted from Toro Silva, 2001).

**Task 1B - Evaluation of Attenuation Relationships and Occurrence Rates:** Seismic attenuation relationships and occurrence rates are key input parameters for generation of bedrock ground motions. The following attenuation equations were used using equal weights: Atkinson and Boore (1995); Toro et al. (1997); Frankel et al. (1996); Pezeshk, et al. (1998); and Campbell and Bozorgnia (2003).

**Task 1C - Generation of Artificial Earthquakes and Bedrock Motions:** In this task, maps of appropriate scales were used to determine the seismic hazard and the probabilistic consistent magnitude and epicentral distance. The consistent magnitude and epicentral distance as well as attenuation relationships and occurrence rates determined in Task 1B were used to generate artificial earthquakes with a corresponding bedrock time history at each of the four sites considered. The computer program SMSIM (Boore 2003) was used to generate artificial time
histories for this study. Spectral values were generated for 5 Hz (0.2 seconds), 1 Hz (1 second), and peak rock accelerations for return periods of 1000 and 2500 years at the selected four sites.

6.3 Task 2 - Site Response Analysis

Task 2 consists of generating peak ground accelerations, 1-second response spectrum accelerations, and 0.2-second response spectrum accelerations for the selected four sites by transmitting the seismic waves at the bedrock through soil. This task includes two sub-tasks as briefly described below.

Task 2A – Site Studied: Site-specific studies were performed for four bridge sites in Illinois:

- Bridge 1. Located in Johnson County (37.433°N, 88.869°W)
- Bridge 2. Located in St. Clair County (38.588°N, 89.912°W)
- Bridge 3. Located in Pulaski County (37.200°N, 89.152°W)
- Bridge 4. Madison County bridge related in Pulaski County (37.283°N, 89.150°W)

Locations of these bridge sites are shown in Figure 6.4 and Figure 6.5.

Task 2B – Determination of Acceleration Coefficients: Site response analyses were performed to obtain representative response spectra at the ground surface based on the propagated NEHRP B-C boundary time histories and soil properties obtained from soil boring information at each bridge. The shear wave velocities for the upper soil strata were obtained from the standard penetration test (SPT) using the procedure outlined in Pezeshk, et al. (1998) and Wei, et al. (1999). The shear wave velocities for the remaining depth of soil/rock to the bottom of the soil boring were determined based on the shear wave velocity profile as outlined in Pezeshk, et al. (1998) and Romero and Rix (2001). The shear modulus, $G_{\text{max}}$, corresponding to a very small shear strain (lower than about $3 \times 10^{-4}$ percent) was determined based on the in-situ shear wave velocities. The shear modulus degradation curves and damping ratio curves used were taken from Pezeshk, et al. (1998).

To determine a better estimate of site characterization for Bridges 3 and 4, Mr. Bob Bauer of Illinois Geological Survey was contacted. Mr. Bauer identified the Weldon Wells borehole (SS#
13430) in Section 15 of T15S, R1W closest to Bridge 3 and Bridge 4 sites. For Bridges 3 and 4, the boring logs provided at the bridge site were used for the upper strata. The Weldon Wells borehole is used for the remaining depth of soil/rock to the bottom of the soil boring at the bridge sites.

Once the required input data were collected, site response analyses were performed for the four selected sites using commercial site response software. Utilizing these data, the program SHAKE91 (Idriss and Sun, 1992) was used to conduct equivalent linear seismic response analyses of the assumed horizontally layered soil deposits. The information produced by SHAKE91 included the relevant data for this study which consisted of the response spectra at the surface for the 0.2 second and 1.0 second spectral accelerations and the surface time histories.

The spectral accelerations at 0.2 second and 1.0 second were determined from the appropriate response spectrum based on the attenuation equations used in the probabilistic seismic hazard analysis (PSHA). Using these values, the smooth, uniform hazard response spectrum at the ground surface was generated for design ground motions with 1,000 and 2,500 year-return periods and damping of 5 percent.

### 6.4 Results

Table 6.1 and Figures 6.4 and 6.5 provide three sets of acceleration coefficients: (1) USGS 1996 acceleration coefficients, (2) Toro and Silva (2001) acceleration coefficients, and (3) acceleration coefficients from this study. Also, the site amplification factors for 0.2-second and 1.0-second spectral for the MCE (2% probability of exceedance in 50 years) are provided in Table 6.1. It is observed that the USGS 1996 and this study have comparable acceleration coefficients when a 2500-year return period is considered, with a slightly lower acceleration coefficient found in the study for Bridges 1, 3, and 4 and slightly higher PGA found in this study for Bridge 2. Furthermore, Toro and Silva acceleration coefficients, which are for rock sites, are much smaller than the other two studies. In general, this study results in higher acceleration coefficients than USGS 1996 acceleration coefficients for a 1000-year return period ground motions. Per request from the Technical Review Panel, the latest USGS acceleration coefficients made available in 2002 are also given in Table 6.1. However, there has been no attempt to compare the results of
this study with the 2002 USGS acceleration coefficients because the seismic response spectra in the proposed NCHRP Specification are based on the 1996 USGS hazard maps (not 2002).
Figure 6.1 – The New Madrid Seismic Zone and Other Seismic Sources Used by Toro Silva (2001)
Figure 6.2 – Background Seismic Sources (Adopted from Toro and Silva, 2001).
Figure 6.3 – Three Zones Used to Represent Wabash Seismic Zone (Adopted from Toro and Silva, 2001).
Figure 6.4 – Locations of bridges studied and the corresponding 0.2-second, 1-second spectral accelerations, and PGA comparisons with the USGS 1996 hazard maps, Toro and Silva (2001) for a return period of 2500 years.
Figure 6.5 – Locations of bridges studied and the corresponding 0.2-second, 1-second spectral accelerations, and PGA comparisons with the USGS 1996 hazard maps for a return period of 1000 years.
Table 6.1 – Summary of Results of the Ground Motion Study at Four Southern Illinois Bridge Sites

<table>
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<tr>
<th>Return Period (Years)</th>
<th>Coordinates</th>
<th>National Hazard Maps</th>
<th>Toro and Silva (2001)</th>
<th>This Study</th>
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<td></td>
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<td>Longitude</td>
<td>PGA 0.2 sec 1 sec PGA 0.2 sec 1 sec PGA 0.2 sec 1 sec</td>
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<td>0.530 0.909 0.210</td>
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<td>0.530 0.909 0.210</td>
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7. SUMMARY AND CONCLUSIONS

7.1 Summary

The technical basis of the seismic provisions within both the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for Highway Bridges* (AASHTO 1996) and the *AASHTO LRFD Bridge Design Specifications* (AASHTO 1998) are essentially the same as that of the seismic design guidelines published over two decades ago by the Applied Technology Council (ATC 1981). In 1998, the National Cooperative Highway Research Program initiated a study to develop a new set of seismic design provisions for highway bridges (NCHRP Project 12-49) for possible incorporation into the future AASHTO LRFD bridge design specifications. The recommended specification provided the technical basis for a stand-alone set of provisions entitled “Recommended LRFD Guidelines for the Seismic Design of Highway Bridges” developed by a joint venture of the ATC and the Multidisciplinary Center for Earthquake Engineering Research, MCEER (ATC/MCEER 2002). The primary objective of the Guidelines was to develop seismic design provisions that reflect the latest research findings, design philosophies, and design approaches.

In comparison with the AASHTO Specifications, the proposed NCHRP Specification has adopted a dual-criteria strategy of two-level design earthquakes. One based on an expected or Frequent Earthquake (FE) with a 50% probability of exceedance in the 75-year design life of a bridge (with a return period of 108 years), and the second based on a rare or Maximum Considered Earthquake (MCE) with a 3% probability of exceedance in 75 years (with an approximate return period of 2500 years). Current AASHTO Specifications consider a single-level earthquake with a 10% probability of exceedance in 50 years (with a return period of 475 years). Also, the proposed NCHRP Specification defines two seismic performance levels in terms of the anticipated performance of the bridge in the rare earthquake event: Life Safety – which means the bridge should not collapse (partial or complete replacement may be required) and serious personal injury or loss of life should be avoided, and Operational – which means the bridge will be functional immediately after a rare earthquake. New US Geological Survey (USGS) maps developed by A.D. Frankel and E.V. Leyendecker (2000) are used in the proposed
NCHRP Specification instead of the USGS maps developed for the 1988 NEHRP provisions used in the current AASHTO specifications (AASHTO 1996 and 1998). The seismic design procedure used in the proposed NCHRP Specification is summarized in Chapter 2.

As a part of a research project sponsored by Illinois Transportation Research Center (ITRC), the substructures of four existing bridges in southern Illinois were redesigned according to the proposed NCHRP Specification (ATC/MCEER 2002) and the AASHTO Specifications (AASHTO 1996). The results of the design of first three bridges located in Johnson County, St. Clair County, and Pulaski County are presented in Chapters 3, 4, and 5, respectively, where the impact of using the new proposed NCHRP Specification on the materials and construction costs are also included. The corresponding engineering computations are given in Appendices A through F. The result of a similar study for the fourth bridge (currently in progress) will be submitted to the project Technical Review Panel (TRP) as an addendum report due to termination of the project effective June 30, 2004, as a result of both the appropriation and re-appropriation of the ITRC not being included in the Illinois Department of Transportation (IDOT) budget for FY 2005.

According to the current AASHTO Specifications, Division 1A, the Seismic Performance Category (SPC) of the Johnson County bridge (originally designed in 1970) and St. Clair County bridge (originally designed in 2000) are specified as “B” where the Acceleration Coefficient (A) of 0.15g and 0.1125g and Soil Profile Type of I and III, Site Coefficient (S) of 1.0 and 1.5 are used, respectively. For the Pulaski County bridge (originally designed in 1965) with Site Soil Profile Type III (S=1.5), the SPC of “C” is selected where A = 0.22g. It is noted that since the St. Clair County bridge was recently designed according to the AASHTO Specification, its substructure members were only checked for strength adequacy.

According to the proposed NCHRP Specification and with the provided geotechnical information, the bridge sites in Johnson County, St. Clair County, and Pulaski County are classified as Site Class “D” with Seismic Hazard Level of IV where Seismic Design and Analysis Procedure (SDAP) “D” and Seismic Detailing Requirements (SDR) of 6 are used to
obtain the “Operational” performance objective. No potential for liquefaction exists at the given sites.

For both AASHTO design and NCHRP design, the bridges are idealized as a “stick figure” model using SAP 2000 beam elements (CSI 1998), where the stiffnesses and masses of the members of the superstructure and substructure are converted into equivalent beam elements. Cracked section properties are used for substructure elements. The foundation of the bridge is modeled using springs, whose stiffnesses are calculated based on the pile stiffnesses. Note that the foundation model requires iteration, since the piles are not designed initially and the spring stiffnesses, and hence the finite element analysis, has to be updated throughout the design phase. The structure is analyzed independently in both the longitudinal and transverse directions where an adequate number of modes are included in each direction to obtain at least 90% mass participation.

The substructures of these bridges consisted of interior bents comprised of reinforced concrete solid walls or multi-column piers, and non-integral reinforced concrete abutments. The foundations consisted of concrete or steel piles. It is noted that for comparison purposes in all Illinois bridges investigated, similar earthquake resisting systems are used in the NCHRP design as in the AASHTO design, as requested by the project TRP. The superstructures of these bridges consisted of reinforced concrete slabs supported on two-, three-, and four-span continuous steel wide flange beams or plate girders.

Since using the proposed NCHRP Specification can result in a significant increase in the level of earthquake forces in southern Illinois (as opposed to the current AASHTO Specifications), the realism of the new USGS accelerations for the four bridge sites in southern Illinois was also evaluated in this study. The acceleration coefficients for both bedrock and ground surface levels were regenerated where at each bridge site by an independent procedure using existing source paths and site models, and attenuation relationships supplemented by new developments to produce synthetic time histories, response spectra values at frequencies of interest, uniform hazard spectra, and site amplification factors. The ground motion acceleration results are presented and compared with the USGS values in Chapter 7.
7.2 Observations and Concluding Remarks

AASHTO seismic forces are uniformly based on a “Life Safety” design philosophy. The NCHRP guidelines are based on a two-tier approach, allowing the bridge owners to choose a “Life Safety” or an “Operational” level of performance, depending on how critical the roadway is in terms of emergency response, economic recovery, and other intangible factors. In this research, as requested by IDOT, the AASHTO Specifications were compared to an “Operational” NCHRP performance level. Thus, it is expected that the seismic forces obtained from the AASHTO Specifications in Illinois bridges would be lower than those obtained from the proposed NCHRP Specification when this approach is used.

Comparison of the design earthquake response spectrum for the MCE, which governs the NCHRP design forces and moments in substructure members in cases investigated, indicates that the peak values of the spectrum are significantly higher (2.9 to 5.7 times) than the corresponding values obtained from the AASHTO design earthquake response spectrum at the bridge sites given in three southern Illinois counties (see Figures 3.2, 4.3, and 5.4). Also, the AASHTO response modification factors for all bridges investigated were significantly larger (2.0 to 3.3 times) than the corresponding values used in the NCHRP designs for the MCE since the “Operational” performance objective was specified for the NCHRP design.

Due to the above reasons, the NCHRP design forces and moments for the substructure elements and foundations were significantly higher than the corresponding values used in AASHTO Specifications (see Tables 3.1, 3.2, 4.1, 4.2, 5.1, and 5.2). Consequently, larger amounts of reinforcing steel and/or concrete were used in the NCHRP wall or multi-column bents where an extensive number of reinforced concrete or steel H piles were needed in the foundations in comparison to the AASHTO design (see Figures 3.12, 3.13, 4.8, 4.9, 4.10, 5.9, 5.10, and 5.11).

More specifically, the concrete volume and reinforcing steel weight required in the NCHRP design are 3.16 and 2.39 times the corresponding values obtained using AASHTO Specifications in the Johnson County bridge while it is estimated that the total cost of the interior bents
designed using the NCHRP Specification is 1.96 times the value obtained using the AASHTO Specifications.

For the St. Clair County Bridge, the concrete volume and reinforcing steel weight needed for the substructures designed according to the proposed NCHRP Specification are 3.65 and 4.30 times the corresponding values obtained using AASHTO Specifications while it is observed that the NCHRP substructure bent design cost is 5.23 times the corresponding value obtained in the AASHTO design.

In the Pulaski County bridge, the concrete volume and reinforcing steel weight needed for the substructures designed according to the proposed NCHRP Specification are 4.88 and 5.61 times the corresponding values obtained using AASHTO Specifications while it is observed that the NCHRP substructure bent design cost is 5.43 times the corresponding value obtained in the AASHTO design. It is noted that the construction cost impact of using the proposed NCHRP Specification is higher in the case of the Pulaski County bridge in comparison with the St. Clair County bridge (5.43 vs 5.23) both with similar the AASHTO Site Soil Profile Type III and the NCHRP Site Class of “D” while the product of the maximum spectrum acceleration coefficient ratio of \( \frac{S_{\text{max (MCE)}}}{S_{\text{max (AASHTO)}}} \) and the response modification factor ratio of \( \frac{R_{\text{AASHTO}}}{R_{\text{MCE}}} \) is also higher in the case of the Pulaski County bridge.

The size of the footings and number of piles needed for the NCHRP designed substructure in this study are obviously impractical to actually incorporate into the final design plans. These were shown for comparison of like foundation types. It is recommended that in practical design the bridge engineer take advantage of the flexibility provided by the proposed NCHRP Specification by using other methods of handling the increased seismic demand on the substructure such as the use of seismic isolation bearings, seismic lock-up devices, or a change in the earthquake resisting system and the foundation type when necessary.

Comparison of the ground motion accelerations obtained in this study with the ones obtained from the USGS maps (1996) indicates that they are comparable when a 2500-year return period is considered. Slightly lower acceleration coefficients were found in the study for the Bridges 1,
3, and 4 and a slightly higher acceleration coefficient was found for Bridge 2 (see Figure 6.4 and Table 6.1 for bridge locations and summary of the ground motion results).

### 7.3 Future Research

For further study of the impact of construction cost on southern Illinois highway bridges using the proposed NCHRP Specification, we propose that several additional cases be considered where the added flexibility provided in the proposed NCHRP Specifications is fully utilized by investigating more innovative methods of handling the increased seismic demands such as using lead rubber seismic isolators (Arab et al. 2004); or changing the earthquake resisting systems such as using more flexible piers to reduce the seismic loads (by increasing the fundamental period of the bridge) or using integral abutments to resist the seismic loads where appropriate (Emami and Panahshahi 2004). Also, additional investigations can be conducted using the 1000-year seismic event as the MCE instead of the 2500-year event used in the proposed NCHRP Specifications. Future research using NCHRP “Life Safety” performance objective could also provide valuable information for cost impact study of nonessential Illinois bridges.

For the ground motion study of the southern Illinois region, another software packages named DEEPSOIL has been recently developed by Professor Hashash of the University of Illinois and his students (Hashash and Park, 2001; Hashash and Park, 2002). The software package DEEPSOIL, which considers the full nonlinear behavior of soil, is based on new research being conducted for the Mid-America Earthquake Center (MAEC). For future ground motion research in southern Illinois, we propose further investigation is done using the computer package DEEPSOIL and compare the results with SHAKE91 results.
REFERENCES


Portland Cement Association (PCA) (1999), *PCACOL: Design and Investigation of Reinforced


Concrete Column Sections, PCA, Illinois.


### Subject File

**Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois**

<table>
<thead>
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<th>Title</th>
<th>By</th>
<th>Date</th>
<th>Checked Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Johnson County Bridge</td>
<td>BE</td>
<td>01/24/03</td>
<td></td>
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</tbody>
</table>

**Appendix A -- Detailed Computations for Seismic Analysis and Design of Johnson County Bridge using Proposed NCHRP Specification**
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Site Class = Medium Stiff Clay

Site Class D MCEER Guide Spec. 3.4.2.1

Type I per AASHTO Spec.

\[ s := 1.0 \] AASHTO Table 3.5.1

\[ A := 0.15 \, g \]

- Design Response Spectrum Development - MCE (MCEER Guide Spec. 3.4.1)

\[ S_s := 1.75 \] 0.2- second period spectral acceleration

\[ S_1 := 0.5061 \] 1- second period spectral acceleration

\[ F_a := 1 \] Site coefficient for short period

\[ F_v := 1.5 \] Site coefficient for long period

\[ S_{DS} := F_a S_s \]

\[ S_{DS} = 1.75 \] Design earthquake response spectral acceleration at short period

\[ S_{D1} := F_v S_1 \]

\[ S_{D1} = 0.759 \] Design earthquake response spectral acceleration at long period

\[ 0.4 S_{DS} = 0.7 \]

\[ T_s := \frac{S_{D1}}{S_{DS}} \] Period at the end of construction design spectral acceleration plateau

\[ T_s = 0.434 \, \text{Sec} \]

\[ T_0 := 0.2 \, T_s \] Period at the beginning of construction design spectral acceleration plateau

\[ T_0 = 0.087 \, \text{Sec} \]

\[ F_a S_s = 1.75 > 0.6 \]

**Seismic Hazard Level IV** (Guide Spec. Table 3.7-2)
- Design Response Spectrum Development - FE (MCEER Guide Spec. 3.4.1)

\[ S_s_{\text{fe}} := 0.1033 \]  
0.2-second period spectral acceleration

\[ S_1_{\text{fe}} := 0.0205 \]  
1-second period spectral acceleration

\[ F_a_{\text{fe}} := 1.6 \]  
Site coefficient for short period

\[ F_v_{\text{fe}} := 2.4 \]  
Site coefficient for long period

\[ S_{DS_{\text{fe}}} := F_a_{\text{fe}} S_s_{\text{fe}} \]

\[ S_{DS_{\text{fe}}} = 0.165 \]  
Design earthquake response spectral acceleration at short period

\[ S_{D1_{\text{fe}}} := F_v_{\text{fe}} S_1_{\text{fe}} \]

\[ S_{D1_{\text{fe}}} = 0.049 \]  
Design earthquake response spectral acceleration at long period

\[ 0.4 S_{DS_{\text{fe}}} = 0.066 \]

\[ T_{s_{\text{fe}}} := \frac{S_{D1_{\text{fe}}}}{S_{DS_{\text{fe}}}} \]  
Period at the end of construction design spectral acceleration plateau

\[ T_{s_{\text{fe}}} = 0.298 \text{ sec} \]

\[ T_{0_{\text{fe}}} := 0.2 \cdot T_{s_{\text{fe}}} \]  
Period at the beginning of construction design spectral acceleration plateau

\[ T_{0_{\text{fe}}} = 0.06 \text{ Sec} \]

\[ F_a_{\text{fe}} S_s_{\text{fe}} = 0.165 \]
Superstructure Section Properties

Rolled beam 27W94

\[ L_{\text{bridge}} := 109.33 \text{ ft} \]

Length of the bridge

\[ A_{\text{girder}} := 27.7 \text{ in}^2 \]

Area of the girders

\[ d_{\text{girder}} := 26.9 \text{ in} \]

Depth of the girders

\[ I_x := 3270 \text{ in}^4 \]

\[ S_x := 243 \text{ in}^3 \]

\[ I_y := 124 \text{ in}^4 \]

\[ S_y := 24.8 \text{ in}^3 \]

Fig 1. Design response spectrum (MCEER Guide Spec. Fig 3.4.1-1)
6 girder at 7’ 2”
Slab thickness = 8”
Hung = 1”

\( N := 8 \) Number of longitudinal reinforcements
\( \text{hung} := 1 \) in
\( b_{\text{slab}} := 42 \times 12 \)
\( b_{\text{slab}} = 504 \) in

Effective width of the composite section (AASHTO LRFD 4.6.2.6.2)

\[ L_1 := \frac{32 \times 12}{4} \] 1/4 of the span length
\[ L_1 = 96 \] in
\[ L_2 := 7.16 \times 12 \] Girder spacing
\[ L_2 = 85.92 \] in
\[ L_3 := 8 \times 12 + \max(0.49, 0.5, 0.745) \]
\[ L_3 = 96.49 \] in
\[ b_{\text{effective}} := \min(L_1, L_2, L_3) \]
\[ b_{\text{effective}} = 85.92 \] in

Effective width of the slab in composite section

\( h_{\text{slab}} := 8 \) in

Thickness of the slab

\( A_{\text{slab}} := b_{\text{slab}} \frac{h_{\text{slab}}}{N} \)
\( A_{\text{slab}} = 504 \) in^2
\( I_{x, \text{slab}} := b_{\text{slab}} \frac{h_{\text{slab}}^3}{12 \cdot N} \)
\( I_{x, \text{slab}} = 2.688 \times 10^3 \) in^4
\( Y_{\text{slab}} := \frac{h_{\text{slab}}}{2} \)
\[ Y_{\text{slab}} = 4 \text{ in} \]
\[ Y_{x_{\text{comp}}} := \frac{A_{\text{slab}} Y_{\text{slab}} + 6A_{\text{girder}} \left( \frac{d_{\text{girder}}}{2} + 2 Y_{\text{slab}} + \text{hung} \right)}{\left(6A_{\text{girder}} + A_{\text{slab}}\right)} \]
\[ Y_{x_{\text{comp}}} = 8.575 \text{ in} \quad \text{From top} \]
\[ I_{x_{\text{comp}}} := 6I_{x} + I_{x_{\text{slab}}} + 6A_{\text{girder}} \left( \frac{d_{\text{girder}}}{2} + 2 Y_{\text{slab}} + \text{hung} - Y_{x_{\text{comp}}} \right)^2 + A_{\text{slab}} \left(Y_{\text{slab}} - Y_{x_{\text{comp}}} \right)^2 \]
\[ I_{x_{\text{comp}}} = 6.485 \times 10^4 \text{ in}^4 \]
\[ S_{x_{\text{comp}}} := \frac{I_{x_{\text{comp}}}}{Y_{x_{\text{comp}}}} \]
\[ S_{x_{\text{comp}}} = 7.563 \times 10^3 \text{ in}^4 \]
\[ A_{\text{comp}} := A_{\text{girder}} 6 + A_{\text{slab}} \]
\[ A_{\text{comp}} = 670.2 \text{ in}^2 \]
\[ r_{x} := \frac{I_{x_{\text{comp}}}}{\sqrt{A_{\text{comp}}}} \]
\[ r_{x} = 9.837 \text{ in} \]
\[ I_{y_{\text{slab}}} := b_{\text{slab}}^3 \frac{h_{\text{slab}}}{12N} \]
\[ I_{y_{\text{slab}}} = 1.067 \times 10^7 \text{ in}^4 \]
\[ I_{y_{\text{comp}}} := 6I_{y} + 2A_{\text{girder}} \left(3.58^2 + 10.74^2 + 17.9^2 \right) + I_{y_{\text{slab}}} \]
\[ I_{y_{\text{comp}}} = 1.069 \times 10^7 \text{ in}^4 \]
\[ S_{y_{\text{comp}}} := \frac{I_{y_{\text{comp}}}}{42.5 \cdot 12} \]
\[ S_{y\_comp} = 4.244 \times 10^4 \text{ in}^3 \]

\[ r_y := \sqrt{\frac{l_{y\_comp}}{A_{comp}}} \]

\[ r_y = 126.32 \text{ in} \]

\[ A_{shear} := 0.8 A_{comp} \]

\[ A_{shear} = 536.16 \text{ in}^2 \]

\[ J := b_{slab} \frac{h_{slab}^3}{12 \text{ N}} + b_{slab} h_{slab} \frac{b_{slab}^3}{12 \text{ N}} \]

\[ J = 1.067 \times 10^7 \]

\[ S_{x\_plastic} := S_{x\_comp}^{1.5} \]

\[ S_{x\_plastic} = 1.134 \times 10^4 \text{ in}^3 \]

\[ S_{y\_plastic} := S_{y\_comp}^{1.5} \]

\[ S_{y\_plastic} = 6.366 \times 10^4 \text{ in}^3 \]

**Mass Calculation**

\[ W_{slab} := 42 \frac{8 \cdot L_{bridge}}{12} \cdot 1.15 \]

\[ W_{slab} = 459.186 \text{ kips} \]

\[ W_{girders} := \frac{94}{1000} \cdot 6 \cdot L_{bridge} \cdot 1.15 \]

\[ W_{girders} = 70.911 \text{ kips} \]

\[ \text{Bar} := 5.2 \cdot L_{bridge} \cdot 1.15 \]
According to the section properties calculation, the finite element model of the bridge is completed by SAP2000. Figure 4 presents the finite element model of the bridge. In this model the bottom of wall piers are fixed and according to the base shears in this analysis, the final model contains springs at the bottom of walls.

\[
\begin{align*}
\text{Bar} &= 85.277 \text{ kips} \\
\text{FWS} &= \frac{35}{1000} \cdot 0.40 \cdot L_{\text{bridge}} \\
\text{FWS} &= 153.062 \text{ kips} & \text{Feature wearing surface} \\
W_{\text{total}} &= W_{\text{slab}} + W_{\text{girders}} + \text{Bar} + \text{FWS} \\
W_{\text{total}} &= 768.437 \text{ kips} \\
w_{\text{total}} &= \frac{W_{\text{total}}}{109.33} \\
w_{\text{total}} &= 7.029 \text{ klf} \\
W_{\text{pilecap}} &= 7.5 \cdot 27.5 \cdot 3 \cdot 0.15 \\
W_{\text{pilecap}} &= 92.813 \text{ kips}
\end{align*}
\]

Fig 2. First Mode Shape (T=0.4902 Sec)
Fig 3. Displacement Diagram for MCE

Fig 4. Finite element model of the bridge
### Table 1. Periods of the bridge

**Modal Participating Mass Ratios**

<table>
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<th>MODE</th>
<th>PERIOD</th>
<th>INDIVIDUAL MODE (%)</th>
<th>UX</th>
<th>UY</th>
<th>UZ</th>
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<td>0.53</td>
<td>0.00</td>
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</table>

**With Fix Base**

99.54 | 99.07 | 98.21
Spring Stiffness based on Design Example 8 and Transportation Research Record 1736 (State of Practice for Design of Group of Laterally Loaded Drilled Shafts).
In this step, stiffness of the piles will be calculated based on P-multipliers curves with the coefficients of FHWA.

\[ L_{\text{pile}} := 450 \text{ in} \]  
\[ D := 3 \text{ ft} \]  
\[ n := 8 \]  
\[ N_{\text{fix}} := 18 \]  
\[ N_{\text{exp}} := 14 \]  
\[ A_b := 1 \]  
\[ A_{\text{axial}} := \left(\frac{3.14}{4}\right) D^2 \]  
\[ A_{\text{axial}} = 7.065 \text{ ft}^2 \]  
\[ L := \frac{L_{\text{pile}}}{12} \]  
\[ L = 37.5 \text{ ft} \]  
\[ E_c := 3600 \cdot 144 \text{ ksf} \]  
\[ E_c = 5.184 \times 10^5 \]  
\[ K_{\text{axial}} := A_{\text{axial}} \frac{E_c}{L} \text{ (Guide Spec. 8.4.3.3)} \]  
\[ K_{\text{axial}} = 9.767 \times 10^4 \frac{k}{\text{ft}} \]
Group pile stiffness for expansion bent

Fig 5. Pile group at the expansion bent

Fig 6. P-multiplier = 0.3 for the third row (shear vs. depth of the pile)

Fig 7. P-multiplier = 0.4 for the second row (shear vs. depth of the pile)
**Fig 8.** P-multiplier = 0.8 for the leading row (shear vs. depth of the pile)

**Fig 9.** P-multiplier curve for longitudinal direction of bridge (displacement vs. shear)

**Fig 10.** P-multiplier curve for transverse direction of bridge (displacement vs. shear)
\[ \Delta_x := \frac{283}{12} \quad \Delta_x = 0.024 \text{ ft} \] Displacement of the pile cap in longitudinal direction

\[ F_x := 200 \text{ kips} \] Shear from Fig 9 for 0.283" displacement of the pile cap in longitudinal direction

\[ n_x := 2 \] Rows of piles in longitudinal direction

\[ K_{pilex} := \frac{F_x}{\Delta_x n_x} \]

\[ K_{pilex} = 4.24 \times 10^3 \frac{k}{\text{ft}} \]

\[ K_{piley} := 2366 \frac{k}{\text{ft}} \] From Fig 10 and the 0.71" displacement of the pile cap in transverse direction (average of 3 pile)

\[ K_{pile} := \frac{K_{pilex} + K_{piley}}{2} \]

\[ K_{pile} = 3.303 \times 10^3 \frac{k}{\text{ft}} \]

\[ N_{pile} := 6 \] Number of the piles

\[ D := 3 \text{ ft} \]

\[ n := 8 \]

\[ N_{exp} = 14 \] Number of longitudinal reinforcement

\[ A_b := 1 \]

\[ A_{axial} := \left( \frac{3.14}{4} \right) D^2 + (n-1)N_{exp} \frac{A_b}{144} \]

\[ A_{axial} = 7.746 \text{ ft}^2 \]

\[ L := \frac{L_{pile}}{12} \]

\[ L = 37.5 \text{ ft} \]

\[ E_c := 3600 \times 144 \text{ ksf} \]

\[ E_c = 5.184 \times 10^5 \]
\[ K_{\text{axial}} := \frac{E_C A_{\text{axial}}}{L} \]

\[ K_{\text{axial}} = 1.071 \times 10^5 \quad \text{kft}^{-1} \]

\[ K_{x_{\text{total}}} := N_{\text{pile}} K_{\text{pilex}} \]

\[ K_{y_{\text{total}}} := N_{\text{pile}} K_{\text{piley}} \]

\[ K_{\text{axial}} := N_{\text{pile}} K_{\text{axial}} \]

\[ K_{r_{y}} := 6 K_{\text{axial}} 4.5^2 \]

\[ K_{r_{x}} := 4 K_{\text{axial}} 9^2 \]

\[ K_{r_{z}} := K_{\text{pile}} \left( 6 \cdot 4.5^2 + 4 \cdot 9^2 \right) \]

\[ K_{x_{\text{total}}} = 2.544 \times 10^4 \quad \text{kft}^{-1} \]

\[ K_{y_{\text{total}}} = 1.42 \times 10^4 \quad \text{kft}^{-1} \]

\[ K_{\text{axial}} = 6.424 \times 10^5 \quad \text{kft}^{-1} \]

\[ K_{r_{x}} = 2.082 \times 10^8 \quad \text{kft}^{-1} \]

\[ K_{r_{y}} = 7.806 \times 10^7 \quad \text{kft}^{-1} \]

\[ K_{r_{z}} = 1.472 \times 10^6 \quad \text{kft}^{-1} \]
Group pile stiffness for fixed bent

Fig 11. Pile group at the fixed bent

Fig 12. P-multiplier = 0.8 for the leading row (shear vs. depth of the pile)

Fig 13. P-multiplier = 0.4 for the second row (shear vs. depth of the pile)
Fig 14. P-multiplier = 0.3 for the third row (shear vs. depth of the pile)

Fig 15. P-multiplier curve for transverse direction of bridge (displacement vs. shear)

Fig 16. P-multiplier curve for longitudinal direction of bridge (displacement vs. shear)
From Fig 16 and the 1.27" displacement of the pile cap in longitudinal direction (average of 3 pile)

From Fig 15 and the 0.63" displacement of the pile cap in transverse direction (average of 4 pile)

\[ K_{\text{pile}} := \frac{K_{\text{pilex}} + K_{\text{piley}}}{2} \]
\[ K_{\text{pile}} = 2.025 \times 10^3 \frac{k}{\text{ft}} \]

\[ N_{\text{pile}} := 12 \]
\[ D := 3 \text{ ft for the piles} \]
\[ n := 8 \]

\[ N_{\text{fix}} = 18 \]
\[ A_b := 1 \]
\[ A_{\text{axial}} := \left( \frac{3.14}{4} \right) D^2 + (n - 1)N_{\text{fix}} A_b \frac{2}{144} \]
\[ A_{\text{axial}} = 7.94 \text{ ft}^2 \]
\[ L := \frac{L_{\text{pile}}}{12} \]
\[ L = 37.5 \text{ ft} \]
\[ E_c := 3600 \text{ ksf} \]
\[ E_c = 5.184 \times 10^5 \]
\[ K_{\text{axial}} := \frac{A_{\text{axial}} E_c}{L} \]
\[ K_{\text{axial}} = 1.098 \times 10^5 \frac{k}{\text{ft}} \]

\[ K_{X \text{\_total}} := N_{\text{pile}} \times K_{\text{piley}} \]

\[ K_{Y \text{\_total}} := N_{\text{pile}} \times K_{\text{piley}} \]

\[ K_{\text{axial}} := N_{\text{pile}} \times K_{\text{axial}} \]

\[ K_{r \_y} := 8K_{\text{axial}} \cdot 9^2 \]

\[ K_{r \_x} := 6K_{\text{axial}} \left(13.5^2 + 4.5^2\right) \]

\[ K_{r \_z} := K_{\text{pile}} \left(8 \cdot 9^2 + 6 \cdot 13.5^2 + 6 \cdot 4.5^2\right) \]

\[ K_{X \text{\_total}} = 2.004 \times 10^4 \frac{k}{\text{ft}} \]

\[ K_{Y \text{\_total}} = 2.856 \times 10^4 \frac{k}{\text{ft}} \]

\[ K_{\text{axial}} = 1.317 \times 10^6 \frac{k}{\text{ft}} \]

\[ K_{r \_x} = 1.6 \times 10^9 \frac{k}{\text{ft}} \]

\[ K_{r \_y} = 8.535 \times 10^8 \frac{k}{\text{ft}} \]

\[ K_{r \_z} = 3.773 \times 10^6 \frac{k}{\text{ft}} \]
### Table 2. Periods of the bridge: (a) MCE  (b) FE

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<thead>
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<th>MODE</th>
<th>PERIOD</th>
<th>INDIVIDUAL MODE (%)</th>
<th>(a) MCE</th>
<th>PERIOD</th>
<th>INDIVIDUAL MODE (%)</th>
<th>(b) FE</th>
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MCEER Guide Spec. Table 4.7-1

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<td>All elements for FE</td>
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<td>Superstructure to abutment</td>
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<td>Column and pile to cap beam</td>
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\[ T_s = 0.434 \text{ Sec} \]
\[ T := 0.4035 \text{ Sec} \]
\[ R_{col} := 1 + (1.5 - 1) \left( \frac{T}{1.25 \cdot T_s} \right) \quad \text{MCEER Guide Spec. 4.7} \]
\[ R_{col} = 1.372 < 1.5 \]
\[ R_{wall} := 1 + (1 - 1) \left( \frac{T}{1.25 \cdot T_s} \right) \]
\[ R_{wall} = 1 \]
\[ R_{FE} := 1 + (0.9 - 1) \left( \frac{T}{1.25 \cdot T_s} \right) \]
\[ R_{FE} = 0.926 > 0.9 \]
\[ R_{FE} := 0.9 \]

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### 100% + 40% Forces & Moments - EQ

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### Subject File

Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

#### Title
Johnson County Bridge

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#### Displacement

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Fig 17: Interaction diagram for the fixed pier

Fig 18: Interaction diagram for the expansion pier
Fixed pier

In this step the maximum displacement of the top of the piles (from the SAP2000 model) are applied to the pile in L-pile program

\[ \Delta := 1.2 \text{ in} \]

displacement of the pile (includes the combination of 100% + 40%)

Fig 19. Displacement diagram of the pile

Fig 20. Shear diagram of the pile
Figure 22 shows that by using 18#9 longitudinal reinforcement the pile will be in plastic range so we will increase the reinforcement to 26#9.
Figure 23 indicates that by using 26#9 (2.5%) longitudinal reinforcement, pile will remain in elastic range under the MCE.
Expansion pier

In this step the maximum displacement of the top of the piles (from the SAP2000 model) are applied to the pile in L-pile program

\[ \Delta = 0.89 \text{ in} \]

displacement of the pile (includes the combination of 100% + 40%)

![Shear diagram of the pile](image1)

Fig 25. Shear diagram of the pile

![Bending moment diagram of the pile](image2)

Fig 26. Bending moment diagram of the pile
Figure 27 shows that by using 14#9 longitudinal reinforcement the pile will remain in elastic range under the MCE.

Figure 28 shows that by using 19#9 longitudinal reinforcement the pile will remain in elastic range under the MCE.
Fig 29: Interaction diagram of the pile
Pile transverse reinforcement (fix bent)

L=37.5' D=3' 26#9 \( \rho = 2.5\% \)

\[ R := 0.8 \quad \text{Response modification factor for the connection (MCEER Guide Spec. Table 4.7-2)} \]

\[ \varepsilon_y := .00207 \]

D := 36 \quad \text{in} \quad \text{Diameter of the pile}

cover := 1.5 \quad \text{in} \quad \text{Clear cover}

d_5 := .625 \quad \text{in} \quad \text{Diameter of spirals}

d_9 := 1.128 \quad \text{in} \quad \text{Diameter of longitudinal reinforments}

\[ D_p := D - 2\left(\text{cover} + \frac{d_5}{2} + \frac{d_9}{2}\right) \quad \text{Circle diameter of longitudinal reinforcement (see Design Example 8 design step 8.2.1)} \]

\[ D_p = 30.622 \quad \text{in} \]

\[ D_{pp} := D - 2\left(\text{cover} + \frac{d_5}{2}\right) \quad \text{Spiral diameter} \]

\[ D_{pp} = 32.375 \quad \text{in} \]

L := 37.5\cdot12 \quad \text{Length of the pile}

\[ L = 450 \quad \text{in} \]

\[ A_b := 1 \quad \text{in}^2 \quad \text{Longitudinal rebar diameter} \]

\[ \Lambda := 2 \quad \text{fixed - fixed \quad fixity factor for circle section (MCEER Guide Spec. 8.8.2.3.a)} \]

P_d := 78 \quad \text{kips} \quad \text{Axial dead load of the pile}

P_{er} := 0 \quad \text{kips for earthquake effect in longitudinal direction}

\[ A_g := \frac{3.14}{4} \cdot D^2 \]
\[ A_g = 1.017 \times 10^3 \text{ in}^2 \]

\[ A_{cc} := \frac{3.14}{4} \cdot D_{pp}^2 \]

\[ A_{cc} = 822.79 \text{ in}^2 \]

\[ K_{\text{shape}} := 0.32 \]

\[ V := 225 \text{ kips} \]

\[ f_c := 3500 \text{ psi} \]

\[ f_{yh} := 60 \text{ ksi} \]

\[ A_v := 0.8A_g \]

\[ A_v = 813.888 \text{ in}^2 \]

\[ OS := 1.5 \]

\[ \phi := 0.85 \]

\[ \alpha := 25 \text{ degree} \]

\[ \tan \alpha := 0.466 \]

\[ H_c := 36 \text{ in} \]

\[ N := 26 \text{ number of rebars in pile} \]

\[ \rho_1 := N \cdot \frac{A_b}{A_g} \]

\[ \rho_1 = 0.026 \]
Method 2: Explicit Approach MCEER Guide Spec. 8.8.2.3

\[ L = 450 \text{ in} \]
\[ D_p = 30.622 \text{ in} \]
\[ \frac{D_p}{L} = 0.068 \]
\[ \tan \alpha = 0.466 \]
\[ P_e := 0.8 P_d \] Effect dead load with the effect of uplift
\[ V_p := \frac{\Lambda}{2} P_e \frac{D_p}{L} \] The contribution due to arch action
\[ V_p = 4.246 \text{ kips} \]
\[ \phi = 0.85 \]
\[ V_c := \frac{\frac{\sqrt{\phi}}{\phi}}{1000} \frac{\text{Acc}}{\phi} \]
\[ V_c = 97.354 \text{ kips} \]
\[ V_u := \frac{V}{R} \]
\[ V_u = 281.25 \text{ kips} \]
\[ V_{s-rq} := \left[ \frac{V_u}{\phi} - (V_p + V_c) \right] \]
\[ V_{s-rq} = 229.282 \text{ kips} \]
\[ K_{\text{shape}} = 0.32 \]
\[ f_{yh} = 60 \text{ ksi} \]
\[ f_{su} := 1.5 \cdot f_{yh} \]
\[ f_{su} = 90 \text{ ksi} \]
\[ A_v = 813.888 \text{ in}^2 \]
\[ \rho_t = 0.026 \]
\[ s := 6 \text{ in} < 6 \text{ Maximum spacing (NCHRP 5.10.6.2) OK} \]
\[ A_{sh} := .31 \text{ in}^2 \#5 \]
\[ \rho_{v\_pr} := \frac{2 \cdot A_{sh}}{s \cdot D_{pp}} \]
\[ \rho_{v\_pr} = 3.192 \times 10^{-3} \]
\[ \tan\theta := \left( \frac{1.6 \cdot \rho_{v\_pr} \cdot A_v}{A \cdot \rho_t \cdot A_g} \right)^{0.25} \]
\[ \tan\theta = 0.532 \]
\[ \tan\theta := \max(\tan\theta, 0.466) \]
\[ \tan\theta = 0.532 \]
\[ V_{s\_pr} := \frac{3.14 \cdot A_{sh} \cdot f_{yh} \cdot D_{pp}}{2 \cdot s \cdot \tan\theta} \]
\[ V_{s\_pr} = 296.342 \text{ kips} \]
\[ V_{s\_rq} = 229.282 \text{ kips OK} \]
Method 1: Implicit Shear Detailing Approach MCEER Guide Spec. 8.8.2.3

\[ s = 6 \text{ in} \]
\[ A_{sh} = 0.31 \text{ in}^2 \]

\[ \rho_{v \_rq} := K_{shape} \frac{A_t}{A_v} \frac{f_{su}}{f_{yh}} A_g \tan \alpha \tan \theta \]
\[ \rho_{v \_rq} = 8.94 \times 10^{-3} \]
\[ \rho_{v \_rq}^{\_nonplastic} := \rho_{v \_rq} - 2 \frac{\sqrt{f_c}}{1000 f_{yh}} \]
\[ \rho_{v \_rq}^{\_nonplastic} = 6.968 \times 10^{-3} \]
\[ \rho_{v \_pr} = 3.192 \times 10^{-3} \]

\[ A_{sh\_method1} := .31 \]
\[ s_{method1} := 2 \frac{A_{sh\_method1}}{D_{pp} \rho_{v \_rq}^{\_nonplastic}} \]
\[ s_{method1} = 2.748 \text{ in} \quad < \text{Method 2, use Method 2} \]

\[ \frac{\rho_{v \_pr}}{\rho_{v \_rq}^{\_nonplastic}} = 0.458 \]

**Summery of transverse reinforcement**

Use #5@6° spiral
Pile transverse reinforcement (expansion bent)

\[ L = 37.5' \quad D = 3' \quad 19\# 9 \quad \rho = 1.87\% \]

\[ d_4 := 0.5 \quad \text{in} \]
\[ d_9 := 1.128 \quad \text{in} \]
\[ D_p := D - 2 \left( \text{cover} + d_4 + \frac{d_9}{2} \right) \]
\[ D_p = 30.872 \quad \text{in} \]
\[ D_{pp} := D - 2 \left( \text{cover} + \frac{d_4}{2} \right) \]
\[ D_{pp} = 32.5 \quad \text{in} \]

\[ V := 190 \quad \text{kips} \]

\[ N := 19 \quad \text{number of rebars in pile} \]

\[ \rho_t := N \cdot \frac{A_b}{A_g} \]
\[ \rho_t = 0.019 \]
Method 2: Explicit Approach

\[ V_u := \frac{V}{R} \]

\[ V_u = 237.5 \text{ kips} \]

\[ V_{s rq} := \left[ \frac{V_u}{\phi} - (V_p + V_c) \right] \]

\[ V_{s rq} = 177.812 \text{ kips} \]

\[ \rho_t = 0.019 \]

\[ s := 6 \text{ in} \quad \text{< 6 Maximum spacing (NCHRP 5.10.6.2) OK} \]

\[ A_{sh} := 0.2 \text{ in}^2 \quad \#4 \]

\[ \rho_{v pr} := \frac{2}{s D_{pp}} A_{sh} \]

\[ \rho_{v pr} = 2.051 \times 10^{-3} \]

\[ \tan \theta := \left( 1.6 \cdot \rho_{v pr} \left( A_v - A_{\rho_t A_g} \right) \right)^{25} \]

\[ \tan \theta = 0.515 \]

\[ \tan \theta := \max(\tan \theta, 0.466) \]

\[ \tan \theta = 0.515 \]

\[ V_{s pr} := \frac{3.14 \cdot A_{sh} \cdot f_{shy} \cdot D_{pp}}{2 \cdot \rho_t \cdot \tan \theta} \]

\[ V_{s pr} = 198.19 \text{ kips} \]

\[ V_{s rq} = 177.812 \text{ kips} \quad \text{OK} \]
Method 1: Implicit Shear Detailing Approach

\[ s = 6 \text{ in} \]
\[ A_{sh} = 0.2 \text{ in}^2 \ #4 \]

\[ \rho_{v, rq} := K_{shape} \cdot \frac{\rho_t}{\phi} \frac{f_{su}}{f_{yh}} \frac{A_g}{A_v} \cdot \tan\alpha \cdot \tan\theta \]
\[ \rho_{v, rq} = 6.326 \times 10^{-3} \]

\[ \rho_{v, rq, \text{nonplastic}} := \rho_{v, rq} - \frac{2 \sqrt{f_c}}{1000 f_{yh}} \]
\[ \rho_{v, rq, \text{nonplastic}} = 4.354 \times 10^{-3} \]

\[ \rho_{v, pr} = 2.051 \times 10^{-3} \]
\[ A_{sh, \text{method1}} := 0.2 \]

\[ s_{\text{method1}} := \frac{A_{sh, \text{method1}}}{D_{pp} \cdot \rho_{v, rq, \text{nonplastic}}} \]
\[ s_{\text{method1}} = 2.826 \text{ in} \ < \text{Method 2, use Method 2} \]

\[ \frac{\rho_{v, pr}}{\rho_{v, rq, \text{nonplastic}}} = 0.471 \]

**Summary of transverse reinforcement**
Use #4@6" spiral
Joint transverse reinforcement for the expansion bent

\( L = 37.5 \, \text{ft} \)  \( D = 3 \, \text{ft} \)  \( \rho = 1.87\% \)

\( H_c = 36 \, \text{in} \)

\( R = 0.8 \)

\( \varepsilon_y = 2.07 \times 10^{-3} \)

\( D = 36 \, \text{in} \)

\( \rho_t = 0.019 \)

\( L = 450 \)

\( A_b = 1 \, \text{in}^2 \)  Longitudinal rebar diameter

\( \Lambda = 2 \)  \text{ fixed - fixed}  

\( P_d = 78 \)  \text{kips}  

\( D_p = 30.872 \, \text{in} \)  

\( D_{pp} = 32.5 \, \text{in} \)  

\( \phi = 0.85 \)

\( A_g = 1.017 \times 10^3 \, \text{in}^2 \)  

\( A_v = 813.888 \, \text{in}^2 \)  

\( f_{yh} = 60 \)  \text{ksi}  

\( f_{su} := 1.5 \cdot f_{yh} \)  

\( f_c = 3.5 \times 10^3 \)  \text{psi}
\[
P_{\text{max}} := P_d + P_{\text{er}}
\]

\[P_{\text{max}} = 78 \text{ kips}
\]

\[A_{\text{sc}} := N - A_b
\]

\[A_{\text{sc}} = 19 \text{ in}^2 \quad \text{Longitudinal reinforcement in pile}
\]

\[K_{\text{shape}} = 0.32
\]

\[M_n := 1110 \text{ k - ft}
\]
Explicit approach for joint design

\[ f_h := 0 \]

\[ P_{\text{max}} = 78 \text{ kips} \]

\[ M_p := OS \cdot M_n \]

\[ OS = 1.5 \]

\[ M_n = 1.11 \times 10^3 \text{ k} - \text{ft} \]

\[ M_p = 1.665 \times 10^3 \text{ k} - \text{ft} \]

\[ b_b := 12.5\cdot12 \text{ in} \]

\[ D = 36 \text{ in} \]

\[ L_{\text{mid\_dept\_jt}} := D + H_c \]

\[ L_{\text{mid\_dept\_jt}} = 72 \text{ in} \]

\[ A_{\text{mid}} := b_b \cdot L_{\text{mid\_dept\_jt}} \]

\[ A_{\text{mid}} = 1.08 \times 10^4 \text{ in}^2 \]

\[ f_v := \frac{P_{\text{max}}}{A_{\text{mid}}} \]

\[ f_v = 7.222 \times 10^{-3} \text{ ksi} \]

\[ h_b := H_c \]

\[ h_b = 36 \text{ in} \]

\[ h_c := D \]

\[ h_c = 36 \text{ in} \]

\[ b_{je} := \sqrt{2} \cdot D \]
\[ b_{je} = 50.912 \text{ in} \]

\[ v_{hv} := \frac{M_n \cdot 12}{h_b \cdot b_c \cdot b_{je}} \]

\[ v_{hv} = 0.202 \text{ ksi} \]

\[ p_r := \frac{\left( f_{h} + f_{v} \right)}{2} - \sqrt{\left[ \frac{\left( f_{h} - f_{v} \right)}{2} \right]^2 + v_{hv}^2} \]

\[ p_r = -0.198 \]

\[ p_{t\_max} := \frac{3.5}{1000} \sqrt{f_c} \]

\[ p_{t\_max} = 0.207 \quad \text{OK, No extra reinforcement is necessary} \]

\[ p_c := \frac{\left( f_{h} + f_{v} \right)}{2} + \sqrt{\left[ \frac{\left( f_{h} - f_{v} \right)}{2} \right]^2 + v_{hv}^2} \]

\[ p_c = 0.206 \]

\[ p_{c\_max} := \frac{25}{1000} f_c \]

\[ p_{c\_max} = 0.875 \quad \text{OK, No extra reinforcement is necessary} \]

\[ \rho_{s\_min} := \frac{3.5}{1000} \frac{\sqrt{f_c}}{f_{yh}} \]

\[ \rho_{s\_min} = 3.451 \times 10^{-3} \quad \text{OK} \]

\[ A_{sh} := .2 \]

\[ s_{\text{max}} := \frac{4 \cdot A_{sh}}{D_{pp} \cdot \rho_{s\_min}} \]

\[ s_{\text{max}} = 7.133 \quad \text{Use #4@6} \]

A-44
Joint transverse reinforcement for the fixed bent

L=37.5' D=3' 26#9 ρ=2.5%

H_c = 36 in

R = 0.8

ε_y = 2.07 × 10^{-3}

D = 36 in

ρ_t := .0255

L = 450

A_b = 1 in^2 Longitudinal rebar diameter

Λ = 2 fixed - fixed

P_d = 78 kips

D_p = 30.872 in

D_pp = 32.5 in

ϕ = 0.85

A_g = 1.017 × 10^3 in^2

A_v = 813.888 in^2

f_yh = 60 ksi

f_{su} := 1.5 f_yh

f_c = 3.5 × 10^3 psi
### Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

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### Calculations

\[
\begin{align*}
P_{\text{max}} & := P_d + P_{\text{er}} \\
N & := 26 \\
A_{sc} & := N \cdot A_b \\
A_{sc} & = 26 \text{ in}^2 \quad \text{Longitudinal reinforcement in pile} \\
K_{\text{shape}} & = 0.32 \\
M_n & := 1340 \text{ k – ft} 
\end{align*}
\]
Explicit approach for joint design

\[ f_h := 0 \]

\[ P_{\text{max}} = 78 \text{ kips} \]

\[ M_p := OS \cdot M_n \]

\[ OS = 1.5 \]

\[ M_n = 1.34 \times 10^3 \text{ k \cdot ft} \]

\[ M_p = 2.01 \times 10^3 \text{ k \cdot ft} \]

\[ b_h := 12.5-12 \text{ in} \]

\[ D = 36 \text{ in} \]

\[ L_{\text{mid_dept jt}} := D + H_c \]

\[ L_{\text{mid_dept jt}} = 72 \text{ in} \]

\[ A_{\text{mid}} := b_h \cdot L_{\text{mid_dept jt}} \]

\[ A_{\text{mid}} = 1.08 \times 10^4 \text{ in}^2 \]

\[ f_v := \frac{P_{\text{max}}}{A_{\text{mid}}} \]

\[ f_v = 7.222 \times 10^{-3} \text{ ksi} \]

\[ h_b := H_c \]

\[ h_b = 36 \text{ in} \]

\[ h_c := D \]

\[ h_c = 36 \text{ in} \]

\[ b_{je} := \sqrt{2} \cdot D \]
b_{je} = 50.912 
\nu_{hv} := \frac{M_{n} \cdot 12}{h_{b} \cdot h_{c} \cdot b_{je}} 
\nu_{hv} = 0.244 
\nu_{hv} \text{ ksi}

p_{c} := \frac{f_{h} + f_{v}}{2} - \sqrt{\left[\frac{(f_{h} - f_{v})}{2}\right]^{2} + \nu_{hv}^{2}}
\nu_{v} = -0.24

pt_{max} := \frac{3.5}{1000} \sqrt{f_{c}}
pt_{max} = 0.207 
\text{Extra reinforcement is required}

p_{c} := \frac{f_{h} + f_{v}}{2} + \sqrt{\left[\frac{(f_{h} - f_{v})}{2}\right]^{2} + \nu_{hv}^{2}}
\nu_{v} = 0.247

p_{c,\text{max}} := \frac{25}{1000} f_{c}
p_{c,\text{max}} = 0.875 
\text{OK}

A_{j\nu} := 0.16 A_{sc}
\text{Vertical reinforcement (Stirrups) (Guide Spec. 8.8.4.3.2)}

A_{j\nu} = 4.16 \text{ in}^{2}

Use #5

A_{j\nu} .31^{\frac{1}{2}} = 13.419 
\text{#5 vertical legs in (development length) each side}

A_{\text{clamp}} := 0.08 A_{sc}
\text{Vertical reinforcement (Clamping reinforcement) (Guide Spec. 8.8.4.3.2)}

A_{\text{clamp}} = 2.08
\[ \frac{A_{\text{clamp}}}{.31} = 6.71 \text{ in} \]

\[ l_{d8} := 0.63 \cdot \frac{f_{yh}}{\sqrt{\frac{f_c}{1000}}} \]

Development length for #9 (AASHTO LRFD 5.11.2.2.1-1)

\[ 0.3 \cdot 1.128 \cdot f_{yh} = 20.304 \text{ in} \]

\[ l_{d8} = 20.205 \text{ in} \]

\[ A_h := 0.08 \cdot \frac{A_{sc}}{.79} \]

Horizontal reinforcement (Guide Spec. 8.8.4.3.3)

\[ A_h = 2.633 \text{ in}^2 \]

\[ l_{d9} := 1.25 \cdot 1.128 \cdot \frac{f_{yh}}{\sqrt{\frac{f_c}{1000}}} \]

\[ l_{d9} = 45.221 \text{ in} \]

\[ .4 \cdot 1.41 \cdot f_{yh} = 33.84 \]

\[ \rho_s := .4 \cdot \frac{A_{sc}}{l_{d9}^2} \]

Hoops (Guide Spec. 8.8.4.3.4)

\[ \rho_s = 5.086 \times 10^{-3} \]

\[ \rho_{s \text{ min}} := \frac{3.5 \cdot \sqrt{f_c}}{1000 \cdot f_{yh}} \]

Minimum required horizontal reinforcement (Guide Spec. 8.8.4.2).

\[ \rho_{s \text{ min}} = 3.451 \times 10^{-3} \text{ OK} \]

\[ A_{sh} := .31 \text{ in}^2 \]

\[ s = 6 \text{ in} \]

\[ s_{\text{max}} := \frac{2 \cdot A_{sh}}{D_{pp} \rho_s} \]

\[ s_{\text{max}} = 3.751 \text{ Use #5@3.5} \]
Deep beam design (Source: Reinforcement concrete, Nawy, 3rd edition, section 6.9)

Shear design (fixed pier)

\[ f_c := 3500 \text{ psi} \]

\[ f_y := 60000 \text{ psi} \]

\[ h := 25.66 \text{ ft} \]

\[ b := 3 \text{ ft} \]

\[ d := 0.9 h \]

\[ d = 23.094 \text{ ft} \]

\[ l := 17.75 \text{ ft} \]

\[ \frac{l}{d} = 0.769 < 5 \text{ so design anthology applies} \]

\[ \Phi := 0.85 \text{ for shear} \]

\[ V := 1276 \text{ kips} \]

\[ M := 21504 \text{ k} - \text{ft} \]

\[ \Phi \left( 8 \sqrt{f_c} b d \frac{144}{1000} \right) = 4.014 \times 10^3 \text{ kips} \Phi V_n \]

\[ A_1 := \frac{M}{V d} \]

\[ A_1 = 0.73 \]

\[ A_2 := 3.5 - 2.5 \frac{M}{V d} \]
A2 = 1.676  
A2 < 3.5  OK

\[ \rho := 0.0253 \]

\[ V_c := A_2 \left( 1.9 \sqrt{f_c} + 2500 \rho \frac{V_d}{M} \right) b d \frac{144}{1000} \]

\[ V_c = 3.328 \times 10^3 \text{ kips} \]

\[ V_{c_{\text{max}}} := 6 \sqrt{f_c} b d \frac{144}{1000} \]

\[ V_{c_{\text{max}}} = 3.541 \times 10^3 \text{ kips} < V_c \text{ (controls)} \]

\[ V_c := V_{c_{\text{max}}} \]

\[ V_c = 3.541 \times 10^3 \text{ kips} \]

\[ V_s := \frac{V}{\phi} - \phi \cdot V_c \]

\[ V_s = -1.509 \times 10^3 \text{ Shear reinforcement is not required} \]
Flexure design for deep beam

\[ \Phi := 1 \]

\[ M = 2.15 \times 10^4 \text{ k \text{ ft}} \]

\[ \frac{1}{h} = 0.692 \quad < 1 \quad \text{so } j d = 0.6l \]

\[ j := \frac{6(l)}{d} \]

\[ j = 0.461 \]

\[ j \cdot d = 10.65 \text{ ft} \]

\[ A_s := \frac{M \cdot 12000}{\Phi \cdot b \cdot f_y \cdot j \cdot d} \]

\[ A_s = 134.61 \text{ in}^2 \quad < 274.56 \text{ in}^2 \text{ OK} \]

\[ A_{s\text{min}} := \max \left( 3 \cdot \frac{\sqrt{f_c}}{f_y} \cdot b \cdot d \cdot 144, 200 \cdot b \cdot \frac{d \cdot 144}{f_y} \right) \]

\[ A_{s\text{min}} = 33.255 \text{ in}^2 \quad < 274.56 \text{ in}^2 \text{ OK} \]
Deep beam design (Source: Reinforcement concrete, Nayy, 3rd edition, section 6.9)

Shear design (expansion pier)

\[
f_c := 3500 \text{ psi}
\]

\[
f_y := 60000 \text{ psi}
\]

\[
h := 25.66 \text{ ft}
\]

\[
b := 3 \text{ ft}
\]

\[
d := 0.9h
\]

\[
d = 23.094 \text{ ft}
\]

\[
l := 17.75 \text{ ft}
\]

\[
l \over d = 0.769 < 1 \text{ so design antology applys}
\]

\[
\Phi := 0.85 \text{ for shear}
\]

\[
V := 678 \text{ kips}
\]

\[
M := 12780 \text{ k-ft}
\]

\[
\Phi \left(8 \sqrt{f_c b d} \frac{144}{1000}\right) = 4.014 \times 10^3 \text{ kips} \quad \Phi V n
\]

\[
A_1 := \frac{M}{V \cdot d}
\]

\[
A_1 = 0.816
\]

\[
A_2 := 3.5 - 2.5 \frac{M}{V \cdot d}
\]
A2 = 1.459  \quad A2 < 3.5 \quad \text{OK}

\rho := .0062

V_c := A2 \left( 1.9 \sqrt{f_c} + 2500 \cdot \rho \cdot \frac{V_d}{M} \right) \cdot \frac{b \cdot d}{144} \cdot \frac{1000}{1000}

V_c = 1.913 \times 10^3 \quad \text{kips}

V_{c_{\text{max}}} := 6 \cdot \sqrt{f_c} \cdot b \cdot d \cdot \frac{144}{1000}

V_{c_{\text{max}}} = 3.541 \times 10^3 \quad \text{> Vc so use Vc}

V_c = 1.913 \times 10^3 \quad \text{kips}

V_s := \frac{V}{\Phi} - \Phi \cdot V_c

V_s = -828.574 \quad \text{Shear reinforcement is not required}
Flexure design for deep beam

\[ \Phi := 1 \]

\[ M = 1.278 \times 10^4 \quad \text{k-ft} \]

\[ l = 17.75 \quad \text{ft} \]

\[ \frac{l}{h} = 0.692 < 1 \quad \text{so } jd=0.6l \]

\[ j := \frac{.6(l)}{d} \]

\[ j = 0.461 \]

\[ jd = 10.65 \quad \text{ft} \]

\[ A_s := \frac{M-12000}{\Phi \cdot b \cdot f_y \cdot jd} \]

\[ A_s = 80 \quad \text{in}^2 > 50.56 \text{in}^2 , \text{so we use } 80\#9 \text{ (1% steel, } A_s=80.01 \text{ in}^2) \]

\[ A_{smin} := \max \left( 3 \cdot \frac{\sqrt{f_c}}{f_y} \cdot b \cdot d \cdot 144, 200 \cdot b \cdot \frac{d \cdot 144}{f_y} \right) \]

\[ A_{smin} = 33.255 \quad \text{in}^2 < 80 \text{ in}^2 \text{ OK} \]
For the fixed bent

Plastic hinge zone length (this length will modify after the reinforcement design)
MCEER Guide Spec. 4.9.1

\[ h := 36 \text{ in} \]

1-

\[ D := h - 3 \text{ in} \]

Controls but last check should be done after design

\[ D = 33 \text{ in} \]

2-

\[ H := 201 \text{ in} \]

\[ \frac{1}{6} H = 33.5 \text{ in} \]

3-

\[ 18 \text{ in} \]

4-

\[ \varepsilon_y := 0.00207 \]

\[ d_b := 1.41 \text{ in} \]

for #11

\[ M := 21504 \text{ k-ft} \]

\[ V := 1276 \text{ k} \]

\[ 1.5 \left( 0.08 \cdot M \cdot \frac{12}{V} + 4400 \cdot \varepsilon_y \cdot d_b \right) = 43.531 \text{ in} \]

Controls (3.6')
Wall pier transverse reinforcement
Method 2: Explicit Approach
(Guide Spec 8.8.2.3)

Fixed pier
308"X36" (2.53% steel, 84#11 at top and 6#11 each side)

\[ N := 180 \quad \text{number of longitudinal reinforcement} \]
\[ R := 0.8 \quad \text{R-factor for joint (table 3.1-2)} \]
\[ L := 201 \quad \text{in} \quad \text{Height of the pier} \]
\[ A_b := 1.56 \quad \text{in}^2 \quad \text{Area of rebars for longitudinal rebar} \]
\[ \Lambda := 1 \quad \text{fixed - free} \quad \text{For longitudinal case which is govern for this bridge. For transverse direction, value of 2 is used for the case of fixed - fixed.} \]
\[ P_d := 534 \quad \text{kips} \quad \text{Axial dead load in the pier} \]
\[ P_e := 59 \quad \text{kips} \quad \text{Axial earthquake force} \]
\[ M_{topL} := 0 \quad \text{k \text{-} ft} \]
\[ M_{botL} := M \]
\[ OS := 1.5 \quad \text{Over strength factor} \]
\[ M_{p\_topL} := OS \cdot M_{topL} \]
\[ M_{p\_topL} = 0 \quad \text{k \text{-} ft} \]
\[ M_{p\_botL} := OS \cdot M_{botL} \]
\[ M_{p\_botL} = 3.226 \times 10^4 \quad \text{k \text{-} ft} \]
\[ V_{uL} := \frac{(M_{p\_topL} + M_{p\_botL}) \cdot 12}{L \cdot R} \]
\[ V_{uL} = 2.407 \times 10^3 \quad \text{kips} \]
\[ V_{u\_analysis} := \frac{V}{R} \quad \text{kips} \]
\[ V_u := \max(V_{uL}, V_{u\_analysis}) \]
\[ V_u = 2.407 \times 10^3 \quad \text{kips} \]
Vp := 4.55 kips

Contribution due to arch action

φ := 0.85

For shear

Vu := 2.407 × 10^3 kips

Vs := \( \frac{Vu}{\phi} - (Vp + Vc) \)

Vs := 2.467 × 10^3 kips

Kshape := .5

For wall in weak axis (MCEER 8.8.2.3)

fyh := 60 ksi

For the transverse and longitudinal steel

fsu := 1.5fyh

Ultimate tensile stress of the longitudinal reinforcement

fsu := 90 ksi
\[
\rho_v := \frac{\phi}{f_{yh}} \frac{A_g}{L} \cdot \frac{D_p}{f_{su}} \cdot \frac{A_v}{\Lambda} \cdot \frac{\rho_t}{K_{shape}^2 A} \cdot \tan \theta
\]

For fixed - fixed case of transverse direction

\[
\rho_v = 4.68 \times 10^{-3}
\]
\[
A_{vs} := \rho_{v} \cdot b_{w} \cdot s
\]

\[
\text{legs\_new} := 30
\]

\[
\frac{A_{vs}}{\text{legs\_new}} = 0.192 \text{ in}^2 < \text{Ash provided OK}
\]

**Outside the plastic hinge zone**

**Method 2: Explicit Approach**

8.8.2.3

\[
\text{legs\_outplastic} := 14
\]

\[
V_{c} = 2 \cdot \sqrt{f_{c} \cdot b_{w} \cdot \frac{d}{1000}} \quad \text{Shear resistance of concrete out side of the plastic hinge zone}
\]

\[
V_{c} = 1.203 \times 10^4 \text{ ips}
\]

\[
V_{p} = 4.55
\]

\[
V_{s} := \frac{V_{u}}{\phi} - (V_{p} + V_{c})
\]

\[
V_{s} = 1.625 \times 10^3 \text{ kips}
\]

\[
s_{m} := 6 \text{ in} \quad \text{Maximum spacing (MCEER 8.8.2.6)}
\]

\[
A_{shm} := .2 \quad \#4
\]

\[
\rho_{v\_pr} := \frac{\text{legs\_outplastic} \cdot A_{shm}}{b_{w} \cdot s_{m}}
\]

\[
\rho_{v\_pr} = 1.515 \times 10^{-3}
\]

\[
A_{g} = 1.109 \times 10^4
\]

\[
\tan\theta := \left(1.6 \cdot \rho_{v\_pr} \cdot \frac{A_{v}}{A \cdot \rho_{t} \cdot A_{g}}\right)^{25}
\]

\[
\tan\theta = 0.547
\]

\[
D_{pp} = 32 \text{ in}
\]

\[
A_{vs} := \frac{V_{s}}{f_{yh} \cdot D_{pp} \cdot \tan\theta}
\]

\[
A_{vs} := \rho_{v} \cdot b_{w} \cdot s
\]
\[
\frac{A_{Vs}}{\text{legs\_outplastic}} = 0.198 \text{ in}^2 < \text{Ash OK}
\]

**Method 1: Implicit Shear Detailing Approach**

\[\text{legs\_new := 26}\]

\[f_{yh} = 60\]

\[\rho_{\text{vstar}} := \rho_v - \frac{2 \cdot \sqrt{f_c}}{f_{yh} \cdot 1000}\]

\[\rho_{\text{vstar}} = 2.708 \times 10^{-3}\]

\[\rho_{pr} := \frac{\text{legs\_new} \cdot A_{shm}}{b_w \cdot s_m}\]

\[\rho_{pr} = 2.814 \times 10^{-3}\] Use method 2

**Transverse reinforcement for confinement at plastic hinges (Pmax column) (Guide Spec. 8.8.2.4)**

\[\text{legs} = 17\]

\[f_{yh} = 60 \text{ ksi}\]

\[U_{sf} := 15.95 \text{ ksi} \quad \text{Strain energy capacity}\]

\[A_{sh} = 0.2 \text{ in}^2\]

\[A_c := (b_w - 6) \cdot (h - 6)\]

\[A_c = 9.06 \times 10^3 \text{ in}^2\]

\[s := 4 \text{ in} < 4'' \text{ OK (MCEER 8.8.2.4)}\]

\[\rho_{s1} := \frac{A_{sh}}{D_{pp} s^4}\]

\[\rho_{s1} = 0.027\]

\[\rho_t = 0.025\]

\[A_g = 1.109 \times 10^4\]
\[ f_c = 3.5 \times 10^3 \]

\[ P_e := P_d + P_e \]

\[ P_e = 593 \text{ kips} \quad \text{Factored axial load include seismic effect} \]

\[ \rho_{s2} := 0.008 \frac{f_c}{1000U_{sf}} \left[ 15 \left( \frac{1000P_e}{f_cA_g} + \rho_t \frac{1000-f_{yh}}{f_c} \right) \left( \frac{A_g}{A_c} \right)^2 - 1 \right] \]

\[ \rho_{s2} = 5.895 \times 10^{-3} \quad \text{OK} \]

\[ D_p = 31 \]

\[ \text{legs} = 17 \]

\[ \text{legs}_{p\text{max}} := 17 \]

\[ \frac{\text{legs}_{p\text{max}}A_{sh}}{sD_p} = 0.027 \quad \text{Include the area of total legs in both direction of cross section (Guide Spec. 8.8.2.4)} \]

\[ s_{\text{max}} := \frac{\text{legs}_{p\text{max}}A_{sh}}{D_p\rho_{s2}} \]

\[ s_{\text{max}} = 18.024 \quad \text{in} \quad \text{OK} \]

\[ \rho_s := \max(\rho_{s1}, \rho_{s2}) \]

\[ \rho_s = 0.027 \]

\[ s = 4 \quad \text{in} \]

**Explicit Shear Detailing Approach (P_{\text{min Column}})**

(Guide Spec. 8.8.2.3)

In longitudinal direction

\[ P_d = 534 \quad \text{kips} \]

\[ P_{\text{eminL}} := P_d \cdot 0.8 \quad \text{kips} \quad \text{For the uplift effect} \]
\[ P_{\text{eminL}} = 427.2 \text{ kips} \]
\[ M_{\text{botL}} = 2.15 \times 10^4 \text{ k-ft} \]
\[ OS = 1.5 \]
\[ P_{\text{eL}} := P_d + OS(P_{\text{eminL}} - P_d) \]
\[ P_{\text{eL}} = 373.8 \text{ kips} \]
\[ M_{\text{botL}} := OS \cdot M_{\text{botL}} \]
\[ V_u := \frac{M_{\text{botL}}12}{L \cdot R} \]
\[ V_u = 2.407 \times 10^3 \text{ ips} \]
\[ A_v = 1.016 \times 10^4 \]
\[ V_c := 0.6 \sqrt{\frac{A_v}{1000}} \]
\[ V_c = 360.786 \text{ kips} \]
\[ V_p := A \cdot P_{\text{eL}} \cdot \frac{D_p}{L \cdot 2} \]
\[ V_p = 28.825 \text{ kips} \]
\[ V_s := \frac{V_u}{\phi} - V_c - V_p \]
\[ V_s = 2.442 \times 10^3 \]
\[ s := 4 \text{ in} \]
\[ A_{\text{sh}} = 0.2 \text{ in}^2 \]
\[ \rho_v := \text{legs} \cdot \frac{A_{\text{sh}}}{b_w \cdot s} \]
\[
\rho_v = 2.76 \times 10^{-3}
\]
\[
\tan \theta := \left[ 1.6 \cdot \rho_v \cdot \frac{A_v}{(A \cdot \rho_t \cdot A_g)} \right]^{25}
\]
\[
\tan \theta = 0.636
\]
\[
\tan \alpha := \frac{D_p}{L}
\]
\[
\tan \alpha = 0.154 \quad \text{OK}
\]
\[
A_s = \frac{V_s \cdot s \cdot \tan \theta}{f_{yh} \cdot D_{pp}}
\]
\[
\frac{A_s}{\text{legs}} = 0.19 \quad \text{OK} < \text{Ash} \quad \text{OK}
\]

**Anti buckling steel**

*(Guide Spec. 8.8.2.5)*

\[
\text{legs}_{\text{antibuckling}} := 82
\]
\[
d_b = 1.41 \quad \text{in} \quad \text{Diameter of longitudinal rebars}
\]
\[
s_{\text{new}} := 6 \cdot d_b \quad \text{Maximum spacing of ties}
\]
\[
s_{\text{new}} = 8.46 \quad \text{in} \quad \text{Required spacing of anti-buckling}
\]
\[
s = 4 \quad \text{in}
\]
\[
A_{sh} := .31 \quad \text{#5}
\]
\[
A_b := 1.56-N
\]
\[
A_{bh} := \frac{A_b}{0.09} \cdot \frac{60}{60}
\]
\[
\frac{A_{bh}}{\text{legs}_{\text{antibuckling}}} = 0.308
\]
\[
s_{\text{max}} := .25 \cdot 60
\]
\[
s_{\text{max}} = 15 \quad \text{OK}
\]
Final check for the plastic hinge zone
(Guide Spec. 4.9.1)
\[
\tan \theta = 0.636
\]
\[
L_p := \frac{(h - 4)}{12} \left( \frac{1}{\tan \theta} + 0.5 \cdot \tan \theta \right)
\]
\[
L_p = 5.043 \text{ ft}
\]
\[
V_u = 2.407 \times 10^3
\]
\[
M_{p\_botL} = 3.226 \times 10^4 \text{ k - ft}
\]
\[
V_{uL} = 2.407 \times 10^3 \text{ kips}
\]
\[
\frac{M_{p\_botL}}{V_u} \left[ 1 - 0.85 \left( \frac{M_{p\_botL}}{1.5 M_{p\_botL}} \right) \right] = 5.807 \text{ ft controls}
\]
\[
\frac{1.5}{12} \left( 4400 \cdot \varepsilon_y \cdot d_b + 0.08 \frac{12 M_{p\_botL}}{V_u} \right) = 3.213
\]

Summary of transverse reinforcement for central column
#5@4" with 82 legs in 6' of top, #4@6" with 14 legs in the rest of the wall

Shear design in transverse direction
MCEER Guide Spec. 8.8.3
\[
\rho_{h\text{min}} := 0.0025
\]
\[
A_{sh\_rq} := \rho_{h\text{min}} (b_w - 4) \cdot h
\]
\[
A_{sh\_rq} = 27.36 \text{ in}^2
\]
\[
A_{sh} = 0.31 \#5
\]
\[ N_h := \frac{A_{sh rq}}{A_{sh}} \]

\[ N_h = 88.258 \quad \text{Number of horizontal reinforcement} \]

\[ s_h := 4 \quad \text{in} \]

\[ V_r := \frac{3}{1000} \sqrt{f_c} (b_w - 3) \cdot h \]

\[ V_r = 1.949 \times 10^3 \quad \text{kips} \]

\[ V_n := \phi \left( 0.756 \cdot \sqrt{f_c} + \rho_{hmin} \cdot f_{yh} \cdot 1000 \right) (b_w - 3) \cdot \frac{h}{1000} \]

\[ V_n = 1.817 \times 10^3 \quad \text{kips} \]

\[ V := \min(V_n, V_r) \]

\[ V = 1.817 \times 10^3 \quad \text{kips} \]

\[ V_{rq} := 1379 \quad \text{kips (So minimum horizontal reinforcement is adequate for this pier)} \]
For the expansion bent

Plastic hinge zone length (this length will modify after the reinforcement design)
MCEER Guide Spec. 4.91

\[ h := 26 \text{ in} \quad \text{thickness of the wall} \]

1-

\[ D := h - 3 \text{ in} \quad \text{controls but last check should be done after design} \]

\[ D = 23 \text{ in} \]

2-

\[ H := 213 \text{ in} \]

\[ \frac{1}{6}H = 35.5 \text{ in} \]

3-

\[ 18 \text{ in} \]

4-

\[ \varepsilon_y := 0.00207 \]

\[ d_b := 1 \text{ in} \quad \text{for #8} \]

\[ M := 3050 \text{ k - ft} \]

\[ V := 271 \text{ k} \]

\[ 1.5 \left( 0.08 \cdot M \cdot \frac{12}{V} + 4400 \cdot \varepsilon_y \cdot d_b \right) = 29.869 \text{ in} \quad \text{Controls (2.7')} \]
Wall pier transverse reinforcement
Method 2: Explicit Approach (Guide Spec 8.8.2.3)
Fixed pier
308"X26" (1.01% steel, 38#9 at top and 2#9 each side)

\[ N := 80 \quad \text{number of longitudinal reinforcement} \]

\[ R := 0.8 \quad \text{R-factor for joint (table 3.1-2)} \]

\[ L := 213 \quad \text{in} \quad \text{Height of the pier} \]

\[ A_b := 1 \times \frac{2}{\text{in}^2} \quad \text{Area of rebar for longitudinal rebar} \]

\[ A := 1 \quad \text{fixed-free} \quad \text{For longitudinal case which governs for this bridge. For transverse direction, value of 2 is used for the case of fixed-free.} \]

\[ P_d := 487 \quad \text{kips} \quad \text{Axial dead load in the pier} \]

\[ P_e := 60 \quad \text{kips} \quad \text{Axial earthquake force} \]

\[ M_{\text{topL}} := 0 \quad \text{k-ft} \]

\[ M_{\text{botL}} := M \]

\[ OS := 1.5 \quad \text{Over strength factor} \]

\[ M_{\text{p, topL}} := OS \cdot M_{\text{topL}} \]

\[ M_{\text{p, topL}} := 0 \quad \text{k-ft} \]

\[ M_{\text{p, botL}} := OS \cdot M_{\text{botL}} \]

\[ M_{\text{p, botL}} = 4.575 \times 10^3 \quad \text{k-ft} \]

\[ V_{uL} := \frac{(M_{\text{p, topL}} + M_{\text{p, botL}})}{L \cdot R} \cdot 12 \]

\[ V_{uL} = 322.183 \quad \text{kips} \]

\[ V_{\text{u, analysis}} := \frac{V}{R} \quad \text{kips} \]

\[ V_u := \max(V_{uL}, V_{\text{u, analysis}}) \]

\[ V_u = 338.75 \quad \text{kips} \]
\[ V_p = 2.958 \text{ kips} \quad \text{Contribution due to arch action} \]
\[ \phi = 0.85 \quad \text{For shear} \]
\[ V_u = 338.75 \text{ kips} \]
\[ V_s := \frac{V_u}{\phi} - \left( V_p + V_c \right) \]
\[ V_s = 144.115 \text{ kips} \]
\[ K_{shape} := 0.5 \quad \text{For wall in weak axis (MCEER 8.8.2.3)} \]
\[ f_{yh} := 60 \text{ ksi} \quad \text{For the transverse and longitudinal steel} \]
\[ f_{su} := 1.5 \cdot f_{yh} \quad \text{Ultimate tensile stress of the longitudinal reinforcement} \]
\[ f_{su} = 90 \text{ ksi} \]
\[ \rho_v = 8.744 \times 10^{-4} \]

For fixed - fixed case of transverse direction

\[ \rho_v = K_{shape} \cdot \frac{A_v}{A_v} \cdot \frac{f_{yh}}{f_{yu}} \cdot \frac{A_v}{L} \cdot \tan \theta \]

\[ \rho_v = 8.744 \times 10^{-4} \]
\[ A_{vs} := \rho_{v} \cdot b_{w} \cdot s \]

\[ \text{legs}_{\text{new}} := 10 \]

\[ \frac{A_{vs}}{\text{legs}_{\text{new}}} = 0.108 \text{ in}^2 \quad < \text{Ash provided OK} \]

Outside the plastic hinge zone

Method 2: Explicit Approach

8.8.2.3

\[ \text{legs}_{\text{outplastic}} := 2 \]

\[ V_{c} := 2 \sqrt{f_{c} \cdot b_{w} \cdot \frac{d}{1000}} \quad \text{Shear resistance of concrete out side of the plastic hinge zone} \]

\[ V_{c} = 838.19 \text{ kips} \]

\[ V_{p} = 2.958 \]

\[ V_{s} := \frac{V_{u}}{\phi} - (V_{p} + V_{c}) \]

\[ V_{s} = -442.619 \text{ kips} \]

\[ s_{m} := 6 \text{ in} \quad \text{Maximum spacing (MCEER 8.8.2.6)} \]

\[ A_{shm} := .11 \quad \#3 \]

\[ \rho_{v \cdot pr} := \frac{A_{shm}}{b_{w} \cdot s_{m}} \]

\[ \rho_{v \cdot pr} = 1.19 \times 10^{-4} \]

\[ A_{g} = 8.008 \times 10^{3} \]

\[ \tan \theta := \left( 1.6 \cdot \rho_{v \cdot pr} \cdot \frac{A_{v}}{A_{p} \cdot A_{g}} \right)^{25} \]

\[ \tan \theta = 0.359 \]

\[ D_{pp} = 22 \text{ in} \]

\[ A_{vs} := s_{m} \cdot \frac{V_{s}}{f_{y \cdot h} \cdot D_{pp}} \cdot \tan \theta \]
\[ \frac{A_{vS}}{\text{legs_outplastic}} = -0.362 \text{ in}^2 \] No reinforcement is required

Method 1: Implicit Shear Detailing Approach

\[ f_{yh} = 60 \] ksi

\[ \rho_{vstar} := \rho_v - \frac{2 \cdot \sqrt{f_c}}{f_{yh} \cdot 1000} \] No reinforcement is required

\[ \rho_{vstar} = -1.098 \times 10^{-3} \]

\[ \rho_{pr} := \text{legs_new} \times \frac{A_{shm}}{b_w \cdot s_m} \]

\[ \rho_{pr} = 1.19 \times 10^{-4} \]

Transverse reinforcement for confinement at plastic hinges (Pmax column)

(\text{Guide Spec. 8.8.2.4})

\[ \text{legs} = 3 \]

\[ f_{yh} = 60 \] ksi

\[ U_{sf} := 15.95 \] ksi Strain energy capacity

\[ A_{sh} = 0.11 \text{ in}^2 \]

\[ A_c := (b_w - 6) \cdot (h - 6) \]

\[ A_c = 6.04 \times 10^3 \text{ in}^2 \]

\[ s := 4 \text{ in} < 4'' \text{ OK (MCEER 8.8.2.4)} \]

\[ \rho_{s1} := \text{legs} \times \frac{A_{sh}}{D_{pp} \cdot s} \]

\[ \rho_{s1} = 3.75 \times 10^{-3} \]

\[ \rho_t = 0.01 \]

\[ A_g = 8.008 \times 10^3 \]
\( f_c = 3.5 \times 10^3 \) 

\( P_e := P_d + P_e \)

\( P_e = 547 \) kips  Factored axial load include seismic effect

\[
\rho_{s2} := \frac{0.008 \cdot \frac{f_c}{1000 U_{sf}}}{15 \left( \frac{1000 P_e}{f_c \cdot A_g} + \rho_t \cdot \frac{1000 f_{yh}}{f_c} \right)^2 \left( \frac{A_g}{A_c} \right)^2 - 1} 
\]

\( \rho_{s2} = -3.741 \times 10^{-5} \)  No reinforcement is required

\( D_p = 21 \)

\( \text{legs} = 3 \)

\( \text{legs}_{\text{pmax}} := 17 \)

\[
\frac{\text{legs}_{\text{pmax}} \cdot A_{sh}}{s \cdot D_p} = 0.022 \quad \text{Include the area of total legs in both direction of cross section (Guide 6)} 
\]

\[
s_{\text{max}} := \frac{\text{legs}_{\text{pmax}}}{D_p} \cdot \rho_{s2} 
\]

\( s_{\text{max}} = -2.272 \times 10^3 \)  OK

\( \rho_s := \max(\rho_{s1}, \rho_{s2}) \)

\( \rho_s = 3.75 \times 10^{-3} \)

\( s = 4 \)  in

Explicit Shear Detailing Approach (Pmin Column)  
(Guide Spec. 8.8.2.3)

In longitudinal direction 

\( P_d = 487 \) kips

\( P_{\text{eminL}} := P_d \cdot 0.8 \) kips

A-73
\[ P_{\text{eminL}} = 389.6 \text{ kips} \]
\[ M_{p,\text{top}} := 0 \text{ k - ft} \]
\[ M_{\text{botL}} = 3.05 \times 10^3 \text{ k - ft} \]
\[ OS = 1.5 \]
\[ P_{eL} := P_d + OS(P_{\text{eminL}} - P_d) \]
\[ P_{eL} = 340.9 \text{ kips} \]
\[ M_{p,\text{bot}} := OS \cdot M_{\text{botL}} \]
\[ V_u := \frac{M_{p,\text{bot}} \cdot 12}{L \cdot R} \]
\[ V_u = 322.183 \text{ kips} \]
\[ A_v = 7.084 \times 10^3 \]
\[ V_c := 0.6\sqrt{f_c \cdot \frac{A_v}{1000}} \]
\[ V_c = 251.457 \text{ kips} \]
\[ V_p := \Lambda \cdot P_{eL} \cdot \frac{D_p}{L \cdot 2} \]
\[ V_p = 16.805 \text{ kips} \]
\[ V_s := \frac{V_u}{\phi} - V_c - V_p \]
\[ V_s = 110.777 \]
\[ s := 4 \text{ in} \]
\[ A_{sh} = 0.11 \text{ in}^2 \]
\[ \rho_v := \text{legs} \cdot \frac{A_{sh}}{b_w \cdot s} \]
\[ \rho_v = 2.679 \times 10^{-4} \]

\[ \tan \theta := [1.6 \cdot \rho_v \cdot \frac{A_v}{(A \cdot \rho_t \cdot A_g)}]^{25} \]

\[ \tan \theta = 0.44 \]

\[ \tan \alpha := \frac{D_p}{L} \]

\[ \tan \alpha = 0.099 \quad \text{OK} \]

\[ A_s := \frac{V_s \cdot s \cdot \tan \theta}{f_{yh} \cdot D_{pp}} \]

\[ \frac{A_s}{\text{legs}} = 0.049 \quad \text{OK} \quad < \text{Ash OK} \]

**Anti buckling steel**  
*(Guide Spec. 8.8.2.5)*

\[ \text{legs\_antibuckling} := 29 \]

\[ d_b := 1.128 \quad \text{in} \quad \text{Diameter of longitudinal rebars} \]

\[ s_{\text{new}} := 6 \cdot d_b \quad \text{Maximum spacing of ties} \]

\[ s_{\text{new}} = 6.768 \quad \text{in} \quad \text{Required spacing for antibuckling reinforcement} \]

\[ s = 4 \quad \text{in} \]

\[ A_{sh} := .2 \quad \#4 \]

\[ A_b := 0.79 \cdot \text{N} \]

\[ A_{bh} := A_b \cdot 0.09 \cdot \frac{60}{60} \]

\[ \frac{A_{bh}}{\text{legs\_antibuckling}} = 0.196 \]

\[ s_{\text{max}} := .25 \cdot 60 \]

\[ s_{\text{max}} = 15 \quad \text{OK} \]
Final check for the plastic hinge zone
(Guide Spec. 4.9.1)

\[ \tan \theta = 0.44 \]

\[ L_p := \frac{(h - 4)}{12} \left( \frac{1}{\tan \theta} + 0.5 \cdot \tan \theta \right) \]

\[ L_p = 4.569 \text{ ft} \]

\[ M_p_{\text{botL}} = 4.575 \times 10^3 \text{ k - ft} \]

\[ V_u L = 322.183 \text{ kips} \]

\[ \frac{M_p_{\text{botL}}}{V_u} \left[ 1 - 0.85 \left( \frac{M_p_{\text{botL}}}{1.5 M_p_{\text{botL}}} \right) \right] = 6.153 \text{ ft controls} \]

\[ \frac{1.5}{12} \left( 4400 \cdot \varepsilon_y \cdot d_b + 0.08 \frac{12 M_p_{\text{botL}}}{V_u} \right) = 2.988 \]

**Summary of transverse reinforcement for central column**

#4@4" with 29 legs in 6.2' of bottom of the wall

**Shear design in transverse direction**

MCEER Guide Spec. 8.8.3

\[ \rho_{\text{hmin}} := 0.0025 \]

\[ A_{sh rq} := \rho_{\text{hmin}} \left( b_w - 4 \right) h \]

\[ A_{sh rq} = 19.76 \text{ in}^2 \]

\[ A_{sh} = 0.2 \]
\[ N_h := \frac{A_{sh rq}}{A_{sh}} \]

\[ N_h = 98.8 \quad \text{Number of horizontal reinforcement} \]

\[ s_h := 4 \quad \text{in} \]

\[ V_r := \frac{3}{1000} \sqrt{f_c} (b_w - 4) h \]

\[ V_r = 1.403 \times 10^3 \quad \text{kips} \]

\[ V_n := \phi (0.756 \sqrt{f_c} + \rho_{hmin} f_y h) \frac{b_w - 4}{1000} h \]

\[ V_n = 1.308 \times 10^3 \quad \text{kips} \]

\[ V := \min(V_n, V_r) \]

\[ V = 1.308 \times 10^3 \quad \text{kips} \]

\[ V_{rq} := 811 \quad \text{kips (So minimum horizontal reinforcement is adequate for this pier)} \]
Fixed Bent

Wall pier connection design

Method 1: Implicit Shear Detailing Approach

(Guide spec. 8.8.4.1, 8.8.2.3, 8.8.2.4)

Fixed pier

308"X36" (2.53% steel, 84#11 at top and 6#11 each side)

\[ N := 180 \quad \text{number of longitudinal reinforcement} \]

\[ b_w := 36 \quad \text{in} \quad \text{Height of pile cap} \]

\[ H_c := 36 \quad \text{in} \quad \text{Height of the joint} \]

\[ h := 36 \quad \text{in} \quad \text{Dimension of the column} \]

\[ D := h - 3 \quad \text{in} \]

\[ D = 33 \quad \text{in} \]

\[ R := 0.8 \quad \text{R-factor for joint (table 3.1-2)} \]

\[ \varepsilon_y := .00207 \quad \text{For the longitudinal steel} \]

\[ L := 201 \quad \text{in} \quad \text{Height of the pier} \]

\[ A_b := 1.56 \quad \text{in}^2 \quad \text{Area of rebars for longitudinal rebar} \]

\[ P_d := 534 \quad \text{kips} \quad \text{Axial dead load in the pier} \]

\[ P_e := 59 \quad \text{kips} \quad \text{Axial earthquake force} \]

\[ M_{botL} := 21504 \quad \text{k – ft} \]

\[ OS := 1.5 \quad \text{Over strength factor} \]

\[ M_{p_botL} := OS \cdot M_{botL} \]

\[ M_{p_botL} = 3.226 \times 10^4 \quad \text{k – ft} \]
\[ f_c := 3500 \text{ psi} \]
\[ b_w := 308 \text{ in} \]
\[ d := D \text{ in} \]
\[ L = 201 \text{ in} \]
\[ D_p := 36 - 4 \] Width of the column core
\[ D_p = 32 \text{ in} \]
\[ \phi := 0.85 \] For shear
\[ f_{yh} := 60 \text{ ksi} \] For the transverse and longitudinal steel
\[ f_{su} := 1.5 \cdot f_{yh} \] Ultimate tensile stress of the longitudinal reinforcement
\[ f_{su} = 90 \text{ ksi} \]
\[ A_v := b_w \cdot d \] Shear area
\[ A_v = 1.016 \times 10^4 \text{ in}^2 \]
\[ \rho_t := 0.0253 \] Longitudinal reinforcement ratio
\[ s := 4 \text{ in} \] < 4" OK (MCEER 8.8.2.4) for plastic hinge zone
\[ A_{sh} := .2 \#4 \] Area of transverse reinforcement (ties)
\[ \text{legs} := 16 \] Number of legs
\[ \rho_{v\_pr} := \text{legs} \cdot \frac{A_{sh}}{b_w \cdot s} \]
\[ \rho_{v\_pr} = 2.597 \times 10^{-3} \]
\[ A_g := b_w \cdot h \]
\[ A_g = 1.109 \times 10^4 \text{ in}^2 \]
L = 201 \text{ in} \quad \text{Length of the column}

A_b := 1.56 \text{ in}^2 \quad \text{For longitudinal rebar}

D_p = 32 \text{ in} \quad \text{Width of the column core}

\phi := 0.85 \quad \text{For shear}

A_v := D_p^2 \quad \text{Shear area } A_v = bw*d

A_g := D^2

f_yh := 60 \text{ ksi}

f_{su} := 1.5 \cdot f_yh

D_{pp} := 31 \quad \text{Core dimension of tied column in the direction under construction}

A_{sc} := N \cdot A_b \quad \text{Longitudinal reinforcement in column}

A_{sc} = 280.8 \text{ in}^2
Explicit approach for joint design
(Guide Spec. 8.8.4.2, C8.8.4.2, 8.8.4.2.2, 8.8.4.3.1, 8.8.4.3.2, 8.8.4.3.3)

Bottom of the wall in longitudinal direction

\[ f_h := 0 \] Average axial stress in the horizontal direction

\[ P := P_d + P_e \]

\[ P = 593 \text{ kips} \] Maximum axial force include earthquake effect

\[ M_{p_{botL}} = 3.226 \times 10^4 \text{ k} \cdot \text{ft} \] From push over analysis

\[ b_b := 25.12 \text{ in} \] Width of pile cap

\[ L_{mid\_dept\_jt} := D + H_c \]

\[ L_{mid\_dept\_jt} = 69 \text{ in} \]

\[ A_{mid} := b_b \cdot L_{mid\_dept\_jt} \]

\[ A_{mid} = 2.07 \times 10^4 \text{ in}^2 \]

\[ f_v := \frac{P_e}{A_{mid}} \] Average axial stress in the vertical direction

\[ f_v = 2.85 \times 10^{-3} \text{ ksi} \]

\[ h_b := H_c \] Joint depth

\[ h_b = 36 \text{ in} \]

\[ h_c := D \] Dimension of column

\[ h_c = 33 \text{ in} \]

\[ b_{je} := 2h_c \] effective joint width for shear stress calculations

\[ b_{je} = 66 \text{ in} \]
Vertical reinforcement (Clamping reinforcement) (Guide Spec. 8.8.4.3.2)

\[ A_{\text{clamp}} = 0.08A_{\text{sc}} \]

\[ A_{\text{clamp}} = 22.464 \]

\[ A_{\text{clamp}} \cdot 0.6 = 37.44 \text{ 38#7 in 36" of the joint} \]

Vertical reinforcement (Stirrups) (Guide Spec. 8.8.4.3.2)

\[ A_{\text{v}} = 0.16A_{\text{sc}} \]

\[ A_{\text{v}} = 44.928 \text{ in}^2 \]

Use #7

\[ A_{\text{v}} \cdot 0.6 = 74.88 \text{ 75#7 vertical legs in (development length) each side} \]
Development length for #9 (AASHTO LRFD 5.11.2.2.1-1)

\[ l_{d9} := \frac{f_{yh}}{f_c} \times \sqrt{\frac{f_c}{1000}} \]

\[ 0.3 \times 1.128 \times f_{yh} = 20.304 \text{ in} \]

\[ l_{d9} = 22.791 \text{ in} \]

Horizontal reinforcement (Guide Spec. 8.8.4.3.3)

\[ A_h := 0.08 A_{sc} \]

\[ A_h = 22.464 \text{ in}^2 \]

\[ l_{d11} := 1.25 \times 1.56 \times \frac{f_{yh}}{f_c} \times \sqrt{\frac{f_c}{1000}} \]

\[ l_{d11} = 62.539 \text{ in} \]

\[ 0.4 \times 1.41 \times f_{yh} = 33.84 \]

\[ \rho_s := 0.4 \frac{A_{sc}}{l_{d11}^2} \]

\[ \rho_s = 0.029 \]

Minimum required horizontal reinforcement (Guide Spec. 8.8.4.2.2)

\[ \rho_{s_{min}} := \frac{3.5}{1000} \times \frac{f_c}{f_{yh}} \]

\[ \rho_{s_{min}} = 3.451 \times 10^{-3} \text{ OK} \]

\[ A_{sh} := 0.2 \text{ in}^2 \]

\[ s = 4 \text{ in} \]

legs := 41

\[ s_{max} := \text{legs} \times \frac{A_{sh}}{D_{pp} \rho_s} \]

\[ s_{max} = 9.211 \text{ Use #4@6 with 41 legs} \]
Expansion Bent

Wall pier connection design
Method 1: Implicit Shear Detailing Approach
(Guide spec. 8.8.4.1, 8.8.2.3, 8.8.2.4)

308"X26" (1.01% steel, 38#9 at top and 2#9 each side)

\[ N := 80 \] number of longitudinal reinforcement

\[ b_w := 36 \text{ in} \] Height of pile cap

\[ H_c := 36 \text{ in} \] Height of the joint

\[ h := 26 \text{ in} \] Dimension of the column

\[ D := h - 3 \text{ in} \]

\[ D = 23 \text{ in} \]

\[ R := 0.8 \] R-factor for joint (table 3.1-2)

\[ \varepsilon_y := .00207 \] For the longitudinal steel

\[ L := 213 \text{ in} \] Height of the pier

\[ A_b := 1 \text{ in}^2 \] Area of rebars for longitudinal rebar

\[ \Lambda := 1 \] fixed - free For longitudinal case which is govern for this bridge. For transverse direction, value of 2 is used for the case of fixed - fixed.

\[ P_d := 486 \text{ kips} \] Axial dead load in the pier

\[ P_e := 60 \text{ kips} \] Axial earthquake force

\[ M_{botL} := 3050 \text{ k - ft} \]

\[ OS := 1.5 \] Over strength factor

\[ M_{p,botL} := OS \cdot M_{botL} \]

\[ M_{p,botL} = 4.575 \times 10^3 \text{ k - ft} \]
\( f_c := 3500 \text{ psi} \)
\( b_w := 308 \text{ in} \)
\( d := D \text{ in} \)
\( L = 213 \text{ in} \)
\( D_p := h - 4 \) Width of the column core
\( D_p = 22 \text{ in} \)
\( \phi := 0.85 \) For shear
\( f_{yh} := 60 \text{ ksi} \) For the transverse and longitudinal steel
\( f_{su} := 1.5 \cdot f_{yh} \) Ultimate tensile stress of the longitudinal reinforcement
\( f_{su} = 90 \text{ ksi} \)
\( A_v := b_w \cdot d \) Shear area
\( A_v = 7.084 \times 10^3 \text{ in}^2 \)
\( \rho_l := 0.0101 \) Longitudinal reinforcement ratio
\( s := 4 \text{ in} < 4" \text{ OK (MCEER 8.8.2.4) for plastic hinge zone} \)
\( A_{sh} := 0.2 \#4 \) Area of transverse reinforcement (ties)
\( \text{legs} := 16 \) Number of legs
\( \rho_{v-pr} := \text{legs} \cdot \frac{A_{sh}}{b_w \cdot s} \)
\( \rho_{v-pr} = 2.597 \times 10^{-3} \)
\( A_g := b_w \cdot h \)
\( A_g = 8.008 \times 10^3 \text{ in}^2 \)
L = 213 in Length of the column

\( A_b = 1 \text{ in}^2 \) For longitudinal rebar

\( D_p = 22 \text{ in} \) Width of the column core

\( \phi := 0.85 \) For shear

\( A_v := D_p^2 \) Shear area \( A_v = bw \cdot d \)

\( A_g := D^2 \)

\( f_{yh} := 60 \text{ ksi} \)

\( f_{su} := 1.5 \cdot f_{yh} \)

\( D_{pp} := h - 5 \) Core dimension of tied column in the direction under construction

\( A_{sc} := N \cdot A_b \) Longitudinal reinforcement in column

\( A_{sc} = 80 \text{ in}^2 \)
Explicit approach for joint design
(Guide Spec. 8.8.4.2, C8.8.4.2, 8.8.4.2.2, 8.8.4.3.1, 8.8.4.3.2, 8.8.4.3.3)

Bottom of the column in longitudinal direction

\[ f_h := 0 \]

Average axial stress in the horizontal direction

\[ P := P_d + P_e \]

\[ P = 546 \text{ kips} \]

Maximum axial force include earthquake effect

\[ M_{p_{\text{botL}}} = 4.575 \times 10^3 \text{ k-ft} \]

From push over analysis

\[ b_b := 25.12 \text{ in} \]

Width of pile cap

\[ L_{\text{mid dept jt}} := D + H_c \]

\[ L_{\text{mid dept jt}} = 59 \text{ in} \]

\[ A_{\text{mid}} := b_b \cdot L_{\text{mid dept jt}} \]

\[ A_{\text{mid}} = 1.77 \times 10^4 \text{ in}^2 \]

\[ f_v := \frac{P_e}{A_{\text{mid}}} \]

Average axial stress in the vertical direction

\[ f_v = 3.39 \times 10^{-3} \text{ ksi} \]

\[ h_b := H_c \]

Joint depth

\[ h_b = 36 \text{ in} \]

\[ h_c := D \]

Dimension of column

\[ h_c = 23 \text{ in} \]

\[ b_{je} := 2h_c \]

effective joint width for shear stress calculations

\[ b_{je} = 46 \text{ in} \]
Joint shear stress

\[
\nu_{hv} := \frac{M_{p,botL}}{b_h b_c b_{jc}}
\]

\[
\nu_{hv} = 1.441 \text{ ksi}
\]

Principal tension stress

\[
p_t := \frac{f_h + f_v}{2} - \sqrt{\left[\frac{f_h - f_v}{2}\right]^2 + \nu_{hv}^2}
\]

\[
p_t = -1.44
\]

Maximum tension stress

\[
p_{t,\text{max}} := \frac{3.5}{1000} \sqrt{f_c}
\]

\[
p_{t,\text{max}} = 0.207 \text{ Not OK, we should provide some horizontal reinforcement}
\]

Principal compression stress

\[
p_c := \frac{f_h + f_v}{2} + \sqrt{\left[\frac{f_h - f_v}{2}\right]^2 + \nu_{hv}^2}
\]

\[
p_c = 1.443
\]

Maximum compression stress

\[
p_{c,\text{max}} := \frac{25}{1000} f_c
\]

\[
p_{c,\text{max}} = 0.875 \text{ OK}
\]

Vertical reinforcement (Stirrups) (Guide Spec. 8.8.4.3.2)

\[
A_{jv} := 0.16 A_{sc}
\]

\[
A_{jv} = 12.8 \text{ in}^2
\]

Use #7

\[
A_{jv,\text{sd}} = 21.333 \text{ 22#7 vertical legs in (development length) each side}
\]

Vertical reinforcement (Clamping reinforcement) (Guide Spec. 8.8.4.3.2)

\[
A_{\text{clamp}} := 0.08 A_{sc}
\]

\[
A_{\text{clamp}} = 6.4
\]

\[
A_{\text{clamp},\text{sd}} = 10.667 \text{ 1#7 in 36\degree of the joint}
\]
Development length for #9 (AASHTO LRFD 5.11.2.1.1-1)

\[ l_{d9} := 0.63 \cdot 1.128 \cdot \frac{f_{yh}}{\sqrt{\frac{f_c}{1000}}} \]

\[ 0.3 \cdot 1.128 \cdot f_{yh} = 20.304 \text{ in} \]

\[ l_{d9} = 22.791 \text{ in} \]

\[ A_h := 0.08 \cdot A_{sc} \]

\[ A_h = 6.4 \text{in}^2 \]

Horizontal reinforcement (Guide Spec. 8.8.4.3.3)

\[ l_{d11} := 1.25 \cdot 1.56 \cdot \frac{f_{yh}}{\sqrt{\frac{f_c}{1000}}} \]

\[ l_{d11} = 62.539 \]

\[ 0.4 \cdot 1.41 \cdot f_{yh} = 33.84 \]

\[ \rho_s := \frac{1}{2} \frac{A_{sc}}{l_{d11}} \]

\[ \rho_s = 8.182 \times 10^{-3} \]

Minimum required horizontal reinforcement (Guide Spec. 8.8.4.2.2)

\[ \rho_{s \text{min}} := \frac{3.5}{1000} \sqrt{\frac{f_c}{f_{yh}}} \]

\[ \rho_{s \text{min}} = 3.451 \times 10^{-3} \text{ OK} \]

\[ A_{sh} := 0.11 \text{ in}^2 \]

\[ s := 6 \text{ in} \]

\[ \text{legs} := 19 \]

\[ s_{\text{max}} := \text{legs} \frac{A_{sh}}{D_{pp} \rho_s} \]

\[ s_{\text{max}} = 12.164 \text{ Use #3@6" with 19 legs} \]
P-Δ Requirements
Guide Spec. 8.3.4

Fix Pier

W := 400 kips Average axial force at column
V := 1283 kips

H := \frac{201}{12}
H = 16.75 ft

C := \frac{V}{W}
C = 3.208

Δ_{limit} := .25 \cdot C \cdot H
Δ_{limit} = 13.431 ft
Δ := .28 ft < Δ_{limit} OK

Expansion Pier

W := 400 kips Average axial force at column
V := 678 kips

H := \frac{201}{12}
H = 16.75 ft

C := \frac{V}{W}
C = 1.695

Δ_{limit} := .25 \cdot C \cdot H
Δ_{limit} = 7.098 ft
Δ := .28 ft < Δ_{limit} OK
Minimum seat requirement

\[ L := 109.33 \cdot 0.3048 \]
\[ L = 33.324 \text{ m} \quad \text{Length of bridge} \]

\[ H := \frac{213}{12} \cdot 0.3048 \]
\[ H = 5.41 \text{ m} \quad \text{Height of the tallest pier} \]

\[ B := 42 \cdot 0.3048 \]
\[ B = 12.802 \text{ m} \quad \text{Width of superstructure} \]

\[ F_V := 1.5 \]

\[ S_1 := 0.759 \]

\[ \alpha := 0 \]

\[ N := \left[ 0.1 + 0.0017 \cdot L + 0.007 \cdot H + 0.05 \sqrt{H} \cdot \sqrt{1 + \left( \frac{2 \cdot B}{L} \right)^2} \right] \cdot (1 + 1.25 \cdot F_V \cdot S_1) \]

\[ N = 0.827 \text{ m} \]

\[ N := \frac{N}{0.3048} \]

\[ N = 2.712 \text{ ft} \quad \text{33” Minimum seat width} \]
Pile cap design procedure for Johnson County bridge

\[ F_c := 4000 \text{ psi} \]
\[ F_y := 60000 \text{ psi} \]

Moments and Forces acting on piles:

<table>
<thead>
<tr>
<th></th>
<th>Axial (kips)</th>
<th>M (kip-ft) Trans (case1)</th>
<th>M (kip-ft) Long (case2)</th>
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<td>Fixed Long.</td>
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<td>8602</td>
<td>21504</td>
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<tr>
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<td>988</td>
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<td>8734</td>
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<tr>
<td>Exp. Long.</td>
<td>764</td>
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</tr>
<tr>
<td>Pier Trans.</td>
<td>728</td>
<td>12780</td>
<td>5112</td>
</tr>
</tbody>
</table>

Dimensions of Fixed pier

\[ W := 2.17 \text{ ft} \]  Width of Wall (ft)
\[ L := 25.67 \text{ ft} \]  Length of wall (ft)
\[ T := 3 \text{ ft} \]  Thickness of pile cap (ft)

\[ s := 9 \]  Number of rows
\[ m_1 := 3 \]  Number of piles in the rows
\[ n_1 := 4 \]  Sum of the squares of the distances to each pile from the center or gravity of piles
\[ \Sigma d_l := \frac{s^2}{12} n_1 \left( n_1^2 - 1 \right) m_1 \]

\[ \Sigma d_l = 1.215 \times 10^3 \text{ ft}^2 \]  \[ \Sigma d_l := 6 \cdot 13.5^2 + 6 \cdot 4.5^2 \]
n2 := 3  
Number of rows 
m2 := 4  
Number of piles in the rows 
\[ \Sigma d_2 := \frac{n2^2}{12} \cdot n2 \cdot (n2^2 - 1) \cdot m2 \]
\[ \Sigma d_2 := 8 \cdot 9^2 \]
\[ \Sigma d_2 = 648 \]
\[ \text{ft}^2 \] 
Or 
M1 := 21504  
kip – ft 
M2 := 8734  
kip – ft 
V := 1014  
kip 
Piles Coordinates:

\[
\begin{align*}
  x_1 & := 13.5 & x_5 & := 9 & x_9 & := 13.5 \\
  y_1 & := 9 & y_5 & := 0 & y_9 & := -9 \\
  x_2 & := 4.5 & x_6 & := 4.5 & x_{10} & := 4.5 \\
  y_2 & := 9 & y_6 & := 0 & y_{10} & := -9 \\
  & := -4.5 & x_7 & := -4.5 & x_{11} & := -4.5 \\
  & := 9 & y_7 & := 0 & y_{11} & := -9 \\
  \ldots & := -13.5 & x_8 & := -13.5 & x_{12} & := -13.5 \\
  y_4 & := 9 & y_8 & := 0 & y_{12} & := -9
\end{align*}
\]
A := 81  
\text{ft}^2  
Area for each pile 
\[ \delta := 0.12 \]
\[ \text{kip} \] 
Soil unit weight 
h := 2  
\text{ft}  
soil height 
p := A \cdot \delta \cdot h 
p = 19.44  
kips  
Soil pressure for each pile 
n := 12  
Number of piles in the group
\begin{align*}
P_1 &= \frac{V}{n} + M_1 \cdot \frac{x_1}{\Sigma d_1} + M_2 \cdot \frac{y_1}{\Sigma d_2} + p \\
P_1 &= 464.179 \text{ kips} \\
P_2 &= \frac{V}{n} + M_1 \cdot \frac{x_2}{\Sigma d_1} + M_2 \cdot \frac{y_2}{\Sigma d_2} + p \\
P_2 &= 304.89 \text{ kips} \\
P_3 &= \frac{V}{n} + M_1 \cdot \frac{x_3}{\Sigma d_1} + M_2 \cdot \frac{y_3}{\Sigma d_2} + p \\
P_3 &= 145.601 \text{ kips} \\
P_4 &= \frac{V}{n} + M_1 \cdot \frac{x_4}{\Sigma d_1} + M_2 \cdot \frac{y_4}{\Sigma d_2} + p \\
P_4 &= -13.688 \text{ kips} \\
P_5 &= \frac{V}{n} + M_1 \cdot \frac{x_5}{\Sigma d_1} + M_2 \cdot \frac{y_5}{\Sigma d_2} + p \\
P_5 &= 263.229 \text{ kips} \\
P_6 &= \frac{V}{n} + M_1 \cdot \frac{x_6}{\Sigma d_1} + M_2 \cdot \frac{y_6}{\Sigma d_2} + p \\
P_6 &= 183.584 \text{ kips} \\
P_7 &= \frac{V}{n} + M_1 \cdot \frac{x_7}{\Sigma d_1} + M_2 \cdot \frac{y_7}{\Sigma d_2} + p \\
P_7 &= 24.296 \text{ kips} \\
P_8 &= \frac{V}{n} + M_1 \cdot \frac{x_8}{\Sigma d_1} + M_2 \cdot \frac{y_8}{\Sigma d_2} + p
\end{align*}
P8 = -134.993  
kips

P9 := \frac{V}{n} + M_1 \cdot \frac{x^9}{\Sigma d_1} + M_2 \cdot \frac{y^9}{\Sigma d_2} + p

P9 = 221.568  
kips

P10 := \frac{V}{n} + M_1 \cdot \frac{x^{10}}{\Sigma d_1} + M_2 \cdot \frac{y^{10}}{\Sigma d_2} + p

P10 = 62.279  
kips

P11 := \frac{V}{n} + M_1 \cdot \frac{x^{11}}{\Sigma d_1} + M_2 \cdot \frac{y^{11}}{\Sigma d_2} + p

P11 = -97.01  
kips

P12 := \frac{V}{n} + M_1 \cdot \frac{x^{12}}{\Sigma d_1} + M_2 \cdot \frac{y^{12}}{\Sigma d_2} + p

P12 = -256.299  
kips

**Pile punching shear check:**

Vu := P

Vu = 464.179  
Kips
d := 33  
in
b := 69\pi  
in

Vc := 4\sqrt{\frac{F_c \cdot b}{1000}} \cdot \frac{d}{1000}

Vc = 1.81 \times 10^3  
kips

\phi := 0.85

\frac{Vu}{\phi} = 546.093  \quad \frac{Vu}{\phi} < Vc  \quad \text{OK}
Minimum Reinforcement will apply in both directions.

Asmin is divided in two parts for top and bottom of pile cap:

In short direction: 48 # 6

Asmin 20.995 $= 0.0018 \times 36 \times 324 = 27.994$ in²

In long direction: 64 # 6

Asmin 20.995 $= 0.0018 \times 36 \times 432 = 36.07 \times 10^3$ in²

As $= 4.449 \times 10^3$ in²

Asmin $= 0.0018 \times 36 \times 432 = 27.994$

In long direction: 64 # 6

Asmin $= 0.0018 \times 36 \times 324 = 20.995$

In short direction: 48 # 6

Minimum Reinforcement will apply in both directions.

Asmin is divided in two parts for top and bottom of pile cap.
Dimensions of Exp. pier

- Width of Wall (ft): \( W = 2.17 \)
- Length of wall (ft): \( L = 25.67 \)
- Thickness of pile cap (ft): \( T = 3 \)
- Pile cap width (ft): \( w = 18 \)
- Pile cap Length (ft): \( l = 27 \)

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\( \Sigma d_1 = \frac{s^2}{12} \cdot n_1 \cdot (n_1^2 - 1) \cdot m_1 \)

\( \Sigma d_2 = \frac{s^2}{12} \cdot n_2 \cdot (n_2^2 - 1) \cdot m_2 \)
M1 := 12780 \; \text{kip - ft}

M2 := 1220 \; \text{kip - ft}

V := 764 \; \text{kip}

Piles Coordinates:

\begin{align*}
x1 := 9 & \quad x4 := 9 \\
y1 := 4.5 & \quad y4 := -4.5 \\
x2 := 0 & \quad x5 := 0 \\
y2 := 4.5 & \quad y5 := -4.5 \\
x3 := -9 & \quad x6 := -9 \\
y3 := 4.5 & \quad y6 := -4.5
\end{align*}

P1 := \frac{V}{n} + \frac{M1 \cdot x1}{\Sigma d1} + \frac{M2 \cdot y1}{\Sigma d2} + p

P1 = 546.959 \; \text{kips}

P2 := \frac{V}{n} + \frac{M1 \cdot x2}{\Sigma d1} + \frac{M2 \cdot y2}{\Sigma d2} + p

P2 = 191.959 \; \text{kips}

P3 := \frac{V}{n} + \frac{M1 \cdot x3}{\Sigma d1} + \frac{M2 \cdot y3}{\Sigma d2} + p

P3 = -163.041 \; \text{kips}

P4 := \frac{V}{n} + \frac{M1 \cdot x4}{\Sigma d1} + \frac{M2 \cdot y4}{\Sigma d2} + p

P4 = 456.588 \; \text{kips}

P5 := \frac{V}{n} + \frac{M1 \cdot x5}{\Sigma d1} + \frac{M2 \cdot y5}{\Sigma d2} + p

P5 = 456.588 \; \text{kips}
P5 = 101.588 kips

\[ P6 = \frac{V}{n} + M1 \cdot \frac{x6}{\Sigma d1} + M2 \cdot \frac{y6}{\Sigma d2} + p \]

\[ P6 = -253.412 \text{ kips} \]

**Pile punching shear check:**

\[ Vu := P1 \]
\[ Vu = 546.959 \text{ kips} \]
\[ d := 33 \text{ in} \]
\[ b := 69\pi \text{ in} \]
\[ Vc := 4 \sqrt{Fc \cdot b \cdot \frac{d}{1000}} \]
\[ Vc = 1.81 \times 10^3 \text{ kips} \]
\[ \phi := 0.85 \]
\[ \frac{Vu}{\phi} = 643.481 < \frac{Vu}{\phi} < Vc \text{ OK} \]
\[ d := 45 \]

**One way shear Check:**

\[ V1 := (P1 + P2 + P3) \cdot 0.74 \]
\[ V1 = 426.148 \text{ kips} \]
\[ b2 := 324 \text{ in} \]
\[ Vc2 := 4 \sqrt{Fc \cdot b2 \cdot \frac{d}{1000}} \]
\[ Vc2 = 3.688 \times 10^3 \]
\[ \frac{V1}{\phi} = 501.35 < \frac{V1}{\phi} < Vc2 \text{ OK} \]
Flexure bending design: Only in one direction

\[ \text{Mu} := (P_1 + P_2 + P_3) \times 1000 \times 3.32 \]

\[ \text{Mu} = 1.912 \times 10^6 \]

\[ \text{As} := \frac{\text{Mu}}{\Phi \cdot F_y \cdot 0.9 \cdot d} \]

\[ \text{As} = 0.874 \ \text{in}^2 \]

Asmin := 0.0018 - 36 - 324

Asmin = 20.995

In Long Direction: 48 # 6

Asmin := 0.0018 - 36 - 216

Asmin = 13.997

In short Direction: 32 # 6

Minimum Reinforcement will apply in both directions:

Asmin is divided in two part for top and bottom of pile cap

Due to having uplift in some piles, it is necessary to check the uplift capacity for piles.
Axial Capacity for uplift:

Ds := 3 ft
L1 := 4 ft
L2 := 24 ft
L := 37.5 ft

\[ \text{qu1} := \frac{38,000}{144} \text{ psi} \quad \text{qu1} = 527.778 \]

\[ \text{qu2} := \frac{50,000}{144} \text{ psi} \quad \text{qu2} = 694.444 \]

\[ f1 := 2.5\sqrt{\text{qu1}} \quad f1 = 57.434 \text{ psi} \quad \text{Less than} \quad .15 \text{ qu1} = 79.167 \text{ psi} \]

\[ f2 := 2.5\sqrt{\text{qu2}} \quad f2 = 65.881 \text{ psi} \quad \text{less than} \quad .158 \text{qu2} = 109.722 \text{ psi} \]

\[ \text{Qu1} := \frac{\pi \cdot Ds \cdot L1 \cdot f1 \cdot 144}{1000} \quad \text{Qu1} = 311.788 \text{ kips} \]

\[ \text{Qu2} := \frac{\pi \cdot Ds \cdot L2 \cdot f2 \cdot 144}{1000} \quad \text{Qu2} = 2.146 \times 10^3 \text{ kips} \]

\[ \text{Wpile} := .150 \left( \frac{\pi}{4} \cdot Ds^2 \right) \cdot L \quad \text{Wpile} = 39,761 \text{ kips} \]

\[ \text{Qallow} := 1.33 \cdot \frac{(\text{Qu1} + \text{Qu2} - \text{Wpile})}{2} \quad \text{kips} \]

\[ \text{Qallow} = 1.608 \times 10^3 \]

\[ \text{Qallowuplift} := 1.33 \cdot \frac{(0.7 \text{Qu1} + 0.7 \text{Qu2} + \text{Wpile})}{2} \quad \text{kips} \]

\[ \text{Qallowuplift} = 1.17 \times 10^3 \]
Total settlement of Drilled shaft:

\[ Ac := \frac{\pi}{4} Ds^2 \quad Ac = 7.069 \text{ ft}^2 \]

\[ Ec := 144.57 \sqrt{3500} \quad Ec = 4.856 \times 10^5 \text{ ksf} \]

\[ E\text{core1} := 144.57 \sqrt{695} \quad E\text{core1} = 2.164 \times 10^5 \text{ ksf} \]

\[ E\text{core2} := 144.57 \sqrt{625} \quad E\text{core2} = 2.052 \times 10^5 \text{ ksf} \]

\[ E\text{mass1} := 0.2 \cdot E\text{core1} \quad E\text{mass1} = 4.328 \times 10^4 \text{ ksf} \]

\[ E\text{mass2} := 0.4 \cdot E\text{core2} \quad E\text{mass2} = 8.208 \times 10^4 \text{ ksf} \]

\[ \frac{Ec}{E\text{mass1}} = 11.22 \quad \frac{Ec}{E\text{mass2}} = 5.916 \quad If := 0.4 \]

\[ s_1 := \left[ \frac{(Qu_1-L_1)}{(Ac \cdot Ec)} + \frac{(Qu_1-If)}{(Ds \cdot E\text{mass1})} \right] \cdot 12 \quad s_1 = 0.016 \text{ in} \]

\[ s_2 := \left[ \frac{(Qu_2-L_2)}{(Ac \cdot Ec)} + \frac{(Qu_2-If)}{(Ds \cdot E\text{mass2})} \right] \cdot 12 \quad s_2 = 0.222 \text{ in} \]

\[ S := s_1 + s_2 \]

\[ S = 0.238 \text{ in} \quad \text{Total settlement} \]
Appendix B -- Detailed Computations for Seismic Analysis and Design of the Johnson County Bridge Using AASHTO Specifications
Table of contents of Appendix- B

Overview of SAP 2000 Model.................................................. B-3
LPILE Graphs............................................................................ B-7
Spring Calculations............................................................... B-9
SAP Model Output................................................................. B-12
Pile Design............................................................................ B-15
Wall Pier Design................................................................. B-20
Pile cap............................................................................... B-29
Summary of Reinforcement.................................................. B-32
Section 4.5.4: minimum number of modes equals three times number of spans, or 9 modes this bridge; maximum number of modes equals 25.

View of SAP2000 Model (with concrete extrusions shown):
Enlarged View of Translational and Rotational Springs:
Mode Shape 1 – Period = 0.6981 seconds

Mode Shape 2 – Period = 0.1915 seconds
Mode Shape 3 – Period = .1416 seconds

Mode Shape 5 – Period = .1073 seconds
Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

AASHTO Standard Specifications

Exp. Pier

B-7
Fixed Pier

Fixed Pier

B-8
**Piles Springs For Fixed Pier**

**Pile Information for L-Pile input:**

- \( \phi := 18 \text{ in} \)
- \( \Delta := 0.205 \text{ in} \)
- clearcover := 3 in (to spiral edge)
- 6 #8 bars for vertical reinforcement

**Results from L-Pile:**

- \( M_u := \frac{1150}{12} \) k - ft
- \( M_u = 96 \) k - ft

**Spring Stiffness:**

- \( L_{pile} := \frac{450}{12} \) L_{pile} = 37.5 ft
- \( D := 18 \text{ in} \)
- \( \pi \left( \frac{D}{4} \right)^2 \)
- \( A_{pilie} := \frac{144}{144} \) \( A_{pile} = 1.767 \text{ ft}^2 \)
- \( E_c := \left( 57 \sqrt{3500} \right) \times 144 \quad E_c = 485591.8 \text{ ksf} \)

- \( k_{axial} := \frac{E_c}{L_{pile}} \)
- \( k_{axial} = 2.288 \times 10^4 \frac{k}{\text{ft}} \)

**Using L-Pile:**

- shear := 31.5 kips
- \( \Delta := 0.205 \text{ in} \)

- \( k_{lateral} := \frac{12 \text{shear}}{\Delta} \)
- \( k_{lateral} = 1.844 \times 10^3 \frac{k}{\text{ft}} \)

**Lateral Springs:**

- \( K_x = K_y \Rightarrow \)
- \( K_x := 8 \times k_{lateral} \)
- \( K_x = 1.475 \times 10^4 \frac{k}{\text{ft}} \)

- \( K_{axial} := 8 \times k_{axial} \)
- \( K_{axial} = 1.831 \times 10^5 \frac{k}{\text{ft}} \)
Rotational Springs:

\[ K_{rx} := (k_{axial}) \left[ \frac{8 \cdot (3.75^2)}{144} \right] \]
\[ K_{ry} := (k_{axial}) \left[ \frac{4 \cdot (3.75^2) + 4 \cdot (11.25^2)}{144} \right] \]
\[ K_{rz} := (k_{lateral}) \left[ \frac{4 \cdot (11.859^2) + 2 \cdot (5.303^2)}{144} \right] \]

\[ K_{rx} = 2.574 \times 10^6 \]
\[ K_{ry} = 1.287 \times 10^7 \]
\[ K_{rz} = 1.141 \times 10^6 \]

Piles Springs For Expansion Pier

Pile Information for L-Pile input:
\[ \phi := 18 \text{ in} \]
\[ \Delta := .205 \text{ in} \]
\[ \text{clearcover} := 3 \text{ in (to spiral edge)} \]
\[ 6 \#8 \text{ bars for vertical reinforcement} \]

Results from L-Pile:
\[ M_u := \frac{550}{12} \] \[ M_u = 46 \text{ k - ft} \]

Spring Stiffness:
\[ L_{pile} := \frac{450}{12} \] \[ L_{pile} = 37.5 \text{ ft} \]
\[ D := 18 \text{ in} \]
\[ A_{pile} := \left[ \frac{\pi}{4} \cdot D^2 \right] \]
\[ A_{pile} = 1.767 \text{ ft}^2 \]

\[ E_c := (57 \sqrt{3500}) \cdot 144 \] \[ E_c = 485591.8 \text{ ksf} \]

\[ k_{axial} := \frac{A_{pile} \cdot E_c}{L_{pile}} \] \[ k_{axial} = 2.288 \times 10^4 \text{ k/sf} \]

Using LPile:
\[ \text{shear} := 17.75 \text{kips} \]
\[ \Delta := .05 \text{ in} \]
\[ k_{lateral} := \frac{12 \cdot \text{shear}}{\Delta} \] \[ k_{lateral} = 4.26 \times 10^3 \text{ k/sf} \]
Johnson County Bridge

AASHTO Standard Specifications

Lateral Springs:

\[ K_x = K_y \Rightarrow \quad K_x := 8 \cdot k_{\text{lateral}} \quad K_x = 3.408 \times 10^4 \quad \text{k} \quad \text{ft} \]

\[ K_{\text{axial}} := 8 \cdot k_{\text{axial}} \quad K_{\text{axial}} = 1.831 \times 10^5 \quad \text{k} \quad \text{ft} \]

Rotational Springs:

\[ K_{rx} := (k_{\text{axial}}) \left[ 8 \cdot (3.75^2) \right] \quad K_{rx} = 2.574 \times 10^6 \]

\[ K_{ry} := (k_{\text{axial}}) \left[ 4 \cdot (3.75^2) + 4 \cdot (11.25^2) \right] \quad K_{ry} = 1.287 \times 10^7 \]

\[ K_{rz} := (k_{\text{lateral}}) \left[ 4 \cdot (11.859^2) + 2 \cdot (5.303^2) \right] \quad K_{rz} = 2.636 \times 10^6 \]
### Modal Participating Mass Ratios

#### With Springs for Division 1-A

<table>
<thead>
<tr>
<th>MODE</th>
<th>PERIOD</th>
<th>INDIVIDUAL MODE (%)</th>
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<td></td>
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### DIVISION 1-A Forces & Moments - $EQ_{\text{trans}}$

<table>
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<tr>
<th>Support / Location</th>
<th>Shear 3 kips</th>
<th>Moment 2 kip-ft</th>
<th>Moment 3 kip-ft</th>
<th>Axial kips</th>
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<tbody>
<tr>
<td>Fix Pier Top</td>
<td>115</td>
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<tr>
<td>Exp Pier Top</td>
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<td>890</td>
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<td>Exp Pier Bottom</td>
<td>56</td>
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### DIVISION 1-A Forces & Moments - $EQ_{\text{long}}$

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<th>Support / Location</th>
<th>Shear 2 kips</th>
<th>Moment 2 kip-ft</th>
<th>Moment 3 kip-ft</th>
<th>Axial kips</th>
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<td>Fix Pier Top</td>
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<td>190</td>
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<td>8</td>
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<td>0</td>
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### DIVISION 1-A Forces & Moments - $EQ_{\text{trans}} / R$

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<th>Moment 2 kip-ft</th>
<th>Moment 3 kip-ft</th>
<th>Axial kips</th>
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<td>0</td>
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<tr>
<td>Exp Pier Top</td>
<td>7.5</td>
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<tr>
<td>Exp Pier Bottom</td>
<td>28</td>
<td>144</td>
<td>0</td>
<td>0</td>
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### DIVISION 1-A Forces & Moments - $EQ_{\text{long}} / R$

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<th>Support / Location</th>
<th>Shear 2 kips</th>
<th>Moment 2 kip-ft</th>
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<th>Axial kips</th>
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<tr>
<td>Fix Pier Top</td>
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<tr>
<td>Exp Pier Bottom</td>
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### Dead Load

<table>
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<th>Location</th>
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<tr>
<td>Fix Pier</td>
<td>306</td>
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<tr>
<td>Exp Pier</td>
<td>468</td>
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Total: 479
**DIVISION 1-A**

### Forces & Moments - Load Case 1

(1.00*\(E_{\text{trans}}\) + 0.30*\(E_{\text{long}}\) + 1*DL)

<table>
<thead>
<tr>
<th>Support / Location</th>
<th>Shear 2 kips</th>
<th>Shear 3 kip</th>
<th>Moment 3 kip-ft</th>
<th>Moment 2 kip-ft</th>
<th>Axial kips</th>
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<tbody>
<tr>
<td>Fix Pier Top</td>
<td>32</td>
<td>58</td>
<td>29</td>
<td>268</td>
<td>307</td>
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<td>Fix Pier Bottom</td>
<td>34</td>
<td>27</td>
<td>543</td>
<td>431</td>
<td>469</td>
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<tr>
<td>Exp Pier Top</td>
<td>1</td>
<td>8</td>
<td>0</td>
<td>445</td>
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<tr>
<td>Exp Pier Bottom</td>
<td>9</td>
<td>28</td>
<td>112</td>
<td>144</td>
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### Forces & Moments - Load Case 2

(1.00*\(E_{\text{long}}\) + 0.30*\(E_{\text{trans}}\) + 1*DL)

<table>
<thead>
<tr>
<th>Support / Location</th>
<th>Shear 2 kips</th>
<th>Shear 3 kip</th>
<th>Moment 3 kip-ft</th>
<th>Moment 2 kip-ft</th>
<th>Axial kips</th>
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</thead>
<tbody>
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<td>Fix Pier Top</td>
<td>105</td>
<td>17</td>
<td>95</td>
<td>80</td>
<td>306</td>
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<tr>
<td>Fix Pier Bottom</td>
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<td>1810</td>
<td>129</td>
<td>468</td>
</tr>
<tr>
<td>Exp Pier Top</td>
<td>3</td>
<td>2</td>
<td>0</td>
<td>134</td>
<td>308</td>
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<tr>
<td>Exp Pier Bottom</td>
<td>30</td>
<td>8</td>
<td>375</td>
<td>43</td>
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**Displacement**

<table>
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<th>Longitudinal Direction (in)</th>
<th>Transverse Direction (in)</th>
<th>Max (in)</th>
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<tr>
<td>Bearing of Fixed Pier</td>
<td>1.16924</td>
<td>0.04026</td>
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</tr>
<tr>
<td>Top of Fixed Pier</td>
<td>1.10282</td>
<td>0.04102</td>
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<tr>
<td>Bottom of Fixed Pier Pile Cap</td>
<td>0.18865</td>
<td>0.05776</td>
<td>0.197</td>
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<tr>
<td>Bearing of Expansion Pier</td>
<td>1.17857</td>
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<tr>
<td>Top of Expansion Pier</td>
<td>0.2202</td>
<td>0.03352</td>
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</tr>
<tr>
<td>Bottom of Expansion Pier Pile Cap</td>
<td>0.02909</td>
<td>0.03396</td>
<td>0.045</td>
</tr>
</tbody>
</table>
Axial Capacity (Side Resistance) of Limestone and Shale:

\[ B_r := 1.5 \text{ ft} \quad L_1 := 5 \text{ ft (Shale)} \]
\[ L_2 := 11.5 \text{ ft (Hard Limestone w/ occasional shale seams)} \]

\[ C_{o1} = \frac{2000}{144} \quad C_{o2} = \frac{50 	imes 2000}{144} \]

\[ C_{o1} = 52.778 \text{ psi} \quad (\text{tons} \cdot \text{ft}^2) \cdot (1 \text{ ft}^2) \cdot (144 \text{ in}^2) \cdot (2000 \text{ lb/ton}) = \text{psi} \]

\[ C_{o2} = 694.444 \text{ psi} \]

From Figure 4.6.5.3.1A:

\[ q_{sr1} := 0 \quad q_{sr2} := 75 \text{ psi} \]

\[ Q_{sr1} := \pi \cdot B_r \cdot L_1 \cdot \frac{144}{1000} \quad Q_{sr1} = 0 \text{ kips} \]

\[ Q_{sr2} := \pi \cdot B_r \cdot L_2 \cdot \frac{q_{sr2} \cdot 144}{1000} \quad Q_{sr2} = 585 \text{ kips} \]

\[ W_{\text{pile}} := 0.150 \left( \frac{\pi}{4} \cdot B_r^2 \right) \cdot 37.5 \quad W_{\text{pile}} = 9.9 \text{ kips} \]

Note: 1.33 increase in allowable load; Factor of Safety = 2.5 (AASHTO 4.6.5.4)

\[ Q_{\text{allow}} := 1.33 \cdot \frac{Q_{sr2} - W_{\text{pile}}}{2.5} \]

\[ Q_{\text{allow}} = 306 \text{ kips} \quad \text{(Allowable Downward Force)} \]

\[ Q_{\text{allowup}} := 1.33 \cdot \frac{0.7 \cdot Q_{sr2} + W_{\text{pile}}}{2.5} \]

\[ Q_{\text{allowup}} = 223 \text{ kips} \quad \text{(Allowable Uplift Force)} \]

Note: \( Q_{\text{ult}} = Q.s + Q.t - W_{\text{self}} \) (Compression) and \( 0.7 \cdot Q.s + W \) (uplift); \( Q_{\text{allow}} = Q_{\text{ult}} / FS \)
Total Settlement of a Drilled Shaft (4.6.5.5.2):

\[ Q := 240 \text{kips} \]
\[ l_{ps} := .55 \]
\[ B_r = 1.5 \text{ ft} \]
\[ E_m := 4 \times 10^6 \times \frac{144}{1000} \]
\[ E_m = 5.76 \times 10^5 \text{ ksf} \]
\[ A := \frac{\pi}{4} \times 1.5^2 \quad A = 1.767 \text{ ft}^2 \]
\[ D_r := 37.5 \text{ ft} \]
\[ E_c := 57000 \times \sqrt{3500} \times \frac{144}{1000} \]
\[ E_c = 485592 \text{ ksf} \]
\[ \rho_s := \frac{Q \left( \frac{l_{ps}}{B_r E_m} + \frac{D_r}{A \cdot E_c} \right)}{Q} \quad \rho_s = 0.01064 \text{ ft} \quad 12 \cdot \rho_s = 0.128 \text{ in} \]

\text{Value of } E(m) \text{ is approximate}

\text{Under the ultimate load of 236 kips (118 tons), we will easily be under .4": Thus the shaft does not experience excessive settlement.}
Axial Force On A Pile (Fixed Pier):

\[ P_w := 712 \text{ kips} \]

\[ M_{eql} := 4260 \text{ k} \cdot \text{ft} \quad M_{eqt} := 251 \text{ k} \cdot \text{ft} \]

\[ d_1 := 3.75 \quad d_2 := 3.75 \quad d_3 := 11.25 \]

(distant from center of pile group to center of closest pile center)

\[ \Sigma d_1 := 8 \left( d_1^2 \right) \quad \Sigma d_1 = 112.5 \text{ ft}^2 \] (weak axis)

\[ \Sigma d_2 := 4 \left( d_2^2 \right) + 4 \left( d_3^2 \right) \quad \Sigma d_2 = 562.5 \text{ ft}^2 \] (strong axis)

(Sum of squares of the distances to each pile from the center of the pile group)

Piles Loads:

\[ F_{pile1} := \left( \frac{P_w}{8} \right) + \left( M_{eql} \cdot \frac{d_1}{\Sigma d_1} \right) + \left( M_{eqt} \cdot \frac{d_3}{\Sigma d_2} \right) \]

\[ F_{pile1} = 236 \text{ kips} \]

\[ F_{pile4} := \left( \frac{P_w}{8} \right) + \left( M_{eql} \cdot \frac{d_1}{\Sigma d_1} \right) - \left( M_{eqt} \cdot \frac{d_3}{\Sigma d_2} \right) \]

\[ F_{pile4} = 226 \text{ kips} \]

\[ F_{pile5} := \left( \frac{P_w}{8} \right) - \left( M_{eql} \cdot \frac{d_1}{\Sigma d_1} \right) + \left( M_{eqt} \cdot \frac{d_3}{\Sigma d_2} \right) \]

\[ F_{pile5} = -48 \text{ kips} \]

\[ F_{pile8} := \left( \frac{P_w}{8} \right) - \left( M_{eql} \cdot \frac{d_1}{\Sigma d_1} \right) - \left( M_{eqt} \cdot \frac{d_3}{\Sigma d_2} \right) \]

\[ F_{pile8} = -58 \text{ kips} \]

\[ F_{piledeadload} := .150 \left( \frac{\pi}{4} \cdot 1.5^2 \right) \cdot 37.5 \]

\[ F_{piledeadload} = 10 \text{ kips} \]

Note: \( Q_{ult} = Q.s + Q.t - W.self \) (Compression) and \( .7*Q.s + W \) (uplift);

\( Q_{allow} = Q_{ult} / FS \)
Axial Force On A Pile (Expansion Pier):

\[ P_w := 619 \text{ kips} \]

\[ M_{eql} := 942 \text{ k} - \text{ft} \quad M_{eqt} := 41 \text{ k} - \text{ft} \]

\[ d1 := 3.75 \quad d2 := 3.75 \quad d3 := 11.25 \]

(distant from center of pile group to center of closest pile center)

\[ \Sigma d1 := 8 \left( d1^2 \right) \quad \Sigma d1 = 112.5 \text{ ft}^2 \quad \text{(weak axis)} \]

\[ \Sigma d2 := 4 \left( d2^2 \right) + 4 \left( d3^2 \right) \quad \Sigma d2 = 562.5 \text{ ft}^2 \quad \text{(strong axis)} \]

(Sum of squares of the distances to each pile from the center of the pile group)

Piles Loads:

\[ F_{pile1} := \left( \frac{P_w}{8} \right) + \left( \frac{M_{eql}}{\Sigma d1} \right) \sum d1 + \left( \frac{M_{eqt}}{\Sigma d2} \right) \sum d2 \]

\[ F_{pile1} = 110 \text{ kips} \]

\[ F_{pile4} := \left( \frac{P_w}{8} \right) + \left( \frac{M_{eql}}{\Sigma d1} \right) \sum d1 - \left( \frac{M_{eqt}}{\Sigma d2} \right) \sum d2 \]

\[ F_{pile4} = 108 \text{ kips} \]

\[ F_{pile5} := \left( \frac{P_w}{8} \right) - \left( \frac{M_{eql}}{\Sigma d1} \right) \sum d1 + \left( \frac{M_{eqt}}{\Sigma d2} \right) \sum d2 \]

\[ F_{pile5} = 47 \text{ kips} \]

\[ F_{pile8} := \left( \frac{P_w}{8} \right) - \left( \frac{M_{eql}}{\Sigma d1} \right) \sum d1 - \left( \frac{M_{eqt}}{\Sigma d2} \right) \sum d2 \]

\[ F_{pile8} = 45 \text{ kips} \]
PCACOL Output. Piles

MATERIAL:
\( f'c = 3.5 \text{ ksi} \)
\( E_c = 3372.17 \text{ ksi} \)
\( f_c = 2.976 \text{ ksi} \)
\( \beta_0 = 0.05 \)
\( \gamma_y = 60 \text{ ksi} \)
\( E_o = 290000 \text{ ksi} \)

SECTION:
\( A_g = 254.459 \text{ in}^2 \)
\( l_c = 5153 \text{ in}^4 \)
\( I_y = 5153 \text{ in}^4 \)
\( X_0 = 0 \text{ in} \)
\( y_0 = 9 \text{ in} \)

REINFORCEMENT:
8 @ 8.6 bars @ 1.38\% 
As = 3.52 in²
Conf. Spiral Clear Cover = 3.675 in
**Important Quantities:**

\[
\begin{align*}
 f_c & = 3.5 \text{ ksi} \\
 E_c & = 1820 \sqrt{f_c} \\
 E_c & = 3.405 \times 10^3 \text{ ksi} \\
 f_r & = 0.24 \sqrt{f_c} \\
 f_r & = 0.449 \text{ ksi} \\
 f_s & = 60 \text{ ksi} \\
 E_s & = 29000 \text{ ksi} \\
 n & = \frac{E_s}{E_c} \\
 n & = 8.517
\end{align*}
\]

**Division 1-A: Section 3**

3.4 acceleration equals .15g  
SPC is category B ( IC can be I or II )

3.9 Load Case 1: 100% Longitudinal + 30% Transverse
Load Case 2: 30% Longitudinal + 100% Transverse

**Division 1-A: Section 6**

6.2.1 Group Load equals 1.0(D + B + SF + E + EQM)

6.3.1 Design Displacements
\[
\begin{align*}
 L & = 46.5 \\
 H & = 17.75 \\
 S & = 0 \\
 N & = (8 + 0.02 \cdot L + 0.08 \cdot H) \cdot \left(1 + 0.000125 \cdot S^2\right) \\
 N & = 10.35 \\
 \text{This is the Required Length of Seat per side in Inches}
\end{align*}
\]

Available Seat Width for entire expansion Pier (Pier 2 on each bridge) equals 26 inches

6.6.2 Special Requirements when designed as Columns ( N/A if designed as pier )
Can use 7.6.3 as a suggested minimum for vertical and horizontal reinforcement of .25%.

**Wall Design for Seismic:**

From SAP2000 Model (fixed pier forces only):

\[
\begin{align*}
P_1 & = 472 \quad \text{Axial Load on pier for LC1} \\
M_{w1} & = 1810 \quad \text{Moment about strong axis (bending in weak direction) of wall for LC1} \\
M_{s1} & = 129 \quad \text{Moment about weak axis (bending in strong direction) of wall for LC1} \\
P_2 & = 470 \quad \text{Axial Load on pier for LC2} \\
M_{w2} & = 543 \quad \text{Moment about strong axis of wall for LC2} \\
M_{s2} & = 431 \quad \text{Moment about weak axis of wall for LC2}
\end{align*}
\]
Division 1: Section 8

8.16 Strength Design Method

8.16.1.2.2

Calculation of the \( \phi \) factor:

\[
f_c = 3.5 \quad A_g := 308\cdot26 \quad A_g = 8.008 \times 10^3
\]

\[
P_u := 473 \quad \phi P_n := P_u \quad \phi := .9 - 2 \frac{\phi P_n}{f_c A_g} \quad \phi = 0.87
\]

8.16.5 Slenderness Effects in Compression Members

\[
r := .3\cdot26 \quad r = 7.8 \quad \text{in} \quad 8.16.5.2.2
\]

\[
k := 2 \quad 8.16.5.2.3
\]

\[
l_u := (3 + 2 + 11)\cdot12 + 9 \quad l_u = 201 \quad \text{in}
\]

\[
\text{slenderness} := k \cdot \frac{l_u}{r} \quad \text{slenderness} = 51.538 \quad \text{Moment Magnifier is required.}
\]

8.16.5.2.7

\[
\beta_d := 1 \quad l_s := \left(0.79\cdot0.9^2\right)\cdot50 \quad l_s = 3.2 \times 10^3
\]

\[
l_g := \frac{1}{12} \cdot 308\cdot26^3 \quad l_g = 4.511 \times 10^5
\]

\[
E_l := \frac{\left[(E_c l_g) / 5 + E_s l_s\right]}{1 + \beta_d} \quad E_l = 199994062
\]

\[
P_c := \frac{\left(\pi^2 \cdot E_l\right)}{\left(2 \cdot l_u\right)^2} \quad P_c = 12214
\]

\[
\phi P_c := \phi \cdot P_c \quad \phi P_c = 10581
\]

\[
P_u := 473.5
\]

\[
\delta_b := \frac{1}{P_u} \quad \delta_b = 1.05
\]
Clear cover of 3 in. was used for each wall, measured to side of transverse rebars, assuming #6 bars were the appropriate size for transverse reinforcement.

Vertical reinforcement is 46 #6 for .25%. 22 bars spaced evenly along each long face, with one additional vertical bar placed in the center on each short side.

Expansion:
Vertical reinforcement is 46 #9 for .57%. 22 bars spaced evenly along each long face, with one additional vertical bar placed in the center on each short side.

Using PCACOL:
Vertical reinforcement is 46 #9 for .57%. 22 bars spaced evenly along each long face, with one additional vertical bar placed in the center on each short side.

Minimum Steel Required:
Using Division 1-A (7.6.3): Piers must have a $\rho_{(min)} = 0.025$ for both $\rho_h$ and $\rho_n$.

$(.0025)(308.26) = 20.02 \text{ in}^2$ (minimum amount of vertical steel)

B-22
Johnson County Bridge – Fixed Pier
Johnson County Bridge – Expansion Pier
8.16.2.3.1 - Transverse Reinforcement:

\[ \rho_h := .0025 \quad A_h := \rho_h \cdot 12 \cdot 26 \quad A_h = 0.78 \quad \text{in}^2 \]

\[ A_b := .79 \quad s := 12 \cdot 2 \cdot \frac{A_b}{A_h} \quad s = 24.308 \quad \text{Spacing for #8 bars.} \]

\[ A_b := .60 \quad s := 12 \cdot 2 \cdot \frac{A_b}{A_h} \quad s = 18.462 \quad \text{Spacing for #7 bars.} \]

\[ A_b := .44 \quad s := 12 \cdot 2 \cdot \frac{A_b}{A_h} \quad s = 13.538 \quad \text{Spacing for #6 bars; Use these at 12" o.c.} \]

\[ A_b := .31 \quad s := 12 \cdot 2 \cdot \frac{A_b}{A_h} \quad s = 9.538 \quad \text{Spacing for #5 bars.} \]

\[ A_h := 12 \cdot 2 \cdot \frac{.44}{12} \quad A_h = 0.88 \quad \rho_h := \frac{A_h}{12 \cdot 26} \quad \rho_h = 0.0028 \]

8.18.2.3.4: spacing for "cross ties" can not be more than 24"

Cross ties will restrain the width of the wall at every third vertical in the fixed pier and every other vertical in the expansion pier. Every other row as you go up the height of the wall will be restrained in this manner. Since all bars are #10 or smaller, #4 ties are the appropriate size. The ties will have a 90 deg. hook on one end and a 180 deg. hook on the other; the ties will be continuous through the width of the pier and will alternate hook types as you go along the width or height of the wall.

An additional note: all transverse reinforcement will have a U-shaped piece spliced to each end to go around the short ends of each wall. These bars will be of the same size and spacing as the horizontal bars, and will have appropriate splice lengths.

8.25.1: \[ l_d := 1.7 \left( 0.44 \cdot \frac{60000}{\sqrt{3500}} \right) \quad l_d = 30.344 \quad \text{Use 31" for development length of U's} \]

8.27.3: \[ \text{Use 31" for splice length of horizontal #6's} \]

8.23.2: Hooks should have a diameter of 9" for #9 bars and 4.5" for #6 bars; these will be used with the appropriate development length to create dowels for vertical rebars.
8.25.1: *Use 18" development length for a #6 bar hook, and 41" development length for #8 bar hook.*

8.23.1: *Extension at free end of standard hook is 13.5" for #8 bar, and 9" for #6 bar*

8.16.6.2.2 Shear Strength

\[ f_c := 3500 \]
\[ \rho_h := 0.003 \]
\[ f_y := 60000 \]

\[ R = 1.0 \]

Div 1A (7.6.3): 

\[ \phi_c := 2 \cdot \sqrt{f_c} \]
\[ \phi_c = 118 \text{ psi} \]

**Fix Pier:** 

\[ V_{w1} := 105 \text{ kip} \]
\[ V_{s1} := 8 \text{ kip} \]
\[ V_{w2} := 31.5 \text{ kip} \]
\[ V_{s2} := 27 \text{ kip} \]

\[ V_{R1} := \sqrt{V_{w1}^2 + V_{s1}^2} \]
\[ V_{R1} = 105 \text{ kip} \]

\[ V_{R2} := \sqrt{V_{w2}^2 + V_{s2}^2} \]
\[ V_{R2} = 41 \text{ kip} \]

**Expansion Pier:** *We are using the same minimum amount of horizontal steel for the fixed pier as for the expansion pier; the fixed pier has higher shear forces, so this design will work for both piers.*

\[ \nu_r := \frac{V_{R1} \cdot 1000}{308 \cdot 26} \]
\[ \nu_r = 13 \text{ psi} \]

**Applied shear stress at wall/pilecap connection**

\[ \phi v_c := 0.85 v_c \]
\[ \phi v_c = 101 \text{ psi} \]

*Concrete is adequate for applied shear force in the pier wall.*
Foundation Design for Seismic:

Design of Piles (using the max deflection of cap under fixed pier):

Using SAP2000 Model we have these given deflections (inches) at the tops of the piles due to loading:

\[ \Delta_1 := 0.189 \quad \Delta_2 := 0.058 \quad \Delta_{\text{max}} := \sqrt{\left( \Delta_1^2 \right) + \left( \Delta_2^2 \right)} \quad \Delta_{\text{max}} = 0.198 \text{ inches} \]

From LPile:

\[ v_{\text{max}} := 31.5 \]

Shear Stress:

\[ v_{\text{stress}} := \frac{v_{\text{max}}}{18 \cdot 14.125} \cdot \frac{1000}{1.33} \quad v_{\text{stress}} = 93.153 \text{ psi} \]

\[ v_{C} := \left( 0.95 \sqrt{3500} \right) \quad v_{C} = 56.203 \text{ psi} \]

The Pile requires shear reinforcement since \( V_{\text{stress}} \) is more than \( V_{C} \).

\[(8-7): \quad f_{S1} := \left( v_{\text{stress}} - v_{C} \right) \cdot 18 \cdot \frac{g}{31} \quad f_{S1} = 19310 \text{ psi} \]

The limit for \( f_{S} = 24000 \text{ psi}; \text{ so 9" pitch on a \#5 spiral will work.} \)

\[ v_{\text{noshear}} := \left( 0.5 \cdot v_{C} \right) \cdot \frac{18 \cdot 14.125}{1000} \quad v_{\text{noshear}} = 7 \text{ kips} \]

At around 7 kips (13 feet below the cap), the concrete alone can handle the shear, so no shear reinforcement will be needed below that point, just a minimum required for adequate lateral reinforcement, spacing will be 9" (See Division 1-A, 6.4.2(C) ).

Using LPile, the forces due to the deflections were:

\[ M := 97 \text{ kip – ft} \]

Using PCACOL: a 18" diameter column bent about one axis, with the given loading needs:

8 #6's for vertical reinforcement which has a \( \rho = 0.00138; \text{ minimum is 0.5% (D1-A, 6.4.2(C) )} \).
The cap is 3' thick. The pile longitudinal reinforcement will go up into the cap for a distance of 14", then the ends of the #6 bars will have 90 degree hooks with 9" extensions.

Use a #5 spiral with a 3" pitch for the first 13' of the pile below the cap and the bottom 2' feet of the pile; also continue spiral up into the cap and terminate at the tops of the verticals with an appropriate hook.

Spiral Laps: 48 \cdot d_b = 30 \text{ in} \quad \text{(controls)}

- or -

Use a 30" lap splice to connect two pieces of spiral together.

Development Lengths:

for flexure: effective depth = 14.125" (controls)

15 \cdot \text{d.b} = 11.25"
Pile cap:  

Moment:

\[ M_u := 3465 \text{ k-ft} \]
\[ b_{ef} := (.8 \cdot 2.67) + 3.75 \quad b_{ef} = 5.89 \text{ ft} \]  
(Actual spacing = 7.5'; so 5.89' controls)
\[ R := \left( \frac{M_u \cdot 12000}{.9 \cdot 26 \cdot 12 \cdot 32^2} \right) \quad R = 145 \text{ psi} \]

\[ \omega := .0025 \left( \frac{60000}{3500} \right) \quad R_d := \omega \cdot 3500 \cdot (1 - .59 \cdot \omega) \quad R_d = 146.207 \text{ psi} \]

\[ A_s := (.0025 \cdot 12 \cdot .32) \quad A_s = 0.96 \frac{\text{in}^2}{\text{ft}} \text{ of footing (transverse steel)} \]

Transverse rebar for footing is chosen to be #6 bars @ 5.5" o.c.

8.17.1.1:

\[ f_r := 7.5 \cdot \sqrt{3500} \quad f_r = 444 \text{ psi} \quad l_g := 12 \cdot (12 \cdot b_{ef}) \cdot 36^3 \quad l_g = 39544879 \text{ in}^4 \]
\[ y_t := 18 \text{ in} \]
\[ M_{cr} := \frac{(f_r \cdot l_g)}{(y_t)} \quad \left( 1.2 \cdot M_{cr} \right) \cdot \frac{1}{12000} = 97479 \text{ k-ft} \]
\[ R_{cr} := \left( \frac{1.2 \cdot M_{cr}}{(.9 \cdot 26 \cdot 12 \cdot 32^2)} \right) \quad R_{cr} = 4068 \text{ psi} \]

This number is off the charts; but it is permissible to waive this requirement if a 33% increase in the required area of steel is provided.

\[ A_s := (.0025 \cdot 12 \cdot .32) \cdot 1.33 \quad A_s = 1.277 \frac{\text{in}^2}{\text{ft}} \]

Use #6 bars at 4" o.c. for transverse reinforcement

\[ A_{sl} := (.0018 \cdot 12 \cdot .32) \quad A_{sl} = 0.691 \frac{\text{in}^2}{\text{ft}} \]

Use #6 bars at 7.5" o.c. for longitudinal reinforcement
Pile cap works in two-way shear.

Two-Way (8.15.5.6):

\[ b_0 := \pi \cdot 2 \cdot (9 + 16) \quad b_0 = 157 \text{ in} \]

\[ v_{2\text{way}} := \frac{236 \cdot 1000}{(b_0 \cdot 32)} \cdot \frac{1}{1.33} \quad v_{2\text{way}} = 35 \text{ psi} \]

\[ v_c = 53 \text{ psi} \]

Pile cap works in two-way shear.
Pullout of Piles in Tension:

\[ A_s := 8.44 \quad A_s = 3.52 \quad \text{in}^2 \]

\[ f_y := 24000 \]

\[ FS := 2 \quad \text{(Also: 33\% increase in allowable stresses)} \]

\[ \frac{1.33 \left( A_s f_y \right)}{FS} \cdot \frac{1}{1000} = 56 \quad \text{kips} \quad \text{(Allowable Uplift of a Single Pile)} \]

\[ F_{uplift} := \frac{58}{.8} \quad \text{(Divide Ultimate Load by R = .8 for connections)} \]

\[ F_{uplift} = 73 \quad \text{kips} \quad \text{(Largest Uplift Force on a Single Pile)} \]

\[ A_{s2} := 11.44 \quad A_{s2} = 4.84 \quad \text{in}^2 \]

\[ \frac{1.33 \left( A_{s2} f_y \right)}{FS} \cdot \frac{1}{1000} = 77 \quad \text{kips} \]

We need 3 (or 4) #6 bar dowels of appropriate length to resist pullout failure

8.25.1:

\[ l_d := .04.44 \cdot \frac{60000}{\sqrt{3500}} \quad l_d = 17.85 \quad \text{in} \]

8.25.3.3:

\[ .75 \cdot l_d = 13.387 \quad \text{Use a development length of 13.5” into the main part of the pile for the #6 dowels} \]

8.29.2:

\[ l_{hb} := 1200 \cdot \frac{.750}{\sqrt{3500}} \quad l_{hb} = 15.213 \quad \text{in} \]

The #6 dowels and #6 verticals will terminate in the slab with an \( L(hb) \) of 11", and a 90 degree hook with a 9" extension.
Shear at the Connection (assume concrete is cracked):

From LPile: \[ v_{\text{max}} = \frac{31.5}{8} \] (Use R factor of .8 for connections)

Shear Stress: \[ v_{\text{stress}} = \frac{v_{\text{max}}}{18 \cdot 14.125} \cdot \frac{1000}{1.33} \]
\[ v_{\text{stress}} = 116.442 \text{ psi} \]

(Use 33% increase in allowable stresses)

\[ (8-7): \quad f_s = \left( v_{\text{stress}} \right) \cdot 18 \cdot \frac{3}{31} \quad f_s = 20283 \text{ psi} \]

The limit for \( f(s) = 24000 \text{ psi} \); so 3" pitch on a #5 spiral will work when no concrete shear strength is apparent.

Summary of Reinforcement

**Pile Cap**
Longitudinal (Strong Axis) - #6 bars @ 7.5" o.c. = 18
Transverse (Weak Axis) - #6 bars @ 4" o.c. = 78

**Piles**
Vertical – 8 #6 bars + 4 #6 dowels
Lateral - #5 bar (16.5’ at 3” pitch; 22.5’ at 9” pitch)

**Fixed Pier**
Verticals – 46 #9 bars (22 per long side; 1 per short side)
Transverse - #6 bars @ 12” centers = 12
  - #6 U-bars @ 12” centers = 24

Cross Ties - #4 bars (every other row; and at every third vertical) = 42

**Expansion Pier**
Verticals – 46 #6 bars (22 per long side; 1 per short side)
Transverse - #6 bars @ 12” centers = 13
  - #6 U-bars @ 12” centers = 26

Cross Ties - #4 bars (every other row; and at every other vertical) = 66
Appendix C -- Detailed Computations for Seismic Analysis and Design of the St. Clair County Bridge using Proposed NCHRP Specification
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TITLE:
Saint Clair County Bridge

SUBJECT FILE:
Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

NCHRP Guidelines

Pertinent Information about Materials used, etc. :

\[ p_{\text{conv}} := (1.45 \times 10^{-4}) \quad \text{This is the N/m}^2 \text{ to psi conversion factor.} \]

\[ \text{feet} := \frac{1}{0.3048} \quad \text{feet} = 3.281 \quad \text{This is the meter to feet conversion.} \]

\[ \text{inch} := \text{feet} \times 12 \quad \text{inch} = 39.37 \quad \text{This will convert meters to inches.} \]

\[ f_c := 24 \left(10^6 \right) p_{\text{conv}} \quad f_c = 3480.000 \]

\[ f_y := 400 \left(10^6 \right) p_{\text{conv}} \quad f_y = 5.8 \times 10^4 \quad \text{(Reinforcement)(psi)} \]

\[ F_{y1} := 345 \left(10^6 \right) p_{\text{conv}} \quad F_{y1} = 5.003 \times 10^4 \quad \text{(Structural) (psi)} \]

\[ F_{y2} := 250 \left(10^6 \right) p_{\text{conv}} \quad F_{y2} = 3.625 \times 10^4 \quad \text{(Structural) (psi)} \]

\[ E_c := 57000 \left(\sqrt{f_c} \right) \quad E_c = 3.363 \times 10^6 \quad \text{Modulus of Elasticity of concrete.} \]

\[ E_s := 29000000 \quad E_s = 2.9 \times 10^7 \quad \text{Modulus of Elasticity of steel.} \]

\[ n := \frac{E_s}{E_c} \quad n = 8.624 \quad \text{Modular Ratio} \]

\[ \text{boltD} := 24 \left(10^{-3} \right) \text{inch} \quad \text{boltD} = 0.945 \quad \text{Bolt Diameter (in.)} \]

\[ \text{Skew} := 26 + \frac{31}{60} + \left(\frac{40}{3600}\right) \quad \text{Skew} = 26.528 \quad \text{Skew Angle (degrees)} \]

Calculations for Moment of Inertia for Superstructure

\[ L_{\text{eff}} := 41.50 \text{-inch} \quad L_{\text{eff}} = 1.634 \times 10^3 \quad \text{Effective length of a span (in)} \]

\[ t_{\text{avg}} := .195 \text{-inch} \quad t_{\text{avg}} = 7.677 \quad \text{Slab thickness (in)} \]

\[ t_w := .014 \text{-inch} \quad t_w = 0.551 \quad \text{Web thickness (in)} \]

\[ b_{f1} := .300 \text{-inch} \quad b_{f1} = 11.811 \quad \text{Top flange width outerline (in)} \]

\[ b_{f2} := .600 \text{-inch} \quad b_{f2} = 23.622 \quad \text{Top flange width innerline (in)} \]

\[ gspace := 1.905 \text{-inch} \quad gspace = 75 \quad \text{Girderline spacing (in)} \]
1. Effective Flange Width of interior maybe be taken as least of:

\[
efw_1 := 0.25 \cdot L_{eff} \\
efw_2 := (12 \cdot t_{avg}) + t_w \\
efw_3 := (12 \cdot t_{avg}) + 0.5 \cdot b_f_1 \\
efw_3b := (12 \cdot t_{avg}) + 0.5 \cdot b_f_2 \\
efw_4 := gspace \\
\text{EFW}_{inside} := gspace \\
\text{EFW}_{inside} = 75
\]

\[
efw_5 := \frac{1}{8} \cdot L_{eff} \\
efw_6 := (6 \cdot t_{avg}) + 0.5 \cdot t_w \\
efw_7 := (6 \cdot t_{avg}) + 0.25 \cdot b_f_1 \\
efw_8 := (6 \cdot t_{avg}) + 0.25 \cdot b_f_2 \\
ohw := 0.925 \text{-inch} \\
\text{EFW}_{outside} := \left(0.5 \cdot \text{EFW}_{inside}\right) + ohw \\
\text{EFW}_{outside} = 73.917
\]

We can use all of the slab for moment of inertia.

For Moment of Inertia Calculations of the superstructure.

\[
I_{yy1} := 36.2084 \\
I_{zz1} := 3676.5792 \\
I_{yy2} := 61.4485 \\
I_{zz2} := 5343.1700
\]
Fig.1. Cross-section of superstructure

Torsional Properties:

We assume only the deck will resist torsion.

\[
b_{\text{deck}} := 20.8\text{-feet} \quad \quad b_{\text{deck}} = 68.241 \text{ (ft)}
\]
\[
b_{\text{deck}} := .195\text{-feet} \quad \quad h_{\text{deck}} = 0.64 \text{ (ft)}
\]
\[
J := \left( b_{\text{deck}} \right) \frac{\left( h_{\text{deck}} \right)^3}{3} \quad \quad J = 5.956 \text{ (ft}^4\text{)}
\]

Cross-Sectional Area of Superstructure:

\[
b_{\text{slab}} := 20.8\frac{\text{feet}}{n} \quad \quad n
\]
\[
b_{\text{slab}} = 7.913
\]
\[
h_{\text{slab}} := .195\text{-feet} \quad \quad h_{\text{slab}} = 0.64
\]
\[
\text{area}_{\text{slab}} := h_{\text{slab}} b_{\text{slab}} \quad \quad \text{area}_{\text{slab}} = 5.062
\]
area_{girder1} := 11\cdot[(.02\cdot.3) + (.02\cdot.6) + (1.372\cdot.014)]\cdot\text{feet}^{2} \quad \text{area}_{girder1} = 4.406 \\
area_{girder2} := 11\cdot[(.045\cdot.6) + (.045\cdot.6) + (1.372\cdot.014)]\cdot\text{feet}^{2} \quad \text{area}_{girder2} = 8.668 \\
area_{super1} := \left(area_{slab} + area_{girder1}\right) \quad \text{area}_{super1} = 9.468 \quad \text{ft}^{2} \\
area_{super2} := \left(area_{slab} + area_{girder2}\right) \quad \text{area}_{super2} = 13.73 \quad \text{ft}^{2} \\
area_{barrier} := [(2\cdot.535) + (0.65\cdot.535) + (0.32\cdot.330) + (0.5\cdot.18\cdot.330)]\cdot\text{feet} \quad \text{area}_{barrier} = 2.982 \\
area_{median} := \left[.150\cdot\left(\frac{1.3 + 1.236}{2}\right)\right]\cdot\text{feet} \quad \text{area}_{median} = 2.047 \\

**Loads:**

**Dead Loads of the superstructure**

\text{w}_{\text{barrier}} := 2\cdot\text{area}_{\text{barrier}}\cdot150 \quad \text{w}_{\text{barrier}} = 895 \quad \text{plf} \\
\text{w}_{\text{slab}} := (19.05\cdot\text{feet})\cdot(.195\cdot\text{feet})\cdot150 \quad \text{w}_{\text{slab}} = 5998 \quad \text{plf} \\
\text{w}_{\text{girder1}} := 11\left[(1.372\cdot\text{feet})\cdot(.014\cdot\text{feet}) + (.020\cdot\text{feet})\cdot(.900\cdot\text{feet})\right]\cdot490 \quad \text{w}_{\text{girder1}} = 2159 \quad \text{plf} \\
\text{w}_{\text{girder2}} := 11\left[(1.372\cdot\text{feet})\cdot(.014\cdot\text{feet}) + (.045\cdot\text{feet})\cdot(1.200\cdot\text{feet})\right]\cdot490 \quad \text{w}_{\text{girder2}} = 4247 \quad \text{plf} \\
\text{w}_{\text{median}} := \text{area}_{\text{median}}\cdot190 \quad \text{w}_{\text{median}} = 0.389 \quad \text{plf} \\

**Interior Cross-frames:**

2 - L4x4x5/16 @ 2546 mm \\
1 - L4x4x5/16 @ 1904 mm \\
weight(lb.) per linear foot = 8.16 \\
10 spaces underneath bridge \quad 13 cross frames per length \quad total number = 130


Future Wearing Surface was 2.4 kN/m^2 or about 50 psf.

\[
w_{frame} := 13 \cdot 10 \cdot \left[ (2.546 + 2.546 + 1.904) \cdot \text{feet} \right] \cdot \frac{8.16}{272}, \quad w_{frame} = 89.516 \text{ plf}
\]

\[
w_{bridge} := (3 + 4.2 + .3 + 3.9 + 4.2 + 3) \cdot \text{feet}, \quad w_{bridge} = 61.024 \text{ ft}
\]

\[
w_{fws} := 2.4 \cdot 1000 \cdot \text{pconv} \cdot 144 \cdot w_{bridge}, \quad w_{fws} = 3.058 \times 10^3 \text{ ft}
\]

Adding the individual pieces together we get:

\[
w_{outer} := w_{barrier} + w_{median} + w_{frame} + w_{slab} + w_{girder1} + w_{fws}, \quad w_{outer} = 12199 \text{ plf}
\]

\[
w_{inner} := w_{barrier} + w_{median} + w_{frame} + w_{slab} + w_{girder2} + w_{fws}, \quad w_{inner} = 14288 \text{ plf}
\]

\(W(outer)\) is the uniform dead load along the superstructure from the abutments to the place where the field splices are located. \(W(inner)\) occupy the center of each girderline which are the continuous part over the central pier.
Response Spectrum:

Site Class = Medium Stiff Clay

Site Class D (MCEER Guide Spec. 3.4.2.1)

SDR = 6

SDAP = D

Type III per AASHTO Spec.

\[ s := 1.5 \quad \text{AASHTO Table 3.5.1} \]

- Design Response Spectrum Development - MCE

Using USGS website to find probabilistic ground motion values, in \( \%g \), at nearest grid point area (2% PE in 50 year):

\[ S_s := 0.6261940 \quad \text{0.2-second period spectral acceleration} \]

\[ S_1 := 0.196946 \quad \text{1-second period spectral acceleration} \]

\[ F_a := 1.3 \quad \text{Site coefficient for short period} \]

\[ F_v := 2.0 \quad \text{Site coefficient for long period} \]

\[ S_{DS} := F_a S_s \]

\[ S_{DS} = 0.814 \quad \text{Design earthquake response spectral acceleration at short period} \]

\[ S_{D1} := F_v S_1 \]

\[ S_{D1} = 0.394 \quad \text{Design earthquake response spectral acceleration at long period} \]

\[ 0.4 S_{DS} = 0.326 \]

\[ T_s := \frac{S_{D1}}{S_{DS}} \quad \text{Period at the end of construction design spectral acceleration plateau} \]
\[ T_S = 0.484 \text{ Sec} \]
\[ T_0 := 0.2 \cdot T_S \quad \text{Period at the beginning of construction design spectral acceleration plateau} \]
\[ T_0 = 0.097 \text{ Sec} \]
\[ F_a \cdot S_s = 0.814 \quad 0.6 < F_a S_s \]

**Seismic Hazard Level IV** (Guide Spec. Table 3.7-1)

- Design Response Spectrum Development - FE

- Design Response Spectrum Development - MCE

Using USGS website to find probabilistic ground motion values, in \( %g \), at nearest grid point area (2% PE in 50 year):

\[ S_{s_{\text{fe}}} := 0.06763 \quad 0.2\text{-second period spectral acceleration} \]
\[ S_{1_{\text{fe}}} := 0.01485 \quad 1\text{-second period spectral acceleration} \]
\[ F_{a_{\text{fe}}} := 1.6 \quad \text{Site coefficient for short period} \]
\[ F_{v_{\text{fe}}} := 2.4 \quad \text{Site coefficient for long period} \]

\[ S_{DS_{\text{fe}}} := F_{a_{\text{fe}}} \cdot S_{s_{\text{fe}}} \]
\[ S_{DS_{\text{fe}}} = 0.108 \quad \text{Design earthquake response spectral acceleration at short period} \]

\[ S_{D1_{\text{fe}}} := F_{v_{\text{fe}}} \cdot S_{1_{\text{fe}}} \]
\[ S_{D1_{\text{fe}}} = 0.036 \quad \text{Design earthquake response spectral acceleration at long period} \]

\[ 0.4 \cdot S_{DS_{\text{fe}}} = 0.043 \]
TS_fe := \frac{S_{D1_fe}}{S_{DS_fe}} \quad \text{Period at the end of construction design spectral acceleration plateau}

T_{S_fe} = 0.329 \text{ Sec}

T_{0_fe} := 0.2 \cdot T_{S_fe} \quad \text{Period at the beginning of construction design spectral acceleration plateau}

T_{0_fe} = 0.066 \text{ Sec}

F_{a_fe} S_{s_fe} = 0.108 \quad F_{a_fe} S_{s_fe} \leq 0.15

F_{v_fe} S_{1_fe} = 0.036 \quad F_{v_fe} S_{1_fe} \leq 0.15

\textbf{Seismic Hazard Level I} (Guide Spec. Table 3.7-1)

\begin{center}
\includegraphics[width=\textwidth]{graph.png}
\end{center}

\textbf{Graph 1.} response Spectrum (MCE and FE)
Response Modification Factor for MCE:

\[ T_S = 0.484 \]
\[ T := 0.7720 \quad \text{Period of vibration of Bridge} \]
\[ R_B := 1.5 \]
\[ R_{col} := 1 + \left( R_B - 1 \right) \frac{T}{1.25 \cdot T_S} \quad \text{Response Modification Factor for substructure} \]
\[ R_{col} = 1.638 < R_B \]
\[ \text{So} \quad R_{col} := 1.5 \]

Response Modification Factor for FE:

\[ T_S := 0.329 \]
\[ T := 0.62169 \quad \text{Period of vibration of Bridge} \]
\[ R_B := 0.9 \]
\[ R_{col} := 1 + \left( R_B - 1 \right) \frac{T}{1.25 \cdot T_S} \quad \text{Response Modification Factor for substructure} \]
\[ R_{col} = 0.849 < R_B \quad \text{ok} \]
\[ \text{So} \quad R_{col} := 0.849 \]
SAP 2000 model:

Fig. 2. SAP modeling of Saint Clair Bridge with section extrusion.

Fig. 3. First Mode shape, longitudinal direction (T=0.77220 Sec)
Table 1. Modal Participating Mass Ratios

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<tr>
<th>OutputCase</th>
<th>Step Type</th>
<th>Step Num</th>
<th>Period</th>
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<th>UY</th>
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Table 1. Modal Participating Mass Ratios
Group Pile Spring Stiffness (roller at abutments in transverse direction)

Maximum Allowable Driving Loads (driven piles)
AASHTO LRFD, 10.7.1.16

Compression \[ 0.9 \phi F_y \cdot A_g \]
Tension \[ 0.9 \phi F_y \cdot A_n \]

Compressive resistance
AASHTO LRFD, 6.9.2.1&2
\[ \phi_c = 0.9 \quad \text{AASHTO LRFD 6.5.4.2} \]

Use HP 14X117 for the NCHRP design
A=34.4 in^2
lxx=1220 in^4 \quad Zxx=194 in^3
lxy=443 in^4 \quad Zyy=91.4 in^3
Depth=14.2 in
Width=14.9 in

\[ E := \frac{29000000 \cdot 144}{1000} \quad \text{ksf} \]
\[ N := 105 \quad \text{21X5 (75\" in long and 75\" spacing in transverse direction)} \]

\[ K_{\text{long\_single}} := 45 \cdot \frac{12}{6} \]
\[ K_{\text{long\_single}} = 900 \quad \frac{k}{ft} \]
\[ K_x := K_{\text{long\_single}} \cdot N \]
\[ K_x = 9.45 \times 10^4 \quad \frac{k}{ft} \]
\[ L := 80.417 \quad \text{ft} \]
\[ K_{\text{tran\_single}} := 51 \cdot \frac{12}{1.4} \]
\[ K_{\text{tran\_single}} = 437.143 \]
\[ K_y := K_{\text{tran\_single}}N \]

\[ K_y = 4.59 \times 10^4 \quad \text{k} \quad \text{ft} \]

\[ A := \frac{34.4}{144} \]

\[ A = 0.239 \quad \text{ft}^2 \]

\[ K_{\text{axial}} := A \cdot \frac{E}{L} \]

\[ K_{\text{axial}} = 1.241 \times 10^4 \quad \text{k} \quad \text{ft} \]

\[ K_z := K_{\text{axial}}N \]

\[ K_z = 1.303 \times 10^6 \quad \text{k} \quad \text{ft} \]

\[ S_1 := \frac{10}{144} \left[ (1.75)^2 + (2.75)^2 + (3.75)^2 + (4.75)^2 + (5.75)^2 + (6.75)^2 + (7.75)^2 + (8.75)^2 + (9.75)^2 + (10.75)^2 \right] \]

\[ K_{rx} := K_{\text{axial}}S_1 \quad \text{S}_2 := \frac{2}{144}N \left( 1.75^2 + 2.75^2 \right) \]

\[ K_{ry} := K_{\text{axial}}S_2 \]

\[ K_{rz} := \left( \frac{K_{rx}}{K_{\text{axial}}} \right) + \left( \frac{K_{ry}}{K_{\text{axial}}} \right) \left( K_{\text{long\_single}} + K_{\text{tran\_single}} \right) \]

\[ K_x = 9.45 \times 10^4 \]

\[ K_y = 4.59 \times 10^4 \]

\[ K_z = 1.303 \times 10^6 \]
$K_{rx} = 1.866 \times 10^9$

$K_{ry} = 6.106 \times 10^7$

$K_{rz} = 1.038 \times 10^8$

Total DL = 3295 kips

Axial capacity (IDOT Bridge Manual 3.8.1) = 9000 psi

$Cap := A \cdot 9000$

$Cap = 2.15 \times 10^3$ kips

$N = 105$

$V := 3295$ kips

$M_{long} := 57415$ k – ft

$M_{tran} := 81932$ k – ft

$P_1 := \frac{V}{N} + M_{long} \frac{2.75}{12 \cdot S_2} + M_{tran} \frac{10.75}{12 \cdot S_1}$

$P_1 = 211.246$ kips

$P_{al} := A \cdot 9.144$

$P_{al} = 309.6$ kips Almost equal to $P_1$ say OK

$Dis_L := .5$ in

$M_1 := 2830$ k – in

$Dis_T := 1.18$ in

$M_2 := 2950$ k – in

C-16
EQ in transverse direction

\[ M_{\text{long}} = 43800 \quad \text{k} - \text{ft} \]
\[ M_{\text{tran}} = 107000 \quad \text{k} - \text{ft} \]

\[
P_2 := \frac{V}{N} + \frac{M_{\text{long}}}{12S_2} + \frac{M_{\text{tran}}}{12S_1} \cdot 10.75 \]

\[ P_2 = 187.087 \quad \text{kips} \]

\[ \text{Dis}_T = 1.55 \quad \text{in} \]
\[ \text{Dis}_L = 0.39 \quad \text{in} \]
\[ M_3 := 2350 \quad \text{k} - \text{in} \]
\[ M_4 := 3560 \quad \text{k} - \text{in} \]

\[
\frac{P_1}{0.85 \cdot 50 \cdot 34.4} + \frac{8}{9} \left( \frac{M_1}{50 \cdot 194} + \frac{M_2}{50 \cdot 91.4} \right) = 0.978
\]

Fig 5. Piles Arrangement


\[
\frac{P_2}{0.85 \cdot 50 \cdot 34.4} + \frac{8}{9} \left( \frac{M_3}{50 \cdot 194} + \frac{M_4}{50 \cdot 91.4} \right) = 1.036
\]

Moment = 2350 kip-in for Displacement = 0.39 in

Moment = 2830 kip-in for Displacement = 0.50 in

Moment = 2950 kip-in for Displacement = 1.18 in

Moment = 3560 kip-in for Displacement = 1.55 in
### Forces and Moments from SAP2000 (MCE):

#### NCHRP Forces & Moments-Trans(Y) EQ 100%Trans+40%Long

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<th>Moment 2 kip-ft</th>
<th>Moment 3 kip-ft</th>
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#### NCHRP Forces & Moments-Long(X) EQ 100%Long+40%Trans

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### Final Forces and Moments due to MCE and DL by applying Reduction Factor of 1.5 for designing columns:

#### NCHRP - MCE

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<th>Shear 3 kips</th>
<th>Moment 2 kip-ft</th>
<th>Moment 3 kip-ft</th>
<th>Axial kips</th>
<th>Final Design Forces and Moments (EQ+DL) By R=1.5</th>
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</table>

C-19
Forces and Moments from SAP2000 (FE):

<table>
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<tbody>
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<td></td>
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</tr>
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<td>Middle Column</td>
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<td></td>
<td>bottom</td>
</tr>
<tr>
<td>Central Column</td>
<td>top</td>
</tr>
<tr>
<td></td>
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</table>

<table>
<thead>
<tr>
<th>Location</th>
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</tr>
</thead>
<tbody>
<tr>
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<tr>
<td>Central Column</td>
<td>top</td>
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</table>

Final Forces and Moments due to FE and DL by applying Reduction Factor of 0.849 for designing columns:

<table>
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<th>Location</th>
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<tr>
<td>Central Column</td>
<td>top</td>
</tr>
<tr>
<td></td>
<td>bottom</td>
</tr>
</tbody>
</table>
Design of Columns using PCACOL:

- Forces taken from table above.
  - Bottom of column: M2= 1299 k-ft, M3= 5083 k-ft, Axial Load=964 kip
  - Top of Column: M2=5070 k-ft, M3= 997 k-ft, Axial Load=517 kip
- $\phi$ is 1 for column design
- Sections after trial and error process are: (56 x 35.4 for Bottom of Column) and (70 x 35.4 for top of the column)
- Ratio of longitudinal reinforcement in columns for 42#11 rebars are: (3.31% for Bottom section) and (2.64% for Top section)

Bottom Section:
TRANVERSE REINFORCEMENT IN COLUMNS:

**Method 1: Implicit Shear Detailing Approach [Guide Spec, Article 8.8.2.3]**

\[
\text{[56 X 35.4 in ratio=3.31%]}
\]

\[
K_{\text{shape}} := 0.375 \\
\Lambda := 1 \\
\rho_1 := 0.0331 \\
\phi := 0.9 \\
f_{yh} := 60000 \text{ psi}
\]

Rectangular Section

Fixity Factor

Longitudinal Steel Content

Strength Reduction

Yeild Strength of Ties

C-22
\[ f_{su} := 1.5 f_{yh} \]
\[ f_{su} = 9 \times 10^4 \text{ psi} \]

\[ L := 147.6 \text{ in} \]
Column Height

\[ D_p := 35.4 - 2 \left( 2 + \frac{1.41}{2} \right) \text{ in} \]
The distance b/w the outer layers of the longitudinal steel.

\[ d := 56 - 2 \]

\[ \alpha := \frac{D_p}{L} \quad \alpha = 0.203 \]
Geometric aspect ratio

\[ \tan(\alpha \text{-deg}) = 3.546 \times 10^{-3} \]

\[ A_g := 35.4 \cdot 56 \]
Gross Section Area

\[ A_g = 1.982 \times 10^3 \text{ in}^2 \]

\[ b_w := 35.4 \]

\[ A_V := b_w \cdot d \]
Shear Area of Concrete

\[ A_V = 1.912 \times 10^3 \text{ in}^2 \]

\[ b := 35.4 \text{ in} \]
Width of column section

\[ h := 56 \text{ in} \]
Height of column section

\[ \text{Try} \quad s := \min \left( 10, \frac{\min(b, h)}{2} \right) \quad s = 10 \text{ in} \]

\[ A_{sh} := 2.0.31 \text{ in}^2 \]
No 5
\[ \rho_V := \frac{A_{sh}}{b_w s} \]

Ratio of Transverse Reinforcement (8.8.2.3-2)

\[ \rho_V = 1.751 \times 10^{-3} \]

\[ \tan \theta := \left( \frac{1.6 \rho_V A_v}{\Lambda \rho_t A_g} \right)^{0.25} \]

\[ \tan \theta = 0.535 \]

\[ \rho_V := K_{shape} \frac{\rho_t}{\phi} \frac{f_{su}}{f_{yh}} \frac{A_g}{A_v} \tan(\alpha \text{-deg}) \tan \theta \]

Ratio of Transverse Reinforcement (8.8.2.3-1)

\[ A_{sh} := b_w s \rho_V \]

\[ \rho_V = 4.067 \times 10^{-5} \]

\[ A_{sh} = 0.014 \]

Use No 5 @10 in for plastic hinge zone

Outside the plastic hinge zone, the amount reinforcement can be reduced to account for some contribution of concrete in shear resistance.

\[ f_c := 3.48 \times 10^3 \text{ psi} \]

Compressive Strength of Concrete

\[ V_c := 2 \sqrt{f_c} \]

Shear Strength of Concrete

\[ V_c = 117.983 \text{ ksi} \]

\[ \rho := \rho_V - 0.17 \frac{f_c}{f_{yh}} \]

Ratio of transverse reinforcement outside the potential plastic hinge zone (8.8.2.3-5)
\[ \rho = -1.265 \times 10^{-4} \]

Because the amount is negative the contribution of concrete is more than enough to carry the shear. No. 5 at 10 inches would be adequate to satisfy the implicit detailing in the plastic hinge zone only. As will be seen, the confinement and anti buckling provisions will control over the shear requirement.

**Method 2: Explicit Approach [Guide Spec, Article 8.8.2.3]**

\[ L := 147.6 \text{ in} \quad \text{Column Height} \]

\[ A_b := 1.56 \text{ in}^2 \quad \text{Area of rebars for Long reinforcement} \]

Approximate the overstrength effects simply as 1.5 times the forces from the verification.

\[ P_d := 538 \text{ kips} \quad \text{Axial load} \]

\[ OS := 1.5 \quad \text{Overstrength Factor} \]

\[ P_e := 750 \text{ kips} \quad \text{Axial load due to EQ} \]

\[ P_e := P_d + OS \cdot (P_e - P_d) \quad \text{Axial load due to EQ} \]

\[ P_e = 856 \text{ kips} \]

\[ M_{p\_top} := 5070 \text{ kip - ft} \quad \text{Moment at the bottom of column} \]

\[ M_{p\_bot} := 5083 \text{ kip - ft} \quad \text{Moment at the top of column} \]

\[ M_{p\_top} := OS \cdot M_{p\_top} \]

\[ M_{p\_bot} := OS \cdot M_{p\_bot} \]

\[ M_{p\_top} = 7.605 \times 10^3 \text{ kip - ft} \]

\[ M_{p\_bot} = 7.625 \times 10^3 \text{ kip - ft} \]
L := \frac{L}{12} \quad L = 12.3 \quad \text{ft} \\
V_u := \frac{(M_{p\text{\ bot}} + M_{p\text{\ top}})}{L} \quad \text{Ultimate shear strength} \\
V_u = 1.238 \times 10^3 \quad \text{kip} \\
V_c := 0.6 \sqrt{\frac{A_v}{1000}} \quad \text{Shear strength of Concrete} \\
V_c = 67.661 \quad \text{kip} \\
V_p := \frac{A_e \cdot \tan(\alpha\cdot \text{deg})}{2} \quad \text{The contribution due to arch action (8.8.2.3-7)} \\
V_p = 1.518 \quad \text{kip} \\
V_s := \frac{V_u}{\phi} - V_c - V_p \quad \text{Shear resistance provided by transvers reinforcing} \\
V_s = 1.307 \times 10^3 \quad \text{kip} \\
\phi := 0.9 \\
A_{sh} := 0.31 \cdot 2 \quad 2 \text{ legs No. 5} \\
A_{sh} = 0.62 \quad \text{in}^2 \\
\rho_v := \frac{A_{sh}}{s \cdot b_w} \quad \text{Ratio of Transvers Reinforcement (8.8.2.3-2)} \\
\rho_v = 1.751 \times 10^{-3} \\
\tan\theta := \left( \frac{1.6 \cdot \rho_v \cdot A_v}{A \cdot \rho_t \cdot A_g} \right)^{0.25} \quad (8.8.2.3-4)
\[
\tan \theta = 0.535
\]

Because \( \tan \theta \) is greater than \( \tan \alpha \), use \( \tan \theta \) to find \( A_{sh} \):

\[
A_{vs} := s \cdot \frac{V_s}{f_y h b_w} \cdot \tan \theta \quad \text{(8.8.2.3-17)}
\]

\[
A_{vs} = 3.288 \times 10^{-3}
\]

**Use #5 @ 10 in.**

### Transverse Reinforcement for confinement at plastic hinge Pmax column [Guide Spec, Article 8.8.2.4]

- \( f_y := 60 \) ksi: yield strength of reinforcing bars
- \( U_{sf} := 15.95 \) ksi: Strain energy capacity (module of toughness) of the transverse reinforcement
- \( A_{sh} := 0.312 \) 2 legs No. 5
- \( h := 54 \) in: Height of section
- \( A_c := (b - 5) \cdot (h - 5) \) Area of column core concrete

\[
A_c = 1.49 \times 10^3 \text{ in}^2
\]

\[
\rho_s := 0.008 \frac{f_y}{U_{sf} \cdot 1000} \left[ 15 \left( \frac{P_c \cdot 1000}{f_c \cdot A_g} + \rho_t \frac{f_y \cdot 1000}{f_c} \right) \cdot \left( \frac{A_g}{A_c} \right)^2 - 1 \right]
\]

\[
\rho_s = 0.021 \quad \text{The volumetric ratio}
\]

| legs | 4 |
s := \frac{\text{legs} \cdot A_{sh}}{\rho_{s} \cdot b} \quad (8.8.2.3-2)

s = 3.395 \quad \text{s max}=3 \text{ in} \quad \text{OK}

\textbf{Method 2: Explicit Shear detailing approach- Pmin Column [Guide Spec, Article 8.8.2.3]}

\begin{align*}
L & := 147.6 \quad \text{Column Hieght} \\
A_{b} & := 1.56 \quad \text{in}^{2} \quad \text{Area of rebars for Long reinforcement} \\
P_{d} & := 517 \quad \text{kips} \quad \text{Axial load} \\
P_{e} & := 314 \quad \text{kips} \quad \text{Axial load due to EQ (Lower Compression)} \\
P_{e} & := P_{d} + OS \cdot (P_{e} - P_{d}) \quad \text{Axial load due to EQ} \\
P_{e} & := 212.5 \quad \text{kips} \\
M_{p\_top} & := 341 \quad \text{kip - ft} \quad \text{Moment at the bottom of column} \\
M_{p\_bot} & := 853 \quad \text{kip - ft} \quad \text{Moment at the top of column} \\
OS & := 1.5 \\
M_{p\_top} & := OS \cdot M_{p\_top} \quad M_{p\_top} = 511.5 \quad \text{kip - ft} \\
M_{p\_bot} & := OS \cdot M_{p\_bot} \quad M_{p\_bot} = 1.28 \times 10^{3} \quad \text{kip - ft} \\
L & := \frac{L}{12} \quad L = 12.3 \quad \text{ft}
\end{align*}
Because \( \tan \theta \) is greater than \( \tan \alpha \), use \( \tan \theta \) to find \( A_{sh} \):

\[
\tan \theta := \left( \frac{1.6 \rho_v A_v}{A \rho_t A_g} \right)^{0.25}
\]

\[
\tan \theta = 0.491
\]
\[ A_{vs} := \frac{s \cdot V_s \cdot \tan\theta}{f_{y} \cdot h \cdot b_{w}} \]  
(8.8.2.3-17)

\[ A_{vs} = 1.792 \times 10^{-4} \]

Use 2#3 @ 10 in. it is more than required

**Transvers reinforcement for confinement at plastic hinges Pmin Column [Guide Spec, Article 8.8.2.4]:**

\[ f_{y} := 60 \text{ ksi} \]  
Yield strength of reinforcing bars

\[ U_{sf} := 15.95 \text{ ksi} \]  
Strain energy capacity (module of toughness) of the transverse reinforcement

\[ A_{sh} := 0.312 \]  
No. 5

\[ h := 56 \text{ in} \]  
Height of section

\[ A_{c} := (b - 5)(h - 5) \]  
Area of column core concrete

\[ A_{c} = 1.55 \times 10^{3} \text{ in}^2 \]

\[ \rho_{t} := 0.0331 \]  
The volumetric ratio

\[
\rho_{s} := 0.008 \frac{f_{c}}{U_{sf} \cdot 1000} \left[ 15 \left( \frac{P_{e} \cdot 1000}{f_{c} \cdot A_{g}} + \rho_{t} \frac{f_{y} \cdot 1000}{f_{c}} \right) + \rho_{t} \frac{f_{y} \cdot 1000}{f_{c}} \right] \left( \frac{A_{g}}{A_{c}} \right)^{2} - 1 \]

\[ \rho_{s} = 0.014 \]

legs := 3
\[ s := \frac{\text{legs} \cdot A_{sh}}{\rho_s \cdot b_w} \]  \hspace{1cm} (8.8.2.3-2)

\[ s = 3.824 \quad \text{s max}=4 \text{ in} \]

\[ s \text{ No.5 } @ 4 \text{ in for confinement in the end region of the Pmin column.} \]

**Anti-Buckling steel [Guide Spec, Article 8.8.2.5]:**

\[ d_b := 1.41 \text{ in} \quad \text{bar diameter for No. 11} \]

\[ s_{new} := 6 \cdot d_b \]

\[ s_{new} = 8.46 \text{ in} \quad \text{Required spacing of anti-buckling} \]

\[ s := 8.46 \text{ in} \]

\[ A_b := 1.56 \cdot 42 \]

\[ A_{bh} := 0.09 \cdot A_b \cdot \frac{f_y}{f_{yh}} \left( \frac{1000}{1} \right) \]

\[ A_{bh} = 5.897 \]

\[ \text{legs_antibuckling} := 8 \quad \text{Legs required for anti buckling} \]

\[ \frac{A_{bh}}{\text{legs_antibuckling}} = 0.369 \]

\[ S := 4 \text{ in} \]

\[ \text{Use No.6 } @ 4 \text{ in for anti buckling} \]
Extend of Shear Steel, Confinement and Anti-buckling Steel [Guide Spec, Article 8.8.2.6 and 4.9]:

\[ D := \frac{h - 4}{12} \]  
\[ D = 4.333 \text{ ft} \]  
Maximum cross-sectional dimantion of column

\[ M_u := 5058 \text{ kip – ft} \]  
Maximum Column Moment

\[ M_{p\_bot} := 8418 \text{ kip – ft} \]  
Column plastic overstrength moment

\[ V_u = 145.61 \text{ kip} \]  
Maximum column shear

\[ M_y := \frac{0.85 \cdot M_{p\_bot}}{1.5} \text{ kip – ft} \]  
Column yield moment

\[ d_b := 1.41 \text{ in} \]  
Longitudinal bar diameter

\[ \varepsilon_y := 0.00207 \text{ in} \]  
Yield strain of the longitudinal Reinforcement

\[ L_1 := D \left( \frac{1}{\tan \theta} + 0.5 \cdot \tan \theta \right) \]  
\[ L_1 = 9.896 \text{ ft} \]

\[ L_2 := 1.5 \left( 0.08 \frac{M_u}{V_u} + 4400 \cdot \varepsilon_y \frac{d_b}{12} \right) \]  
\[ L_2 = 5.774 \text{ ft} \]

\[ L_3 := \frac{M_u}{V_u} \left( 1 - \frac{M_y}{M_{p\_bot}} \right) \]  
\[ L_3 = 15.053 \text{ ft} \]

\[ L := \max(L_1, L_2, L_3) \]

\[ L = 15.053 \text{ ft} \]  
Controls

Plastic zone length= 15 ft
Connections:

a) Connection at the top of columns to Cap beam:

\[70 \times 35.4 \text{ in ratio=2.64%}\]

a) Confinement reinforcement per Guide Spec 8.8.2.4.

No.5 @ 3 in

b) Anti Buckling reinforcement per Guide Spec 8.8.2.5.

No.6 @ 4 in

c) Shear Reinforcement per Guide Spec 8.8.2.3.

\[
\begin{align*}
B1 & := 35.4 - 5 - 0.625 & B1 &= 29.775 \text{ in} \\
D1 & := 70 - 5 - 0.625 & D1 &= 64.375 \text{ in} \\
B2 & := 35.4 - 5 - 2 \times 0.625 & B2 &= 29.15 \text{ in} \\
D2 & := 70 - 5 - 2 \times 0.625 & D2 &= 63.75 \text{ in} \\
\phi & := 0.90 & \\
f_{yh} & := 60000 \text{ psi} & \\
f_{su} & := 1.5 \times f_{yh} & f_{su} &= 9 \times 10^4 \text{ psi} & \\
\rho_t & := 0.0264 & \\
A_g & := 70 \times 35.4 & A_g &= 2.478 \times 10^3 \text{ in}^2 \\
A_c & := B1 \times D1 & A_c &= 1.917 \times 10^3 \text{ in}^2 \\
D & := 35.4 \text{ in} & \\
H_c & := 43.3 \text{ in} & \\
\end{align*}
\]

Strength Reduction Factor for shear

Yeild Strength of Ties

Longitudinal Steel Content

Cross-Sectional Area of Columns

Cross-Sectional Area of Column core

Width of the column framing into the joint

Height of the cap beam
\[
\tan \alpha := \frac{D}{H_c}
\]
\[
\rho := \frac{B1}{D1} + \frac{0.5 \cdot \rho t \cdot f_{su} \cdot A_g \cdot \left(\frac{D}{H_c}\right)^2}{2 \cdot B1 - 2 \cdot \phi \cdot f_{yh} \cdot A_c}
\]

For a rectangular column, minimum ratio \(\rho = 0.011\)

\[A_{sh} := 2.31\]

\[s := \frac{A_{sh}}{\rho \cdot B2} \quad s = 1.865\]

\begin{tabular}{|c|}
\hline
No.5 @ 2 in \hline
\end{tabular}

Use No. 5 @ 3

**Explicit Approach for the joint Design [Guide Spec, Article 8.8.4.2]:**

**Case 1:** from the transverse displacement capacity verification with overstrenght:

\[f_h := 0 \quad \text{ksi} \]
\[P_{max} := 943 \quad \text{kip} \]
\[M_p := 5330 \quad \text{kip} \cdot \text{ft} \]
\[b_b := 39.37 \quad \text{in} \]
\[H_c := 43.3 \quad \text{in} \]
\[L_{mid\_depth\_jt} := D + H_c \]
\[L_{mid\_depth\_jt} = 78.7 \]
\[A_{mid\_depth\_jt} := b_b \cdot L_{mid\_depth\_jt} \]
\[A_{mid\_depth\_jt} = 3.098 \times 10^3 \quad \text{in}^2 \]
\[ f_v := \frac{P_{\text{max}}}{A_{\text{mid_depth_jt}}} \]

Average axial stress in the horizontal direction

\[ f_v = 0.304 \quad \text{ksi} \]

\[ h_b := 43.3 \quad \text{in} \]

Cap beam Depth

\[ b_{je} := b_b \]

The effective joint width

\[ h_c := 70 \]

The column lateral dimension

\[ v_{hv} := \frac{M_p \cdot 12}{h_b \cdot h_c \cdot b_{je}} \]

The average shear stress within the plane of the connection.

\[ v_{hv} = 0.536 \quad \text{ksi} \]

\[ f_c := 3.48 \quad \text{ksi} \]

Compressive Strength of Concrete

\[ p_t := \frac{f_h + f_v}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{hv}^2} \]

Principle tension stress (8.8.4.2-1).

\[ p_t = -0.405 \quad \text{ksi} \]

Maximum tension stress

\[ P_{t_{\text{max}}} := 3.5 \sqrt{\frac{f_c}{1000}} \]

\[ P_{t_{\text{max}}} = 0.206 \quad \text{ksi} < p_t \]

Therefore must use Guide spec 8.8.4.3.

\[ p_c := \frac{f_h + f_v}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{hv}^2} \]

Principle compression stress (8.8.4.2-2).

\[ p_c = 0.709 \quad \text{ksi} \]
By inspection, this will produce smaller principal stresses than Case 1.

**Case 2:**

\[
P_{\text{min}} := 341 \text{ kip}
\]

\[
M_p := 3216 \text{ kip} \cdot \text{ft}
\]

Obviously it will produce amall principal stresses than Case 1.

**Case 3:**

\[
P_{\text{DL}} := 570 \text{ kip}
\]

Axial Load due to Dead Load

\[
M_p := 4258 \text{ kip} \cdot \text{ft}
\]

In the longitudinal direction

By inspection, this will produce smaller principal stresses than Case 1.

design Reinforcement for joint force transfer Guide Spec (8.8.4.3)
Stirrup Guide Spec (8.8.4.3.2)

**Column = 42 # 11**

\[
N := 42
\]

Area of one longitudinal reinforcement

\[
A_{\text{bar}} := 1.56 \text{ in}^2
\]

Total Area of Longitudinal steel

\[
A_{\text{st}} := N \cdot A_{\text{bar}}
\]

Area of the steel located within the distance 0.5D or 0.5h from the column or pier wall face.

\[
A_{\text{juv}} := 0.16 \cdot A_{\text{st}}
\]

\[
A_{\text{juv}} = 10.483 \text{ in}^2
\]

Using 2 legs No. 5 Stirrups

\[
A_{\text{stirrup}_\text{leg}} := 2 \cdot 0.31 \text{ in}^2
\]
\[
\frac{A_{jy}}{A_{stirrup\_leg}} = 16.908
\]
9 No.5 Stirrups on Each Side

\[
A_{clamp} := 0.08 \cdot A_{st}
\]
Clamping Guide Spec (8.8.4.3.2)

\[
\frac{A_{clamp}}{A_{stirrup\_leg}} = 8.454
\]
9 #5 legs required in joint core

Horizontal reinforcement Guide Spec (8.8.4.3.2)

\[
A_h := 0.08 \cdot A_{st}
\]

\[
A_{bar} := 1.0 \text{ in}^2
\]
Using No.9 bars,

\[
f_y := 60 \text{ ksi}
\]

\[
f_c := 3.48 \text{ ksi}
\]

\[
\frac{A_h}{A_{bar}} = 5.242
\]

\[
L_d := 1.25 \cdot A_{bar} \frac{f_y}{\sqrt{f_c}}
\]
No more than

\[
L_d := 0.4 \cdot f_y \cdot d_b
\]

\[
L_d = 40.204
\]

Hoop Reinforcement Guide Spec (8.8.4.3.4)

\[
f_y := 60 \text{ ksi}
\]

\[
f_c := 3.48 \text{ ksi}
\]

\[
A_{bar} := 1.56 \text{ in}^2
\]
Cross-Section Area of bar #11

\[
l_{ac} := 1.25 \cdot A_{bar} \frac{f_y}{\sqrt{f_c}}
\]
Development length of #11 per AASHTO LRFD 5.11.2.1

\[
l_{ac} = 62.719 \text{ in}
\]
\( \rho_s := 0.4 \cdot \frac{A_{st}}{l_{ac}} \)

Guide Spec (8.8.4.3-1)

\( \rho_s = 6.663 \times 10^{-3} \)

\( A_{\text{hoop}} := 0.31 \cdot 2 \text{ in}^2 \)

2 legs No.5 for hoops

\( s := 4 \cdot \frac{A_{\text{hoop}}}{\rho_s \cdot D_2} \)

\( s = 5.839 \)

2 No. 5 @ 5 3/4 in pitch required in the joint

b) Connection at the bottom of columns to Footing:

\( [56 \times 35.4 \text{ in ratio=3.31%}] \)

a) Confinement reinforcement per Guide Spec 8.8.2.4.

No.5 @ 3 in

b) Anti Buckling reinforcement per Guide spec 8.8.2.5.

No.6 @ 4 in

c) Shear Reinforcement per Guide Spec 8.8.2.3.

\( B_1 := 35.4 - 5 - 0.625 \quad B_1 = 29.775 \text{ in} \)

\( D_1 := 56 - 5 - 0.625 \quad D_1 = 50.375 \text{ in} \)

\( B_2 := 35.4 - 5 - 2 \cdot 0.625 \quad B_2 = 29.15 \text{ in} \)
D2 := \( 56 - 5 - 2 \cdot 0.625 \)  \( D2 = 49.75 \) in

\( \phi := 0.90 \)

\( f_{yh} := 60000 \) psi

\( f_{su} := 1.5 \cdot f_{yh} \)

\( f_{su} = 9 \times 10^4 \) psi

\( \rho_t := 0.0331 \)

\( A_g := 56 \cdot 35.4 \)

\( A_g = 1.982 \times 10^3 \) in\(^2\)

\( A_c := B2 \cdot D2 \)

\( A_c = 1.45 \times 10^3 \) in\(^2\)

\( D := 35.4 \) in

\( H_c := 66.93 \) in

\( \tan \alpha := \frac{D}{H_c} \)

\( \rho := \frac{2 \cdot B1}{D1} + 0.5 \cdot \rho_t \cdot f_{su} \cdot A_g \left( \frac{D}{H_c} \right)^2 \)

\( 2 \cdot B1 \)

\( D1 + 2 \cdot \phi \cdot f_{yh} \cdot A_c \left( \frac{D}{H_c} \right) \)

\( \rho = 6.329 \times 10^{-3} \)

\( A_{sh} := 2.31 \)

\( s := \frac{A_{sh}}{\rho \cdot B2} \)

\( s = 3.361 \) maximum

\( \frac{2 \text{ No.5 @ 3 1/4 in}}{\text{Use No. 5 @ 3 in}} \)
Explicit Approach for the joint Design [Guide Spec, Article 8.8.4.2]:

**Case 1:** from the transverse displacement capacity verification with overstrength:

\[ f_h := 0 \text{ ksi} \]

\[ P_{\text{max}} := 962 \text{ kip} \]

\[ M_p := 5058 \text{ kip} - \text{ft} \]

\[ b_b := 39.37 \text{ in} \]

\[ H_c := 66.93 \text{ in} \]

\[ L_{\text{mid_depth_jt}} := D + H_c \]

\[ L_{\text{mid_depth_jt}} = 102.33 \text{ in} \]

\[ A_{\text{mid_depth_jt}} := b_b \times L_{\text{mid_depth_jt}} \]

\[ A_{\text{mid_depth_jt}} = 4.029 \times 10^3 \text{ in}^2 \]

\[ f_v := \frac{P_{\text{max}}}{A_{\text{mid_depth_jt}}} \]

\[ f_v = 0.239 \text{ ksi} \]

\[ h_p := 43.3 \text{ in} \]

\[ b_{jc} := b_b \]

\[ h_c := 56 \]

The column lateral dimension
The average shear stress within the plane of the connection.

\[ v_{hv} := \frac{M_p 12}{h_b h_c \cdot b_{je}} \]

\[ v_{hv} = 0.636 \text{ ksi} \]

\[ f_c := 3.48 \text{ ksi} \]

Compressive Strength of Concrete

\[ p_t := \frac{\left( f_h + f_v \right)}{2} - \sqrt{\left( \frac{f_h - f_v}{2} \right)^2 + v_{hv}^2} \]

\[ p_t = -0.528 \text{ ksi} \]

Principle tension stress (8.8.4.2-1).

\[ P_{t\_max} := 3.5 \cdot \frac{f_c}{1000} \]

\[ P_{t\_max} = 0.206 \text{ ksi} < p_t \]

Therefore must use Guide spec 8.8.4.3.

\[ p_c := \frac{\left( f_h + f_v \right)}{2} + \sqrt{\left( \frac{f_h - f_v}{2} \right)^2 + v_{hv}^2} \]

\[ p_c = 0.766 \text{ ksi} \]

\[ p_{c\_max} := 0.25 \cdot f_c \]

Maximum Compression stress [Guide spec, Article 8.8.4.2.3]

\[ p_{c\_max} = 0.87 \text{ ksi} > p_c \text{ OK} \]

Case 2:

\[ P_{\text{min}} := 359 \text{ kip} \]

\[ M_p := 3778 \text{ kip-ft} \]

Obviously it will produce smaller principle stresses than Case 1.
Case 3:

\[ p_{DL} := 570 \text{ kip} \]
\[ M_p := 853 \text{ kip ft} \]

Axial Load due to Dead Load
In the longitudinal direction

By inspection, this will produce smaller principal stresses than Case 1.

design Reinforcement for joint force transfer Guide Spec (8.8.4.3)
Stirrup Guide Spec (8.8.4.3.2)

Column = 42 # 11 \hspace{1cm} N := 42

\[ A_{bar} := 1.56 \text{ in}^2 \]
\[ A_{st} := N \cdot A_{bar} \]
\[ A_{st} = 65.52 \text{ in}^2 \]
\[ A_{jv} := 0.16 \cdot A_{st} \]
\[ A_{jv} = 10.483 \text{ in}^2 \]
\[ A_{stirrup\_leg} := 0.31 \cdot 2 \text{ in}^2 \]

Using No. 5 Stirrups

\[ \frac{A_{jv}}{A_{stirrup\_leg}} = 16.908 \]
\[ 9 \# 5 \text{ legs required} \]

\[ A_{clamp} := 0.08 \cdot A_{st} \]
\[ A_{clamp} = 8.454 \]
\[ 9 \# 5 \text{ legs required in joint core} \]
Horizontal reinforcement Guide Spec (8.8.4.3.2)

\[ A_h := 0.08 \cdot A_{st} \]
\[ A_{bar} := 1.0 \text{ in}^2 \]
\[ \frac{A_h}{A_{bar}} = 5.242 \]
\[ L_d := 1.25 \cdot A_{bar} \cdot \frac{f_y}{\sqrt{f_c}} \]
\[ \text{Using No.9 bars,} \]
\[ f_y := 60 \text{ ksi} \]
\[ f_c := 3.48 \text{ ksi} \]
\[ d_h := 1.128 \text{ in for No.9} \]
\[ L_d := 40.204 \text{ in} \]

6 \#9 in footing

Hoop Reinforcement Guide Spec (8.8.4.3.4)

\[ f_y := 60 \text{ ksi} \]
\[ f_c := 3.48 \text{ ksi} \]
\[ A_{bar} := 1.56 \text{ in}^2 \]
\[ l_{ac} := 1.25 \cdot A_{bar} \cdot \frac{f_y}{\sqrt{f_c}} \]
\[ l_{ac} := 62.719 \text{ in} \]
\[ \rho_s := 0.4 \cdot \frac{A_{st}}{l_{ac}} \]
\[ \rho_s := 6.663 \times 10^{-3} \]
\[ A_{hoop} := 0.312 \text{ in}^2 \]
\[ s := 4 \cdot \frac{A_{hoop}}{\rho_s \cdot D_2} \]
\[ s = 7.482 \]

2 No. 5 @ 7 1/4 in pitch required in the joint
Footing design procedure for St. Clair County bridge

Fc := 3480 psi
Fy := 60000 psi

Moments and Forces acting on piles:

M1 := 25116 kip – ft   Moments in Transvers direction
M2 := 29260 kip – ft   Moments in Longitudinal direction
V := 2987 kip          Moments in Transvers direction

Dimensions of pier

W := 3.28 Width of Wall (ft)
L := 64.08 Length of wall (ft)
T := 4 Thickness of pile cap (ft)
s := 6.25 ft  

m1 := 5  
n1 := 21  

\[ \Sigma d_1 := \frac{s^2}{12} \cdot n_1 (n_1^2 - 1) \cdot m_1 \]  

Sum of the squares of the distances to each pile from the center or gravity of piles  

\[ \Sigma d_1 = 1.504 \times 10^5 \text{ pile ft}^2 \]  

m2 := 21  
n2 := 5  

\[ \Sigma d_2 := \frac{s^2}{12} \cdot n_2 (n_2^2 - 1) \cdot m_2 \]  

\[ \Sigma d_2 = 8.203 \times 10^3 \text{ pile ft}^2 \]  

M1 := 25116  

kip – ft  

M2 := 29260  

kip – ft  

V := 2987  

kip  

**Piles Coordinates:**  

**1st row**  

x1 := 62.5  
x4 := 43.75  
x7 := 25  
x10 := 6.25  
x13 := -12.5  
x16 := -31.25  
x19 := -50  
y1 := 12.5  
y4 := 12.5  
y7 := 12.5  
y10 := 12.5  
y13 := 12.5  
y16 := 12.5  
y19 := 12.5  
x2 := 56.25  
x5 := 37.5  
x8 := 18.75  
x11 := 0  
x14 := -18.75  
x17 := -37.5  
x20 := -56.25  
y2 := 12.5  
y5 := 12.5  
y8 := 12.5  
y11 := 12.5  
y14 := 12.5  
y17 := 12.5  
y20 := 12.5
A 39.063 =

\[
A = 39.063
\]

\[
\delta := 0.12
\]

\[
h := 2
\]

\[
p := A \cdot \delta \cdot h
\]

\[
p = 9.375 kips
\]

\[
n := 105
\]

**Axial forces in each pile**

\[
P1 := \frac{V}{n} + M1 \cdot \frac{x1}{\Sigma d1} + M2 \cdot \frac{y1}{\Sigma d2} + p
\]

\[
P1 = 92.847 kips
\]

\[
P2 := \frac{V}{n} + M1 \cdot \frac{x2}{\Sigma d1} + M2 \cdot \frac{y2}{\Sigma d2} + p
\]

\[
P2 = 91.803 kips
\]
P3 := \frac{V}{n} + M1 \cdot \frac{x3}{\Sigma d1} + M2 \cdot \frac{y3}{\Sigma d2} + p

P3 = 90.76 kips

P4 := \frac{V}{n} + M1 \cdot \frac{x4}{\Sigma d1} + M2 \cdot \frac{y4}{\Sigma d2} + p

P4 = 89.716

P5 := \frac{V}{n} + M1 \cdot \frac{x5}{\Sigma d1} + M2 \cdot \frac{y5}{\Sigma d2} + p

P5 = 88.672 kips

P6 := \frac{V}{n} + M1 \cdot \frac{x6}{\Sigma d1} + M2 \cdot \frac{y6}{\Sigma d2} + p

P6 = 87.628

P7 := \frac{V}{n} + M1 \cdot \frac{x7}{\Sigma d1} + M2 \cdot \frac{y7}{\Sigma d2} + p

P7 = 86.584 kips

P8 := \frac{V}{n} + M1 \cdot \frac{x8}{\Sigma d1} + M2 \cdot \frac{y8}{\Sigma d2} + p

P8 = 85.541

P9 := \frac{V}{n} + M1 \cdot \frac{x9}{\Sigma d1} + M2 \cdot \frac{y9}{\Sigma d2} + p

P9 = 84.497 kips

P10 := \frac{V}{n} + M1 \cdot \frac{x10}{\Sigma d1} + M2 \cdot \frac{y10}{\Sigma d2} + p

P10 = 83.453

P11 := \frac{V}{n} + M1 \cdot \frac{x11}{\Sigma d1} + M2 \cdot \frac{y11}{\Sigma d2} + p

P11 = 82.409 kips

P12 := \frac{V}{n} + M1 \cdot \frac{x12}{\Sigma d1} + M2 \cdot \frac{y12}{\Sigma d2} + p

P12 = 81.366

P13 := \frac{V}{n} + M1 \cdot \frac{x13}{\Sigma d1} + M2 \cdot \frac{y13}{\Sigma d2} + p

P13 = 80.322

P14 := \frac{V}{n} + M1 \cdot \frac{x14}{\Sigma d1} + M2 \cdot \frac{y14}{\Sigma d2} + p

P14 = 79.278

P15 := \frac{V}{n} + M1 \cdot \frac{x15}{\Sigma d1} + M2 \cdot \frac{y15}{\Sigma d2} + p

P15 = 78.234 kips

P16 := \frac{V}{n} + M1 \cdot \frac{x16}{\Sigma d1} + M2 \cdot \frac{y16}{\Sigma d2} + p

P16 = 77.19
P17 := \( \frac{V}{n} + \frac{M_1 x_{17} + M_2 y_{17} + p}{\Sigma d_1} \)  

\[ P17 = 76.147 \text{ kips} \]

P18 := \( \frac{V}{n} + \frac{M_1 x_{18} + M_2 y_{18} + p}{\Sigma d_2} \)  

\[ P18 = 75.103 \text{ kips} \]

P19 := \( \frac{V}{n} + \frac{M_1 x_{19} + M_2 y_{19} + p}{\Sigma d_1} \)  

\[ P19 = 74.059 \text{ kips} \]

P20 := \( \frac{V}{n} + \frac{M_1 x_{20} + M_2 y_{20} + p}{\Sigma d_2} \)  

\[ P20 = 73.015 \text{ kips} \]

P21 := \( \frac{V}{n} + \frac{M_1 x_{21} + M_2 y_{21} + p}{\Sigma d_1} \)  

\[ P21 = 71.971 \text{ kips} \]

P22 := \( \frac{V}{n} + \frac{M_1 x_{22} + M_2 y_{22} + p}{\Sigma d_2} \)  

\[ P22 = 70.554 \text{ kips} \]

P23 := \( \frac{V}{n} + \frac{M_1 x_{23} + M_2 y_{23} + p}{\Sigma d_1} \)  

\[ P23 = 69.51 \text{ kips} \]

P24 := \( \frac{V}{n} + \frac{M_1 x_{24} + M_2 y_{24} + p}{\Sigma d_2} \)  

\[ P24 = 68.466 \text{ kips} \]

P25 := \( \frac{V}{n} + \frac{M_1 x_{25} + M_2 y_{25} + p}{\Sigma d_1} \)  

\[ P25 = 67.422 \text{ kips} \]

P26 := \( \frac{V}{n} + \frac{M_1 x_{26} + M_2 y_{26} + p}{\Sigma d_2} \)  

\[ P26 = 66.379 \text{ kips} \]

P27 := \( \frac{V}{n} + \frac{M_1 x_{27} + M_2 y_{27} + p}{\Sigma d_1} \)  

\[ P27 = 65.335 \text{ kips} \]

P28 := \( \frac{V}{n} + \frac{M_1 x_{28} + M_2 y_{28} + p}{\Sigma d_2} \)  

\[ P28 = 64.291 \text{ kips} \]
Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

NCHRP Guidelines

P29 := \( \frac{V}{n} + \frac{M_1 \cdot x_{29}}{\Sigma d_1} + \frac{M_2 \cdot y_{29}}{\Sigma d_2} + p \)

\( P29 = 63.247 \) kips

P31 := \( \frac{V}{n} + \frac{M_1 \cdot x_{31}}{\Sigma d_1} + \frac{M_2 \cdot y_{31}}{\Sigma d_2} + p \)

\( P31 = 61.16 \) kips

P33 := \( \frac{V}{n} + \frac{M_1 \cdot x_{33}}{\Sigma d_1} + \frac{M_2 \cdot y_{33}}{\Sigma d_2} + p \)

\( P33 = 59.072 \) kips

P35 := \( \frac{V}{n} + \frac{M_1 \cdot x_{35}}{\Sigma d_1} + \frac{M_2 \cdot y_{35}}{\Sigma d_2} + p \)

\( P35 = 56.985 \) kips

P37 := \( \frac{V}{n} + \frac{M_1 \cdot x_{37}}{\Sigma d_1} + \frac{M_2 \cdot y_{37}}{\Sigma d_2} + p \)

\( P37 = 54.897 \) kips

P39 := \( \frac{V}{n} + \frac{M_1 \cdot x_{39}}{\Sigma d_1} + \frac{M_2 \cdot y_{39}}{\Sigma d_2} + p \)

\( P39 = 52.809 \) kips

P30 := \( \frac{V}{n} + \frac{M_1 \cdot x_{30}}{\Sigma d_1} + \frac{M_2 \cdot y_{30}}{\Sigma d_2} + p \)

\( P30 = 62.204 \) kips

P32 := \( \frac{V}{n} + \frac{M_1 \cdot x_{32}}{\Sigma d_1} + \frac{M_2 \cdot y_{32}}{\Sigma d_2} + p \)

\( P32 = 60.116 \) kips

P34 := \( \frac{V}{n} + \frac{M_1 \cdot x_{34}}{\Sigma d_1} + \frac{M_2 \cdot y_{34}}{\Sigma d_2} + p \)

\( P34 = 58.028 \) kips

P36 := \( \frac{V}{n} + \frac{M_1 \cdot x_{36}}{\Sigma d_1} + \frac{M_2 \cdot y_{36}}{\Sigma d_2} + p \)

\( P36 = 55.941 \) kips

P38 := \( \frac{V}{n} + \frac{M_1 \cdot x_{38}}{\Sigma d_1} + \frac{M_2 \cdot y_{38}}{\Sigma d_2} + p \)

\( P38 = 53.853 \) kips

P40 := \( \frac{V}{n} + \frac{M_1 \cdot x_{40}}{\Sigma d_1} + \frac{M_2 \cdot y_{40}}{\Sigma d_2} + p \)

\( P40 = 51.766 \) kips
P41 := \frac{V}{n} + M_1 \frac{x41}{\Sigma d1} + M_2 \frac{y41}{\Sigma d2} + p

P41 = 50.722 kips

P42 := \frac{V}{n} + M_1 \frac{x42}{\Sigma d1} + M_2 \frac{y42}{\Sigma d2} + p

P42 = 49.678 kips

P43 := \frac{V}{n} + M_1 \frac{x43}{\Sigma d1} + M_2 \frac{y43}{\Sigma d2} + p

P43 = 48.26 kips

P44 := \frac{V}{n} + M_1 \frac{x44}{\Sigma d1} + M_2 \frac{y44}{\Sigma d2} + p

P44 = 47.217 kips

P45 := \frac{V}{n} + M_1 \frac{x45}{\Sigma d1} + M_2 \frac{y45}{\Sigma d2} + p

P45 = 46.173 kips

P46 := \frac{V}{n} + M_1 \frac{x46}{\Sigma d1} + M_2 \frac{y46}{\Sigma d2} + p

P46 = 45.129 kips

P47 := \frac{V}{n} + M_1 \frac{x47}{\Sigma d1} + M_2 \frac{y47}{\Sigma d2} + p

P47 = 44.085 kips

P48 := \frac{V}{n} + M_1 \frac{x48}{\Sigma d1} + M_2 \frac{y48}{\Sigma d2} + p

P48 = 43.042 kips

P49 := \frac{V}{n} + M_1 \frac{x49}{\Sigma d1} + M_2 \frac{y49}{\Sigma d2} + p

P49 = 41.998 kips

P50 := \frac{V}{n} + M_1 \frac{x50}{\Sigma d1} + M_2 \frac{y50}{\Sigma d2} + p

P50 = 40.954 kips

P51 := \frac{V}{n} + M_1 \frac{x51}{\Sigma d1} + M_2 \frac{y51}{\Sigma d2} + p

P51 = 39.91 kips

P52 := \frac{V}{n} + M_1 \frac{x52}{\Sigma d1} + M_2 \frac{y52}{\Sigma d2} + p

P52 = 38.866 kips

P53 := \frac{V}{n} + M_1 \frac{x53}{\Sigma d1} + M_2 \frac{y53}{\Sigma d2} + p

P54 := \frac{V}{n} + M_1 \frac{x54}{\Sigma d1} + M_2 \frac{y54}{\Sigma d2} + p
P53 = 37.823 kips
P55 := \frac{V}{n} + M1 \cdot \frac{x55}{\Sigma d1} + M2 \cdot \frac{y55}{\Sigma d2} + p
P55 = 35.735 kips

P57 := \frac{V}{n} + M1 \cdot \frac{x57}{\Sigma d1} + M2 \cdot \frac{y57}{\Sigma d2} + p
P57 = 33.647 kips

P59 := \frac{V}{n} + M1 \cdot \frac{x59}{\Sigma d1} + M2 \cdot \frac{y59}{\Sigma d2} + p
P59 = 31.56 kips

P61 := \frac{V}{n} + M1 \cdot \frac{x61}{\Sigma d1} + M2 \cdot \frac{y61}{\Sigma d2} + p
P61 = 29.472 kips

P63 := \frac{V}{n} + M1 \cdot \frac{x63}{\Sigma d1} + M2 \cdot \frac{y63}{\Sigma d2} + p
P63 = 27.385 kips

P65 := \frac{V}{n} + M1 \cdot \frac{x65}{\Sigma d1} + M2 \cdot \frac{y65}{\Sigma d2} + p
P65 = 24.923 kips

P54 = 36.779 kips
P56 := \frac{V}{n} + M1 \cdot \frac{x56}{\Sigma d1} + M2 \cdot \frac{y56}{\Sigma d2} + p
P56 = 34.691 kips

P58 := \frac{V}{n} + M1 \cdot \frac{x58}{\Sigma d1} + M2 \cdot \frac{y58}{\Sigma d2} + p
P58 = 32.604 kips

P60 := \frac{V}{n} + M1 \cdot \frac{x60}{\Sigma d1} + M2 \cdot \frac{y60}{\Sigma d2} + p
P60 = 30.516 kips

P62 := \frac{V}{n} + M1 \cdot \frac{x62}{\Sigma d1} + M2 \cdot \frac{y62}{\Sigma d2} + p
P62 = 28.429 kips

P64 := \frac{V}{n} + M1 \cdot \frac{x64}{\Sigma d1} + M2 \cdot \frac{y64}{\Sigma d2} + p
P64 = 25.967 kips

P66 := \frac{V}{n} + M1 \cdot \frac{x66}{\Sigma d1} + M2 \cdot \frac{y66}{\Sigma d2} + p
P66 = 23.88 kips
<table>
<thead>
<tr>
<th>P67 (= \frac{V}{n} + \frac{M_1}{\Sigma d_1} + \frac{M_2}{\Sigma d_2} + p )</th>
<th>P68 (= \frac{V}{n} + \frac{M_1}{\Sigma d_1} + \frac{M_2}{\Sigma d_2} + p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>(= 22.836 ) kips</td>
<td>(= 21.792 ) kips</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>P69 (= \frac{V}{n} + \frac{M_1}{\Sigma d_1} + \frac{M_2}{\Sigma d_2} + p )</th>
<th>P70 (= \frac{V}{n} + \frac{M_1}{\Sigma d_1} + \frac{M_2}{\Sigma d_2} + p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>(= 20.748 ) kips</td>
<td>(= 19.704 ) kips</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>P71 (= \frac{V}{n} + \frac{M_1}{\Sigma d_1} + \frac{M_2}{\Sigma d_2} + p )</th>
<th>P72 (= \frac{V}{n} + \frac{M_1}{\Sigma d_1} + \frac{M_2}{\Sigma d_2} + p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>(= 18.661 ) kips</td>
<td>(= 17.617 ) kips</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>P73 (= \frac{V}{n} + \frac{M_1}{\Sigma d_1} + \frac{M_2}{\Sigma d_2} + p )</th>
<th>P74 (= \frac{V}{n} + \frac{M_1}{\Sigma d_1} + \frac{M_2}{\Sigma d_2} + p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>(= 16.573 ) kips</td>
<td>(= 15.529 ) kips</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>P75 (= \frac{V}{n} + \frac{M_1}{\Sigma d_1} + \frac{M_2}{\Sigma d_2} + p )</th>
<th>P76 (= \frac{V}{n} + \frac{M_1}{\Sigma d_1} + \frac{M_2}{\Sigma d_2} + p )</th>
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</thead>
<tbody>
<tr>
<td>(= 14.486 ) kips</td>
<td>(= 13.442 ) kips</td>
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<thead>
<tr>
<th>P77 (= \frac{V}{n} + \frac{M_1}{\Sigma d_1} + \frac{M_2}{\Sigma d_2} + p )</th>
<th>P78 (= \frac{V}{n} + \frac{M_1}{\Sigma d_1} + \frac{M_2}{\Sigma d_2} + p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>(= 12.398 ) kips</td>
<td>(= 11.354 ) kips</td>
</tr>
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</table>
P79 := \frac{V}{n} + M_1 \frac{x_{79}}{\Sigma d_1} + M_2 \frac{y_{79}}{\Sigma d_2} + p \\
P79 = 10.31 \text{ kips}

P80 := \frac{V}{n} + M_1 \frac{x_{80}}{\Sigma d_1} + M_2 \frac{y_{80}}{\Sigma d_2} + p \\
P80 = 9.267 \text{ kips}

P81 := \frac{V}{n} + M_1 \frac{x_{81}}{\Sigma d_1} + M_2 \frac{y_{81}}{\Sigma d_2} + p \\
P81 = 8.223 \text{ kips}

P82 := \frac{V}{n} + M_1 \frac{x_{82}}{\Sigma d_1} + M_2 \frac{y_{82}}{\Sigma d_2} + p \\
P82 = 7.179 \text{ kips}

P83 := \frac{V}{n} + M_1 \frac{x_{83}}{\Sigma d_1} + M_2 \frac{y_{83}}{\Sigma d_2} + p \\
P83 = 6.135 \text{ kips}

P84 := \frac{V}{n} + M_1 \frac{x_{84}}{\Sigma d_1} + M_2 \frac{y_{84}}{\Sigma d_2} + p \\
P84 = 5.091 \text{ kips}

P85 := \frac{V}{n} + M_1 \frac{x_{85}}{\Sigma d_1} + M_2 \frac{y_{85}}{\Sigma d_2} + p \\
P85 = 3.674 \text{ kips}

P86 := \frac{V}{n} + M_1 \frac{x_{86}}{\Sigma d_1} + M_2 \frac{y_{86}}{\Sigma d_2} + p \\
P86 = 2.63 \text{ kips}

P87 := \frac{V}{n} + M_1 \frac{x_{87}}{\Sigma d_1} + M_2 \frac{y_{87}}{\Sigma d_2} + p \\
P87 = 1.586 \text{ kips}

P88 := \frac{V}{n} + M_1 \frac{x_{88}}{\Sigma d_1} + M_2 \frac{y_{88}}{\Sigma d_2} + p \\
P88 = 0.542 \text{ kips}

P89 := \frac{V}{n} + M_1 \frac{x_{89}}{\Sigma d_1} + M_2 \frac{y_{89}}{\Sigma d_2} + p \\
P89 = -0.501 \text{ kips}

P90 := \frac{V}{n} + M_1 \frac{x_{90}}{\Sigma d_1} + M_2 \frac{y_{90}}{\Sigma d_2} + p \\
P90 = -1.545 \text{ kips}
P91 := \frac{V}{n} + \frac{M_1 x^{91}}{\Sigma d_1} + \frac{M_2 y^{91}}{\Sigma d_2} + p

P91 = -2.589 kips

P93 := \frac{V}{n} + \frac{M_1 x^{93}}{\Sigma d_1} + \frac{M_2 y^{93}}{\Sigma d_2} + p

P93 = -4.676 kips

P95 := \frac{V}{n} + \frac{M_1 x^{95}}{\Sigma d_1} + \frac{M_2 y^{95}}{\Sigma d_2} + p

P95 = -6.764 kips

P97 := \frac{V}{n} + \frac{M_1 x^{97}}{\Sigma d_1} + \frac{M_2 y^{97}}{\Sigma d_2} + p

P97 = -8.852 kips

P99 := \frac{V}{n} + \frac{M_1 x^{99}}{\Sigma d_1} + \frac{M_2 y^{99}}{\Sigma d_2} + p

P99 = -10.939 kips

P101 := \frac{V}{n} + \frac{M_1 x^{101}}{\Sigma d_1} + \frac{M_2 y^{101}}{\Sigma d_2} + p

P101 = -13.027 kips

P102 := \frac{V}{n} + \frac{M_1 x^{102}}{\Sigma d_1} + \frac{M_2 y^{102}}{\Sigma d_2} + p

P102 = -14.071 kips
Pile punching shear check:

\[ V_{105} := \frac{V}{n} + M_1 \frac{x_{105}}{\Sigma d_1} + M_2 \frac{y_{105}}{\Sigma d_2} + p \]

\[ P105 = -17.202 \text{ kips} \]

\[ K := F \cdot b \cdot \frac{d}{1000} \]

\[ V_c := 4 \cdot \sqrt{F_c \cdot b \cdot d} \]

\[ V_c := P1 \]

\[ V_u := 92.847 \text{ kips} \quad \text{Maximum axial load in piles} \]

\[ d := 45 \text{ in} \quad \text{Concrete slab thickness for 3 ft footing} \]

\[ b := 191.6 \text{ in} \quad \text{Perimeter acting in punching shear} \]
\[ V_c = 2.034 \times 10^3 \text{ kips} \]

\[ \phi := 0.85 \]

**Strength reduction factor**

\[ \frac{V_u}{\phi} = 109.232 \quad \frac{V_u}{\phi} < V_c \quad \text{OK} \]

**One way shear Check:**

\[ V_1 := 2.933 \times 10^3 \text{ Kips} \quad V_1 \text{ is equal to sum of P1 to P42} \]

\[ V_2 := 549.898 \text{ Kips} \quad V_2 \text{ is equal to sum of P43 to P105} \]

\[ b_2 := 1575 \text{ in} \quad \text{Length of pilecap} \]

\[ V_{c2} := 4 \sqrt{F_c \cdot b_2 \cdot \frac{d}{1000}} \]
\[ V_{c2} = 1.672 \times 10^4 \]
\[ \frac{V_1}{\phi} = 3.451 \times 10^3 \]
\[ \frac{V_1}{\phi} < V_{c2} \quad \text{OK} \]

\[ v_1 := (P_1 + P_2 + P_3 + P_4 + P_5) \]
\[ v_2 := (P_{22} + P_{23} + P_{24} + P_{25} + P_{26}) \]
\[ v_3 := (P_{43} + P_{44} + P_{45} + P_{46} + P_{47}) \]
\[ v_4 := (P_{64} + P_{65} + P_{66} + P_{67} + P_{68}) \]
\[ v_5 := (P_{85} + P_{86} + P_{87} + P_{88} + P_{89}) \]
\[ v := v_1 + v_2 + v_3 + v_4 + v_5 \]
\[ v = 1.154 \times 10^3 \]
\begin{align*}
v = \frac{0.85}{1.358 \times 10^3} = 0.000733 \\
b_3 &= 375 \\
F_c &= 3480 \\
d &= 36 \\
V_{c3} &= 4\sqrt{F_c \cdot b_3 \cdot \frac{d}{1000}} \\
V_{c3} &= 3.186 \times 10^3 \\
\frac{v}{0.85} < V_{c3} &\quad \text{OK}
\end{align*}

Two-way shear check:

\begin{align*}
V_{u} &= V_1 + V_2 \\
V_{u} &= 3.483 \times 10^3
\end{align*}
b₀ := 801.91 \cdot 72.37 \\
\text{Perimeter of inside rectangular}

b₀ = 5.803 \times 10^4 \text{ in}

V_c := 4\sqrt{F_c \cdot b₀ \cdot d}

V_c = 4.93 \times 10^8

\phi := 0.85 \text{ Strength reduction factor}

\frac{V_u}{\phi} = 4.098 \times 10^3 < V_c \text{ ok}

\textbf{Flexure bending design:}
Use 31 # 9 in long direction at the top of footing
SECTION B-B

\[ V_1 := 1.731 \times 10^3 \]

\[ V_2 := 1.262 \times 10^3 \]

\[ M_1 := (V_1) \times 55 \times 1000 \quad \text{V1 is equal to sumation of P1 to P21} \]

\[ M_2 := (V_2) \times 130 \times 1000 \quad \text{V2 is equal to sumation of P22 to P42} \]

\[ \mu := M_1 + M_2 \]

\[ \mu = 2.593 \times 10^8 \quad \text{lb} \quad \text{in} \]

\[ \Phi := 0.9 \quad \text{Strength reduction factor} \]

\[ As := \frac{\mu}{\Phi \times F_y \times 0.9 \times d} \quad \text{Area of steel} \]

\[ As = 148.185 \quad \text{in}^2 \]

\[ As_{\text{min}} := 0.0018 \times 45 \times 1575 \]

\[ As_{\text{min}} = 127.575 \quad \text{Minimum reinforcement} \]

\[ As_{\text{min}} = 127.575 \quad \text{in}^2 \quad \text{less than As} \quad \text{OK} \]

In short direction: 149 # 9

Use 103 # 9 in short direction at the top of footing
CAP Beam Design:

For Cap-beam design, a simple 2D model under developed. (11 concentrated loads applying on the beam as shown below).

![2D model of Cap-beam]

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<th>Forces &amp; Moments-dead load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>Shear 2 kips</td>
</tr>
<tr>
<td>Beam</td>
<td>49</td>
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</table>

<table>
<thead>
<tr>
<th>NCHRP</th>
<th>Forces &amp; Moments-Trans(Y) EQ 100%Trans+40%Long</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>Shear 2 kips</td>
</tr>
<tr>
<td>Beam</td>
<td>593</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>NCHRP</th>
<th>Forces &amp; Moments-Long(X) EQ 100%Long+40%Trans</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>Shear 2 kips</td>
</tr>
<tr>
<td>Beam</td>
<td>783</td>
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</tbody>
</table>
Final Design Forces and Moments:

<table>
<thead>
<tr>
<th>Location</th>
<th>Shear 2</th>
<th>Shear 3</th>
<th>Moment 2</th>
<th>Moment 3</th>
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<tbody>
<tr>
<td>Beam</td>
<td>642</td>
<td>5</td>
<td>8</td>
<td>5730</td>
</tr>
<tr>
<td></td>
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</tbody>
</table>

Table 1

Reinforcement Design for continuous deep beam:

Deep beam design (Source: Reinforcement concrete, Nawy, 3rd edition, section 6.9)

Shear reinforcement:

\[ f_c = 3480 \text{ psi} \]

Compressive Strength of Concrete

\[ f_y = 60000 \text{ psi} \]

Yield strength of reinforcing bars

\[ l = 175.19 \text{ in} \]

span

\[ l_n = 105.2 \text{ in} \]

clear span

\[ h = 56.5 \text{ in} \]

Height of the beam

\[ b_w = 39.37 \text{ in} \]

Width of beam

\[ d = 0.9 \cdot h \]

\[ d = 50.85 \text{ in} \]
\[
\frac{l_n}{d} = 2.069 \quad < 5 \text{, hence treat as a deep beam}
\]

\[
0.15 \cdot l_n = 15.78 \quad \text{in} \quad \text{Distance of the critical section (6.17a)}
\]

\[
V := 642 \quad \text{kip} \quad \text{Design shear force from table 1}
\]

\[
V_a := \frac{V \cdot l_n}{2} - V \cdot \frac{0.15 \cdot l_n}{12}
\]

\[
V_a = 3.292 \times 10^4 \quad \text{kip}
\]

\[
\phi := 0.85 \quad \text{Strength reduction factor}
\]

\[
\phi V_n := 0.8 \cdot 8 \cdot \sqrt{f_c b_w d}
\]

\[
\phi V_n = 8.031 \times 10^5 \quad \text{kip} \quad > V_a \quad \text{OK}
\]

\[
M_a := V \cdot \left( \frac{l_n}{12} \right) - V \cdot \left( \frac{0.15 \cdot l_n}{12} \right)
\]

\[
M_a = 3.145 \times 10^3
\]

\[
3.5 - 2.5 \cdot \frac{M_a}{V_a d} = 3.495 \quad > 2.5 \quad \text{Use 2.5}
\]

\[
\rho_w = \frac{9 \cdot 0.79}{b_w d} \quad \text{Steel Ratio in tension (9 No.8)}
\]

\[
\rho_w = 3.552 \times 10^{-3}
\]

C-65
\[ V_c := 2.5 \left( 1.9 \sqrt{f_c} + 2500 \cdot \rho_w \cdot \frac{V_u \cdot d}{M_u} \right) \cdot b_w \cdot d \]

\[ V_c = 2.421 \times 10^7 \quad \text{lb} \]

\[ V_c := 6 \sqrt{f_c} \cdot b_w \cdot d = \left( 7.086 \times 10^5 \right) \quad \text{Controls} \]

Assume No.5 bars placed both horizontally and vertically on both faces

\[ V_s := \frac{V_u}{\phi} - \frac{V_c}{1000} \]

\[ V_s = 3.803 \times 10^4 \]

\[ A_v := .314 \quad 4 \text{ No. 5} \]

\[ A_{vh} := 6 \cdot 0.44 \quad 6 \text{ No. 6 in horizontal position} \]

\[ A_{vh} = 2.64 \]

Try \quad S_1 := 18

\[ V_s := \left( \frac{A_v}{S_1 \cdot 12} + \frac{A_{vh}}{S_1 \cdot 12} \right) \cdot f_y \cdot d \]

\[ V_s = 3.868 \times 10^5 \]

\[ \frac{d}{f} = 10.17 \quad \text{or 18 in} \quad \text{Maximum Spacing} \]
\[
\frac{d}{3} = 16.95 \quad \text{or 18 in}
\]

\[
S := 10 \quad \text{Controls}
\]

\[
A_v := 0.0015 \cdot b_w \cdot S
\]

Minimum reinforcement

\[
A_v = 0.591 \quad \text{less than } Av=0.62 \quad \text{ok}
\]

\[
A_{vh} := 0.0025 \cdot b_w \cdot S
\]

Minimum reinforcement

\[
A_{vh} = 0.984 \quad \text{less than } Avh=3.52. \quad \text{ok}
\]

**Flexural Steel:**

\[
M := 5370 \quad \text{kip} - \text{ft}
\]

\[
\phi := 0.9 \quad \text{Strength reduction factor}
\]

\[
M_n := \frac{M_u}{\phi}
\]

\[
jd := 0.2(l + 1.5 \cdot h) \quad \text{Lever arm}
\]

\[
jd = 51.988 \quad \text{in}
\]

\[
A_s := \frac{M_n \cdot 12000}{f_y \cdot jd}
\]

Area of tension steel

\[
A_s = 13.445 \quad \text{in}^2
\]
Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

NCHRP Guidelines

Use 14 # 9 as flexural reinforcement

AASHTO LRFD SEC.5.7.3.3.2:

1) \( \rho_{\text{min}} := 0.03 \frac{f_c}{f_y} \)

\( \rho_{\text{min}} = 1.74 \times 10^{-3} \)

\( \rho_{\text{min}} := \frac{A_s}{b_w d} \)

\( \rho_{\text{min}} = 6.716 \times 10^{-3} \)

\( \rho_{\text{min}} > \rho_{\text{min}} \) \text{ OK}

\( A_{s1} := \frac{200}{f_y} b_w d \)

\( A_{s2} := \frac{3 \sqrt{f_c}}{f_y} b_w d \)

\( A_{s1} = 6.673 \)

\( A_{s2} = 5.905 \) Controls

\( A_{\text{min}} := \max (A_{s1}, A_{s2}) \)

\( A_{\text{min}} = 6.673 \) in\(^2\) less than \( A_{s} \) OK

\( A_{s} = 13.445 \) in\(^2\)
2)

\[ F_r := 7.5 \sqrt{f_c} \]

\[ I_g := \frac{b_w \cdot h^3}{12} \]

Moment of Inertia

\[ y_t := \frac{h}{2} \]

\[ M_{cr} := \frac{F_r \cdot I_g}{12000} \]

Cracking Moment

\[ M_{cr} = 772.291 \]

\[ a := A_s \cdot \frac{f_y}{0.85 \cdot f_c \cdot b_w} \]

\[ a = 6.927 \]

\[ M_u := \phi \cdot A_s \cdot f_y \left( d - \frac{a}{2} \right) \cdot \frac{1}{12000} \]

\[ M_u = 2.867 \times 10^3 \]

\[ M_u \geq 1.2 \cdot M_{cr} \quad \text{OK} \]
Fig. 5. bent front view with dimensions

Fig. 6. bent side view with dimensions
Fig. 7. Reinforcement details for Columns, Footing and Cap beam (front view)

Fig. 8. Reinforcement details for Footing and Cap beam (Side view)
Appendix D -- Detailed Computations for Seismic Analysis and Design Check of the St. Clair County Bridge Using AASHTO Specifications
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<tr>
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<td></td>
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<tr>
<td>Pertinent Information about Materials used, etc</td>
<td>D-24</td>
</tr>
<tr>
<td>Moment of inertia for superstructure</td>
<td>D-25</td>
</tr>
<tr>
<td>Torsional properties</td>
<td>D-26</td>
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<tr>
<td>Cross sectional Area</td>
<td>D-26</td>
</tr>
<tr>
<td>Loads</td>
<td>D-27</td>
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<td>Columns</td>
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<td>Cap beam</td>
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<tr>
<td>Earthquake Data</td>
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<tr>
<td>Design Checks</td>
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<tr>
<td>Moment-Columns</td>
<td>D-31</td>
</tr>
<tr>
<td>Moment-Crashwall</td>
<td>D-31</td>
</tr>
<tr>
<td>Shear-Columns</td>
<td>D-31</td>
</tr>
<tr>
<td>Shear-Crashwall</td>
<td>D-32</td>
</tr>
<tr>
<td>Slenderness Effects</td>
<td>D-32</td>
</tr>
</tbody>
</table>
SAP 2000 Model

View of SAP 2000 Model (with concrete extrusion shown):
Enlarged View of Translational and Rotational Springs:
Deformed shape; Mode 1 - 1.0392 sec.

Deformed shape; Mode 2 - 0.6622 sec.
Deformed shape; Mode 6 - 0.3609 sec.

Deformed shape; Mode 8 - 0.3168 sec.
## TABLE: Modal Participating Mass Ratios

<table>
<thead>
<tr>
<th>Mode Number</th>
<th>Period Sec</th>
<th>UX %</th>
<th>UY %</th>
<th>UZ %</th>
</tr>
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<tbody>
<tr>
<td>1</td>
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<td>6</td>
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<td>16</td>
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<td>0.03</td>
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<tr>
<td>25</td>
<td>0.0205</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

100.00  99.23  100.00
Piles (L-pile results)

Longitudinal direction of the bridge:
Transverse direction of the bridge:

![Graph showing bending moment and shear force](image-url)
Assumed location of Global and Local coordinate systems (due to the skew of the Pier):

- Longitudinal - X axis direction;
- Transverse - Y axis direction.
Axial Force On A Pile:

\[ \phi := 26.528 \text{ degrees} \]

\[ R_{Sc} := 5 \]

\[ R_r := R_{Sc} \times 0.5 \]

Skew of the bridge

Response Modification Factor for a pile bent:

\[ M_{Y\text{glob}} := \frac{21913}{R_r} \]

\[ M_{Y\text{glob}} = 8.765 \times 10^3 \text{ k – ft} \]

Longitudinal

\[ M_{Y\text{localY}} := M_{Y\text{glob}} \times \sin(\phi) \]

\[ M_{Y\text{localY}} = 8.631 \times 10^3 \text{ k – ft} \]

\[ M_{X\text{glob}} := \frac{5531}{R_r} \]

\[ M_{X\text{glob}} = 2.212 \times 10^3 \text{ k – ft} \]

Transverse

\[ M_{X\text{localY}} := M_{X\text{glob}} \times \cos(\phi) \]

\[ M_{X\text{localY}} = 1.531 \times 10^3 \text{ k – ft} \]

\[ M_{X\text{localX}} := M_{X\text{glob}} \times \sin(\phi) \]

\[ M_{Y\text{locax}} := M_{X\text{glob}} \times \cos(\phi) \]

\[ M_{eql} := M_{Y\text{localY}} + M_{Y\text{locax}} \]

\[ M_{eql} = 9.017 \times 10^3 \text{ k – ft} \]

Longitudinal

\[ M_{eqt} := M_{X\text{localY}} + M_{X\text{localX}} \]

\[ M_{eqt} = 3.709 \times 10^3 \text{ k – ft} \]

Transverse

\[ P_w := 3291 \text{ kips} \]

The distance from center line of foundation to center of pile (ft):

\[ d_1 := 3.281 \quad d_2 := 3.002 \quad d_3 := 9.006 \quad d_4 := 14.747 \quad d_5 := 20.488 \quad d_6 := 26.229 \quad d_7 := 31.97 \]
Sum of squares of the distances to each pile from the center line of the foundation:

\[ \Sigma d_1 := 24 \left( d_1^2 \right) \]
\[ \Sigma d_1 = 258.359 \text{ ft}^2 \quad \text{(longitudinal)} \]

\[ \Sigma d_2 := 6 \left( d_2^2 \right) + 6 \left( d_3^2 \right) + 6 \left( d_4^2 \right) + 6 \left( d_5^2 \right) + 6 \left( d_6^2 \right) + 6 \left( d_7^2 \right) \]
\[ \Sigma d_2 = 1.462 \times 10^4 \text{ ft}^2 \quad \text{(transverse)} \]

Piles Loads:

\[ F_{\text{pile}1} := \left( \frac{P_w}{36} \right) + \left( \frac{M_{\text{eqL}}}{\Delta d_1} \right) \left( \frac{d_1}{\Delta d_1} \right) + \left( \frac{M_{\text{eqT}}}{\Delta d_2} \right) \left( \frac{d_2}{\Delta d_2} \right) \quad F_{\text{pile}1} = 214 \text{ kips} \]

\[ F_{\text{pile}12} := \left( \frac{P_w}{36} \right) + \left( \frac{M_{\text{eqL}}}{\Delta d_1} \right) - \left( \frac{M_{\text{eqT}}}{\Delta d_2} \right) \left( \frac{d_2}{\Delta d_2} \right) \quad F_{\text{pile}12} = 197.82 \text{ kips} \]

\[ F_{\text{pile}25} := \left( \frac{P_w}{36} \right) - \left( \frac{M_{\text{eqL}}}{\Delta d_1} \right) + \left( \frac{M_{\text{eqT}}}{\Delta d_2} \right) \left( \frac{d_2}{\Delta d_2} \right) \quad F_{\text{pile}25} = -14.98 \text{ kips} \]

\[ F_{\text{pile}36} := \left( \frac{P_w}{36} \right) - \left( \frac{M_{\text{eqL}}}{\Delta d_1} \right) - \left( \frac{M_{\text{eqT}}}{\Delta d_2} \right) \left( \frac{d_2}{\Delta d_2} \right) \quad F_{\text{pile}36} = -31.2 \text{ kips} \]

Note: \( Q_{\text{ult}} = Q_s + Q_t - W_{\text{self}} \) (Compression) and \( 0.7Q_s + W \) (uplift);
\( Q_{\text{allow}} = Q_{\text{ult}} / FS \)

Forces from SAP2000 model:

COMB 1: \( F_{\text{longitud.}} = 1367.04 \text{ kips}, F_{\text{transverse}} = 1327.71 \text{ kips}, \)
\( M_{\text{longitud.}} = 5530.71 \text{ k-ft}, M_{\text{transverse}} = 21912.78 \text{ k-ft} \)

COMB 2: \( F_{\text{longitud.}} = 1148.89 \text{ kips}, F_{\text{transverse}} = 1021.76 \text{ kips}, \)
\( M_{\text{longitud.}} = 2988.43 \text{ k-ft}, M_{\text{transverse}} = 11482.57 \text{ k-ft} \)
Combined axial load and bending (AASHTO sixteenth edition): Article 10.54.2.1; equation 10-156.

HP 12 x 53 pile properties:

- \( F_y \) := 36 ksi
- \( A_{pile} \) := 15.5 in\(^2\)
- \( S_x \) := 21.1 in\(^3\)
- \( S_y \) := 66.7 in\(^3\)
- \( Z_x \) := 74 in\(^3\)
- \( Z_y \) := 32.2 in\(^3\)

\[
M_{px} := F_y Z_x
\]

\[ M_{px} = 2.664 \times 10^3 \text{ k-in} \]  
full plastic moment about X-axis

\[
M_{py} := F_y Z_y
\]

\[ M_{py} = 1.159 \times 10^3 \text{ k-in} \]  
full plastic moment about Y-axis

\( P_{pile} := 214 \text{ k} \)  
arxial load at top of pile

LPile output (local coordinates):

- \( M_{xlpile} := 1623 \text{ k-in} \)  
moment about X-axis (single pile)
- \( M_{ylpile} := 948.2 \text{ k-in} \)  
moment about Y-axis (single pile)
- \( V_{xlpile} := 35.7 \text{ k} \)  
longitudinal lateral load (single pile)
- \( V_{ylpile} := 27.8 \text{ k} \)  
transverse lateral load (single pile)
Capacity of the piles (deflection limited to 0.5in):

\[ n_p := 36 \]
\[ V_{Xfull} := V_{xpile} \times n_p \]
\[ V_{Xfull} = 1.285 \times 10^3 \text{ k} \] longitudinal direction
\[ V_{Yfull} := V_{ypile} \times n_p \]
\[ V_{Yfull} = 1.001 \times 10^3 \text{ k} \] transverse direction

Maximum Loads (SAP 2000 Model output):

\[ \phi := 26.528 \text{ degrees} \] skew of the bridge
\[ V_{Xglob} := 1367 \text{ k} \]
\[ V_{Yglob} := 1327 \text{ k} \]

Converting loads from Global to Local coordinate system:

\[ V_{XlocalY} := V_{Yglob} \times \cos(\phi) \]
\[ V_{XlocalY} = 231.744 \text{ k} \] X component (local coord. syst.) of transverse load in global coord. syst.
\[ V_{YlocalY} := V_{Yglob} \times \sin(\phi) \]
\[ V_{YlocalY} = 1.307 \times 10^3 \text{ k} \] Y component (local coord. syst.) of transverse load in global coord. syst.
\[ V_{XlocalX} := V_{Xglob} \times \sin(\phi) \]
\[ V_{XlocalX} = 1.346 \times 10^3 \text{ k} \] X component (local coord. syst.) of longitudinal load in global coord. syst.
\[ V_{YlocalX} := V_{Xglob} \times \cos(\phi) \]
\[ V_{YlocalX} = 238.729 \text{ k} \] Y component (local coord. syst.) of longitudinal load in global coord. syst.
\[ V_{Xmax} := V_{XlocalY} + V_{XlocalX} \]
\[ V_{Xmax} = 1.578 \times 10^3 \text{ k} \] transverse load (local coordinates)
\[ V_{Ymax} := V_{YlocalY} + V_{YlocalX} \]
\[ V_{Ymax} = 1.545 \times 10^3 \text{ k} \] longitudinal load (local coordinates)
**Response Modification Factor (R) for vertical piles (AASHTO Division A1, table 3.7) Can be increased by factor of 0.5 (Article 6.2.2 Design forces for foundations)**

$$R_{pm} := R_p \cdot 0.5$$  

**Response Modification Factor (R) with factor of 0.5 applied:**

$$M_x := \frac{\left( \frac{V_{x,\text{max}}}{V_{x,\text{full}}} \right)}{R_{pm}} \quad M_x = 796.971 \quad \text{k-in}$$

$$M_y := \frac{\left( \frac{V_{y,\text{max}}}{V_{y,\text{full}}} \right)}{R_{pm}} \quad M_y = 585.647 \quad \text{k-in}$$

$$\left( \frac{P_{\text{pile}}}{0.85 \cdot A_{\text{pile}} \cdot F_y} \right) + \frac{8}{9} \left[ \left( \frac{M_x}{M_{\text{px}}} \right) + \left( \frac{M_y}{M_{\text{py}}} \right) \right] = 1.166$$

*In order to satisfy AASHTO design requirements the resultant of this equation must be less than or equal 1.*

*We used 5 here because AASHTO recommends R=5 for a multi-column bent. To be conservative, a 3 could be used, if the bent considered a single column for X-axis bending.*
Springs (at bottom of foundation)

*Actual L-Pile input - output information is given in the Appendix A.*

<table>
<thead>
<tr>
<th>Pile Information for L-Pile input:</th>
<th></th>
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</thead>
<tbody>
<tr>
<td>HP 12x53 - piles</td>
<td>ft := 12 in</td>
</tr>
<tr>
<td>L_pile := 936 in</td>
<td>k := 1000 lb</td>
</tr>
<tr>
<td>L_pile = 78 ft</td>
<td>m = 3.281 length</td>
</tr>
<tr>
<td>A_pile := 15.5 in^2</td>
<td>Apile := 0.108 ft^2</td>
</tr>
<tr>
<td>E_S := 29000000 lb/in^2</td>
<td>E_S = 4176000000 lb/ft^2</td>
</tr>
</tbody>
</table>

Δ := 0.5 in top of pile displacement limit.

<table>
<thead>
<tr>
<th>Results from L-Pile (displacement is limited to 0.5 in):</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>M_{utrans} := 9.482 x 10^5 lb in</td>
<td>M_{utrans} = 7.902 x 10^4 lb ft Transverse</td>
</tr>
<tr>
<td>V_{utrans} := 2.7803 x 10^4 lb</td>
<td>V_{utrans} = 2.78 x 10^4 lb Transverse</td>
</tr>
<tr>
<td>M_{ulong} := 1.623 x 10^6 lb in</td>
<td>M_{ulong} = 1.353 x 10^5 lb ft Longitudinal</td>
</tr>
<tr>
<td>V_{ulong} := 3.5646 x 10^4 lb</td>
<td>V_{ulong} = 3.565 x 10^4 lb Longitudinal</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Spring Stiffness:</th>
<th></th>
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<tbody>
<tr>
<td>k_{axial} := A_pile \frac{E_S}{L_pile}</td>
<td>k_{axial} = 5.763 x 10^6 lb ft Axial</td>
</tr>
<tr>
<td>k_{long} := \frac{V_{ulong}}{\Delta}</td>
<td>k_{long} = 8.555 x 10^5 lb ft Longitudinal</td>
</tr>
<tr>
<td>k_{trans} := \frac{V_{utrans}}{\Delta}</td>
<td>k_{trans} = 6.673 x 10^5 lb ft Transverse</td>
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</tbody>
</table>
Lateral Springs:

\[ k_{\text{longB}} := 173.8 \times 10^3 \frac{\text{lb}}{\text{ft}} \text{ for battered pile} \]

\[ K_{\text{long}} := 24 \left( k_{\text{longB}} + k_{\text{long}} \right) + 12k_{\text{long}} \quad K_1 := 3.497 \times 10^4 \frac{k}{\text{ft}} \]

\[ K_{\text{trans}} := 36 \cdot k_{\text{trans}} \quad K_2 := 2.402 \times 10^4 \frac{k}{\text{ft}} \]

Axial Springs:

\[ K_{\text{axial}} := 36 \cdot k_{\text{axial}} \quad K_3 := 2.075 \times 10^5 \frac{k}{\text{ft}} \]

Rotational Springs:

<table>
<thead>
<tr>
<th>Distance between piles (transverse)</th>
<th>Distance between footing C.L. and piles C.L.</th>
<th>Distance between piles (longitudinal)</th>
<th>Distance from center of footing to center of piles</th>
</tr>
</thead>
<tbody>
<tr>
<td>( l_1 := 3.002 \text{ ft} )</td>
<td>( d_1 := 3.002 \text{ ft} )</td>
<td>( w := 3.28 \text{ ft} )</td>
<td>( n_1 := \sqrt{d_1^2 + w^2} ) ( n_1 = 4.446 \text{ ft} )</td>
</tr>
<tr>
<td>( l_2 := 6.004 \text{ ft} )</td>
<td>( d_2 := 9.006 \text{ ft} )</td>
<td>( n_2 := \sqrt{d_2^2 + w^2} )</td>
<td>( n_2 = 9.585 \text{ ft} )</td>
</tr>
<tr>
<td>( l_3 := 5.741 \text{ ft} )</td>
<td>( d_3 := 14.747 \text{ ft} )</td>
<td>( n_3 := \sqrt{d_3^2 + w^2} )</td>
<td>( n_3 = 15.107 \text{ ft} )</td>
</tr>
<tr>
<td>( l_4 := 5.741 \text{ ft} )</td>
<td>( d_4 := 20.488 \text{ ft} )</td>
<td>( n_4 := \sqrt{d_4^2 + w^2} )</td>
<td>( n_4 = 20.749 \text{ ft} )</td>
</tr>
<tr>
<td>( l_5 := 5.741 \text{ ft} )</td>
<td>( d_5 := 26.229 \text{ ft} )</td>
<td>( n_5 := \sqrt{d_5^2 + w^2} )</td>
<td>( n_5 = 26.433 \text{ ft} )</td>
</tr>
<tr>
<td>( l_6 := 5.741 \text{ ft} )</td>
<td>( d_6 := 31.97 \text{ ft} )</td>
<td>( n_6 := \sqrt{d_6^2 + w^2} )</td>
<td>( n_6 = 32.138 \text{ ft} )</td>
</tr>
</tbody>
</table>
**Title:**
St. Clair County Bridge

**Subject File:**
Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

**AASHTO Standard Specifications**

---

\[
K_{rx} := k_{axial} \cdot 6 \left[ (d_1)^2 + (d_2)^2 + (d_3)^2 + (d_4)^2 + (d_5)^2 + (d_6)^2 \right]
\]

\[
K_{r1} := 8.428 \times 10^7 \frac{k}{ft} \quad \text{about X axis}
\]

\[
K_{ry} := k_{axial} \cdot 33 \cdot 3.28^2 \quad \quad K_{ry} = 2.046 \times 10^9 \text{ mass length}^{-1}
\]

\[
K_{r2} := 24.55 \times 10^6 \frac{k}{ft} \quad \text{about Y axis}
\]

\[
K_{rz} := k_{long} \left[ 4 \left[ (n_1)^2 + (n_2)^2 + (n_3)^2 + (n_4)^2 + (n_5)^2 + (n_6)^2 \right] \cdots \right] \\
\quad + 2 \left[ (d_1)^2 + (d_2)^2 + (d_3)^2 + (d_4)^2 + (d_5)^2 + (d_6)^2 \right]
\]

\[
K_{r3} := 1.273 \times 10^7 \frac{k}{ft} \quad \text{about Z axis}
\]
Substructure

Supports (both abutments): Longitudinal - Roller; Transverse - Pin

<table>
<thead>
<tr>
<th>Support/Location</th>
<th>Longitudinal (Weak Direction of Pier)</th>
<th>Transverse (Strong Direction of Pier)</th>
<th>Axial</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>Shear X (kips) V3</td>
<td>Moment Y (kip-ft) M2</td>
<td>Shear Y (kips) V2</td>
</tr>
<tr>
<td>Top of Cap Beam</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1(external)</td>
<td>80.00</td>
<td>26.67</td>
<td>285.00</td>
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<tr>
<td>3(middle)</td>
<td>199.00</td>
<td>66.33</td>
<td>708.00</td>
</tr>
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<td>Top of column</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>1(external)</td>
<td>127.00</td>
<td>42.33</td>
<td>1041.00</td>
</tr>
<tr>
<td>3(middle)</td>
<td>166.00</td>
<td>55.33</td>
<td>1521.00</td>
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<tr>
<td>Bottom of column</td>
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<td></td>
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<tr>
<td>1(external)</td>
<td>127.00</td>
<td>42.33</td>
<td>515.00</td>
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<tr>
<td>3(middle)</td>
<td>166.00</td>
<td>55.33</td>
<td>518.00</td>
</tr>
<tr>
<td>Bottom of wall</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>1(external)</td>
<td>127.00</td>
<td>42.33</td>
<td>1244.00</td>
</tr>
<tr>
<td>3(middle)</td>
<td>167.00</td>
<td>55.67</td>
<td>1478.00</td>
</tr>
</tbody>
</table>

R = 2.0 (Wall-type Pier); (Substructure)
R = 3.0 (Single Columns); (Substructure)
R = 5.0 (Multiple column bent); (Substructure)
R = 1.0 (Columns, piers or pile bents to cap beam or superstructure); (Connections)
R = 0.8 (Superstructure to abutment); (Connections)

<table>
<thead>
<tr>
<th>Support/Location</th>
<th>Longitudinal (Weak Direction of Pier)</th>
<th>Transverse (Strong Direction of Pier)</th>
<th>Axial</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear X (kips) V3</td>
<td>Moment Y (kip-ft) M2</td>
<td>Shear Y (kips) V2</td>
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<tr>
<td>Top of Cap Beam</td>
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<tr>
<td>1(external)</td>
<td>46.00</td>
<td>15.33</td>
<td>162.00</td>
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<tr>
<td>3(middle)</td>
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<td>3(middle)</td>
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<tr>
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<tr>
<td>1(external)</td>
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<tr>
<td>3(middle)</td>
<td>154.00</td>
<td>51.33</td>
<td>1376.00</td>
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<table>
<thead>
<tr>
<th>Support/Location</th>
<th>Longitudinal (Weak Direction of Pier)</th>
<th>Transverse (Strong Direction of Pier)</th>
<th>Axial</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear X (kips)</td>
<td>Moment Y (kip-ft)</td>
<td>Shear Y (kips)</td>
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<tr>
<td>3</td>
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<tr>
<td>Bottom of wall</td>
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### TABLE: Joint Displacements

<table>
<thead>
<tr>
<th>Joint</th>
<th>OutputCase</th>
<th>CaseType</th>
<th>StepType</th>
<th>U1</th>
<th>U2</th>
<th>U3</th>
<th>R1</th>
<th>R2</th>
<th>R3</th>
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<tbody>
<tr>
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<td>Max</td>
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<td>-0.6920</td>
<td>0.0037</td>
<td>0.0003</td>
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<td>Combination</td>
<td>Max</td>
<td>0.3950</td>
<td>0.4745</td>
<td>-1.2310</td>
<td>0.0020</td>
<td>0.0001</td>
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<tr>
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<td>0.0037</td>
<td>0.0003</td>
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<td>0.4744</td>
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### TABLE: Base Reactions

<table>
<thead>
<tr>
<th>Number</th>
<th>OutputCase</th>
<th>CaseType</th>
<th>StepType</th>
<th>GlobalFX</th>
<th>GlobalFY</th>
<th>GlobalFZ</th>
<th>GlobalMX</th>
<th>GlobalMY</th>
<th>GlobalMZ</th>
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<tbody>
<tr>
<td></td>
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<td></td>
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<td>Kip</td>
<td>Kip</td>
<td>Kip</td>
<td>Kip-ft</td>
<td>Kip-ft</td>
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<td>LinStatic</td>
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<td>COMB2</td>
<td>Combination</td>
<td>Max</td>
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<td>1750</td>
<td>4545</td>
<td>31323</td>
<td>13564</td>
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</tbody>
</table>

### TABLE: Joint Reactions - Spring Forces

<table>
<thead>
<tr>
<th>Joint</th>
<th>OutputCase</th>
<th>CaseType</th>
<th>StepType</th>
<th>U1</th>
<th>U2</th>
<th>U3</th>
<th>R1</th>
<th>R2</th>
<th>R3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td>Kip</td>
<td>Kip-ft</td>
<td>Kip-ft</td>
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<tr>
<td>J3</td>
<td>DEAD</td>
<td>LinStatic</td>
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<td>COMB1</td>
<td>Combination</td>
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<td>1328</td>
<td>3225</td>
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<td>J3</td>
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<td>Combination</td>
<td>Max</td>
<td>1149</td>
<td>1022</td>
<td>3225</td>
<td>2988</td>
<td>11483</td>
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</tr>
</tbody>
</table>
Top of a typical column

**MATERIAL:**
- $f_c = 3.5$ ksi
- $E_c = 33721.7$ ksi
- $f_y = 2.975$ ksi
- $E_y = 60$ ksi
- $f_s = 23600$ ksi

**SECTION:**
- $A_g = 2411.5$ in$^2$
- $I_y = 246174$ in$^4$
- $X_u = 0$ in
- $Y_u = 0$ in

**REINFORCEMENT:**
- 78 #9 bars @ 0.517%
- $A_s = 22.12$ in$^2$
- Confinement: Tied
- Clear Cover = 3.49854 in

---

**BY:**
- G.V.I.

**CHECKED:**
- W.N.M.

**DATE:**
- 6/24/2004

Sheet No: 21 of 33
Bottom of a typical column

MATERIAL:
Pc = 3.5 ksi
Ec = 33720 ksi
fc = 2.975 ksi
Beta1 = 6.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
Ag = 1548 in²
bx = 157288 in⁴
by = 248453 in⁴
Xc = 0 in
Yc = 0 in

REINFORCEMENT:
20 #2 bars @ 1.4366%
As = 22.12 in²
Confinement: Tied
Clear Cover = 3.400 in
Bottom of Crash Wall

MATERIAL:
- $f_c = 3.5$ ksi
- $f_y = 2.575$ ksi
- $f_t = 0.85$
- $f_y = 60$ ksi
- $f_y = 29000$ ksi

SECTION:
- $A_g = 299891$ in$^2$
- $I_x = 1.427986e+09$ in$^4$
- $I_y = 3.88138e+06$ in$^4$
- $x = 0$ in
- $y = 0$ in

REINFORCEMENT:
- 262 #5 bars @ 0.304%
- $A_s = 115.28$ in$^2$
- Confinement tied
- Flow @ 5% 95074 in
Section Properties of Superstructure

Pertinent Information about Materials used, etc.:

\[ p_{\text{conv}} := \left(1.45 \times 10^{-4}\right) \quad \text{N/m}^2 \text{ to psi conversion factor.} \]
\[ \text{feet} := \frac{1}{0.3048} \quad \text{feet} = 3.281 \quad \text{meter to feet conversion.} \]
\[ \text{inch} := \text{feet} \cdot 12 \quad \text{inch} = 39.37 \quad \text{meter to inches conversion.} \]

\[ f_c := 24 \left(10^6\right) \cdot p_{\text{conv}} \quad f_c = 3480.000 \]
\[ f_y := 400 \left(10^6\right) \cdot p_{\text{conv}} \quad f_y = 5.8 \times 10^4 \quad \text{(Reinforcement)(psi)} \]
\[ F_{y1} := 345 \left(10^6\right) \cdot p_{\text{conv}} \quad F_{y1} = 5.003 \times 10^4 \quad \text{(Structural)} \]
\[ F_{y2} := 250 \left(10^6\right) \cdot p_{\text{conv}} \quad F_{y2} = 3.625 \times 10^4 \quad \text{(Structural)} \]
\[ E_c := \frac{57000}{\sqrt{f_c}} \quad E_c = 3.363 \times 10^6 \quad \text{Modulus of Elasticity of concrete.} \]
\[ E_s := 29000000 \quad E_s = 2.9 \times 10^7 \quad \text{Modulus of Elasticity of steel.} \]
\[ n := \frac{E_s}{E_c} \quad n = 8.624 \quad \text{Modular Ratio} \]
\[ \text{boltD} := 24 \left(10^{-3}\right) \cdot \text{inch} \quad \text{boltD} = 0.945 \quad \text{Bolt Diameter (in.)} \]
\[ \text{Skew} := 26 + \left(\frac{31}{60}\right) + \left(\frac{40}{3600}\right) \quad \text{Skew} = 26.528 \quad \text{Skew Angle (degrees)} \]
Calculations for Moment of Inertia for Superstructure

\[ L_{\text{eff}} = 41.50 \text{-inch} \]
\[ t_{\text{avg}} = 0.195 \text{-inch} \]
\[ t_w = 0.014 \text{-inch} \]
\[ b_{f1} = 0.300 \text{-inch} \]
\[ b_{f2} = 0.600 \text{-inch} \]
\[ \text{gspace} = 1.905 \text{-inch} \]
\[ L_{\text{eff}} = 1.634 \times 10^3 \text{ inch} \]
\[ t_{\text{avg}} = 7.677 \text{ inch} \]
\[ t_w = 0.551 \text{ inch} \]
\[ b_{f1} = 11.811 \text{ inch} \]
\[ b_{f2} = 23.622 \text{ inch} \]
\[ \text{gspace} = 75 \text{ inch} \]

Effective Flange Width of interior may be taken as least of (in inches):

\[ efw_1 = \frac{1}{4} \cdot L_{\text{eff}} \]
\[ efw_2 = \left(12 \cdot t_{\text{avg}}\right) + t_w \]
\[ efw_3 = \left(12 \cdot t_{\text{avg}}\right) + \left(0.5 \cdot b_{f1}\right) \]
\[ efw_3b = \left(12 \cdot t_{\text{avg}}\right) + \left(0.5 \cdot b_{f2}\right) \]
\[ efw_4 = \text{gspace} \]
\[ efw_1 = 408.465 \text{ inch} \]
\[ efw_2 = 92.677 \text{ inch} \]
\[ efw_3 = 98.031 \text{ inch} \]
\[ efw_3b = 103.937 \text{ inch} \]
\[ efw_4 = 75 \text{ inch} \]

This controls \( EFW_{\text{inside}} = \text{gspace} \)

\[ EFW_{\text{inside}} = 75 \text{ inch} \]

Effective Flange Width of exterior may be taken as .5 EFW plus least of (in inches):

\[ efw_5 = \frac{1}{8} \cdot L_{\text{eff}} \]
\[ efw_6 = \left(6 \cdot t_{\text{avg}}\right) + \left(0.5 \cdot t_w\right) \]
\[ efw_7 = \left(6 \cdot t_{\text{avg}}\right) + \left(0.25 \cdot b_{f1}\right) \]
\[ efw_8 = \left(6 \cdot t_{\text{avg}}\right) + \left(0.25 \cdot b_{f2}\right) \]
\[ ohw = 0.925 \text{-inch} \]
\[ efw_5 = 204.232 \text{ inch} \]
\[ efw_6 = 46.339 \text{ inch} \]
\[ efw_7 = 49.016 \text{ inch} \]
\[ efw_8 = 51.969 \text{ inch} \]
\[ ohw = 36.417 \text{ inch} \]

This controls \( EFW_{\text{outside}} = \left(0.5 \cdot EFW_{\text{inside}}\right) + \text{ohw} \)

\[ EFW_{\text{outside}} = 73.917 \text{ inch} \]
We can use all of the slab for moment of inertia.

\[ l_{yy1} := 36.2084 \quad l_{zz1} := 3676.5792 \]
\[ l_{yy2} := 61.4485 \quad l_{zz2} := 5343.1700 \]

**Torsional Properties:**

We assume only the deck will resist torsion.

\[ b_{\text{deck}} := 20.8 \cdot \text{feet} \quad b_{\text{deck}} = 68.241 \quad \text{ft} \]
\[ h_{\text{deck}} := .195 \cdot \text{feet} \quad h_{\text{deck}} = 0.64 \quad \text{ft} \]
\[ J := \left( b_{\text{deck}} \right) \left( \frac{h_{\text{deck}}}{3} \right)^3 \quad J = 5.956 \quad \text{ft}^4 \]

**Cross-Sectional Area of Superstructure:**

\[ b_{\text{slab}} := 20.8 \cdot \text{feet} \quad b_{\text{slab}} = 7.913 \quad \text{ft} \]
\[ h_{\text{slab}} := .195 \cdot \text{feet} \quad h_{\text{slab}} = 0.64 \quad \text{ft} \]
\[ \text{area}_{\text{slab}} := h_{\text{slab}} \cdot b_{\text{slab}} \quad \text{area}_{\text{slab}} = 5.062 \quad \text{ft}^2 \]
\[ \text{area}_{\text{girder1}} := 11 \cdot \left[ (.02 \cdot .3) + (.02 \cdot .6) + (1.372 \cdot .014) \right] \cdot \text{feet} \cdot \text{feet} \quad \text{area}_{\text{girder1}} = 4.406 \quad \text{ft}^2 \]
\[ \text{area}_{\text{girder2}} := 11 \cdot \left[ (.045 \cdot .6) + (.045 \cdot .6) + (1.372 \cdot .014) \right] \cdot \text{feet} \cdot \text{feet} \quad \text{area}_{\text{girder2}} = 8.668 \quad \text{ft}^2 \]
\[ \text{area}_{\text{super1}} := \left( \text{area}_{\text{slab}} + \text{area}_{\text{girder1}} \right) \quad \text{area}_{\text{super1}} = 9.468 \quad \text{ft}^2 \]
\[ \text{area}_{\text{super2}} := \left( \text{area}_{\text{slab}} + \text{area}_{\text{girder2}} \right) \quad \text{area}_{\text{super2}} = 13.73 \quad \text{ft}^2 \]
St. Clair County Bridge

AASHTO Standard Specifications

**Loads:**

**Dead Loads of the superstructure**

\[
\begin{align*}
\text{w}_{\text{barrier}} & := 2 \cdot \text{area}_{\text{barrier}} \cdot 150 \\
\text{w}_{\text{slab}} & := (19.05 \cdot \text{feet}) \cdot (1.195 \cdot \text{feet}) \cdot 150 \\
\text{w}_{\text{girder1}} & := 11 \left[ (1.372 \cdot \text{feet}) \cdot (0.014 \cdot \text{feet}) + (0.020 \cdot \text{feet}) \cdot (0.014 \cdot \text{feet}) \right] \cdot 490 \\
\text{w}_{\text{girder2}} & := 11 \left[ (1.372 \cdot \text{feet}) \cdot (0.045 \cdot \text{feet}) + (0.045 \cdot \text{feet}) \cdot (1.20 \cdot \text{feet}) \right] \cdot 490 \\
\text{w}_{\text{median}} & := \text{area}_{\text{median}} \cdot 190
\end{align*}
\]

\[
\begin{align*}
\text{area}_{\text{barrier}} & := [(2.535) + (0.065 \cdot 0.535) + (0.320 \cdot 0.330) + (0.5 \cdot 0.330)] \cdot \text{feet} \cdot \text{feet} \\
\text{area}_{\text{median}} & := 0.150 \left\lfloor \frac{1.3 + 1.236}{2} \right\rfloor \cdot \text{feet} \cdot \text{feet}
\end{align*}
\]

**area\_{barrier} = 2.982 ft^2**

**area\_{median} = 2.047 ft^2**

**Interior Cross-frames:**

- 2 - L4x4x5/16 @ 2546 mm
- 1 - L4x4x5/16 @ 1904 mm

weight(lb.) per linear foot = 8.16

10 spaces underneath bridge  
13 cross frames per length  
total number = 130

\[
\text{w}_{\text{cf}} := 13 \cdot 10 \cdot \left( (2.546 + 2.546 + 1.904) \cdot \text{feet} \right) \cdot \frac{8.16}{272} \cdot \text{w}_{\text{cf}} = 89.516 \text{ plf}
\]

Future Wearing Surface was 2.4 kN/m^2 or about 50 psf.

\[
\begin{align*}
\text{w}_{\text{bridge}} & := (3 + 4.2 + 0.3 + 3.9 + 4.2 + 3) \cdot \text{feet} \\
\text{w}_{\text{fws}} & := 2.4 \cdot 1000 \cdot \text{pconv} \cdot 144 \cdot \text{w}_{\text{bridge}}
\end{align*}
\]

\[
\begin{align*}
\text{w}_{\text{bridge}} & = 61.024 \text{ ft} \\
\text{w}_{\text{fws}} & = 3.058 \times 10^3 \text{ plf}
\end{align*}
\]
Dimensions used to model:
column1 ..  
  \( b_{\text{col1}} = 1.75 \text{ feet} \)
  \( b_{\text{col1}} = 5.741 \text{ ft} \)
column2 ..  
  \( b_{\text{col2}} = 1.625 \text{ feet} \)
  \( b_{\text{col2}} = 5.331 \text{ ft} \)
column3 ..  
  \( b_{\text{col3}} = 1.5 \text{ feet} \)
  \( b_{\text{col3}} = 4.921 \text{ ft} \)
column4 ..  
  \( b_{\text{col4}} = 1.375 \text{ feet} \)
  \( b_{\text{col4}} = 4.511 \text{ ft} \)
column5 ..  
  \( b_{\text{col5}} = 1.25 \text{ feet} \)
  \( b_{\text{col5}} = 4.101 \text{ ft} \)
column6 ..  
  \( b_{\text{col6}} = 1.125 \text{ feet} \)
  \( b_{\text{col6}} = 3.691 \text{ ft} \)

All columns are a width of 2.953 ft over their entire length. Columns designed as non-prismatic sections in SAP-2000.
Width is constant across the cap.

\[ h_{\text{capend}} = 3.228 \text{ ft} \]

Section taken at end of cap:

\[ h_{\text{capend}} := .984 \text{ feet} \]

Section taken in middle of cap:

\[ b_{\text{cap}} := 1 \text{ feet} \]

\[ b_{\text{cap}} = 3.281 \text{ ft} \]

\[ h_{\text{cap}} := \text{feet}(.984 + .300) \]

\[ h_{\text{cap}} = 4.213 \text{ ft} \]

Section taken from the top of the cap.

\[ y_{\text{bar}} = 2.072 \text{ ft} \]

Pier Cap Dimensions and properties:

\[ y_{\text{bar}} := \left[ \frac{23.36 \cdot .984}{2} + 2 \cdot (.5 \cdot 1.905 \cdot 3) \cdot (.984 + .1) + (19.55 \cdot 3) \cdot (.984 + \frac{3}{2}) \right] \cdot \text{feet} \]

\[ [(23.36 \cdot .984) + 2 \cdot (.5 \cdot 1.905 \cdot 3) + (19.55 \cdot 3)] \]

\[ y_{\text{bar}} = 2.072 \text{ ft} \]

Measured from the top of the cap.
Saint Clair County Bridge  -  AASHTO Division 1-A  -  Design Checks

Earthquake Data:

Seismic Performance Category (SPC) is B
Bedrock Acceleration Coefficient (A) is .1125g  \[ a := .1125 \]
Site Coefficient is 1.5  \[ sc := 1.5 \]

Since A is between 0.09 and 0.19, we are in Seismic Zone 2.
Thus, the Soil Profile Type is III.
Moment - Columns

R is equal to 3 in weak axis of central pier (centerline of deck + skew)
R is equal to 5 in strong axis of central pier (perpendicular to weak axis)

Moment at bottom of column is okay according to PCA Column
Moment at top of column is okay according to PCA Column

*** Note that top uses exception in Article 8.18.2.1 on the use of a minimum of steel that is less than the typical 1% for longitudinal reinforcement

Moment - Crashwall

R is equal to 2 in both strong and weak axis of pier

Moment at bottom of crashwall is okay according to PCA Column; Use 8.18.2.1 exception

Shear - Columns (8.16.6.2.2)

\[
V_c := 2\left(1 + \frac{593000}{2000\cdot44\cdot35}\right)\sqrt{3500\cdot44\cdot32} \quad V_c = 198672 \text{ lbs} \\
V_{c2} := 2\sqrt{3500\cdot44\cdot32} \quad V_{c2} = 1.666 \times 10^5 \text{ lbs}
\]

Use value of 168 kips since it takes into account the axial load that is present.

\[
v_{col1} := \sqrt{53^2 + 29^2} \quad v_{col1} = 60.415 \text{ kips} \\
v_{col2} := \sqrt{26^2 + 24.5^2} \quad v_{col2} = 35.725 \text{ kips}
\]

Applied shear is less than 2x V_c, thus column satisfies requirements for shear with only minimum reinforcement
Shear - Crashwall

\[ V_{c3} := 2\sqrt{3500 \cdot 769 - 36.4} \quad V_{c3} = 3.312 \times 10^6 \text{ lbs} \]

\[ .85 \cdot V_{c3} = 2.815 \times 10^6 \text{ lbs} \]

*Use value of 2815 kips*

\[ v_{wall1} := \sqrt{79.5^2 + 72.6^2} \quad v_{wall1} = 107.6 \text{ kips} \]

\[ v_{wall2} := \sqrt{39^2 + 61^2} \quad v_{wall2} = 72.4 \text{ kips} \]

*Applied shear is less than 2x V.c, thus wall satisfies requirements for shear with only minimum reinforcement*

**Slenderness Effects (8.16.5)**

\[ r := .3 \cdot 35.4 \quad r = 10.6 \text{ in} \quad k := 2 \]

\[ l_u := 147.6 \text{ in} \quad \text{slenderness} := k \cdot \frac{l_u}{r} \quad \text{slenderness} = 27.8 \]

\[ M_{1b} := -236 \quad k - \text{ft} \]

\[ M_{2b} := 471 \quad k - \text{ft} \]

\[ \beta_d := 1 \quad E_c := 1820 \sqrt{3500} \quad E_s := 29000 \quad C_m := .6 + .4 \cdot \left( \frac{M_{1b}}{M_{2b}} \right) \]

\[ l_g := \frac{1}{12} \cdot 44.3 \cdot 35.4^3 \quad l_g = 1.638 \times 10^5 \text{ in}^4 \quad C_m = 0.4 \]

\[ E_l := \frac{E_c \cdot l_g}{(1 + \beta_d)} \quad E_l = 3.527 \times 10^9 \]
The distance from centerline of bearing to backwall is 474 mm, so the overall seat width is around 950 mm; the seat is adequate.
Appendix E -- Detailed Computation for Seismic Analysis and Design of the Pulaski County Bridge using Proposed NCHRP Specification
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Site Class = Medium Stiff Clay

Site Class D  MCEER Guide Spec. 3.4.2.1

Type III per AASHTO Spec.

\[ s := 1.5 \quad \text{AASHTO Table 3.5.1} \]

\[ A := 0.22 \text{ g} \]

- Design Response Spectrum Development - MCE

\[ S_s := 3.16 \quad \text{0.2-second period spectral acceleration} \]

\[ S_1 := 0.919 \quad \text{1-second period spectral acceleration} \]

\[ F_a := 1 \quad \text{Site coefficient for short period} \quad \text{(MCEER Table 3.4.2.3-1)} \]

\[ F_V := 1.5 \quad \text{Site coefficient for long period} \quad \text{(MCEER Table 3.4.2.3-2)} \]

\[ S_{DS} := F_a S_s \]

\[ S_{DS} = 3.16 \quad \text{Design earthquake response spectral acceleration at short period} \]

\[ S_{D1} := F_V S_1 \]

\[ S_{D1} = 1.379 \quad \text{Design earthquake response spectral acceleration at long period} \]

\[ 0.4 S_{DS} = 1.264 \]

\[ T_S := \frac{S_{D1}}{S_{DS}} \quad \text{Period at the end of construction design spectral acceleration plateau} \]

\[ T_S = 0.436 \quad \text{Sec} \]

\[ T_0 := 0.2 \cdot T_S \quad \text{Period at the beginning of construction design spectral acceleration plateau} \]

\[ T_0 = 0.087 \quad \text{Sec} \]

\[ F_a \cdot S_s = 3.16 > 0.6 \]
Seismic Hazard Level IV (Guide Spec. Table 3.7-2)

SDAP D, SDR 6

- Design Response Spectrum Development - FE

\[ S_s = 0.1026 \] 0.2-second period spectral acceleration

\[ S_1 = 0.0200 \] 1-second period spectral acceleration

\[ F_a = 1.6 \] Site coefficient for short period (MCEER Table 3.4.2.3-1)

\[ F_v = 2.4 \] Site coefficient for long period (MCEER Table 3.4.2.3-2)

\[ S_{DS} = F_a S_s \]

\[ S_{DS} = 0.164 \] Design earthquake response spectral acceleration at short period

\[ S_{D1} = F_v S_1 \]

\[ S_{D1} = 0.048 \] Design earthquake response spectral acceleration at long period

\[ 0.4 S_{DS} = 0.066 \]

\[ T_s = \frac{S_{D1}}{S_{DS}} \] Period at the end of construction design spectral acceleration plateau

\[ T_s = 0.292 \text{ Sec} \]

\[ T_0 = 0.2 \cdot T_s \] Period at the beginning of construction design spectral acceleration plateau

\[ T_0 = 0.058 \text{ Sec} \]
Response Spectrum

Superstructure Section Properties

\[ L_{\text{bridge}} := 232 \]

\[ A_{\text{girder}} := 38.3 \text{ in}^2 \]

\[ d_{\text{girder}} := 33.1 \text{ in} \]

\[ I_x := 6710 \text{ in}^4 \]

\[ S_x := 406 \text{ in}^3 \]

\[ I_y := 218 \text{ in}^4 \]
Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

NCHRP Guidelines

BY: NA
CHECKED: AAA
DATE: 6/22/2004
Sheet No: 6

Title: Pulaski County Bridge

\[ S_y := 37.9 \, \text{in}^3 \]

5 girder at 6' 6''
Slab thickness = 6.5'
Hung = 1''

\[ N := 8 \]
\[ \text{hung} := 1 \, \text{in} \]
\[ b_{\text{slab}} := 30\cdot12 \]
\[ b_{\text{slab}} = 360 \, \text{in} \]

Effective width of the composite section (AASHTO LRFD 4.6.2.6.2)

\[ L_1 := 40.66 \cdot \frac{12}{4} \]
\[ L_1 = 121.98 \, \text{in} \]
\[ L_2 := 6.5\cdot12 \]
\[ L_2 = 78 \, \text{in} \]
\[ L_3 := 6.5\cdot12 + \max(0.58, 0.855) \]
\[ L_3 = 78.58 \, \text{in} \]
\[ b_{\text{effective}} := \min(L_1, L_2, L_3) \]
\[ b_{\text{effective}} = 78 \, \text{in} \]
\[ b_{\text{slab}} := 6.5 \, \text{in} \]
\[ A_{\text{slab}} := b_{\text{slab}} \cdot \frac{h_{\text{slab}}}{N} \]
\[ A_{\text{slab}} = 292.5 \, \text{in}^2 \]
\[ I_{x_{\text{slab}}} := b_{\text{slab}} \frac{h_{\text{slab}}^3}{12 \cdot N} \]

\[ I_{x_{\text{slab}}} = 1.03 \times 10^3 \text{ in}^4 \]

\[ Y_{\text{slab}} := \frac{h_{\text{slab}}}{2} \]

\[ Y_{\text{slab}} = 3.25 \text{ in} \]

\[ Y_{x_{\text{comp}}} := \frac{A_{\text{slab}} Y_{\text{slab}} + 5A_{\text{girder}} \left( \frac{d_{\text{girder}}}{2} + 2Y_{\text{slab}} + \text{hung} \right)}{\left( 5A_{\text{girder}} + A_{\text{slab}} \right)} \]

\[ Y_{x_{\text{comp}}} = 11.48 \text{ in} \] \quad \text{From top} \]

\[ I_{x_{\text{comp}}} := 5I_{x} + I_{x_{\text{slab}}} + 5A_{\text{girder}} \left( \frac{d_{\text{girder}}}{2} + 2Y_{\text{slab}} + \text{hung} - Y_{x_{\text{comp}}} \right)^2 + A_{\text{slab}} \left( Y_{\text{slab}} - Y_{x_{\text{comp}}} \right)^2 \]

\[ I_{x_{\text{comp}}} = 8.465 \times 10^4 \text{ in}^4 \]

\[ S_{x_{\text{comp}}} := \frac{I_{x_{\text{comp}}}}{Y_{x_{\text{comp}}}} \]

\[ S_{x_{\text{comp}}} = 7.374 \times 10^3 \text{ in}^4 \]

\[ A_{\text{comp}} := A_{\text{girder}} \cdot 5 + A_{\text{slab}} \]

\[ A_{\text{comp}} = 484 \text{ in}^2 \]

\[ r_{x} := \sqrt{\frac{I_{x_{\text{comp}}}}{A_{\text{comp}}}} \]

\[ r_{x} = 13.225 \text{ in} \]

\[ I_{y_{\text{slab}}} := h_{\text{slab}} \frac{b_{\text{slab}}^3}{12 \cdot N} \]
Title: Pulaski County Bridge

\[ I_{y_{\text{slab}}} = 3.159 \times 10^6 \text{ in}^4 \]

\[ I_{y_{\text{comp}}} := 5 \cdot I_y + 2 \cdot A_{\text{girder}} \left( 6.5^2 + 13^2 \right) + I_{y_{\text{slab}}} \]

\[ I_{y_{\text{comp}}} = 3.176 \times 10^6 \text{ in}^4 \]

\[ S_{y_{\text{comp}}} := \frac{I_{y_{\text{comp}}}}{42.5 \cdot 12} \]

\[ S_{y_{\text{comp}}} = 1.26 \times 10^4 \text{ in}^3 \]

\[ r_y := \frac{I_{y_{\text{comp}}}}{A_{\text{comp}}} \]

\[ r_y = 81.01 \text{ in} \]

\[ A_{\text{shear}} := 0.8 \cdot A_{\text{comp}} \]

\[ A_{\text{shear}} = 387.2 \text{ in}^2 \]

\[ J := b_{\text{slab}} \frac{h_{\text{slab}}^3}{12 \cdot N} + b_{\text{slab}} \frac{b_{\text{slab}}^3}{12 \cdot N} \]

\[ J = 3.16 \times 10^6 \]

\[ S_{x_{\text{plastic}}} := S_{x_{\text{comp}}}^{1.5} \]

\[ S_{x_{\text{plastic}}} = 1.106 \times 10^4 \text{ in}^3 \]

\[ S_{y_{\text{plastic}}} := S_{y_{\text{comp}}}^{1.5} \]

\[ S_{y_{\text{plastic}}} = 1.891 \times 10^4 \text{ in}^3 \]
Mass Calculation

\[ W_{slab} = 30 \cdot \frac{6.5 \cdot \text{L}_{bridge}}{12} \cdot .15 \]

\[ W_{slab} = 565.5 \text{ kips} \]

\[ W_{girders} = \frac{130}{1000} \cdot 5 \cdot \text{L}_{bridge} \cdot 1.15 \]

\[ W_{girders} = 173.42 \text{ kips} \]

\[ \text{Bar} = 5.2 \cdot \text{L}_{bridge} \cdot .15 \]

\[ \text{Bar} = 180.96 \text{ kips} \]

\[ \text{FWS} = \frac{35}{1000} \cdot 24 \cdot \text{L}_{bridge} \]

\[ \text{FWS} = 194.88 \text{ kips} \]

\[ W_{total} = W_{slab} + W_{girders} + \text{Bar} + \text{FWS} \]

\[ W_{total} = 1.115 \times 10^3 \text{ kips} \]

\[ w_{total} = \frac{W_{total}}{\text{L}_{bridge}} \]

\[ w_{total} = 4.805 \text{ klf} \]
Finite Element Model with fix base
Period = 0.874 Sec
Title: Pulaski County Bridge

Subject File: Evaluation of Comprehensive Siesmic Design of Bridges (LRFD) in Illinois

NCHRP Guidelines

Modal Participating Mass Ratios

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<tr>
<th>MODE</th>
<th>PERIOD</th>
<th>INDIVIDUAL MODE (%)</th>
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</thead>
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<tr>
<td></td>
<td></td>
<td>UX</td>
</tr>
<tr>
<td>1</td>
<td>0.8738</td>
<td>65.68</td>
</tr>
<tr>
<td>2</td>
<td>0.2681</td>
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<tr>
<td>3</td>
<td>0.2116</td>
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<tr>
<td>4</td>
<td>0.1859</td>
<td>0.00</td>
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<tr>
<td>5</td>
<td>0.1594</td>
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<tr>
<td>6</td>
<td>0.1396</td>
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<tr>
<td>7</td>
<td>0.1219</td>
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<td>8</td>
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<td>11</td>
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<tr>
<td>13</td>
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<td>50</td>
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<td>0.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>78.09</td>
</tr>
</tbody>
</table>

Center of gravity on top of the hammer head:

\[
Y := \frac{(2.25 \cdot 5 + 2.25 \cdot 30 \cdot 2.25 + 2.25 \cdot 5 \cdot 12 \cdot 2.25 + 2 \cdot (2.25 \cdot 0.66 \cdot 0.5 \cdot 9 \cdot 2.25))}{(30 \cdot 2.25 + 9 \cdot 2.25 + 12 \cdot 2.25)}
\]

\[
Y = 2.512 \text{ ft from the bottom}
\]
Title: Pulaski County Bridge

SUBJECT FILE:
Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois
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First main mode shape
T = 0.0226 Sec

First main mode shape
T = 0.0209 Sec

Second main mode shape
T = 0.0060 Sec

Second main mode shape
T = 0.0059 Sec

E-12
Title: Pulaski County Bridge

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Third main mode shape
T=0.0034 Sec

Third main mode shape
T=0.0035 Sec

Fourth main mode shape
T=0.0024 Sec

Fourth main mode shape
T=0.0029 Sec
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Evaluation of Comprehensive Siesmic Design of Bridges (LRFD) in Illinois

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Title: Pulaski County Bridge

Fifth main mode shape
T=0.0022 Sec

Sixth main mode shape
T=0.0019 Sec

Fifth main mode shape
T=0.0022 Sec

Sixth main mode shape
T=0.0020 Sec
Title: Pulaski County Bridge

Subject File:
Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

NCHRP Guidelines

BY: NA
CHECKED: AAA
DATE: 6/22/2004

Sheet No: 15

Stiff element from the top of the bearing to the center of gravity of the superstructure
Moment released in the longitudinal direction

PIER 1&3  PIER 2
Title: Pulaski County Bridge

2D Model of Intermediate Bent With Masses at Real Location of Girders

2D Model of Intermediate Bent With Single Vertical Column
Title: Pulaski County Bridge

SUBJECT FILE:

Evaluation of Comprehensive Siesmic Design of Bridges (LRFD) in Illinois

NCHRP Guidelines

---

Without Rotational Mass

\[ M = 2120 \text{ k-ft} \]

With Rotational Mass

\[ M = 2780 \text{ k-ft} \]

With Masses at Location of Girders

\[ M = 2804 \text{ k-ft} \]

Without Rotational Mass

\[ M = 2120 \text{ k-ft} \]
Group Pile Stiffness At Fixed Bent:

Use HP 14X117 for the NCHRP design

\[ A = 34.4 \text{ in}^2 \]
\[ l_{xx} = 1220 \text{ in}^4 \]
\[ l_{yy} = 443 \text{ in}^4 \]
\[ \text{Depth} = 14.2 \text{ in} \]
\[ \text{Width} = 14.9 \text{ in} \]

\[ E := \frac{29000000 \cdot 144}{1000} \text{ ksf} \]

\[ N := 40 \]

**4X10 (70" in long and 70" spacing in transverse direction)**

Results from L-pile:

\[ \Delta_{\text{long}} = 0.56 \text{ in} \]
\[ \Delta_{\text{trans}} = 0.71 \text{ in} \]

Results from SAP model:

\[ V_{\text{long single}} = 96 \text{ Kips} \]
\[ V_{\text{trans single}} = 74 \text{ Kips} \]

\[ K_{\text{long single}} = \frac{V_{\text{long single}}}{\Delta_{\text{long}}} \]

\[ K_{\text{long single}} := 96 \cdot \frac{12}{0.56} \]

\[ K_{\text{long single}} = 2.057 \times 10^3 \frac{k}{\text{ft}} \]
$K_x := k_{\text{long\_single}} N$

$K_x = 8.229 \times 10^4 \frac{k}{\text{ft}}$

$L := 60 \text{ ft}$

$K_{\text{tran\_single}} = \frac{V_{\text{trans\_single}}}{\Delta_{\text{trans}}}$

$K_{\text{tran\_single}} := 74 \frac{12}{.71}$

$K_{\text{tran\_single}} = 1.251 \times 10^3$

$K_y := k_{\text{tran\_single}} N$

$K_y = 5.003 \times 10^4 \frac{k}{\text{ft}}$

$A := \frac{34.4}{144}$

$A = 0.239 \text{ ft}^2$

$K_{\text{axial}} := A \frac{E}{L}$

$K_{\text{axial}} = 1.663 \times 10^4 \frac{k}{\text{ft}}$

$K_z := k_{\text{axial}} N$

$K_z = 6.651 \times 10^5 \frac{k}{\text{ft}}$

$S_1 := \frac{8}{144} \left[ (0.5 \cdot 0.7)^2 + (1.5 \cdot 0.7)^2 + (2.5 \cdot 0.7)^2 + (3.5 \cdot 0.7)^2 + (4.5 \cdot 0.7)^2 \right]$

$S_2 := \frac{2 \cdot N}{144 \cdot 4} \left[ (0.5 \cdot 0.7)^2 + (1.5 \cdot 0.7)^2 \right]$
Title: Pulaski County Bridge

\[ K_{tx} := K_{axial} \cdot S_1 \]
\[ K_{ty} := K_{axial} \cdot S_2 \]
\[ K_{rz} := (K_{long\_single} \cdot S_1 + K_{tran\_single} \cdot S_2) \]
\[ K_x = 8.229 \times 10^4 \frac{k}{ft} \]
\[ K_y = 5.003 \times 10^4 \frac{k}{ft} \]
\[ K_z = 6.651 \times 10^5 \frac{k}{ft} \]
\[ K_{tx} = 1.867 \times 10^8 \frac{ft-Kips}{rad} \]
\[ K_{ty} = 2.829 \times 10^7 \frac{ft-Kips}{rad} \]
\[ K_{rz} = 2.523 \times 10^7 \frac{ft-Kips}{rad} \]

Total DL = 1109 kips

Axial capacity (IDOT Bridge Manual 3.8.1) = 9000 psi

\[ P_{cap} := A \cdot 1.5 \cdot 9.144 \]
\[ P_{cap} = 464.4 \text{ kips} \]
\[ N = 40 \text{ No of piles used} \]
\[ V := 1109 \text{ kips} \]
**Results from SAP:**

\[ M_{\text{long}} := 46161 \text{ k \text{-} ft} \]
\[ M_{\text{tran}} := 15526 \text{ k \text{-} ft} \]

\[ P_1 := \frac{V}{N} + \frac{M_{\text{long}}}{12 \cdot S_2} + \frac{M_{\text{tran}}}{12 \cdot S_1} \cdot 1.5 - \frac{75}{75} \cdot \frac{1.5-75}{12} \]

\[ P_1 = 320.969 \text{ kips} \]

\[ P_1 < P_{\text{cap}} \text{ Ok} \]

**Results from L-pile:**

\[ \text{Dis}_L := .39 \text{ in} \]
\[ M_1 := 4080 \text{ k \text{-} in} \]

\[ \text{Dis}_T := .23 \text{ in} \]
\[ M_2 := 1800 \text{ k \text{-} in} \]

**Compressive Resistance (Combined axial compression and flexure)**

\[ \frac{P_1}{0.85 - 50 - 34.4} + \frac{8}{9} \left( \frac{M_1}{50 - 194} + \frac{M_2}{50 - 91.4} \right) = 0.944 < 1 \text{ OK} \]

\[ f_y = 50 \text{ ksi} \text{ for steel pile} \]

**EQ in transverse direction**

**Results from SAP:**

\[ M_{\text{long}} := 18646 \text{ k \text{-} ft} \]
\[ M_{\text{tran}} := 38811 \text{ k \text{-} ft} \]

\[ P_2 := \frac{V}{N} + \frac{M_{\text{long}}}{12S_2} + \frac{M_{\text{tran}}}{12S_1} \cdot 1.5 \cdot \frac{1.5 - 75}{12} \cdot \frac{4.5 - 75}{12} \]

\[ P_2 = 227.676 \text{ kips} < P_{\text{cap}} \text{ OK} \]

**Results from L-pile:**

\[ \text{Dis}_L := .15 \text{ in} \]
\[ M_3 := 2230 \text{ k \text{-} in} \]

\[ \text{Dis}_T := .57 \text{ in} \]
\[ M_4 := 3055 \text{ k \text{-} in} \]
Spring Stiffness at Expansion Bent

\[ N := 24 \]

\[ K_{\text{long\_single}} = 2.057 \times 10^3 \quad \frac{k}{\text{ft}} \]

\[ K_x := K_{\text{long\_single}}^N \]

\[ K_x = 4.937 \times 10^4 \quad \frac{k}{\text{ft}} \]

\[ L := 60 \quad \text{ft} \]

\[ K_{\text{tran\_single}} = 1.251 \times 10^3 \]

\[ K_y := K_{\text{tran\_single}}^N \]

\[ K_y = 3.002 \times 10^4 \quad \frac{k}{\text{ft}} \]

\[ A := \frac{34.4}{144} \]

\[ A = 0.239 \quad \text{ft}^2 \]

\[ K_{\text{axial}} := A \cdot \frac{E}{L} \]

\[ K_{\text{axial}} = 1.663 \times 10^4 \quad \frac{k}{\text{ft}} \]

\[ K_z := K_{\text{axial}}^N \]
Axial capacity (IDOT Bridge Manual 3.8.1) = 9000 psi
P\text{cap} := A \cdot 1.5 \cdot 9 \cdot 144 \\
P_{\text{cap}} = 464.4 \text{ kips} \\
N = 24 \\
V := 820 \text{ kips}

Results from SAP:
\[ M_{\text{long}} := 8243 \text{ k – ft} \]
\[ M_{\text{tran}} := 15508 \text{ k – ft} \]
\[ P_{\text{1}} := \frac{V}{N} + M_{\text{long}} \cdot \frac{1.75}{12 \cdot S_{2}} + M_{\text{tran}} \cdot \frac{3.5.75}{12 \cdot S_{1}} \]
\[ P_{\text{1}} = 207.915 \text{ kips} \]
\[ P_{\text{1}} < P_{\text{cap}} \quad \text{Ok} \]

Results from L-pile:
\[ \text{Dis}_{L} := .28 \text{ in} \]
\[ \text{Dis}_{T} := .39 \text{ in} \]

Compressive Resistance (Combined axial compression and flexure)
\[ \frac{P_{\text{1}}}{0.85 \cdot 50 \cdot 34.4} + \frac{8}{9} \left( \frac{M_{1}}{50 \cdot 194} + \frac{M_{2}}{50 \cdot 91.4} \right) = 0.935 < 1 \quad \text{Ok} \quad (AASHTO \ LRFD \ 6.9.2.2) \]

Earthquake in transverse direction:
Results from SAP Model:
\[ M_{\text{long}} := 3298 \text{ k – ft} \]
\[ M_{\text{tran}} := 38766 \text{ k – ft} \]
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\[ P_2 := \frac{V}{N} + M_{\text{long}} \frac{1.75}{12S_2} + M_{\text{tran}} \frac{3.5\cdot75}{12S_1} \]

\[ P_2 = 269.812 \text{ kips} < P_{\text{cap}} \text{ OK} \]

Results from L-pile:
\[ \text{Dis}_L := 0.11 \text{ in} \quad M_3 := 1700 \text{ k – in} \]
\[ \text{Dis}_F := 0.75 \text{ in} \quad M_4 := 3380 \text{ k – in} \]

Compressive Resistance (Combined axial compression and flexure)
\[ \frac{P_2}{0.85 \cdot 50 \cdot 34.4} + \frac{8}{9} \left( \frac{M_3}{50 \cdot 194} + \frac{M_4}{50 \cdot 91.4} \right) = 0.998 \quad 1 \text{ Ok} \]
MCEER Guide Spec. Table 4.7-1

<table>
<thead>
<tr>
<th>Base Response Modification Factor (Rb) SDAP D Operational Performance</th>
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<td>Wall pier</td>
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<td>Vertical pile</td>
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<td>All elements for FE</td>
</tr>
<tr>
<td>Superstructure to abutment</td>
</tr>
<tr>
<td>Column and pile to cap beam</td>
</tr>
</tbody>
</table>

\[ T_s := 0.436 \quad \text{Sec} \]
\[ T := 0.874 \quad \text{Sec} \]
\[ R_{col} := 1 + (1.5 - 1) \left( \frac{T}{1.25 \cdot T_s} \right) \quad \text{MCEER Guide Spec. 4.7} \]
\[ R_{col} = 1.802 \quad < 1.5 \]
\[ R_{wall} := 1 + (1 - 1) \left( \frac{T}{1.25 \cdot T_s} \right) \]
\[ R_{wall} = 1 \]
\[ T_s := 0.292 \quad \text{Sec} \]
\[ T := 0.874 \quad \text{Sec} \]
\[ R_{FE} := 1 + (0.9 - 1) \left( \frac{T}{1.25 \cdot T_s} \right) \]
\[ R_{FE} = 0.761 \quad > 0.9 \]
\[ R_{FE} := 0.9 \]

<table>
<thead>
<tr>
<th>Response Modification Factor (R) SDAP D Operational Performance</th>
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### Pulaski County Bridge

**Evaluation of Comprehensive Siesmic Design of Bridges (LRFD) in Illinois**

**NCHRP Guidelines**

<table>
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<tr>
<th>MCE</th>
<th>Forces &amp; Moments - EQ&lt;sub&gt;trans&lt;/sub&gt;</th>
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**Sheet No:** 27
Title: Pulaski County Bridge

SUBJECT FILE:

Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

NCHRP Guidelines

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<td>Moment 2 kip-ft</td>
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<td>33204</td>
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</table>
Interaction diagram for the fixed bent. As the thickness of the wall was inadequate for the maximum moment, the thickness of the wall has been increased to 4’
Maximum moment at the bottom of pier based on 2.5' thick wall.

The finite element model was run for the 4' thick pier. In this analysis moment has increased by 100% by using 48" thick wall and the pier can not be design for this moment. So all of three pier will be considered fixed in longitudinal direction of the bridge.
Group Pile Stiffness (All Bents Fixed)

\[ E := \frac{29000000 \times 144}{1000} \]

\[ E = 4.176 \times 10^6 \text{ ksf} \]

\[ N := 40 \]

\[ 4 \times 10 \text{ (70" in long and 70" spacing in transverse direction)} \]

**Results from SAP model:**

\[ V_{\text{long_single}} = 74 \text{ Kips} \]

\[ V_{\text{trans_single}} = 75 \text{ Kips} \]

**Results from L-pile:**

\[ \Delta_{\text{long}} = 0.24 \text{ in} \]

\[ \Delta_{\text{trans}} = 0.66 \text{ in} \]

\[ K_{\text{long_single}} = \frac{V_{\text{long_single}}}{\Delta_{\text{long}}} \]

\[ K_{\text{long_single}} := 74 \times \frac{12}{24} \]

\[ K_{\text{long_single}} = 3.7 \times 10^3 \frac{\text{kips}}{\text{ft}} \]

\[ K_x := K_{\text{long_single}}^N \]

\[ K_X = 1.48 \times 10^5 \frac{\text{kips}}{\text{ft}} \]

\[ L := 60 \text{ ft} \]

\[ K_{\text{tran_single}} = \frac{V_{\text{trans_single}}}{\Delta_{\text{trans}}} \]
\[ K_{\text{tran\_single}} := 75 \cdot \frac{12}{.66} \]

\[ K_{\text{tran\_single}} = 1.364 \times 10^3 \]

\[ K_y := K_{\text{tran\_single}} \cdot N \]

\[ K_y = 5.455 \times 10^4 \]

\[ A := \frac{34.4}{144} \]

\[ A = 0.239 \text{ ft}^2 \]

\[ K_{\text{axial}} := A \cdot \frac{E}{L} \]

\[ K_{\text{axial}} = 1.663 \times 10^4 \]

\[ K_z := K_{\text{axial}} \cdot N \]

\[ K_z = 6.651 \times 10^5 \cdot \frac{k}{\text{ft}} \]

\[ S_1 := \frac{6}{144} \left[ (.5 \cdot 70)^2 + (1.5 \cdot 70)^2 + (2.5 \cdot 70)^2 + (3.5 \cdot 70)^2 + (4.5 \cdot 70)^2 \right] \]

\[ S_2 := \frac{2}{144} \cdot \frac{N}{4} \left[ (.5 \cdot 70)^2 + (1.5 \cdot 70)^2 \right] \]

\[ K_{\text{tx}} := K_{\text{axial}} \cdot S_1 \]

\[ K_{\text{ty}} := K_{\text{axial}} \cdot S_2 \]

\[ K_{\text{tz}} := (K_{\text{long\_single}} \cdot S_1 + K_{\text{tran\_single}} \cdot S_2) \]
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\[ K_x = 1.48 \times 10^5 \text{ k} / \text{ft} \]
\[ K_y = 5.455 \times 10^4 \text{ k} / \text{ft} \]
\[ K_z = 6.651 \times 10^5 \text{ k} / \text{ft} \]
\[ K_{tx} = 1.4 \times 10^8 \text{ k} / \text{ft} \]
\[ K_{ty} = 2.829 \times 10^7 \text{ k} / \text{ft} \]
\[ K_{tz} = 3.348 \times 10^7 \text{ k} / \text{ft} \]

Total DL = 1065 kips

Axial capacity (IDOT Bridge Manual 3.8.1) = 9000 psi

\[ P_{cap} := A \cdot 1.5 \cdot 9 \cdot 144 \]
A is area of cross-section of single pile

\[ P_{cap} = 464.4 \text{ kips} \]
Ultimate axial capacity of pile

\[ N = 40 \]
No of piles used

\[ V := 1065 \text{ kips} \]

Results from SAP:

\[ M_{long} := 31836 \text{ k} \cdot \text{ft} \]
\[ M_{tran} := 17848 \text{ k} \cdot \text{ft} \]

\[ P_1 := \frac{V}{N} + M_{long} \cdot 1.5 \cdot 70 \cdot 12 \cdot S_2 + M_{tran} \cdot 4.5 \cdot 70 \cdot 12 \cdot S_1 \]
\[ P_1 = 245.983 \text{ kips} \]

\[ P_1 < P_{cap} \text{ Ok} \]
Results from L-pile:

\[ \text{Dis}_L := 0.24 \text{ in} \quad M_1 := 3055 \text{ k - in} \]

\[ \text{Dis}_T := 0.20 \text{ in} \quad M_2 := 1660 \text{ k - in} \]

**Compressive Resistance (Combined axial compression and flexure)**

\[
\frac{P_1}{0.85 \cdot 50 \cdot 34.4} + 8 \left( \frac{M_1}{50 \cdot 194} + \frac{M_2}{50 \cdot 91.4} \right) = 0.771 < 1 \quad \text{OK} \quad \text{(AASHTO LRFD6.9.2.2 - 2)}
\]

**EQ in transverse direction**

Results from SAP:

\[ M_{\text{long}} := 12746 \quad \text{k - ft} \]

\[ M_{\text{tran}} := 44615 \quad \text{k - ft} \]

\[ P_2 := \frac{V}{N} + M_{\text{long}} \frac{1.5 \cdot 75}{12S_2} + M_{\text{tran}} \frac{4.5 \cdot 75}{12S_1} \]

\[ P_2 = 245.851 \quad \text{kips} < P_{\text{cap}} \quad \text{OK} \]

Results from L-pile:

\[ \text{Dis}_L := 0.097 \text{ in} \quad M_3 := 1640 \text{ k - in} \]

\[ \text{Dis}_T := 0.51 \text{ in} \quad M_4 := 2890 \text{ k - in} \]

**Compressive Resistance (Combined axial compression and flexure)**

\[
\frac{P_2}{0.85 \cdot 50 \cdot 34.4} + 8 \left( \frac{M_3}{50 \cdot 194} + \frac{M_4}{50 \cdot 91.4} \right) = 0.88 < 1 \quad \text{OK}
\]
MCEER Guide Spec. Table 4.7-1

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<td>Superstructure to abutment</td>
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<tr>
<td>Column and pile to cap beam</td>
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</table>

\[ T_s := 0.436 \quad \text{Sec} \]
\[ T := 0.8738 \quad \text{Sec} \]

\[ R_{col} := 1 + (1.5 - 1) \left( \frac{T}{1.25 \cdot T_s} \right) \]

MCEER Guide Spec. 4.7

\[ R_{col} = 1.802 < 1.5 \]

\[ R_{wall} := 1 + (1 - 1) \left( \frac{T}{1.25 \cdot T_s} \right) \]

\[ R_{wall} = 1 \]

\[ R_{FE} := 1 + (0.9 - 1) \left( \frac{T}{1.25 \cdot T_s} \right) \]

\[ R_{FE} = 0.84 > 0.9 \]

\[ R_{FE} := 0.9 \]

Response Modification Factor (R) SDAP D Operational Performance

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<td>Column and pile to cap beam</td>
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### Pulaski County Bridge

#### Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

**NCHRP Guidelines**

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### Forces & Moments - EQ<sub>trans</sub>

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<th>Moment 2</th>
<th>Moment 3</th>
<th>Axial</th>
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<td>Pier 2 Bottom</td>
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### Forces & Moments - EQ<sub>long</sub>

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### Dead Load

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<td>Pier 1&amp;3</td>
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### 100% + 40% Forces & Moments - EQ

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### Factored Forces & Moments - EQ by R

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### Final Design Forces & Moments (EQ + DL)

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<td>1199</td>
<td>0</td>
<td>28687</td>
<td>41430</td>
<td>570</td>
<td>480</td>
<td>895</td>
<td>11475</td>
<td>16572</td>
</tr>
<tr>
<td>Pier 1&amp;3 Bottom</td>
<td>469</td>
<td>1651</td>
<td>0</td>
<td>16116</td>
<td>39190</td>
<td>519</td>
<td>660</td>
<td>1172</td>
<td>6446</td>
<td>15676</td>
</tr>
</tbody>
</table>

---

Sheet No: 36
## Pulaski County Bridge

### Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

**NCHRP Guidelines**

<table>
<thead>
<tr>
<th>FE</th>
<th>Forces &amp; Moments - EQ(\text{trans})</th>
<th>(\text{Transverse})</th>
<th>(\text{Longitudinal})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Support / Location</td>
<td>Shear 3 Kips</td>
<td>Moment 2 Kip-ft</td>
</tr>
<tr>
<td>Pier 2</td>
<td>Top</td>
<td>30</td>
<td>393</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>703</td>
<td>0</td>
</tr>
<tr>
<td>Pier 1 &amp; 3</td>
<td>Top</td>
<td>17</td>
<td>245</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>359</td>
<td>0</td>
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</table>

<table>
<thead>
<tr>
<th>FE</th>
<th>Forces &amp; Moments - EQ(\text{long})</th>
<th>(\text{Longitudinal})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Support / Location</td>
<td>Shear 2 Kips</td>
</tr>
<tr>
<td>Pier 2</td>
<td>Top</td>
<td>36</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>0</td>
</tr>
<tr>
<td>Pier 1 &amp; 3</td>
<td>Top</td>
<td>51</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>0</td>
</tr>
</tbody>
</table>

### 100% + 40% Forces & Moments - EQ

<table>
<thead>
<tr>
<th>FE</th>
<th>100% + 40% Forces &amp; Moments - EQ(\text{trans})</th>
<th>(\text{Transverse})</th>
<th>(\text{Longitudinal})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Support / Location</td>
<td>Shear 2 kips</td>
<td>Shear 3 kips</td>
</tr>
<tr>
<td>Pier 2</td>
<td>Top</td>
<td>14</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>703</td>
<td>0</td>
</tr>
<tr>
<td>Pier 1 &amp; 3</td>
<td>Top</td>
<td>21</td>
<td>17</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>359</td>
<td>377</td>
</tr>
</tbody>
</table>

### Factored Forces & Moments - EQ/R

<table>
<thead>
<tr>
<th>FE</th>
<th>Factored Forces &amp; Moments - EQ/R(\text{trans})</th>
<th>(\text{Transverse})</th>
<th>(\text{Longitudinal})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Support / Location</td>
<td>Shear 2 kips</td>
<td>Shear 3 kips</td>
</tr>
<tr>
<td>Pier 2</td>
<td>Top</td>
<td>16</td>
<td>34</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>781</td>
<td>335</td>
</tr>
<tr>
<td>Pier 1 &amp; 3</td>
<td>Top</td>
<td>24</td>
<td>19</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>399</td>
<td>419</td>
</tr>
</tbody>
</table>

### Final Design Forces & Moments - (EQ/R+DL)

<table>
<thead>
<tr>
<th>FE</th>
<th>Final Design Forces &amp; Moments - (EQ/R+DL)(\text{trans})</th>
<th>(\text{Transverse})</th>
<th>(\text{Longitudinal})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Support / Location</td>
<td>Shear 2 kips</td>
<td>Shear 3 kips</td>
</tr>
<tr>
<td>Pier 2</td>
<td>Top</td>
<td>16</td>
<td>34</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>781</td>
<td>335</td>
</tr>
<tr>
<td>Pier 1 &amp; 3</td>
<td>Top</td>
<td>24</td>
<td>19</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>399</td>
<td>419</td>
</tr>
</tbody>
</table>
Therefore, MCE design forces govern.
Forces at 5.5’ from the bottom of the pier. Reinforcement due to this design will be used for the upper part of the piers.

### Dead Load

<table>
<thead>
<tr>
<th>Location</th>
<th>Axial kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier 2</td>
<td>519</td>
</tr>
<tr>
<td>Pier 1&amp;3</td>
<td>421</td>
</tr>
</tbody>
</table>

### 100% + 40% Forces & Moments - EQ

<table>
<thead>
<tr>
<th>Support / Location</th>
<th>Transverse</th>
<th>Longitudinal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier 2 Top Bottom</td>
<td>Shear 3 kips</td>
<td>0</td>
</tr>
<tr>
<td>Pier 1&amp;3 Top Bottom</td>
<td>Shear 3 kips</td>
<td>0</td>
</tr>
</tbody>
</table>

### Factored Forces & Moments - EQ by R

<table>
<thead>
<tr>
<th>Support / Location</th>
<th>Transverse</th>
<th>Longitudinal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier 2 Top Bottom</td>
<td>Shear 3 kips</td>
<td>0</td>
</tr>
<tr>
<td>Pier 1&amp;3 Top Bottom</td>
<td>Shear 3 kips</td>
<td>0</td>
</tr>
</tbody>
</table>

### Final Design Forces & Moments (EQ + DL)

<table>
<thead>
<tr>
<th>Support / Location</th>
<th>Transverse</th>
<th>Longitudinal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier 2 Top Bottom</td>
<td>Shear 3 kips</td>
<td>0</td>
</tr>
<tr>
<td>Pier 1&amp;3 Top Bottom</td>
<td>Shear 3 kips</td>
<td>0</td>
</tr>
</tbody>
</table>
Title: Pulaski County Bridge

SUBJECT FILE:
Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois
NCHRP Guidelines

BY: NA
CHECKED: AAA
DATE: 6/22/2004
Sheet No: 40

MATERIAL:
Pc = 3.5 ksi
Ee = 3372.17 kpsi
Ec = 2.975 ksi
Bt = 0.01
h = 60 kpsi
Es = 25900 kpsi

SECTION:
Ag = 8524.8 in²
b = 1.623666 + 0.006 in⁴
h = 2.2407 × 10⁻⁷ in²
Yo = 0 in
Ye = 0 in

REINFORCEMENT:
3/8 #11 bars @ 2.490% Jr = 213.16 in²
Confinement: Other
Transverse Reinforcement Design
For the Pier 2

Plastic hinge zone length (this length will modify after the reinforcement design)
MCEER Guide Spec. 4.9.1

\[ h := 48 \text{ in} \quad \text{thickness of the wall} \]

1-

\[ D := h - 3 \text{ in} \]

\[ D := 48 \text{ in} \]

2-

\[ H := 256.25 \text{ in} \]

\[ \frac{1}{6}H = 42.708 \text{ in} \]

3-

18 \text{ in} \]

4-

\[ \varepsilon_y := 0.00207 \]

\[ d_b := 1.41 \text{ in} \quad \text{for #11} \]

\[ M := 24711 \text{ k-ft} \]

\[ V := 1199 \text{ k} \]

\[ 1.5 \left( 0.08 \cdot M \cdot \frac{12}{V} + 4400 \cdot \varepsilon_y \cdot d_b \right) = 48.941 \text{ in} \quad \text{Controls but last check should be done after design} \]
Wall pier transverse reinforcement
Method 2: Explicit Approach
(Guide Spec 8.8.2.3)

Pier 2
192"X48" (3.32% steel, 86#11 in two layers at top and 12#11 each side)

N := 196 \quad \text{number of longitudinal reinforcement}

R := 0.8 \quad \text{R-factor for joint (table 3.1-2)}

L := H \quad \text{Height of the pier}

A_b := 1.56 \quad \text{in}^2 \quad \text{Area of rebars for longitudinal rebar}

\lambda := 1 \quad \text{fixed - free} \quad \text{For longitudinal case which is govern for this bridge. For transverse}
\text{direction, value of 2 is used for the case of fixed - fixed.}

P_d := 570 \quad \text{kips} \quad \text{Axial dead load in the pier}

P_e := 0 \quad \text{kips} \quad \text{Axial earthquake force}

M_{topL} := 0 \quad \text{k - ft}

M_{botL} := 24711 \quad \text{k - ft}

OS := 1.5 \quad \text{Over strength factor}

M_{p\_topL} := OS \cdot M_{topL}

M_{p\_topL} = 0 \quad \text{k - ft}

M_{p\_botL} := OS \cdot M_{botL}

M_{p\_botL} = 3.707 \times 10^4 \quad \text{k - ft}

V_{uL} := \frac{(M_{p\_topL} + M_{p\_botL}) \cdot 12}{L \cdot R}

V_{uL} = 2.17 \times 10^3 \quad \text{kips}

V_{u\_analysis} := \frac{V}{R} \quad \text{kips}
$V_u := \max(V_uL, V_{u\text{ analysis}})$

$V_u = 2.17 \times 10^3$ kips

$f_c := 3500$ psi

$b_w := 192$ in

d := D in

$h = 48$ in

$V_c := .6\left[\sqrt{\left(\frac{f_c}{(\psi)}\right)}\right] b_w \frac{d}{1000}$ Shear resistance in the end regions (plastic hinge zone)

$V_c = 327.136$ kips

$L = 256.25$ in

$D_p := h - 5$ Width of the column core

$D_p = 43$ in

$\frac{D_p}{L} = 0.168$

$V_p := \frac{L}{2} P e \frac{D_p}{L}$ Contribution due to arch action

$V_p = 0$ kips

$\phi := 0.90$ For shear

$V_u = 2.17 \times 10^3$ kips

$V_s := \frac{V_u}{\phi} - (V_p + V_c)$

$V_s = 2.084 \times 10^3$ kips

$K_{\text{shape}} := .5$ For wall in weak axis (MCEER 8.8.2.3)
Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

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\[ f_{\text{yh}} = 60 \text{ ksi} \quad \text{For the transverse and longitudinal steel} \]

\[ f_{\text{su}} = 1.5 \cdot f_{\text{yh}} \quad \text{Ultimate tensile stress of the longitudinal reinforcement} \]

\[ f_{\text{su}} = 90 \text{ ksi} \]

\[ A_v := b_w \cdot d \quad \text{Shear area} \]

\[ A_v = 9.216 \times 10^3 \text{ in}^2 \]

\[ \rho_t := 0.0332 \quad \text{Longitudinal reinforcement ratio} \]

\[ s := 4 \text{ in} < 4 \text{ in} \text{ OK (MCEER 8.8.2.4) for plastic hinge zone} \]

\[ A_{\text{sh}} := 0.2 \quad \#4 \quad \text{Area of transverse reinforcement (ties)} \]

\[ \text{legs} := 10 \quad \text{Number of legs} \]

\[ \rho_{v \cdot \text{pr}} := \frac{A_{\text{sh}}}{b_w \cdot s} \]

\[ \rho_{v \cdot \text{pr}} = 2.604 \times 10^{-3} \]

\[ A_g := b_w \cdot h \quad A_c := (b_w - 6) \cdot (h - 6) \quad \text{Area of column core concrete} \]

\[ A_g = 9.216 \times 10^3 \text{ in}^2 \]

\[ A_c = 7.812 \times 10^3 \text{ in}^2 \]

\[ \tan \theta := \left( 1.6 \cdot \rho_{v \cdot \text{pr}} \cdot \frac{A_v}{A \cdot \rho_t \cdot A_g} \right)^{0.25} \]

\[ \tan \theta = 0.595 \]

\[ D_{pp} := h - 4 \quad \text{Width of the perimeter transverse direction} \]

\[ A_{vs} := s \cdot \frac{V_s}{f_{\text{yh}} \cdot D_{pp}} \cdot \tan \theta \]

\[ \frac{A_{vs}}{\text{legs}} = 0.188 \text{ in}^2 < A_{\text{sh}} \text{ OK} \]
Method 1: Implicit Shear Detailing Approach 
(Guide Spec. 8.8.2.3)

\[ \rho_v := \frac{K_{shape} \cdot \rho_t \cdot f_{su} \cdot A_g \cdot D_p \cdot \tan \theta}{L} \]

For fixed - fixed case of transverse direction

\[ \rho_v = 6.52 \times 10^{-3} \]

\[ A_{vs} := \rho_v \cdot b_w \cdot s \]

\[ \text{legs\_new} := 26 \]

\[ \frac{A_{vs}}{\text{legs\_new}} = 0.193 \text{ in}^2 \]

< Ash provided OK

Outside the plastic hinge zone

Method 2: Explicit Approach 8.8.2.3

\[ \text{legs\_outplastic} := 9 \]

\[ V_c := 2 \sqrt{f_c \cdot b_w} \cdot d \cdot \frac{1000}{1000} \]

Shear resistance of concrete out side of the plastic hinge zone

\[ V_c = 1.09 \times 10^3 \text{ kips} \]

\[ V_p = 0 \]

\[ V_s := \frac{V_u}{\phi} - (V_p + V_c) \]

\[ V_s = 1.32 \times 10^3 \text{ kips} \]

\[ s_m := 6 \text{ in} \]

Maximum spacing (MCEER 8.8.2.6)

\[ A_{shm} := .2 \#4 \]

\[ \rho_{v\_pr} := \text{legs\_outplastic} \cdot \frac{A_{shm}}{b_w \cdot s_m} \]

\[ \rho_{v\_pr} = 1.563 \times 10^{-3} \]
\[ A_g = 9.216 \times 10^3 \]
\[ \tan \theta := \left( \frac{1.6 \cdot \rho_{v,pr}}{A_v \cdot \rho_{t,pr} \cdot A_g} \right)^{25} \]
\[ \tan \theta = 0.524 \]
\[ D_{pp} = 44 \text{ in} \]
\[ A_{vs} := \frac{s_m \cdot V_s}{f_yh \cdot D_{pp}} \cdot \tan \theta \]
\[ \frac{A_{vs}}{\text{legs}_{\text{outplastic}}} = 0.175 \text{ in}^2 < \text{Ash OK} \]

**Method 1: Implicit Shear Detailing Approach**

\[ \text{legs}_{\text{new}} := 25 \]

\[ f_{yh} = 60 \]

\[ \rho_{vstar} := \rho_v - \frac{2 \sqrt{f_c}}{f_{yh}^{1000}} \]

\[ \rho_{vstar} = 4.548 \times 10^{-3} \]

\[ \rho_{pr} := \frac{\text{legs}_{\text{new}}}{b_w \cdot s_m} \cdot A_{shm} \]

\[ \rho_{pr} = 4.34 \times 10^{-3} \text{ Use method 2} \]

**Transverse reinforcement for confinement at plastic hinges (Pmax column)**

\[ \text{Guide Spec. 8.8.2.4} \]

\[ \text{legs} = 10 \]

\[ f_{yh} = 60 \text{ ksi} \]

\[ U_{sf} := 15.95 \text{ ksi Strain energy capacity} \]
\[ A_{sh} = 0.2 \text{ in}^2 \]
\[ A_c := (b_w - 6) \cdot (h - 6) \]
\[ A_c = 7.812 \times 10^3 \text{ in}^2 \]
\[ s := 4 \text{ in} < 4" \text{ OK (MCEER 8.8.2.4)} \]
\[ \rho_{s1} := \frac{A_{sh}}{D_p s} \]
\[ \rho_{s1} = 0.011 \]
\[ \rho_t = 0.033 \]
\[ A_g = 9.216 \times 10^3 \]
\[ f_c = 3.5 \times 10^3 \]
\[ P_e := P_d + P_e \]
\[ P_e = 570 \text{ kips} \quad \text{Factored axial load include seismic effect} \]
\[ \rho_{s2} := 0.008 \cdot \frac{f_c}{1000 \text{Usf}} \left[ 15 \cdot \left( \frac{1000 P_e}{f_c A_g} + \rho_t \cdot \frac{1000 \cdot f_{yh}}{f_c} \right) \cdot \left( \frac{A_g}{A_c} \right)^2 - 1 \right] \]
\[ \rho_{s2} = 0.011 \quad \text{OK} \]
\[ D_p = 43 \]
\[ \text{legs} = 10 \]
\[ \text{legs}_{\text{pmax}} := 10 \]
\[ \frac{\text{legs}_{\text{pmax}} \cdot A_{sh}}{s D_p} = 0.012 \quad \text{Include the area of total legs in both direction of cross section (Guide Spec. 8.8.2.4)} \]
\[ s_{\text{max}} := \frac{A_{sh}}{D_p \rho_{s2}} \]
\[ s_{\text{max}} = 4.184 \quad \text{in} \quad \text{OK} \]
\[ \rho_s := \max(\rho_{s1}, \rho_{s2}) \]
\[ \rho_s = 0.011 \]
\[ s = 4 \quad \text{in} \]

Explicit Shear Detailing Approach (Pmin Column) 
(Guide Spec. 8.8.2.3)
In longitudinal direction
\[ P_d = 570 \quad \text{kips} \]
\[ P_{\text{eminL}} := P_d \cdot 0.8 \quad \text{kips} \quad \text{For the uplift effect} \]
\[ P_{\text{eminL}} = 456 \quad \text{kips} \]
\[ M_{p_{\text{top}}} := 0 \quad \text{kft} \]
\[ M_{\text{botL}} = 2.471 \times 10^4 \quad \text{kft} \]
\[ \text{OS} = 1.5 \]
\[ P_{\text{eL}} := P_d + \text{OS} \times (P_{\text{eminL}} - P_d) \]
\[ P_{\text{eL}} = 399 \quad \text{kips} \]
\[ M_{p_{\text{bot}}} := \text{OS} \times M_{\text{botL}} \]
\[ V_u := \frac{M_{p_{\text{bot}}}}{L \cdot R} \]
\[ V_u = 2.17 \times 10^3 \quad \text{kips} \]
\[ A_v = 9.216 \times 10^3 \]
Title: Pulaski County Bridge

\[ V_c := 0.6 \sqrt{\frac{V_c}{1000}} \]

\[ V_c = 327.136 \text{ kips} \]

\[ V_p := \Lambda \cdot p_e \cdot \frac{D_p}{L \cdot 2} \]

\[ V_p = 33.477 \text{ kips} \]

\[ V_s := \frac{V_u}{\phi} - V_c - V_p \]

\[ V_s = 2.05 \times 10^3 \]

\[ s := 4 \text{ in} \]

\[ A_{sh} = 0.2 \text{ in}^2 \]

\[ \rho_v := \frac{A_{sh}}{b_w \cdot s} \]

\[ \rho_v = 2.604 \times 10^{-3} \]

\[ \tan \theta := \left[ 1.6 \cdot \rho_v \cdot \frac{A_v}{(\Lambda \cdot \rho t \cdot A_g)} \right]^{1/25} \]

\[ \tan \theta = 0.595 \]

\[ \tan \alpha := \frac{D_p}{L} \]

\[ \tan \alpha = 0.168 \text{ OK} \]

\[ A_s := V_s \cdot \frac{s \cdot \tan \theta}{f_{yh} \cdot D_{pp}} \]

\[ \frac{A_s}{\text{legs}} = 0.185 \text{ OK < Ash OK} \]
Anti buckling steel  
(Guide Spec. 8.8.2.5)

\[
\text{legs_antibuckling} := 70
\]

\[
d_b = 1.41 \text{ in} \quad \text{Diameter of longitudinal rebars}
\]

\[
s_{\text{new}} := 6 \cdot d_b \quad \text{Maximum spacing of ties}
\]

\[
s_{\text{new}} = 8.46 \text{ in} \quad \text{Required spacing of anti-buckling}
\]

\[
s = 4 \text{ in}
\]

\[
A_{\text{sh}} := 0.44 \quad \#6
\]

\[
A_b := 1.56 \cdot N
\]

\[
A_{bh} := A_b \cdot 0.09 \cdot \frac{60}{60}
\]

\[
\frac{A_{bh}}{\text{legs_antibuckling}} = 0.393 \quad \text{in}^2 < A_{sh} \quad \text{OK}
\]

\[
s_{\text{max}} := 0.25 \cdot 60
\]

\[
s_{\text{max}} = 15 \quad \text{OK}
\]

**Final check for the plastic hinge zone**  
(Guide Spec. 4.9.1)

\[
\tan 0 = 0.595
\]

\[
L_p := \frac{(h - 4)}{12} \left( \frac{1}{\tan 0} + 0.5 \cdot \tan 0 \right)
\]

\[
L_p = 7.252 \quad \text{ft}
\]

\[
V_u = 2.17 \times 10^3
\]

\[
M_{p\text{-botL}} = 3.707 \times 10^4 \quad \text{k-ft}
\]

\[
V_{ulL} = 2.17 \times 10^3 \quad \text{kips}
\]
Deep beam design (Source: Reinforcement concrete, Nawy, 3rd edition, section 6.9)

Shear design (pier 2)

\[
f_c := 3500 \text{ psi} \\
f_y := 60000 \text{ psi} \\
h := 16 \text{ ft} \\
b := 4 \text{ ft} \\
d := 0.9 \cdot h \\
d = 14.4 \text{ ft} \\
l := \frac{H}{12} \text{ ft} \\
l = 21.354 \text{ ft} \\
\frac{l}{d} = 1.483 < 5 \text{ so design anthology applies}
\]
\[ \Phi := 0.90 \quad \text{for shear} \]
\[ V = 1.199 \times 10^3 \quad \text{kips} \]
\[ M := 41430 \quad \text{k} - \text{ft} \]
\[ \Phi \left( 8 \sqrt{f_c b d} \cdot \frac{144}{1000} \right) = 3.533 \times 10^3 \quad \text{kips} \quad \Phi V_n \]
\[ A_1 := \frac{M}{V d} \]
\[ A_1 = 2.4 \]
\[ A_2 := 3.5 - 2.5 \frac{M}{V d} \]
\[ A_2 = -2.499 \quad A_2 < 3.5 \quad \text{OK} \]
\[ \text{But } A_2 < 1 \quad \text{Hence use } A_2 := 1 \]
\[ \rho_t = 0.033 \]
\[ V_c := A_2 \left( 1.9 \sqrt{f_c} + 2500 \rho_t \frac{V d}{M} \right) b d \cdot \frac{144}{1000} \]
\[ V_c = 1.219 \times 10^3 \quad \text{kips} \]
\[ V_{c_{\text{max}}} := 6 \sqrt{f_c b d} \cdot \frac{144}{1000} \]
\[ V_{c_{\text{max}}} = 2.944 \times 10^3 \]
\[ V_c := \min(V_c, V_{c_{\text{max}}}) \]
\[ V_c = 1.219 \times 10^3 \quad \text{kips} \]
\[ V_s := \frac{V}{\Phi} - \Phi V_c \]
VS = 234.91 kips

K1 := \frac{305.75}{4.5} \left( \frac{11 - h}{d} \right) / 12

K2 := \frac{VS \times 1000}{f_y \times d \times 12}

SV := \left( 2.79 \left( 1 + \frac{h}{d} \right) \right) / \left[ 12 \left( K1 - K2 \right) \right]

SV := \left( \frac{12}{2.791 + \frac{h}{d}} \right) \left( K1 - K2 \right)

SV = 249.571 in

SV_{\text{max}} := \min \left( 18, \frac{b \times 12}{3} \right)

SV_{\text{max}} = 16 in

Flexure design for deep beam

\Phi := 1

M = 4.143 \times 10^4 \text{ k - ft}

\frac{1}{h} = 1.335 > 1 \text{ so } j = 0.6l

j := 0.2 \left( \frac{1 + 2h}{d} \right)

j = 0.741
j·d = 10.671 ft

\[ A_s := \frac{M \cdot 12000}{\phi \cdot f_y \cdot j \cdot d - 12} \]

\[ A_s = 64.709 \text{ in}^2 < 307.75 \text{ in}^2 \text{ OK} \]

\[ A_{s_{\text{min}}} := \max \left( 3 \cdot \sqrt{\frac{f_c}{f_y}} \cdot b \cdot d - 144, 200 \cdot b \cdot \frac{d \cdot 144}{f_y} \right) \]

\[ A_{s_{\text{min}}} = 27.648 \text{ in}^2 < 305.75 \text{ in}^2 \text{ OK} \quad b := 2.5 \cdot 12 \]

**Shear design in transverse direction**

MCEER Guide Spec. 8.8.3

\( h := 48 \)

\( \rho_{h_{\text{min}}} := .0025 \)

\( A_{s_{h_{-\text{rq}}} := \rho_{h_{\text{min}}} (b_w - 4) \cdot h} \)

\[ A_{s_{h_{-\text{rq}}} = 22.56 \text{ in}^2} \]

\[ A_{s_{h}} = 0.44 \quad 2 \#6 \]

\( N_{h} := \frac{A_{s_{h_{-\text{rq}}}}}{A_{s_{h}}} \)

\( N_{h} = 51.273 \quad \text{Number of horizontal reinforcement} \)

\( s_{h} := 4 \quad \text{in} \)

\[ V_r := \frac{3}{1000} \sqrt{f_c (b_w - 3)} \cdot h \]

\[ V_r = 1.61 \times 10^3 \text{ kips} \]
V_n := \phi \left( 0.756 \sqrt{f_c} + \rho_{h_{\text{min}}} f_{y_{h}} 1000 \right) \left( b_w - 3 \right) \frac{h}{1000}

V_n = 1.59 \times 10^3 \text{ kips}

V := \min \{ V_n, V_r \}

V = 1.59 \times 10^3 \text{ kips}

V_{rq} := 1651 \text{ kips (So minimum horizontal reinforcement is not adequate for this pier)}

\rho_h := 0.0029

A_{sh_{rq}} := \rho_h (b_w - 4) h

A_{sh_{rq}} = 26.17 \text{ in}^2

A_{sh} := 0.88 \text{ 2 #6}

N_{h} := \frac{A_{sh_{rq}}}{A_{sh}}

N_{h} = 29.738 \text{ Number of horizontal reinforcement}

s_h := 4 \text{ in}

V_n := \phi \left( 0.756 \sqrt{f_c} + \rho_{h} f_{y_{h}} 1000 \right) \left( b_w - 3 \right) \frac{h}{1000}

V_n = 1.786 \times 10^3 \text{ kips}

V_{rq} = 1.651 \times 10^3 \text{ kips (So use 34#8 horizontal reinforcement)}
Transverse Reinforcement Design
For the Pier 1&3

Plastic hinge zone length (this length will modify after the reinforcement design)
MCEER Guide Spec. 4.9.1

1-

\[ h := 48 \text{ in } \text{ thickness of the wall} \]

2-

\[ D := h - 3 \text{ in} \]

\[ D := 48 \text{ in} \]

3-

\[ H := 256.25 \text{ in} \]

\[ \frac{1}{6} \cdot H = 42.708 \text{ in} \]

4-

\[ \varepsilon_{yo} := 0.00207 \]

\[ d_b := 1.41 \text{ in } \text{ for #11} \]

\[ M := 29060 \text{ k - ft} \]

\[ V := 1651 \text{ k} \]

\[ 1.5 \left( 0.08 \cdot M \frac{12}{V} + 4400 \cdot \varepsilon_{yo} \cdot d_b \right) = 44.61 \text{ in } \text{ Controls but last check should be done after design} \]
Wall pier transverse reinforcement  
Method 2: Explicit Approach  
(Guide Spec 8.8.2.3)  
Pier 1&3  
192”X48” (3.32% steel, 86#11 in two layers at top and 12#11 each side)  
\[ N := 196 \]  number of longitudinal reinforcement  
\[ R := 0.8 \]  R-factor for joint (table 3.1-2)  
\[ L := H \text{ in} \]  Height of the pier  
\[ A_b := 1.56 \text{ in}^2 \]  Area of rebars for longitudinal rebar  
\[ A := 1 \]  fixed - free  
For longitudinal case which is govern for this bridge. For transverse direction, value of 2 is used for the case of fixed - fixed.  
\[ P_d := 470 \text{ kips} \]  Axial dead load in the pier  
\[ P_e := 164 \text{ kips} \]  Axial earthquake force  
\[ M_{topL} := 0 \text{ k - ft} \]  
\[ M_{botL} := M \]  
\[ OS := 1.5 \text{ Over strength factor} \]  
\[ M_{p\_topL} := OS \cdot M_{topL} \]  
\[ M_{p\_topL} = 0 \text{ k - ft} \]  
\[ M_{p\_botL} := OS \cdot M_{botL} \]  
\[ M_{p\_botL} = 4.359 \times 10^4 \text{ k - ft} \]  
\[ V_{ul} := \frac{(M_{p\_topL} + M_{p\_botL})}{L \cdot R}^{12} \]  
\[ V_{ul} = 2.552 \times 10^3 \text{ kips} \]  
\[ V_{u\_analysis} := \frac{V}{R} \text{ kips} \]
\[ V_u := \max(V_{uL}, V_{u\text{analysis}}) \]

\[ V_u = 2.552 \times 10^3 \text{ kips} \]

\[ f_c := 3500 \text{ psi} \]

\[ b_w := 192 \text{ in} \]

\[ d := D \text{ in} \]

\[ h = 48 \text{ in} \]

\[ V_c := 0.6\left(\sqrt{f_c}\right) \cdot b_w \cdot \frac{d}{1000} \]

Shear resistance in the end regions (plastic hinge zone)

\[ V_c = 327.136 \text{ kips} \]

\[ L := 253.5 \text{ in} \]

\[ D_p := h - 5 \text{ Width of the column core} \]

\[ D_p := 43 \text{ in} \]

\[ \frac{D_p}{L} = 0.17 \]

\[ V_p := \frac{\Lambda}{2} \cdot P e \cdot \frac{D_p}{L} \]

\[ V_p = 13.909 \text{ kips} \]

Contribution due to arch action

\[ \phi := 0.90 \text{ For shear} \]

\[ V_u = 2.552 \times 10^3 \text{ kips} \]

\[ V_s := \frac{V_u}{\phi} - \left(V_p + V_c\right) \]

\[ V_s = 2.494 \times 10^3 \text{ kips} \]

\[ K_{\text{shape}} := 0.5 \text{ For wall in weak axis (MCEER 8.8.2.3)} \]
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\[ f_{yh} := 60 \text{ ksi} \quad \text{For the transverse and longitudinal steel} \]

\[ f_{su} := 1.5 \cdot f_{yh} \quad \text{Ultimate tensile stress of the longitudinal reinforcement} \]

\[ f_{su} = 90 \text{ ksi} \]

\[ A_v := b_w \cdot d \quad \text{Shear area} \]

\[ A_v = 9.216 \times 10^3 \text{ in}^2 \]

\[ \rho_t := 0.0332 \quad \text{Longitudinal reinforcement ratio} \]

\[ s := 4 \text{ in} < 4" \text{ OK (MCEER 8.8.2.4) for plastic hinge zone} \]

\[ A_{sh} := 0.2 \#4 \text{ Area of transverse reinforcement (ties)} \]

\[ \text{legs} := 13 \quad \text{Number of legs} \]

\[ \rho_{v,pr} := \text{legs} \cdot \frac{A_{sh}}{b_w \cdot s} \]

\[ \rho_{v,pr} = 3.385 \times 10^{-3} \]

\[ A_g := b_w \cdot h \]

\[ A_g = 9.216 \times 10^3 \text{ in}^2 \]

\[ \tan \theta := \left( 1.6 \cdot \rho_{v,pr} \cdot \frac{A_v}{A \cdot \rho_t \cdot A_g} \right)^{25} \]

\[ \tan \theta = 0.636 \]

\[ D_{pp} := h - 4 \quad \text{Width of the perimeter transverse direction} \]

\[ A_{vs} := s \cdot \frac{V_s}{f_{yh} \cdot D_{pp} \cdot \tan \theta} \]

\[ A_{vs} \text{ legs} = 0.185 \text{ in}^2 < Ash \text{ OK} \]
Method 1: Implicit Shear Detailing Approach
(Guide Spec. 8.8.2.3)

\[
\rho_v := K_{\text{shape}}^{-2} \left( \frac{f_{\text{su}}}{f_{\text{yh}}} \frac{A_g A_p}{\phi L \tan \theta} \right)
\]

For fixed - fixed case of transverse direction

\[
\rho_v = 5.965 \times 10^{-3}
\]

\[A_{\text{vs}} := \rho_v b_w s\]

\[\text{legs}_\text{new} := 26\]

\[
\frac{A_{\text{vs}}}{\text{legs}_\text{new}} = 0.176 \text{ \(\text{in}^2\)} < \text{Ash provided OK}
\]

Outside the plastic hinge zone
Method 2: Explicit Approach

8.8.2.3

\[\text{legs}_\text{outplastic} := 13\]

\[V_c := 2 \sqrt{\frac{f_c b_w d}{1000}}\]

Shear resistance of concrete out side of the plastic hinge zone

\[V_c = 1.09 \times 10^3 \text{ \text{kips}}\]

\[V_p = 13.909\]

\[V_s := \frac{V_u}{\phi} - (V_p + V_c)\]

\[V_s = 1.731 \times 10^3 \text{ \text{kips}}\]

\[s_m := 6 \text{ \text{in}}\]

Maximum spacing (MCEER 8.8.2.6)

\[A_{\text{shm}} := .2 \ #4\]

\[\rho_{v_{\text{pr}}} := \text{legs}_\text{outplastic} \frac{A_{\text{shm}}}{b_w s_m}\]

\[\rho_{v_{\text{pr}}} = 2.257 \times 10^{-3}\]
\[ A_g = 9.216 \times 10^3 \]

\[ \tan \theta := \left( 1.6 \cdot \rho_{v,pr} \frac{A_v}{A \cdot \rho_t \cdot A_g} \right)^{25} \]

\[ \tan \theta = 0.574 \]

\[ D_{pp} = 44 \text{ in} \]

\[ A_{vs} := \frac{s_m \cdot V_s}{f_{yh} \cdot D_{pp} \cdot \tan \theta} \]

\[ \frac{A_{vs}}{\text{legs}_{outplastic}} = 0.174 \text{ in}^2 < \text{Ash OK} \]

**Method 1: Implicit Shear Detailing Approach**

\[ \text{legs}_{new} := 27 \]

\[ f_{yh} = 60 \]

\[ \rho_{v,star} := \rho_v - \frac{2 \sqrt{f_c}}{f_{yh} \cdot 1000} \]

\[ \rho_{v,star} = 3.993 \times 10^{-3} \]

\[ \rho_{pr} := \frac{\text{legs}_{new} \cdot A_{shm}}{b_w \cdot s_m} \]

\[ \rho_{pr} = 4.688 \times 10^{-3} \text{ Use method 2} \]

**Transverse reinforcement for confinement at plastic hinges (Pmax column)**

(Guide Spec. 8.8.2.4)

\[ \text{legs} = 13 \]

\[ f_{yh} = 60 \text{ ksi} \]

\[ U_{sf} := 15.95 \text{ ksi \ Strain energy capacity} \]
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\[ A_{sh} = 0.2 \text{ in}^2 \]
\[ A_c := (b_w - 6)(h - 6) \]
\[ A_c = 7.812 \times 10^3 \text{ in}^2 \]
\[ s := 4 \text{ in} < 4" \text{ OK (MCEER 8.8.2.4)} \]
\[ \rho_{s1} := \text{legs} \frac{A_{sh}}{D_{pp} s} \]
\[ \rho_{s1} = 0.015 \]
\[ \rho_t = 0.033 \]
\[ A_g = 9.216 \times 10^3 \]
\[ f_c = 3.5 \times 10^3 \]
\[ P_e := P_d + P_e \]
\[ P_e = 634 \text{ kips} \]

Factored axial load include seismic effect

\[ \rho_{s2} := 0.008 \frac{f_c}{1000U_{sf}} \left[ 15 \left( \frac{1000P_e}{f_cA_g} + \rho_t \frac{1000f_{yh}}{f_c} \right)^2 \left( \frac{A_g}{A_c} \right)^2 - 1 \right] \]
\[ \rho_{s2} = 0.011 \text{ OK} \]
\[ D_p = 43 \]
\[ \text{legs} = 13 \]
\[ \text{legs}_\text{pmax} := 10 \]
\[ \frac{\text{legs}_\text{pmax}A_{sh}}{sD_p} = 0.012 \] Include the area of total legs in both direction of cross section (Guide Spec. 8.8.2.4)

\[ s_{\max} := \text{legs}_\text{pmax} \frac{A_{sh}}{D_{pp} \rho_{s2}} \]
\[ s_{\max} = 4.151 \text{ in} \text{ OK} \]
\[ \rho_s := \max(\rho_{s1}, \rho_{s2}) \]

\[ \rho_s = 0.015 \]

\[ s = 4 \quad \text{in} \]

**Explicit Shear Detailing Approach (Pmin Column)**

*(Guide Spec. 8.8.2.3)*

In longitudinal direction

\[ P_d = 470 \quad \text{kips} \]

\[ P_{eminL} := P_d \cdot 0.8 \quad \text{kips} \quad \text{For the uplift effect} \]

\[ P_{eminL} = 376 \quad \text{kips} \]

\[ M_{p_top} := 0 \quad \text{k} \cdot \text{ft} \]

\[ M_{botL} = 2.906 \times 10^4 \quad \text{k} \cdot \text{ft} \]

\[ OS = 1.5 \]

\[ P_{eL} := P_d + OS \cdot (P_{eminL} - P_d) \]

\[ P_{eL} = 329 \quad \text{kips} \]

\[ M_{p_bot} := OS \cdot M_{botL} \]

\[ V_u := \frac{M_{p_bot} \cdot 12}{L \cdot R} \]

\[ V_u = 2.579 \times 10^3 \quad \text{ips} \]

\[ A_v = 9.216 \times 10^3 \]

\[ V_c := 0.6 \cdot \sqrt{f_c} \cdot \frac{A_v}{1000} \]

\[ V_c = 327.136 \quad \text{kips} \]
Diameter of longitudinal rebars in
\[ d_b = 1.41 \text{ in} \]

legs _antibuckling := 70

\[ \rho_v := \frac{A_{sh}}{b_w \cdot s} \]

\[ \rho_v = 3.385 \times 10^{-3} \]

\[ \tan \theta := \left[ 1.6 \cdot \rho_v \cdot \frac{A_v}{\left( A \cdot \rho_t \cdot A_g \right)} \right]^{0.25} \]

\[ \tan \theta = 0.636 \]

\[ \tan \alpha := \frac{D_p}{L} \]

\[ \tan \alpha = 0.17 \quad \text{OK} \]

\[ A_s := V_s \cdot \frac{s \cdot \tan \theta}{f_{yh} \cdot D_{pp}} \]

\[ \frac{A_s}{\text{legs}} = 0.186 \quad \text{OK} \quad < \text{Ash OK} \]

Anti buckling steel
(Guide Spec. 8.8.2.5)

\[ \text{d_b} = 1.41 \quad \text{in} \]

Diameter of longitudinal rebars


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\[ s_{\text{new}} := 6 \cdot d_b \]

Maximum spacing of ties

\[ s_{\text{new}} = 8.46 \text{ in} \]

Required spacing of anti-buckling

\[ s = 4 \text{ in} \]

\[ A_{\text{sh}} := 0.44 \text{ #6} \]

\[ A_b := 1.56 \cdot \text{N} \]

\[ A_{\text{bh}} := A_b \cdot 0.09 \cdot \frac{60}{60} \]

\[ \frac{A_{\text{bh}}}{\text{legs\_antibuckling}} = 0.393 < \text{Ash \ OK} \]

\[ s_{\text{max}} := 0.25 \cdot 60 \]

\[ s_{\text{max}} = 15 \text{ OK} \]

---

**Final check for the plastic hinge zone**

(\text{Guide Spec.} 4.9.1)

\[ \tan \theta = 0.636 \]

\[ L_p := \frac{(b - 4)}{12} \left( \frac{1}{\tan \theta} + 0.5 \cdot \tan \theta \right) \]

\[ L_p = 6.934 \text{ ft} \]

\[ V_u = 2.579 \times 10^3 \]

\[ M_{P_{\text{botL}}} = 4.359 \times 10^4 \text{ k-ft} \]

\[ V_{ulL} = 2.552 \times 10^3 \text{ kips} \]

\[ \frac{M_{P_{\text{botL}}}}{V_u} \left[ 1 - 0.85 \left( \frac{M_{P_{\text{botL}}}}{1.5} \right) \right] = 7.323 \text{ ft controls} \]
The page contains calculations and design information for a deep beam design. The calculations are as follows:

\[ \frac{1.5}{12} \left( 4400 \cdot e \cdot d_b + 0.08 \cdot \frac{12M_{p_botL}}{V_u} \right) = 3.633 \]

**Summary of transverse reinforcement pier 1 & 3**
- #6@6" with 70 legs in 7.3' of bottom
- #4@4" with 23 legs in 7.3' of bottom
- #4@6" with 23 legs in the rest of the wall

**Deep beam design** (Source: Reinforcement concrete, Nawy, 3rd edition, section 6.9)

Shear design (pier 1&3)

\[ f_c := 3500 \text{ psi} \]
\[ f_y := 60000 \text{ psi} \]
\[ h := 16 \text{ ft} \]
\[ b := 4 \text{ ft} \]
\[ d := 0.9 \cdot h \]
\[ d = 14.4 \text{ ft} \]
\[ l := \frac{L}{12} \text{ ft} \]
\[ l = 21.125 \text{ ft} \]
\[ \frac{1}{d} = 1.467 < 5 \text{ so design anthology applies} \]
\[ \Phi := 0.90 \text{ for shear} \]
\[ V = 1.651 \times 10^3 \text{ kips} \]
\[ M := 39190 \text{ k·ft} \]
\[ \Phi \left( 8 \sqrt{f_c \cdot b \cdot d} \cdot \frac{144}{1000} \right) = 3.533 \times 10^3 \text{kips} \ 
\]

\[ A_1 := \frac{M}{V \cdot d} \]

\[ A_1 = 1.648 \]

\[ A_2 := 3.5 - 2.5 \frac{M}{V \cdot d} \]

\[ A_2 = -0.621 \quad A_2 < 3.5 \quad \text{OK} \]

\[ \rho_t = 0.033 \]

\[ V_c := A_2 \left( 1.9 \sqrt{f_c} + 2500 \cdot \rho_t \frac{V \cdot d}{M} \right) \cdot b \cdot d \cdot \frac{144}{1000} \]

\[ V_c = -838.372 \text{kips} \quad \text{use 0} \]

\[ V_{c_{\max}} := 6 \sqrt{f_c \cdot b \cdot d} \cdot \frac{144}{1000} \]

\[ V_{c_{\max}} = 2.944 \times 10^3 \]

\[ V_c := \min(V_c, V_{c_{\max}}) \]

\[ V_c := 0 \text{kips} \]

\[ V_s := \frac{V}{\Phi} - \Phi \cdot V_c \]

\[ V_s = 1.834 \times 10^3 \text{kips} \quad \text{Shear reinforcement is not required} \]

\[ K_1 := \frac{305.75}{4.5} \left( 11 - \frac{h}{d} \right) \]

\[ K_2 := \frac{V_s \cdot 1000}{f_y \cdot d \cdot 12} \]
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\[ S_v := \frac{2.79 \left( 1 + \frac{h}{d} \right)}{12 \left( K_1 - K_2 \right)} \]

\[ S_v := \frac{12}{2.79 \cdot 1 + \frac{h}{d}} \left( K_1 - K_2 \right) \]

\[ S_v = 248.883 \text{ in} \]

\[ S_{v\text{max}} := \min \left( 18, \frac{b \cdot 12}{3} \right) \]

\[ S_{v\text{max}} = 16 \text{ in} \]

**Flexure design for deep beam**

\[ \Phi := 1 \]

\[ M = 3.919 \times 10^4 \text{ k-ft} \]

\[ \frac{1}{h} = 1.32 > 1 \text{ so } j = 0.6l \]

\[ j := \frac{2(1+2h)}{d} \]

\[ j = 0.738 \]

\[ j \cdot d = 10.625 \text{ ft} \]

\[ A_s := \frac{M \cdot 12000}{\Phi \cdot f_y \cdot j \cdot d \cdot 12} \]

\[ A_s = 61.475 \text{ in}^2 < 307.75 \text{ in}^2 \text{ OK} \]

\[ A_{s\text{min}} := \max \left( 3 \cdot \sqrt{\frac{f_c}{f_y}} \cdot b \cdot d \cdot 144, 200 \cdot b \cdot \frac{d \cdot 144}{f_y} \right) \]

\[ A_{s\text{min}} = 27.648 \text{ in}^2 < 305.75 \text{ in}^2 \text{ OK} \]
Shear design in transverse direction

MCEER Guide Spec. 8.8.3

\[ h := 48 \]

\[ \rho_{h_{\text{min}}} := .0025 \]

\[ A_{\text{sh}\_rq} := \rho_{h_{\text{min}}} (b_w - 4) \cdot h \]

\[ A_{\text{sh}\_rq} = 22.56 \text{ in}^2 \]

\[ A_{\text{sh}} = 0.44 \text{ 2 #6} \]

\[ N_h := \frac{A_{\text{sh}\_rq}}{A_{\text{sh}}} \]

\[ N_h = 51.273 \text{ Number of horizontal reinforcement} \]

\[ s_h := 4 \text{ in} \]

\[ V_r := \frac{3}{1000} \sqrt{f_c (b_w - 3)} \cdot h \]

\[ V_r = 1.61 \times 10^3 \text{ kips} \]

\[ V_n := \phi (0.756 \sqrt{f_c} + \rho_{h_{\text{min}}} f_y h) \cdot (b_w - 3) \cdot \frac{h}{1000} \]

\[ V_n = 1.59 \times 10^3 \text{ kips} \]

\[ V := \min(V_n, V_r) \]

\[ V = 1.59 \times 10^3 \text{ kips} \]

\[ V_{\text{rq}} := 1651 \text{ kips (So minimum horizontal reinforcement is not adequate for this pier)} \]
\[ \rho_h := .0029 \]

\[ A_{sh\_rq} := \rho_h (b_w - 4) h \]

\[ A_{sh\_rq} = 26.17 \text{ in}^2 \]

\[ A_{sh} := 0.88 \]

\[ N_h := \frac{A_{sh\_rq}}{A_{sh}} \]

\[ N_h = 29.738 \quad \text{Number of horizontal reinforcement} \]

\[ s_h := 4 \text{ in} \]

\[ V_n := \phi \left( .756 \cdot \sqrt{f_c} + \rho_h f_y h^\frac{1000}{h} \cdot (b_w - 3) \right) \frac{h}{1000} \]

\[ V_n = 1.786 \times 10^3 \text{ kips} \]

\[ V_{rq} = 1.651 \times 10^3 \text{ kips} \]

(Use 60#6 horizontal reinforcement)
Pier 2

Wall pier connection design
Method 1: Implicit Shear Detailing Approach
(Guide spec. 8.8.4.1, 8.8.2.3, 8.8.2.4)
192"X48" (3.32% steel, 86#11 in two layers at top and 12#11 each side)

\[ N := 196 \quad \text{number of longitudinal reinforcement} \]
\[ b_w := 48 \quad \text{in} \quad \text{Height of pile cap} \]
\[ H_c := 48 \quad \text{in} \quad \text{Height of the joint} \]
\[ h := 48 \quad \text{in} \quad \text{Dimension of the column} \]
\[ D := h - 3 \quad \text{in} \]
\[ D = 45 \quad \text{in} \]
\[ R := 0.8 \quad \text{R-factor for joint (table 3.1-2)} \]
\[ \varepsilon_y := .00207 \quad \text{For the longitudinal steel} \]
\[ L := 256.25 \quad \text{in} \quad \text{Height of the pier} \]
\[ A_b := 1.56 \quad \text{in}^2 \quad \text{Area of rebars for longitudinal rebar} \]
\[ P_d := 570 \quad \text{kips} \quad \text{Axial dead load in the pier} \]
\[ P_e := 0 \quad \text{kips} \quad \text{Axial earthquake force} \]
\[ M_{botL} := 24711 \quad \text{k} \cdot \text{ft} \]
\[ OS := 1.5 \quad \text{Over strength factor} \]
\[ M_{p\_botL} := OS \cdot M_{botL} \]
\[ M_{p\_botL} = 3.707 \times 10^4 \quad \text{k} \cdot \text{ft} \]
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NCHRP Guidelines

BY: NA

CHECKED: AAA

DATE: 6/22/2004

Sheet No: 72

\[ f_c := 3500 \text{ psi} \]

\[ b_w := 192 \text{ in} \]

\[ d := D \text{ in} \]

\[ L = 256.25 \text{ in} \]

\[ D_p := h - 4 \text{ Width of the column core} \]

\[ D_p = 44 \text{ in} \]

\[ \phi := 0.90 \text{ For shear} \]

\[ f_{yh} := 60 \text{ ksi For the transverse and longitudinal steel} \]

\[ f_{su} := 1.5 \cdot f_{yh} \text{ Ultimate tensile stress of the longitudinal reinforcement} \]

\[ f_{su} = 90 \text{ ksi} \]

\[ A_v := b_w \cdot d \text{ Shear area} \]

\[ A_v = 8.64 \times 10^3 \text{ in}^2 \]

\[ \rho_l := 0.0332 \text{ Longitudinal reinforcement ratio} \]

\[ s := 4 \text{ in} < 4" \text{ OK (MCEER 8.8.2.4) for plastic hinge zone} \]

\[ A_{sh} := 0.2 \# 4 \text{ Area of transverse reinforcement (ties)} \]

\[ \text{legs} := 16 \text{ Number of legs} \]

\[ \rho_{v,pr} := \text{legs} \cdot \frac{A_{sh}}{b_w \cdot s} \]

\[ \rho_{v,pr} = 4.167 \times 10^{-3} \]

\[ A_g := b_w \cdot h \]

\[ A_g = 9.216 \times 10^3 \text{ in}^2 \]
L = 256.25 in

\( A_b := 1.56 \text{ in}^2 \) For longitudinal rebar

\( D_p = 44 \text{ in} \) Width of the column core

\( \phi := 0.90 \) For shear

\( A_v := D_p^2 \) Shear area \( A_v = bw \times d \)

\( A_g := D^2 \)

\( f_{yh} := 60 \text{ ksi} \)

\( f_{su} := 1.5 \times f_{yh} \)

\( D_{pp} := 31 \) Core dimension of tied column in the direction under construction

\( A_{sc} := N \times A_b \) Longitudinal reinforcement in column

\( A_{sc} = 305.76 \text{ in}^2 \)

Explicit approach for joint design

(\text{Guide Spec. 8.8.4.2, C8.8.4.2, 8.8.4.2.2, 8.8.4.3.1, 8.8.4.3.2, 8.8.4.3.3})

Bottom of the wall in longitudinal direction

\( f_h := 0 \) Average axial stress in the horizontal direction

\( P := P_d + P_e \)

\( P = 570 \text{ kips} \) Maximum axial force include earthquake effect

\( M_{p\_botL} = 3.707 \times 10^4 \text{ k} - \text{ ft} \) From push over analysis

\( b_b := 20.12 \text{ in} \) Width of pile cap
Title: Pulaski County Bridge

\[
L_{\text{mid_dept_jt}} := D + H_c
\]

\[
L_{\text{mid_dept_jt}} = 93 \text{ in}
\]

\[
A_{\text{mid}} := b_b L_{\text{mid_dept_jt}}
\]

\[
A_{\text{mid}} = 2.232 \times 10^4 \text{ in}^2
\]

\[
f_v := \frac{P_e}{A_{\text{mid}}}
\]

Average axial stress in the vertical direction

\[
f_v = 0 \text{ ksi}
\]

\[
b_b := H_c \text{ Joint depth}
\]

\[
b_b = 48 \text{ in}
\]

\[
h_c := D \text{ Dimension of column}
\]

\[
h_c = 45 \text{ in}
\]

\[
b_{je} := 2h_c \text{ effective joint width for shear stress calculations}
\]

\[
b_{je} = 90 \text{ in}
\]

\[
v_{hv} := \frac{M_{p,botL} \cdot 12}{h_b h_c b_{je}} \text{ Joint shear stress}
\]

\[
v_{hv} = 2.288 \text{ ksi}
\]

\[
p_t := \frac{f_h + f_v}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{hv}^2} \quad \text{Principal tension stress}
\]

\[
p_t = -2.288
\]

\[
p_{t_{\text{max}}} := \frac{3.5}{1000} \sqrt{f_c} \quad \text{Maximum tension stress}
\]
\[ p_{t,\text{max}} = 0.207 \]

Not OK, we should provide some horizontal reinforcement

\[ p_c := \frac{f_h + f_v}{2} + \sqrt{\frac{(f_h - f_v)^2}{2}} + v_h^2 \]

Principal compression stress

\[ p_c = 2.288 \]

\[ p_{c,\text{max}} := \frac{25}{1000} f_c \]

Maximum compression stress

\[ p_{c,\text{max}} = 0.875 \]

Not OK, we should provide some horizontal reinforcement

\[ A_{jv} := 0.16 A_{sc} \]

Vertical reinforcement (Stirrups) (Guide Spec. 8.8.4.3.2)

\[ A_{jv} = 48.922 \text{ in}^2 \]

Use #7

\[ A_{jv} \cdot 0.6 = 81.536 \]

72#7 vertical legs in (development length) each side

\[ A_{\text{clamp}} := 0.08 A_{sc} \]

Vertical reinforcement (Clamping reinforcement) (Guide Spec. 8.8.4.3.2)

\[ A_{\text{clamp}} = 24.461 \]

\[ A_{\text{clamp}} \cdot 0.6 = 40.768 \]

31#7 in 36" of the joint

\[ l_{d9} := 0.63 - 1.128 \frac{f_{yh}}{\sqrt{\frac{f_c}{1000}}} \]

Development length for #9 (AASHTO LRFD 5.11.2.2.1-1)

\[ 0.3 - 1.128 f_{yh} = 20.304 \text{ in} \]

\[ l_{d9} = 22.791 \text{ in} \]

\[ A_h := 0.08 A_{sc} \]

Horizontal reinforcement (Guide Spec. 8.8.4.3.3)

\[ A_h = 24.461 \]

25#9 in 40" length each side
Use #4@4" with 23 legs
Pier 1&3

Wall pier connection design
Method 1: Implicit Shear Detailing Approach
(Guide spec. 8.8.4.1, 8.8.2.3, 8.8.2.4)

192"X48" (3.32% steel, 86#11 in two layers at top and 12#11 each side)

\[ N = 196 \] number of longitudinal reinforcement

\[ b_w := 48 \text{ in} \] Height of pile cap

\[ H_c = 48 \text{ in} \] Height of the joint

\[ h = 48 \text{ in} \] Dimension of the column

\[ D := h - 3 \text{ in} \]

\[ D = 45 \text{ in} \]

\[ R := 0.8 \] R-factor for joint (table 3.1-2)

\[ \varepsilon_y := .00207 \] For the longitudinal steel

\[ L := 253.5 \text{ in} \] Height of the pier

\[ A_b := 1 \text{ in}^2 \] Area of rebars for longitudinal rebar

\[ \Lambda := 1 \] fixed - free For longitudinal case which is govern for this bridge. For transverse direction, value of 2 is used for the case of fixed - fixed.

\[ P_d := 470 \text{ kips} \] Axial dead load in the pier

\[ P_e := 164 \text{ kips} \] Axial earthquake force

\[ M_{botL} := 29060 \text{ k-ft} \]

\[ OS := 1.5 \] Over strength factor

\[ M_{p_botL} := OS \cdot M_{botL} \]

\[ M_{p_botL} = 4.359 \times 10^4 \text{ k-ft} \]
Evaluation of Comprehensive Siesmic Design of Bridges (LRFD) in Illinois

NCHRP Guidelines

Title: Pulaski County Bridge

\[ f_c := 3500 \text{ psi} \]
\[ b_w := 196 \text{ in} \]
\[ d := D \text{ in} \]
\[ L = 253.5 \text{ in} \]
\[ D_p := h - 4 \quad \text{Width of the column core} \]
\[ D_p = 44 \text{ in} \]
\[ \phi := 0.85 \quad \text{For shear} \]
\[ f_{y_h} := 60 \text{ ksi} \quad \text{For the transverse and longitudinal steel} \]
\[ f_{su} := 1.5 f_{y_h} \quad \text{Ultimate tensile stress of the longitudinal reinforcement} \]
\[ f_{su} = 90 \text{ ksi} \]
\[ A_v := b_w d \quad \text{Shear area} \]
\[ A_v = 8.82 \times 10^3 \text{ in}^2 \]
\[ \rho_t = 0.033 \quad \text{Longitudinal reinforcement ratio} \]
\[ s := 4 \text{ in} \quad < 4'' \text{OK (MCEER 8.8.2.4) for plastic hinge zone} \]
\[ A_{sh} := 0.2 \quad \#4 \quad \text{Area of transverse reinforcement (ties)} \]
\[ \text{legs} := 16 \quad \text{Number of legs} \]
\[ \rho_{v,pr} := \text{legs} \frac{A_{sh}}{b_w s} \]
\[ \rho_{v,pr} = 4.082 \times 10^{-3} \]
\[ A_g := b_w h \]
\[ A_g = 9.408 \times 10^3 \text{ in}^2 \]

\[ L = 253.5 \text{ in} \quad \text{Length of the column} \]

\[ A_b = 1 \text{ in}^2 \quad \text{For longitudinal rebar} \]

\[ D_p = 44 \text{ in} \quad \text{Width of the column core} \]

\[ \phi := 0.85 \quad \text{For shear} \]

\[ A_v := D_p^2 \quad \text{Shear area } A_v = b w \cdot d \]

\[ A_g := D^2 \]

\[ f_{yh} := 60 \text{ ksi} \]

\[ f_{su} := 1.5 f_{yh} \]

\[ D_{pp} := h - 5 \quad \text{Core dimension of tied column in the direction under construction} \]

\[ A_{sc} := N \cdot A_b \quad \text{Longitudinal reinforcement in column} \]

\[ A_{sc} = 196 \text{ in}^2 \]
Explicit approach for joint design
(Guide Spec. 8.8.4.2, C8.8.4.2, 8.8.4.2.2, 8.8.4.3.1, 8.8.4.3.2, 8.8.4.3.3)

Bottom of the column in longitudinal direction

\[ f_h := 0 \]

Average axial stress in the horizontal direction

\[ P := P_d + P_e \]

\[ P = 634 \text{ kips} \]

Maximum axial force include earthquake effect

\[ M_{p\_botL} = 4.359 \times 10^4 \text{ k - ft} \]

From push over analysis

\[ b_b := 25.12 \text{ in} \]

Width of pile cap

\[ L_{mid\_dept\_jt} := D + H_c \]

\[ L_{mid\_dept\_jt} = 93 \text{ in} \]

\[ A_{mid} := b_b L_{mid\_dept\_jt} \]

\[ A_{mid} = 2.79 \times 10^4 \text{ in}^2 \]

\[ f_v := \frac{P_e}{A_{mid}} \]

Average axial stress in the vertical direction

\[ f_v = 5.878 \times 10^{-3} \text{ ksi} \]

\[ b_b := H_c \]

Joint depth

\[ b_b = 48 \text{ in} \]

\[ h_c := D \]

Dimension of column

\[ h_c = 45 \text{ in} \]

\[ b_{je} := 2h_c \]

effective joint width for shear stress calculations
Title: Pulaski County Bridge

Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

NCHRP Guidelines

bje = 90 in

\[ v_{hv} := \frac{M_{p, bot} \cdot L}{h_b \cdot h_c \cdot b_{je}} \]  
Joint shear stress

\[ v_{hv} = 2.691 \text{ ksi} \]

\[ p_t := \frac{f_h + f_v}{2} - \sqrt{\left[ \frac{f_h - f_v}{2} \right]^2 + v_{hv}^2} \]  
Principal tension stress

\[ p_t = -2.688 \]

\[ p_{t, max} := \frac{3.5}{1000} \sqrt{f_c} \]  
Maximum tension stress

\[ p_{t, max} = 0.207 \]  
Not OK, we should provide some horizontal reinforcement

\[ p_c := \frac{f_h + f_v}{2} + \sqrt{\left[ \frac{f_h - f_v}{2} \right]^2 + v_{hv}^2} \]  
Principal compression stress

\[ p_c = 2.694 \]

\[ p_{c, max} := \frac{.25}{1000} f_c \]  
Maximum compression stress

\[ p_{c, max} = 0.875 \]  
Not OK, we should provide some horizontal reinforcement

\[ A_{jv} := .16A_{sc} \]  
Vertical reinforcement (Stirrups) (Guide Spec. 8.8.4.3.2)

\[ A_{jv} = 31.36 \text{ in}^2 \]

Use #7

\[ A_{jv} = 52.267 \]  
3#7 vertical legs in (development length) each side

\[ A_{clamp} := .08A_{sc} \]  
Vertical reinforcement (Clamping reinforcement) (Guide Spec. 8.8.4.3.2)

\[ A_{clamp} = 15.68 \]
A_{clamp} = 26.133 \quad 2\#7 in 36\" of the joint

\[ l_d := 0.63 \times 1.128 \frac{f_{yh}}{f_c} \frac{\sqrt{f_c}}{1000} \]

Development length for #9 (AASHTO LRFD 5.11.2.2.1-1)

\[ l_d = 22.791 \text{ in} \]

\[ A_h := 0.08 \times A_{sc} \]

Horizontal reinforcement (Guide Spec. 8.8.4.3.3)

\[ A_h = 15.68 \]

\[ l_{d11} := 1.25 \times 1.56 \frac{f_{yh}}{f_c} \frac{\sqrt{f_c}}{1000} \]

\[ l_{d11} = 62.539 \]

\[ \rho_s := \frac{A_{sc}}{l_{d11}} \]

\[ \rho_s = 0.02 \]

\[ \rho_{s\_min} := \frac{3.5 \sqrt{f_c}}{1000 \times f_{yh}} \]

Minimum required horizontal reinforcement (Guide Spec. 8.8.4.2.2)

\[ \rho_{s\_min} = 3.451 \times 10^{-3} \quad \text{OK} \]

\[ A_{sh} := 0.2 \text{ in}^2 \]

\[ s := 4 \text{ in} \]

\[ \text{legs} := 23 \]

\[ s_{max} := \text{legs} \frac{A_{sh}}{D_{pp} \rho_s} \]

\[ s_{max} = 5.337 \text{ in} \]

Use #4@4" with 23 legs
P-∆ Requirements
(Guide Spec. 8.3.4)

Pier 2

\[ W := 525 + 2 \cdot 308 \text{ kips} \]
\[ V := 3636 \text{ kips from SAP model} \]
\[ H := \frac{256.25}{12} \text{ ft} \]
\[ H = 21.354 \text{ ft} \]
\[ C := \frac{V}{W} \]
\[ C = 3.187 \]
\[ \Delta_{\text{limit}} := 0.25 \cdot C \cdot H \]
\[ \Delta_{\text{limit}} = 17.012 \text{ ft} \]
\[ \Delta := 0.52 \text{ ft} \leq \Delta_{\text{limit}} \text{ OK} \]
Displacement of 0.52 ft is taken from SAP model

Pier 1&3

Average axial force at column

\[ W := 1197 \text{ kips} \]
\[ V := 2448 \text{ kips} \]
\[ H := \frac{L}{12} \text{ ft} \]
\[ H = 21.125 \text{ ft} \]
\[ C := \frac{V}{W} \]
\[ C = 2.045 \]
\[ \Delta_{\text{limit}} := 0.25 \cdot C \cdot H \]
Title: Pulaski County Bridge

Minimum seat requirement

\[ \Delta_{\text{limit}} = 10.801 \text{ ft} \]
\[ \Delta = 0.122 \text{ ft} < \Delta_{\text{limit}} \quad \text{OK} \]

Displacement of 0.122 ft is taken from SAP model

\[ N = 3.541 \text{ ft} \]

\(43^{\circ}\) Minimum seat width
Moment check at the cantilever part of the wall pier

\[ M_{eq} := 99 \text{ k-ft} \]
\[ M_{DL} := 495 \text{ k-ft} \]
\[ R := 1.0 \]
\[ M := \frac{M_{eq}}{R} + M_{DL} \]
\[ M = 594 \text{ k-ft} \]
\[ b := 4.12 \text{ in} \]
\[ h := 4.5 \text{-}12 \text{ in} \]
\[ d := h - 4 \]
\[ A := h \cdot b \text{ Section area at hammer part of the wall pier} \]
\[ A = 2.592 \times 10^3 \text{ in}^2 \]
\[ \rho := \left( \frac{4}{2.5} \right) \cdot (10) \cdot \frac{1}{A} \text{ The existing ratio of steel has increased by the ratio of new wall (4' thick) to the existing wall thickness (2.5')} \]
\[ \rho = 6.173 \times 10^{-3} \]
\[ R := \frac{M \cdot 12000}{b \cdot d^2} \]
\[ R = 59.4 \]
\[ \rho_{req} := 0.0025 \text{ OK} \]
Pile cap design procedure for Pulasky County bridge

\[ f_c = 3500 \text{ psi} \]
\[ F_y = 60000 \text{ psi} \]

Pile punching shear check:

\[ P_{axial} = 245 \text{ kips} \text{ (from sheet 30)} \]

\[ V_u = \frac{P_{axial}}{\phi} \]
\[ V_u = 245 \text{ Kips} \]
\[ d = 45 \text{ in} \]
\[ b = (14.2 + d) \cdot 4 \text{ in} \]

\[ V_c := 4 \sqrt{f_c} \cdot \frac{b \cdot d}{1000} \]
\[ V_c = 2.522 \times 10^3 \text{ kips} \]
\[ \phi = 0.9 \]

\[ \frac{V_u}{\phi} = 272.222 \text{ } < V_c \text{ } \text{OK} \]

One way shear Check:

\[ V_1 := 10 P_{axial} \]
\[ V_1 = 2.45 \times 10^3 \text{ Kips} \]
\[ b_2 := 55 \cdot 12 \text{ in} \]
\[ b_2 = 660 \text{ in} \]

\[ V_{c2} := 4 \sqrt{f_c} \cdot b_2 \cdot \frac{d}{1000} \]
\[ V_{c2} = 7.028 \times 10^3 \]

\[ \frac{V_1}{\phi} = 2.722 \times 10^3 \text{ } < V_{c2} \text{ } \text{OK} \]
Flexure bending design: Only in one direction

\[ \mu := 10 \cdot P_{\text{axial}} 1000 \cdot (1.5 \cdot 70) \]

\[ \mu = 2.572 \times 10^8 \text{ lb in} \]

\[ \phi := 0.9 \]

\[ y_t := \frac{d + 3}{2} \]

\[ y_t = 24 \text{ in} \]

\[ f_r := 7.5 \cdot \frac{f'_c}{f'_t} \]

\[ f_r = 443.706 \]

\[ I_g := \frac{b2 \cdot (d + 3)^3}{12} \]

\[ I_g = 6.083 \times 10^6 \text{ in}^4 \]

\[ M_{cr} := \frac{f_r I_g}{y_t} \]

\[ M_{cr} = 1.125 \times 10^8 \text{ lb in} \]

\[ \mu > 1.2M_{cr} \]

\[ \text{OK} \]

\[ As := \frac{\mu}{\phi \cdot F_y \cdot 0.9 \cdot d} \]

\[ As = 117.627 \text{ in}^2 \]

\[ As_{min} := 0.0018 \cdot (d + 3) \cdot b2 \]

\[ As_{min} = 57.024 \text{ in}^2 \]

\[ N_L := \frac{As}{79} \]

\[ N_L = 148.895 \]

In short Direction: 149 # 8

\[ As_{min} := 0.0018 \cdot (d + 3) \cdot 280 \]

\[ As_{min} = 24.192 \text{ in}^2 \]

\[ N_T := \frac{As_{min}}{44} \]

\[ N_T = 54.982 \]

In long direction: 55 # 6
Axial Capacity for uplift:

\[ S_1 := 11230 \]
\[ S_2 := 1361 \]
\[ M_{long} := 31836 \text{ k-ft} \]
\[ M_{tran} := 17848 \text{ k-ft} \]
\[ N := 40 \]
\[ V := 1065 \text{ kips} \]
\[ P_{uplift} := \frac{V}{N} - \frac{M_{long}1.5}{12S_2} - \frac{M_{tran}4.5}{12S_1} \]
\[ P_{uplift} = -219.771 \text{ kips} \]
\[ L_{pile} := \frac{(14.2\cdot2 + 14.9\cdot2)}{12} \text{ ft} \]
\[ L_{pile} = 4.85 \text{ ft} \]
\[ L_1 := 4 \text{ ft} \]
\[ L_2 := 41 \text{ ft} \]
\[ L := 45 \text{ ft} \]
\[ q_{u1} := \frac{38\cdot2000}{144} \text{ psi} \]
\[ q_{u1} = 527.778 \]
\[ q_{u2} := \frac{50\cdot2000}{144} \text{ psi} \]
Title: Pulaski County Bridge

QU2 = 694.444

f1 := 2.5\sqrt{QU1}  
\text{f1} = 57.434 \text{ psi}  
\text{Less than} .15\cdot QU1 = 79.167 \text{ psi}

f2 := 2.5\sqrt{QU2}  
\text{f2} = 65.881 \text{ psi}  
\text{Less than} .15QU2 = 109.722 \text{ psi}

QU1 := \frac{L\text{-pile}\cdot L1\cdot f1\cdot 144}{1000}  
\text{QU1} = 160.446 \text{ kips}

QU2 := \frac{L\text{-pile}\cdot L2\cdot f2\cdot 144}{1000}  
\text{QU2} = 1.886 \times 10^3 \text{ kips}

W\text{pile} := .490\cdot \left(\frac{34.4}{144}\right)\cdot L  
\text{Wpile} = 5.268 \text{ kips}

Qallow := 1.33\cdot \frac{(QU1 + QU2 - W\text{pile})}{2}

Qallow = 1.358 \times 10^3 \text{ kips}

Qallowuplift := 1.33\cdot \frac{(0.7QU1 + 0.7QU2 + W\text{pile})}{2}

Qallowuplift = 956.336 \text{ kips}

Flexural Design for uplift:

\begin{align*}
\text{Mu} & := 10\cdot \text{Puplift}\cdot 1000\cdot (1.5\cdot 70) \\
\text{Mu} & := 2.308 \times 10^8 \text{ lb in}  \\
\Phi & := 0.9  \\
y_t & := \frac{d + 3}{2}  \\
y_t & = 24 \text{ in}
\end{align*}

\begin{align*}
\text{f}_r & := 7.5\cdot \sqrt{f_c}  \\
f_r & = 443.706
\end{align*}

\begin{align*}
I_g & := \frac{b2(d + 3)^3}{12}  \\
I_g & = 6.083 \times 10^6 \text{ in}^4
\end{align*}
Title: Pulaski County Bridge

\[ M_{cr} := \frac{f_y f_g}{y_t} \]

\[ M_c = 1.125 \times 10^8 \text{ lb - in} \]

\[ \text{Mu} > 1.2 M_{cr} \quad \text{OK} \]

\[ A_s := \frac{\text{Mu}}{0.79} \]

\[ A_s = 105.533 \text{ in}^2 \]

\[ A_{sin} := 0.0018 \cdot (d + 3) \cdot b_2 \]

\[ A_{sin} = 57.024 \text{ in}^2 \]

\[ N_L := \frac{A_s}{0.79} \]

\[ N_L = 133.586 \]

In short Direction: 134 # 8

\[ A_{sin} := 0.0018 \cdot (d + 3) \cdot 280 \]

\[ A_{sin} = 24.192 \text{ in}^2 \]

\[ N_T := \frac{A_{sin}}{0.44} \]

\[ N_T = 54.982 \]

In long direction: 55 # 6
Appendix F -- Detailed Computations for Seismic Analysis and Design of the Pulaski County Bridge Using AASHTO Specifications
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SAP 2000 Model

View of SAP 2000 Model (with concrete extrusion shown):
Enlarged View of Translational and Rotational Springs:
Deformed Shape. Mode # 1. Period - 0.947 sec.

Deformed Shape. Mode # 2. Period - 0.293 sec.
Deformed Shape. Mode # 7. Period - 0.158 sec.

Deformed Shape. Mode # 9. Period - 0.102 sec.
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TABLE: Modal Participating Mass Ratios
Pulaski County Bridge - AASHTO Division 1-A

Earthquake Data:

Seismic Performance Category (SPC) - C
Bedrock Acceleration Coefficient (A) - 0.22\*g

Soil Profile Type - III
Site Coefficient (S) - 1.5

Response Spectrum

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Divisions:
- Division 1-A

Earthquake Data:

Seismic Performance Category (SPC)  -  C
Bedrock Acceleration Coefficient (A)  -  0.22\*g

Soil Profile Type  -  III
Site Coefficient (S) - 1.5

Response Spectrum

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Load Combinations

COMB1 - 100% load in X (longitudinal) direction + 30% load in Y (transverse) direction

COMB2 - 100% load in Y (transverse) direction + 30% load in X (longitudinal) direction

COMD1 - COMB1 + Dead Load

COMD2 - COMB2 + Dead Load
**Piles (L-pile results)**

Longitudinal direction of the bridge: (expansion pier)
Transverse direction of the bridge: (expansion pier)
Longitudinal direction of the bridge: (fixed pier)
Transverse direction of the bridge: (fixed pier)
Pile Capacity:

\[ A_{\text{pile}} := 26.1 \text{ in}^2 \]

Cross sectional area of HP14x89 pile

\[ Q_{\text{pile}} := 9 \cdot A_{\text{pile}}^{1.5} \]

\[ Q_{\text{pile}} = 352.35 \text{ kip} \]

Pile Capacity

Group effect

\[ d_{\text{pile}} := 14 \text{ in} \]

Diameter of HP 14x89 pile

In order to avoid a group effect in the foundation the distance between piles must be:
- no less than:

\[ D_{b,\text{pile}} := 5 \cdot d_{\text{pile}} \]

\[ D_{b,\text{pile}} = 70 \text{ in} \]

- no larger than: 8 ft.

Set \( D_{b,\text{pile}} \) equal to 6 ft - Distance between piles (on center)
Axial force on a pile.

Pier #1 & #3 (expansion)

\[ \phi := 0 \text{ degrees} \]  
Skew of the bridge

\[ R := 1 \]  
Response Modification Factor (applied in both directions)

Load Combination COMD1

\[ M_{x\text{unf}} := 646.0 \text{ k - ft} \]  
Longitudinal (unfactored)

\[ M_{y\text{unf}} := 480.1 \text{ k - ft} \]  
Transverse (unfactored)

\[ P := 516.1 \text{ kips} \]  
Axial load

\[ M_y := \frac{M_{y\text{unf}}}{R} \]  
\[ M_y = 480.1 \text{ k - ft} \]  
Longitudinal (factored)

\[ M_x := \frac{M_{x\text{unf}}}{R} \]  
\[ M_x = 646 \text{ k - ft} \]  
Transverse (factored)
The distance from center line of foundation to center of pile (ft):

Distance between piles is set to avoid group effect

\[ d_1 := 12 \quad d_2 := 6 \quad d_3 := 0 \quad n_3 := 3 \]

Sum of squares of the distances to each pile from the center line of the foundation:

\[ \Sigma d_1 := 10 \cdot \left( n_3^2 \right) \quad \Sigma d_1 = 90 \quad \text{ft}^2 \quad \text{(longitudinal)} \]

\[ \Sigma d_2 := 4 \cdot \left( d_2^2 \right) + 4 \cdot \left( d_1^2 \right) \quad \Sigma d_2 = 720 \quad \text{ft}^2 \quad \text{(transverse)} \]

Piles Loads:

\[ F_{pile1} := \frac{P}{10} + M_y \frac{n_3}{\Sigma d_1} + M_x \frac{d_1}{\Sigma d_2} \quad F_{pile1} = 78.4 \quad \text{kips} \]

\[ F_{pile5} := \frac{P}{10} + M_y \frac{n_3}{\Sigma d_1} - M_x \frac{d_1}{\Sigma d_2} \quad F_{pile5} = 56.85 \quad \text{kips} \]

\[ F_{pile6} := \frac{P}{10} - M_y \frac{n_3}{\Sigma d_1} + M_x \frac{d_1}{\Sigma d_2} \quad F_{pile6} = 46.37 \quad \text{kips} \]

\[ F_{pile10} := \frac{P}{10} - M_y \frac{n_3}{\Sigma d_1} - M_x \frac{d_1}{\Sigma d_2} \quad F_{pile10} = 24.84 \quad \text{kips} \]

Pile Capacity is 352.35 kips
Load Combination COMD2

\[ M_{x\text{unf.}} := 2153.4 \text{ k ft} \quad \text{Longitudinal (unfactored)} \]
\[ M_{y\text{unf.}} := 144.0 \text{ k ft} \quad \text{Transverse (unfactored)} \]
\[ P := 504.2 \text{ kips} \quad \text{Axial load} \]
\[ M_y := \frac{M_{y\text{unf.}}}{R} \quad M_y = 144 \text{ k ft} \quad \text{Longitudinal (factored)} \]
\[ M_x := \frac{M_{x\text{unf.}}}{R} \quad M_x = 2.153 \times 10^3 \text{ k ft} \quad \text{Transverse (factored)} \]

The distance from center line of foundation to center of pile (ft):

\[ d_1 := 12 \quad d_2 := 6 \quad d_3 := 0 \quad n_3 := 3 \]

Sum of squares of the distances to each pile from the center line of the foundation:

\[ \Sigma d_1 := 10 \cdot (n_3^2) \quad \Sigma d_1 = 90 \text{ ft}^2 \quad \text{(longitudinal)} \]
\[ \Sigma d_2 := 4 \cdot (d_2^2) + 4 \cdot (d_1^2) \quad \Sigma d_2 = 720 \text{ ft}^2 \quad \text{(transverse)} \]

Piles Loads:

\[ F_{\text{pile1}} := \frac{P}{10} + M_y \frac{n_3}{\Sigma d_1} + M_x \frac{d_1}{\Sigma d_2} \]
\[ F_{\text{pile5}} := \frac{P}{10} + M_y \frac{n_3}{\Sigma d_1} - M_x \frac{d_1}{\Sigma d_2} \]
\[ F_{\text{pile6}} := \frac{P}{10} - M_y \frac{n_3}{\Sigma d_1} + M_x \frac{d_1}{\Sigma d_2} \]
\[ F_{\text{pile10}} := \frac{P}{10} - M_y \frac{n_3}{\Sigma d_1} - M_x \frac{d_1}{\Sigma d_2} \]

\[ F_{\text{pile1}} = 91.1 \text{ kips} \]
\[ F_{\text{pile5}} = 19.33 \text{ kips} \]
\[ F_{\text{pile6}} = 81.51 \text{ kips} \]
\[ F_{\text{pile10}} = 9.73 \text{ kips} \]

Capacity is 352.35 kips
**Axial Force on a Pile.**

**Pier #2 (fixed)**

\[
\phi := 0 \quad \text{degrees} \quad \text{Skew of the bridge}
\]

\[
R := 1 \quad \text{Response Modification Factor (applied in both directions)}
\]

**Load Combination COMD1**

(from SAP 2000; file: (FINAL) springs4)

- \( M_{x\text{unf}} := 1108.4 \quad \text{k - ft} \quad \text{Longitudinal (unfactored)} \)
- \( M_{y\text{unf}} := 11784.1 \quad \text{k - ft} \quad \text{Transverse (unfactored)} \)
- \( P := 646.9 \quad \text{kips} \quad \text{Axial load} \)

\[
M_y := \frac{M_{y\text{unf}}}{R} \quad M_y = 1.178 \times 10^4 \quad \text{k - ft} \quad \text{Longitudinal (factored)}
\]

\[
M_x := \frac{M_{x\text{unf}}}{R} \quad M_x = 1.108 \times 10^3 \quad \text{k - ft} \quad \text{Transverse (factored)}
\]

- \( L := 27 \quad \text{ft} \)
- \( W := 14.5 \quad \text{ft} \)
The distance from center line of foundation to center of pile (ft):

\[ d_1 := 12 \quad d_2 := 6 \quad d_3 := 0 \quad n_3 := 6 \]

Sum of squares of the distances to each pile from the center line of the foundation:

\[ \Sigma d_1 := 10 \cdot \left( n_3^2 \right) \]
\[ \Sigma d_2 := 6 \cdot \left( d_2^2 \right) + 6 \cdot \left( d_1^2 \right) \]

Piles Loads:

\[ F_{pile1} := \frac{P}{10} + M_y \frac{n_3}{\Sigma d_1} + M_x \frac{d_1}{\Sigma d_2} \]
\[ F_{pile5} := \frac{P}{10} + M_y \frac{n_3}{\Sigma d_1} - M_x \frac{d_1}{\Sigma d_2} \]
\[ F_{pile11} := \frac{P}{10} - M_y \frac{n_3}{\Sigma d_1} + M_x \frac{d_1}{\Sigma d_2} \]
\[ F_{pile15} := \frac{P}{10} - M_y \frac{n_3}{\Sigma d_1} - M_x \frac{d_1}{\Sigma d_2} \]

Pile Capacity is 352.35 kips
Load Combination COMD2 (from SAP 2000; file: (FINAL) springs4)

\[ M_{x\text{unf.}} := 3694.7 \quad \text{k-ft} \quad \text{Longitudinal (unfactored)} \]
\[ M_{y\text{unf.}} := 3535.2 \quad \text{k-ft} \quad \text{Transverse (unfactored)} \]
\[ P := 646.9 \quad \text{kips} \quad \text{Axial load} \]
\[ M_y := \frac{M_{y\text{unf.}}}{R} \quad M_y = 3.535 \times 10^3 \quad \text{k-ft} \quad \text{Longitudinal (factored)} \]
\[ M_x := \frac{M_{x\text{unf.}}}{R} \quad M_x = 3.695 \times 10^3 \quad \text{k-ft} \quad \text{Transverse (factored)} \]

The distance from center line of foundation to center of pile (ft):

**Distance between piles is set to avoid group effect**

\[ d_1 := 12 \quad d_2 := 6 \quad d_3 := 0 \quad n_3 := 6 \]

Sum of squares of the distances to each pile from the center line of the foundation:

\[ \Sigma d_1 := 10 \cdot \left( n_3^2 \right) \quad \Sigma d_1 = 360 \quad \text{ft}^2 \quad \text{(longitudinal)} \]
\[ \Sigma d_2 := 6 \cdot \left( d_2^2 \right) + 6 \cdot \left( d_1^2 \right) \quad \Sigma d_2 = 1.08 \times 11 \text{ft}^2 \quad \text{(transverse)} \]

Piles Loads:

\[ F_{\text{pile1}} := \frac{P}{10} + M_y \frac{n_3}{\Sigma d_1} + M_x \frac{d_1}{\Sigma d_2} \quad F_{\text{pile1}} = 164.7 \quad \text{kips} \]
\[ F_{\text{pile5}} := \frac{P}{10} - M_y \frac{n_3}{\Sigma d_1} - M_x \frac{d_1}{\Sigma d_2} \quad F_{\text{pile5}} = 82.56 \quad \text{kips} \]
\[ F_{\text{pile11}} := \frac{P}{10} - M_y \frac{n_3}{\Sigma d_1} + M_x \frac{d_1}{\Sigma d_2} \quad F_{\text{pile11}} = 46.82 \quad \text{kips} \]
\[ F_{\text{pile15}} := \frac{P}{10} - M_y \frac{n_3}{\Sigma d_1} - M_x \frac{d_1}{\Sigma d_2} \quad F_{\text{pile15}} = -35.282 \quad \text{kips} \]

**Pile Capacity is 352.35 kips**
Combined axial load and bending (AASHTO sixteenth edition):
(Article 10.54.2.1 Maximum Capacity; equation 10-156, p.330)

HP 14x89 pile properties:

\[
F_y := 36 \text{ ksi} \\
A_{\text{pile}} := 26.1 \text{ in}^2 \\
Z_x := 67.7 \text{ in}^3 \\
Z_y := 146.0 \text{ in}^3 \\
M_{px} := F_y Z_x \\
M_{px} = 2.437 \times 10^3 \text{ k-in} \\
M_{py} := F_y Z_y \\
M_{py} = 5.256 \times 10^3 \text{ k-in}
\]

All axes are Global to SAP 2000 model

e.g., designation "longitudinal" and "transverse" refer to location of
Global coordinate system in SAP model.
**Pier #1 & #3 (expansion)**

**Load combination: COMD 1**

Maximum axial force on a pile:

(As determined in "Axial force on a pile")

\[ P := 78.4 \text{ k} \text{ axial load (single pile)} \]

Maximum Loads (from LPILE output):

\[ M_{xLpile} := 32.7 \text{ k-in moment about X-axis (single pile)} \]

\[ M_{yLpile} := 84.0 \text{ k-in moment about Y-axis (single pile)} \]

\[ R_p := 1 \text{ Response Modification Factor for piles (SPC - C)} \]

\[
\left( \frac{P}{0.85 \cdot A_{pile} \cdot F_y} \right) + \left[ \frac{M_{xLpile}}{M_{px}} \right] + \left[ \frac{M_{yLpile}}{M_{py}} \right] = 0.128
\]

*In order to satisfy AASHTO design requirements the resultant of this equation must be less than or equal 1.*
Pier #1 & #3 (expansion)

Load combination: COMD 2

Maximum axial force on a pile:
(as determined in "Axial force on a pile")

\[ P := 91.1 \text{ k } \text{ axial load (single pile)} \]

Maximum Loads (from LPILE output):

\[ M_{xLpile} := 32.7 \text{ k-in } \text{ moment about X-axis (single pile)} \]

\[ M_{yLpile} := 84.0 \text{ k-in } \text{ moment about Y-axis (single pile)} \]

\[ R_p := 1 \] Response Modification Factor for piles (SPC - C)

\[
\left( \frac{P}{0.85 \cdot F_y} \right) + \left( \frac{M_{xLpile}}{M_{px}} \right) + \left( \frac{M_{yLpile}}{M_{py}} \right) = 0.143
\]

In order to satisfy AASHTO design requirements the resultant of this equation must be less than or equal 1.
Pier #2 (fixed)

Load combination: COMD 1

Maximum axial force on a pile:
(as determined in “Axial force on a pile”)

\[ P := 273.4 \text{k} \text{ axial load (single pile)} \]

Maximum Loads (from LPILE output):

\[ M_{x_{Lpile}} := 205.4 \text{k in} \text{ moment about X-axis (single pile)} \]

\[ M_{y_{Lpile}} := 251.6 \text{k in} \text{ moment about Y-axis (single pile)} \]

\[ R_p := 1 \text{ Response Modification Factor for piles (SPC - C)} \]

\[ \left( \frac{P}{0.85 \cdot A_{pile} F_y} \right) + \left[ \left( \frac{M_{x_{Lpile}}}{M_{px}} \right) + \left( \frac{M_{y_{Lpile}}}{M_{py}} \right) \right] = 0.474 \]

*In order to satisfy AASHTO design requirements the resultant of this equation must be less than or equal 1.*
Pier #2 (fixed)

Load combination: COMD 2

Maximum axial force on a pile:
(as determined in "Axial force on a pile")

\[ P := 164.7 \text{ k } \text{axial load (single pile)} \]

Maximum Loads (from LPILE output):

\[ M_{xLpile} := 205.4 \text{ k-in } \text{moment about X-axis (single pile)} \]

\[ M_{yLpile} := 251.6 \text{ k-in } \text{moment about Y-axis (single pile)} \]

\[ R_p := 1 \text{ Response Modification Factor for piles (SPC - C)} \]

\[
\left( \frac{P}{0.85 \cdot A_{pile} \cdot F_y} \right) + \left[ \frac{M_{xLpile}}{M_{px}} \right] + \left[ \frac{M_{yLpile}}{M_{py}} \right] = 0.338
\]

In order to satisfy AASHTO design requirements the resultant of this equation must be less than or equal 1.
Springs

Pier #1 & #3 (expansion)

Pile Information for L-Pile input:

HP 14x89 - piles

\[ L_{\text{pile}} := 540 \text{ in} \]
\[ A_{\text{pile}} := 26.1 \text{ in}^2 \]
\[ E_S := 29000000 \frac{\text{lb}}{\text{in}^2} \]

Results from L-Pile:

Piers #1 & #3 Long.HP 14x89_(iteration3).lpd
Piers #1 & #3 Trans.HP 14x89_(iteration3).lpd

\[ \Delta_t := 0.002 \text{ in} \] Displacement (transverse)
\[ \Delta_l := 0.003 \text{ in} \] Displacement (longitudinal)

\[ V_{\text{utrans}} := 1864.3 \text{ lb} \] Transverse
\[ V_{\text{ulong}} := 3712.2 \text{ lb} \] Longitudinal

Spring Stiffness (per one pile):

\[ k_{\text{axial}} := \frac{A_{\text{pile}} \cdot E_S}{L_{\text{pile}}} \]
\[ k_{\text{axial}} = 1.402 \times 10^6 \frac{\text{lb}}{\text{in}} \] Axial

\[ k_{\text{trans}} := \frac{V_{\text{utrans}}}{\Delta_t} \]
\[ k_{\text{trans}} = 9.322 \times 10^5 \frac{\text{lb}}{\text{in}} \] Transverse

\[ k_{\text{long}} := \frac{V_{\text{ulong}}}{\Delta_l} \]
\[ k_{\text{long}} = 1.237 \times 10^6 \frac{\text{lb}}{\text{in}} \] Longitudinal
Lateral Springs:

\[ K_{\text{long}} := 10 \cdot K_{\text{long}} \]
\[ K_{\text{long}} = 1.237 \times 10^7 \frac{\text{lb}}{\text{in}} \quad \text{Along X axis} \quad (K_u) \]
\[ K_{\text{trans}} := 10 \cdot K_{\text{trans}} \]
\[ K_{\text{trans}} = 9.322 \times 10^6 \frac{\text{lb}}{\text{in}} \quad \text{Along Y axis} \quad (K_u) \]

Axial Springs:

\[ K_{\text{axial}} := 10 \cdot K_{\text{axial}} \]
\[ K_{\text{axial}} = 1.402 \times 10^7 \frac{\text{lb}}{\text{in}} \quad \text{Along Z axis} \quad (K_u) \]

NOTE: Distance between piles is set to avoid group effect
### Rotational Springs:

Distance between piles (transverse) inches | Distance between footing C.L. and piles C.L. (transverse) inches | Distance between footing C.L. and piles C.L. (longitudinal) inches | Distance from center of footing to center of piles; inches
---|---|---|---
\( l_{12} := 72 \) | \( d_1 := 144 \) | \( w := 36 \) | \( n_1 := \sqrt{d_1^2 + w^2} \)
\( l_{23} := 72 \) | \( d_2 := 72 \) | \( n_2 := \sqrt{d_2^2 + w^2} \)
\( d_3 := 0 \) | \( n_3 := \sqrt{d_3^2 + w^2} \)

\( K_{rx} := k_{axial} \left[ 4 \left( d_1 \right)^2 + 4 \left( d_2 \right)^2 \right] \)
\( K_{ry} := k_{axial} \cdot 10 \cdot n_3^2 \)
\( K_{rz} := k_{long} \left( 4 \cdot d_1^2 + 4 \cdot d_2^2 \right) + k_{trans} \left( 10n_3 \right)^2 \)

\( K_{rx} = 1.453 \times 10^{11} \text{ lb-in rad} \) about X axis \( (K_{r1}) \)
\( K_{ry} = 1.817 \times 10^{10} \text{ lb-in rad} \) about Y axis \( (K_{r2}) \)
\( K_{rz} = 2.491 \times 10^{11} \text{ lb-in rad} \) about Z axis \( (K_{r3}) \)
Pier #2 (fixed)

Pile Information for L-Pile input:

\[ \text{HP 14x89 - piles} \]
\[ L_{\text{pile}} := 540 \text{ in} \]
\[ A_{\text{pile}} := 26.1 \text{ in}^2 \]
\[ E_s := 29000000 \frac{\text{lb}}{\text{in}^2} \]

Results from L-Pile:

Pier #2 Long.HP 14x89_(iteration4).lpd
Pier #2 Trans.HP 14x89_(iteration4).lpd

\[ \Delta_t := 0.015 \text{ in} \quad \text{Displacement (transverse)} \]
\[ \Delta_l := 0.01 \text{ in} \quad \text{Displacement (longitudinal)} \]
\[ V_{\text{trans}} := 10250.2 \text{ lb} \quad \text{Transverse} \]
\[ V_{\text{long}} := 10248.6 \text{ lb} \quad \text{Longitudinal} \]

Spring Stiffness (per one pile):

\[ k_{\text{axial}} := A_{\text{pile}} \frac{E_s}{L_{\text{pile}}} \quad k_{\text{axial}} = 1.402 \times 10^6 \frac{\text{lb}}{\text{in}} \quad \text{Axial} \]
\[ k_{\text{trans}} := \frac{V_{\text{trans}}}{\Delta_t} \quad k_{\text{trans}} = 6.833 \times 10^5 \frac{\text{lb}}{\text{in}} \quad \text{Transverse} \]
\[ k_{\text{long}} := \frac{V_{\text{long}}}{\Delta_l} \quad k_{\text{long}} = 1.025 \times 10^6 \frac{\text{lb}}{\text{in}} \quad \text{Longitudinal} \]
Lateral Springs:

\[ K_{\text{long}} := 15 \cdot k_{\text{long}} \quad K_{\text{long}} = 1.537 \times 10^7 \, \text{lb/in} \quad \text{Along X axis} \quad (K_{u1}) \]

\[ K_{\text{trans}} := 15 \cdot k_{\text{trans}} \quad K_{\text{trans}} = 1.025 \times 10^7 \, \text{lb/in} \quad \text{Along Y axis} \quad (K_{u2}) \]

Axial Springs:

\[ K_{\text{axial}} := 15 \cdot k_{\text{axial}} \quad K_{\text{axial}} = 2.103 \times 10^7 \, \text{lb/in} \quad \text{Along Z axis} \quad (K_{u3}) \]

**NOTE:** Distance between piles is set to avoid group effect.
Rotational Springs:

<table>
<thead>
<tr>
<th>Distance between piles (transverse) inches</th>
<th>Distance between footing C.L. and piles C.L. (transverse) inches</th>
<th>Distance between footing C.L. and piles C.L. (longitudinal) inches</th>
<th>Distance from center of footing to center of piles; inches</th>
</tr>
</thead>
<tbody>
<tr>
<td>( l_{12} = 72 )</td>
<td>( d_1 = 144 )</td>
<td>( w = 72 )</td>
<td>( n_1 = \sqrt{d_1^2 + w^2} )</td>
</tr>
<tr>
<td>( l_{23} = 72 )</td>
<td>( d_2 = 72 )</td>
<td></td>
<td>( n_2 = \sqrt{d_2^2 + w^2} )</td>
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<tr>
<td></td>
<td>( d_3 = 0 )</td>
<td></td>
<td>( n_3 = \sqrt{d_3^2 + w^2} )</td>
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</tbody>
</table>

\[
K_{rx} := k_{axial} \left[ 6 \cdot (d_1)^2 + 6 \cdot (d_2)^2 \right] \quad K_{rx} = 2.18 \times 10^{11} \text{ lb-in \ over X axis (}K_{r1}\text{)}
\]

\[
K_{ry} := k_{axial} \cdot 10 \cdot n_3^2 \quad K_{ry} = 7.266 \times 10^{10} \text{ lb-in \ over Y axis (}K_{r2}\text{)}
\]

\[
K_{rz} := k_{long} \left( 6 \cdot d_1^2 + 6 \cdot d_2^2 \right) + k_{trans} \left( 10n_3^2 \right) \quad K_{rz} = 5.136 \times 10^{11} \text{ lb-in \ over Z axis (}K_{r3}\text{)}
\]
Displacements at bottom of foundation

SAP 2000 File: (FINAL)springs5

<table>
<thead>
<tr>
<th>TABLE: Joint Displacements</th>
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</tr>
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<tr>
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</tr>
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<tr>
<td>74</td>
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Joint # 66- bottom of Pier # 1
Joint # 70- bottom of Pier # 3
Joint # 74- bottom of Pier # 2

Displacements are approximately equal to those obtained from LPILIE:
Files:
Piers #1 & #3 Long.HP 14x89_(iteration3).lpd
Piers #1 & #3 Trans.HP 14x89_(iteration3).lpd
Pier #2 Long.HP 14x89_(iteration4).lpd
Pier #2 Trans.HP 14x89_(iteration4).lpd
**L-Pile output**  
See Appendices for actual L-Pile output

| Path to file locations: Z:\ITRC research\Pulaski C.B\Lpile\Design\ |  
| Name of input data file: Piers #1 & #3 Long.HP 14x89_(iteration3).lpd |  
| |  
| BC | Boundary | Boundary | Axial | Pile Head | Maximum | Maximum |  
| Type | Condition | Condition | Load | Deflection | Moment | Shear |  
| 1 | 2 | lbs | in | in-lbs | lbs |  
| 5 | y= | .003000 | S= | 0.000 | 51597.0000 | .003000 | -84023.6868 | 3712.2222 |  
| Path to file locations: Z:\ITRC research\Pulaski C.B\Lpile\Design\ |  
| Name of input data file: Piers #1 & #3 Trans.HP 14x89_(iteration3).lpd |  
| |  
| BC | Boundary | Boundary | Axial | Pile Head | Maximum | Maximum |  
| Type | Condition | Condition | Load | Deflection | Moment | Shear |  
| 1 | 2 | lbs | in | in-lbs | lbs |  
| 5 | y= | .002000 | S= | 0.000 | 51597.0000 | .002000 | -32745.3507 | 1864.3300 |  
| Path to file locations: Z:\ITRC research\Pulaski C.B\Lpile\Design\ |  
| Name of input data file: Pier #2 Long.HP 14x89_(iteration4).lpd |  
| |  
| BC | Boundary | Boundary | Axial | Pile Head | Maximum | Maximum |  
| Type | Condition | Condition | Load | Deflection | Moment | Shear |  
| 1 | 2 | lbs | in | in-lbs | lbs |  
| 5 | y= | .010000 | S= | 0.000 | 43126.0000 | .010000 | -2.516E+05 | 10248.6469 |  
| Path to file locations: Z:\ITRC research\Pulaski C.B\Lpile\Design\ |  
| Name of input data file: Pier #2 Trans.HP 14x89_(iteration4).lpd |  
| |  
| BC | Boundary | Boundary | Axial | Pile Head | Maximum | Maximum |  
| Type | Condition | Condition | Load | Deflection | Moment | Shear |  
| 1 | 2 | lbs | in | in-lbs | lbs |  
| 5 | y= | .015000 | S= | 0.000 | 43126.0000 | .015000 | -2.054E+05 | 10250.2109 |
### Forces in Substructure

**SAP 2000 model: (FINAL)springs4**

**Without Response Modification Factor**

Pulaski County (Pier #1 & #3 at neck Joints #4 & #40; at bottom Joints #5 and #30)

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<th>F2</th>
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Pulaski County (Pier #2 at neck Joint #23; at bottom Joint #14)

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### Pulaski County Bridge

**Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois**

**AASHTO Standard Specifications**

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</table>

**Response Modification Factors:**

- **Piers:**
  - R=1 Shear
  - R=2 Flexure
- **Piles:**
  - R=1 longitudinal and transverse
Spring Forces

SAP 2000 File: (FINAL)springs4

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Joint # 66- bottom of Pier # 1
Joint # 70- bottom of Pier # 3
Joint # 74- bottom of Pier # 2
Design checks for Pier #1 and #3 (expansion)

Load Combination COMD2

Deep beam design (Reinforcement concrete, Nawy, 4rd edition, section 6.9)

Shear design for pier #1& #3 (expansion)

\[ f_c := 3500 \text{ psi} \]
\[ f_y := 60000 \text{ psi} \]
\[ h := 15.5 \text{ ft} \quad \text{-Depth of beam} \]
\[ b := 2.5 \text{ ft} \quad \text{-Width of beam} \]
\[ d := 0.95 \cdot h \quad \text{(assumed)} \]
\[ d = 14.725 \text{ ft} \quad \text{-Distance from reinforcement to extreme compression fiber} \]
\[ l := 18.25 \text{ ft} \quad \text{-Hight of beam} \]
\[ \Phi := 0.85 \quad \text{(for shear)} \]
\[ V := 60.1 \text{ k} \]
\[ M_{\text{un}} := 2183.6 \text{ k-ft unfactored} \]
\[ R := 2 \]
\[ M := \frac{M_{\text{un}}}{R} \quad \text{factored} \]
\[ \frac{1}{d} = 1.239 < 2.5 \quad \text{- criteria determining Deep Beam} \]

For \( \frac{1}{d} < 2.0 \), use following eq.

\[ V_u := \Phi \left( 8 \sqrt{f_c b d} \right) \frac{144}{1000} \]

\[ V_u = 2.133 \times 10^3 \quad \text{k} \quad \text{-Factored shear force} \]

\[ V = 60.1 \quad \text{k} < V_u = 2.133 \times 10^3 \quad \text{k} \quad \text{OK} \]

\[ V_c := 2 \left( \sqrt{f_c b d} \right) \frac{144}{1000} \]

\[ V_c = 627.223 \quad \text{k} \quad \text{-Nominal shear resisting force (controls)} \]

\[ V_{c,\text{max}} := 6 \sqrt{f_c b d} \frac{144}{1000} \]

\[ V_{c,\text{max}} = 1.882 \times 10^3 \quad \text{k} \quad \text{-Maximum nominal shear resisting force} \]

\[ \Phi \cdot V_c = 533.139 \quad \text{k} \]

\[ V_u = 2.133 \times 10^3 \quad \text{must be less or equal than} \quad \Phi \left( V_c + V_s \right) \]

\[ \Phi \cdot V_c < V_u \quad \text{-Shear reinforcement is required} \]

\[ V_u - \Phi \cdot V_c \leq \Phi \cdot V_s \]

\[ V_s := \frac{V_u}{\Phi} - V_c \]

\[ V_s = 1.882 \times 10^3 \quad \text{k} \]
\[ A_{s1} := 1.56 \text{ in}^2 \] Cross sectional area of # 11 rebar

\[ A_l := A_{s1} \frac{2}{144} \] \[ A_l = 0.022 \text{ ft}^2 \] Area of trans. rebars placed at distance \( s_{tr} \)

\[ A_{str} := 0.44 \text{ in}^2 \] Cross sectional area of # 6 rebar

\[ A_{tr} := A_{str} \frac{2}{144} \] \[ A_{tr} = 6.111 \times 10^{-3} \text{ ft}^2 \] Area of long. rebars placed at distance \( s_l \)

\[ s_{trmax} := \frac{d}{5} \] \[ s_{trmax} = 2.945 \text{ ft} \] -Maximum distance between rebars in transverse direction (on center) or 1.5 ft -use this value

\[ s_h := 1.5 \text{ ft} \] -Distance between horizontal (transverse) rebars (on center)

\[ t_r := 10.44 \text{ in} \] -Clear spacing between rebars;

\[ \frac{(t_r + 1.56)}{12} = 1 \]

\[ s_v := 1.0 \text{ ft} \] -Distance between vertical rebars (on center);PCACOL (file:#11BOT13)

\[ \left[ \frac{A_{tr}}{s_h} \right] \left[ \frac{1 + \left( \frac{h}{d} \right)}{12} \right] + \left[ \frac{A_l}{s_v} \right] \left[ \frac{11 - \left( \frac{h}{d} \right)}{12} \right] f_y \cdot d = V_s \]

\[ K_1 := \left( \frac{A_{tr}}{s_h} \right) \left[ \frac{1 + \left( \frac{h}{d} \right)}{12} \right] \] \[ K_1 = 6.969 \times 10^{-4} \]

\[ K_2 := \left( \frac{A_l}{s_v} \right) \left[ \frac{11 - \left( \frac{h}{d} \right)}{12} \right] \] \[ K_2 = 0.018 \]
Check for minimum steel: (pier # 1 & # 3)

Rebars # 6 placed in transverse direction on both faces of the pier

\[ A_{tr} = 6.111 \times 10^{-3} \text{ ft}^2 \] - transverse

\[ A_{trmin} := 0.0015 \cdot b \cdot s_h \]

\[ A_{trmin} = 5.625 \times 10^{-3} \text{ ft}^2 \]

\[ A_{tr} = 6.111 \times 10^{-3} \text{ ft}^2 \]

\[ A_{trmin} < A_{tr} \] - Condition is satisfied, hence \( s_h \)-distance between horizontal # 6 rebars = 1.5 ft

Rebars # 11 placed in longitudinal direction on both faces of the pier

\[ A_l = 0.022 \text{ ft}^2 \] - longitudinal

\[ A_{lmin} := 0.0025 \cdot b \cdot s_h \]

\[ A_{lmin} = 9.375 \times 10^{-3} \text{ ft}^2 \]

\[ A_l = 0.022 \text{ ft}^2 \]

\[ A_{lmin} < A_l \] - Condition is satisfied, hence \( s_v \)-distance between vertical rebars doesn’t need to exceed = 1.5 ft (actual \( s_v \) is 12 in )
Flexure design for deep beam (pier #1 & #3)

\[ \Phi := 0.9 \quad \text{for flexure} \]

\[ M = 1.092 \times 10^3 \quad \text{ft} \cdot \text{lb} \]

\[ \frac{l}{h} = 1.177 \quad > 1 \quad \text{so } j d = 0.2(l + 2h) \]

\[ j := \frac{0.2(l + 2h)}{d} \]

\[ j = 0.669 \]

\[ j \cdot d = 9.85 \quad \text{ft} \]

Minimum reinforcement area

\[ A_s := \frac{M \cdot 144}{\Phi \cdot f_y \cdot j \cdot d \cdot 144 \cdot 1000} \quad A_s = 2.053 \quad \text{in}^2 \]

\[ A_{s\text{min1}} := \max \left( 3 \cdot \sqrt{f_c} \cdot b \cdot d \cdot 144 \cdot f_y \right) \quad A_{s\text{min1}} = 15.681 \quad \text{in}^2 \]

\[ A_{s\text{min2}} := \frac{200 \cdot b \cdot d \cdot 144}{f_y} \quad A_{s\text{min2}} = 17.67 \quad \text{in}^2 \quad \text{-controls} \]

\[ y := 0.25 \cdot h - 0.05 \cdot l \]

\[ y = 2.963 \quad \text{ft} \]

-Depth over which flex. reinf. is to be distributed (in transverse direction)

\[ s_v = 1 \quad \text{ft} \]

-Distance between vertical rebars (on center)

as acquired from PCACOL design (file:#11TBOT13)

\[ n_{tr} := \frac{y}{s_v} \cdot 2 + 2 \]

\[ n_{tr} = 7.925 \]

-number of rebars resisting tension (in transverse dir.) as acquired from PCACOL design (file:#11BOT13)
As1 := 1.56 in²  - cross section area of #11 rebar

As := As1 · n_tr  As = 12.363 in²  - Total cross section area of rebars available to resist flexure

A_s = 12.363 in² must be equal or greater than A_{smin2} = 17.67 in²  not satisfactory

As1 := 1.56 in²  - cross section area of #11 rebar

N_{req} := \frac{A_{smin2}}{A_{s1}}

N_{req} = 11.327  - number of #11 bars to be placed within distance y = 2.963 ft from extreme tension fiber
Pile cap design

Pier # 1 & # 3 (expansion)

- $f_c := 3500$ psi
- $f_y := 60000$ psi
- $P_{axial} := 91.1$ k - Vertical force at top of most loaded pile
- $d_{bar8} := 1.0$ in - Diameter of # 8 bar
- $a_{bar8} := 0.79$ in$^2$ - Area of # 8 bar
- $a_{bar11} := 1.56$ in$^2$ - Area of #11 bar
- $d_p := 13.83$ in - Cross sectional dimensions of HP 14x89
- $b_f := 14.695$ in
- $b_{pc} := 26.5 \cdot 12$ in - Length of pile cap
- $w_{pc} := 8.5 \cdot 12$ in - Width of pile cap
- $h := 36$ in - Thickness of the pile cap
- $d_{emb} := 12$ in - Depth of pile embedment
- $d_{aver} := h - 3 - d_{bar8}$ - Average depth of the footing slab (from top to bottom reinforcement)
- $d_{aver} := 32$ in
- $\phi := 0.85$ - for shear
- $\Phi := 0.9$ - for flexure
Pile punching shear check:

\[ Vu := P_{axial} \]
\[ Vu = 91.1 \text{ k} \]
\[ b_o := \left( d_p + d_{aver} \right)^2 + \left( b_f + d_{aver} \right)^2 \]
\[ b_o = 185.05 \text{ in} \quad \text{Perimeter of critical cross section} \]

\[ \beta_c := \frac{b_f}{d_p} \]
\[ \beta_c = 1.063 \]
\[ V_{c1} := \left( 2 + \frac{4}{\beta_c} \right) \cdot \sqrt{V_c} \cdot b_o \cdot d_{aver} \]
\[ V_{c1} = 2.019 \times 10^6 \text{ k} \quad \text{Nominal shear strength} \]
\[ \alpha := 20 \]
\[ V_{c2} := \left( 2 + \frac{\alpha \cdot d_{aver}}{b_o} \right) \cdot \sqrt{V_c} \cdot b_o \cdot d_{aver} \]
\[ V_{c2} = 1.912 \times 10^6 \text{ k} \quad \text{Nominal shear strength} \]
\[ V_{c,\text{max}} := 4 \cdot \sqrt{V_c} \cdot b_o \cdot d_{aver} \cdot \frac{1000}{1000} \]
\[ V_{c,\text{max}} = 1.401 \times 10^3 \quad \text{Maximum nominal shear strength (controls)} \]

\[ \frac{Vu}{\phi} = 107.176 \text{ k} \]

\[ \frac{Vu}{\phi} < V_{c,\text{max}} \quad \text{OK} \]
One way shear check:

\[ P_{\text{axial}} = 91.1 \text{ kN} \]
\[ V_u := 5P_{\text{axial}} \]
\[ V_u = 455.5 \text{ kN} \]
\[ V_{c1} := 2 \cdot \sqrt{f_c \cdot b_p} \frac{d_{\text{aver}}}{1000} \]
\[ \phi \cdot V_{c1} = 1.023 \times 10^3 \text{ kN} \]
\[ \phi \cdot V_{c1} > V_u \quad \text{OK} \]

Two way shear check:

\[ V_{u1} := 10P_{\text{axial}} \]
\[ V_{u1} = 911 \text{ kN} \]
\[ b_o := 2 \cdot (15.5 + 2.5) \cdot 12 + d_{\text{aver}} \quad \text{in - Perimeter of the failure plane} \]
\[ b_o = 464 \text{ in} \]
\[ V_c := 4 \sqrt{f_c \cdot b_o} \frac{d_{\text{aver}}}{1000} \]
\[ V_c = 3.514 \times 10^3 \text{ kN} \]
\[ \phi \cdot V_c = 2.987 \times 10^3 \text{ kN} \]
\[ \phi \cdot V_c > V_{u1} \quad \text{OK} \]
**Flexure bending design:** (longitudinal direction; short bars)

\[
M_u := 5 \cdot (P_{\text{axial}}) \cdot 18 \cdot 1000
\]

\[
M_u = 8.199 \times 10^6 \text{ lb - in} \quad \Phi := 0.9
\]

\[
A_{s_{\text{min}}} := \frac{(200 \cdot b_{\text{pc}} \cdot d_{\text{aver}})}{f_y}
\]

\[
A_{s_{\text{min}}} = 33.92 \text{ in}^2
\]

\[
a := 0.25 \cdot d_{\text{aver}} \quad \text{(assumed)}
\]

\[
A_s := \frac{M_u}{\Phi \cdot f_y \cdot a}
\]

\[
A_s = 18.979 \text{ in}^2
\]

\[
a := \frac{(A_{s_{\text{min}}} \cdot f_y)}{0.85 \cdot f_c \cdot b_{\text{pc}}}
\]

\[
a = 2.151 \text{ in}
\]

\[
M_n := A_{s_{\text{min}}} \cdot f_y \left( d_{\text{aver}} - \frac{a}{2} \right)
\]

\[
M_n = 6.294 \times 10^7 \text{ lb - in}
\]

\[
\Phi \cdot M_n = 5.664 \times 10^7 \text{ lb - in}
\]

\[
\Phi \cdot M_n > M_u \quad \text{OK}
\]

\[
y_t := \frac{d_{\text{aver}}}{2} \quad y_t = 16 \text{ in}
\]

\[
f_r := 7.5 \cdot \sqrt{f_c} \quad f_r = 443.706 \text{ psi}
\]

\[
l_g := \frac{b_{\text{pc}} \cdot d_{\text{aver}}^3}{12} \quad l_g = 8.684 \times 10^5 \text{ in}^4
\]
\[
M_{cr} := \frac{f_r \cdot l_g}{\gamma_t}
\]

\[M_{cr} = 2.408 \times 10^7 \text{ lb in}\]

\[\phi \cdot M_n = 5.664 \times 10^7 \text{ lb in}\]

\[1.2M_{cr} = 2.89 \times 10^7 \text{ lb in}\]

\[\phi \cdot M_n > 1.2M_{cr} \quad \text{OK}\]

\[N_L := \frac{A_{smin}}{a_{bar8}}\]

\[N_L = 42.937 \quad \text{- Number of short bars}\]

Short Bars use: 43 # 8
**Flexure bending design:** (transverse direction; long bars)

\[ M_u := 2 \cdot P_{\text{axial}} \cdot 4.25 \cdot 12 \cdot 1000 \]

\[ M_u = 9.292 \times 10^6 \text{ lb – in} \quad \Phi := 0.9 \]

\[ A'_{\text{min}} := \frac{(200 \cdot w_{\text{pc}} \cdot d_{\text{aver}})}{f_y} \]

\[ A'_{\text{min}} = 10.88 \text{ in}^2 \]

\[ a := 0.25 \cdot d_{\text{aver}} \quad \text{(assumed)} \]

\[ A_s := \frac{M_u}{\Phi \cdot f_y \cdot a} \]

\[ A_s = 21.51 \text{ in}^2 \quad \text{–governs} \]

\[ a := \frac{(A_s \cdot f_y)}{0.85 \cdot f_c \cdot w_{\text{pc}}} \]

\[ a = 4.253 \text{ in} \]

\[ M_n := 40 \cdot f_y \left( d_{\text{aver}} - \frac{a}{2} \right) \]

\[ M_n = 7.17 \times 10^7 \text{ lb – in} \]

\[ \Phi \cdot M_n = 6.453 \times 10^7 \text{ lb – in} \]

\[ \Phi \cdot M_n > M_u \quad \text{OK} \]

\[ y_t := \frac{d_{\text{aver}}}{2} \quad y_t = 16 \text{ in} \]

\[ f_r := 7.5 \cdot \sqrt{f_c} \quad f_r = 443.706 \text{ psi} \]

\[ l_g := \frac{b_{\text{pc}} \cdot d_{\text{aver}}^3}{12} \quad l_g = 8.684 \times 10^5 \text{ in}^4 \]
$M_{cr} := \frac{f_r' \cdot g}{\gamma_t}$

$M_{cr} = 2.408 \times 10^7 \text{ lb} \cdot \text{in}$

$\Phi \cdot M_n = 6.453 \times 10^7 \text{ lb} \cdot \text{in}$

$1.2M_{cr} = 2.89 \times 10^7 \text{ lb} \cdot \text{in}$

$\Phi \cdot M_n > 1.2M_{cr} \quad \text{OK}$

$N_L := \frac{A_S}{\text{abar}_8}$

$N_L = 27.227 \quad \text{- Number of short bars} \quad \text{Long Bars use: 28 # 8}$

**Top reinforcement**

There is no uplift force on piles at Pier #1 and #3 (expansion), therefore shrinkage reinforcement is sufficient

$b_{pc} := 26.5 \cdot 12 \quad \text{in} \quad \text{- Length of pile cap} \quad b_{pc} = 318 \quad \text{in}$

$w_{pc} := 8.5 \cdot 12 \quad \text{in} \quad \text{- Width of pile cap} \quad w_{pc} = 102 \quad \text{in}$

$h := 36 \quad \text{in} \quad \text{- Thickness of the pile cap}$

$A_g_1 := b_{pc} \cdot h \quad A_g_1 = 1.145 \times 10^4 \text{ in}^2$

$A_{smin1} := 0.0018 \cdot A_g_1 \quad A_{smin1} = 20.606 \text{ in}^2$

$N_{short} := \frac{A_{smin1}}{\text{abar}_8} \quad N_{short} = 26.084$

Provide 27 # 8 bars at 11.5 in (on center) along short direction.
Ag2 := wpc \cdot h \quad Ag2 = 3.672 \times 10^3 \text{ in}^2

Asmin2 := 0.0018 \cdot Ag2

Asmin2 = 6.61 \text{ in}^2

Nlong := \frac{A_{smin2}}{a_{bar8}} \quad N_{long} = 8.367

Provide 9 # 8 bars at 12 in (on center) along short direction.
Design checks for Pier # 2 (fixed)

Deep beam design  (Reinforced concrete, Nawy, 4rd edition, section 6.9)

Load Combination COMD2

\[ f_c := 3500 \text{ psi} \]
\[ f_y := 60000 \text{ psi} \]
\[ h := 16.5 \text{ ft} \quad \text{-Depth of beam} \]
\[ b := 2.5 \text{ ft} \quad \text{-Width of beam} \]
\[ d := 0.95 \cdot h \text{ (assumed)} \]
\[ d = 15.675 \text{ ft} \quad \text{-Distance from reinforcement to extreme compression fiber} \]
\[ l := 21 \text{ ft} \quad \text{-Height of beam} \]
\[ \Phi := 0.85 \quad \text{- for shear} \]
\[ V := 130.7 \text{ k} \]
\[ M_{un} := 3450.6 \text{ k-ft} \]
\[ R := 2 \]
\[ M := \frac{M_{un}}{R} \text{ factored} \]
\[ \frac{l}{d} = 1.34 < 2.5 \quad - \text{criteria determining Deep Beam} \]

For \[ \frac{l}{d} < 2.0 \quad - \text{use following eq.} \]

\[ V_u := \Phi \left( 8 \sqrt{f_c b d} \right) \times \frac{144}{1000} \]

\[ V_u = 2.27 \times 10^3 \quad k \quad - \text{Factored shear force} \]

\[ V = 130.7k < V_u = 2.27 \times 10^3 \quad k \quad \text{OK} \]

\[ V_c := 2 \left( \sqrt{f_c} b d \right) \times \frac{144}{1000} \]

\[ V_c = 667.689 \quad k \quad - \text{Nominal shear resisting force (controls)} \]

\[ V_{cmax} := 6 \sqrt{f_c b d} \times \frac{144}{1000} \]

\[ V_{cmax} = 2.003 \times 10^3 \quad k \quad - \text{Maximum nominal shear resisting force} \]

\[ \Phi \cdot V_c = 567.535 \quad k \]

\[ V_u = 2.27 \times 10^3 \quad \text{must be less or equal than} \quad \Phi \left( V_c + V_s \right) \]

\[ \Phi \cdot V_c < V_u \quad - \text{Shear reinforcement is required} \]

\[ V_u - \Phi \cdot V_c \leq \Phi \cdot V_s \]

\[ V_s := \frac{V_u}{\Phi} - V_c \]

\[ V_s = 2.003 \times 10^3 \quad k \]
\[ A_{s1} := 1.56 \, \text{in}^2 \quad \text{Cross sectional area of # 11 rebar} \]

\[ A_I := \frac{A_{s1}}{144} \quad A_I = 0.022 \, \text{ft}^2 \quad \text{Area of trans. rebars placed at distance} \quad s_{tr} \]

\[ A_{str} := 0.44 \, \text{in}^2 \quad \text{Cross sectional area of # 6 rebar} \]

\[ A_{tr} := \frac{A_{str}}{144} \quad A_{tr} = 6.111 \times 10^{-3} \, \text{ft}^2 \quad \text{Area of long. rebars placed at distance} \quad s_l \]

\[ s_{trmax} := \frac{d}{s_{trmax}} = 3.135 \, \text{ft} \quad \text{-Maximum distance between rebars in transverse direction (on center)} \]

\[ s_h := 1.5 \, \text{ft} \quad \text{-Distance between horizontal (transverse) rebars (on center)} \]

\[ t_r := 10.44 \, \text{in} \quad \text{-Clear spacing between rebars;} \]

\[ \frac{(t_r + 1.56)}{12} = 1 \, \text{ft} \]

\[ s_v := 1.0 \, \text{ft} \quad \text{-Distance between vertical rebars (on center); PCACOL (file: #11BOT2)} \]

\[ \left[ \frac{A_{tr}}{s_h} \right] \frac{1 + \frac{h}{d}}{12} + \left[ \frac{A_I}{s_v} \right] \frac{11 - \frac{h}{d}}{12} \quad f_y \cdot d = V_s \]

\[ K_1 := \left( \frac{A_{tr}}{s_h} \right) \frac{1 + \frac{h}{d}}{12} \quad K_1 = 6.969 \times 10^{-4} \]

\[ K_2 := \left( \frac{A_I}{s_v} \right) \frac{11 - \frac{h}{d}}{12} \quad K_2 = 0.018 \]
Check for minimum steel: (pier # 2)

Rebars # 6 placed in transverse direction on both faces of the pier

\[ A_{tr} = 6.111 \times 10^{-3} \text{ ft}^2 \quad - \text{transverse} \]

\[ A_{trmin} := 0.0015 \cdot b \cdot s_h \]

\[ A_{trmin} = 5.625 \times 10^{-3} \text{ ft}^2 \]

\[ A_{tr} = 6.111 \times 10^{-3} \text{ ft}^2 \]

\[ A_{trmin} < A_{tr} \quad -\text{Condition is satisfied, hence } s_h \text{-distance between horizontal # 6 rebars = 1.5 ft} \]

Rebars # 11 placed in longitudinal direction on both faces of the pier

\[ A_{l} = 0.022 \text{ ft}^2 \quad - \text{longitudinal} \]

\[ A_{lmin} := 0.0025 \cdot b \cdot s_h \]

\[ A_{lmin} = 9.375 \times 10^{-3} \text{ ft}^2 \]

\[ A_{l} = 0.022 \text{ ft}^2 \]

\[ A_{lmin} < A_{l} \quad -\text{Condition is satisfied, hence } s_v \text{-distance between vertical rebars doesn't need to exceed = 1.5 ft (actual } s_v \text{ is 12 in)} \]
Flexure design for deep beam (pier # 2)

\[ \Phi := 0.9 \]

\[ M = 1.725 \times 10^3 \text{ ft} \]

\[ \frac{l}{h} = 1.273 > 1 \text{ so jd=0.2*(l+2h)} \]

\[ j := \frac{0.2(l + 2h)}{d} \]

\[ j = 0.689 \]

\[ j \cdot d = 10.8 \text{ ft} \]

Minimum reinforcement area

\[ A_s := \frac{M \cdot 144}{\Phi \cdot f_y \cdot j \cdot d \cdot 144} \]

\[ A_s = 2.958 \text{ in}^2 \]

\[ A_{smin1} := \max\left(3 \cdot \frac{\sqrt{f_c}}{f_y} \cdot b \cdot d \cdot 144\right) \]

\[ A_{smin1} = 16.692 \text{ in}^2 \]

\[ A_{smin2} := \frac{200 \cdot b \cdot d \cdot 144}{f_y} \]

\[ A_{smin2} = 18.81 \text{ in}^2 \]

- controls

\[ y := 0.25 \cdot h - 0.05 \cdot l \]

- Depth over which flex. reinf. is to be distributed (in transverse direction)

\[ y = 3.075 \text{ ft} \]

\[ s_v = 1 \text{ ft} \]

-as acquired from PCACOL design (file:#11BOT2)

\[ n_{tr} := \frac{y}{s_v} \cdot 2 + 2 \]

- number of rebars resisting tension (in transverse dir.) as acquired from PCACOL design (file:#11BOT2)
$A_{S1} := 1.56 \text{ in}^2$ - cross section area of # 11 rebar

$A_S := A_{S1} \cdot n_{tr}$

$A_S = 12.714 \text{ in}^2$ - Total cross section area of rebars available to resist flexure

$A_S = 12.714 \text{ in}^2$ must be equal or greater than $A_{smin2} = 18.81 \text{ in}^2$ not satisfactory

Number of rebars required for flexure:

$N_{req} := \frac{A_{smin2}}{A_{S1}}$

$N_{req} = 12.058$ - number of #11 bars to be placed within distance $y = 3.075 \text{ ft}$ from extreme tension fiber
Pile cap design

Pier # 2 (fixed)

\[ f_c := 3500 \text{ psi} \]
\[ f_y := 60000 \text{ psi} \]
\[ P_{axial} := 273.4 \text{ k} \]
\[ d_{bar} := 1.0 \text{ in} \]
\[ a_{bar} := 0.79 \text{ in}^2 \]
\[ d_p := 13.83 \text{ in} \]
\[ b_f := 14.695 \text{ in} \]
\[ h := 46 \text{ in} \]
\[ b_{pc} := 27.0 \cdot 12 \text{ in} \]
\[ w_{pc} := 14.5 \cdot 12 \text{ in} \]
\[ d_{emb} := 12 \text{ in} \]
\[ d_{aver} := h - 3 - d_{bar} \]
\[ d_{aver} = 42 \text{ in} \]
\[ \phi := 0.85 \]
\[ \Phi := 0.9 \]

- Vertical force at top of most loaded pile
- Diameter of # 8 bar
- Area of # 8 bar
- Cross sectional dimensions of HP 14x89
- Thickness of the pile cap
- Length of pile cap
- Width of pile cap
- Depth of pile embedment
- Average depth of the footing slab (from top to bottom reinforcement)
- for shear
- for flexure
Pile punching shear check:

\[ V_u := P_{axial} \]
\[ V_u = 273.4 \text{ k} \]

\[ b_o := \left( d_p + d_{aver}\right)^2 + \left( b_f + d_{aver}\right)^2 \]
\[ b_o = 225.05 \text{ in} \quad \text{Perimeter of critical cross section} \]

\[ \beta_c := \frac{b_f}{d_p} \quad \beta_c = 1.063 \]

\[ V_{c1} := \left( 2 + \frac{4}{\beta_c} \right) \sqrt{f_c \cdot b_o \cdot d_{aver}} \]
\[ V_{c1} = 3.223 \times 10^6 \text{ k} \quad \text{-Nominal shear strength} \]

\[ \alpha := 20 \]

\[ V_{c2} := \left( 2 + \frac{\alpha \cdot d_{aver}}{b_o} \right) \sqrt{f_c \cdot b_o \cdot d_{aver}} \]
\[ V_{c2} = 3.206 \times 10^6 \text{ k} \quad \text{-Nominal shear strength} \]

\[ V_{c,\text{max}} := 4 \cdot \sqrt{f_c \cdot b_o \cdot d_{aver}} \frac{1000}{1000} \]
\[ V_{c,\text{max}} = 2.237 \times 10^3 \text{ k} \quad \text{-Maximum nominal shear strength} \quad \text{(controles)} \]

\[ \frac{V_u}{\phi} = 321.647 \text{ k} \]
\[ \frac{V_u}{\phi} < V_{c,\text{max}} \quad \text{OK} \]
One way shear check:

\[ P_{\text{axial}} = 273.4 \text{ k} \]
\[ V_u := 5P_{\text{axial}} \]
\[ V_u = 1.367 \times 10^3 \text{ k} \]
\[ V_{c1} := 2 \cdot \sqrt{f_c \cdot b_{\text{pc}} \cdot d_{\text{aver}} \over 1000} \quad d_{\text{aver}} = 42 \text{ in} \]
\[ \phi \cdot V_{c1} = 1.369 \times 10^3 \text{ k} \]
\[ \phi \cdot V_{c1} > V_u \quad \text{OK} \]

Two way shear check:

\[ V_{u1} := 2 \cdot (6 \cdot P_{\text{axial}}) \]
\[ V_{u1} = 3.281 \times 10^3 \text{ k} \]
\[ b_o := 2 \cdot (16 + 2.5) \cdot 12 + d_{\text{aver}} \text{ in} \quad \text{- Perimeter of the failure plane} \]
\[ b_o = 486 \text{ in} \]
\[ V_c := 4 \cdot \sqrt{f_c \cdot b_o \cdot d_{\text{aver}} \over 1000} \]
\[ V_c = 4.83 \times 10^3 \text{ k} \]
\[ \phi \cdot V_c = 4.106 \times 10^3 \text{ k} \]
\[ \phi \cdot V_c > V_{u1} \quad \text{OK} \]

**Flexure bending design:** (longitudinal direction; short bars)

\[ M_u := (5P_{\text{axial}}) \cdot 1000 \cdot 57 \]
\[ M_u = 7.792 \times 10^7 \text{ lb - in} \quad \phi := 0.9 \]
TITLE:
Pulaski County Bridge

SUBJECT FILE:
Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

AASHTO Standard Specifications

BY: G.V.I.
CHECKED: W.N.M.
DATE: 6/24/2004

Sheet No: 60 of 68

\[ A_{smin} = \frac{200 \cdot b_{pc} \cdot d_{aver}}{f_y} \]

\[ A_{smin} = 45.36 \quad \text{in}^2 \]

\[ a := 0.25 \cdot d_{aver} \quad \text{(assumed)} \]

\[ A_s := \frac{M_u}{\Phi \cdot f_y \cdot a} \]

\[ A_s = 137.423 \quad \text{in}^2 \]

\[ a := \frac{(A_s \cdot f_y)}{0.85 \cdot f_c \cdot b_{pc}} \]

\[ a = 8.554 \quad \text{in} \]

\[ M_n := A_s \cdot f_y \left( d_{aver} - \frac{a}{2} \right) \]

\[ M_n = 3.11 \times 10^8 \quad \text{lb} - \text{in} \]

\[ \Phi \cdot M_n = 2.799 \times 10^8 \quad \text{lb} - \text{in} \]

\[ \Phi \cdot M_n > M_u \quad \text{OK} \]

\[ y_t := \frac{d_{aver}}{2} \quad y_t = 21 \quad \text{in} \]

\[ f_r := 7.5 \cdot \sqrt{f_c} \quad f_r = 443.706 \quad \text{psi} \]

\[ l_g := \frac{b_{pc} \cdot d_{aver}^3}{12} \quad l_g = 2 \times 10^6 \quad \text{in}^4 \]

\[ M_{cr} := \frac{f_r \cdot l_g}{y_t} \]

\[ M_{cr} = 4.227 \times 10^7 \quad \text{lb} - \text{in} \]
### Pulaski County Bridge

**Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois**

**AASHTO Standard Specifications**

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**SUBJECT FILE:**

**Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois**

**BY:**

G.V.I.__________

**CHECKED:**

W.N.M.__________

**DATE:**

6/24/2004

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\[ \Phi \cdot M_n = 2.799 \times 10^8 \quad \text{lb - in} \]

\[ 1.2M_{cr} = 5.072 \times 10^7 \quad \text{lb - in} \]

\[ \Phi \cdot M_n > 1.2M_{cr} \quad \text{OK} \]

\[ N_L := \frac{A_{smin}}{a_{bar8}} \]

\[ N_L = 57.418 \quad \text{- Number of short bars} \]

**Short Bars use: 58 # 8**

**Flexure bending design:** (transverse direction; long bars)

\[ M_u := 3 \cdot P_{axial} \cdot 1000 \cdot 4.25 \cdot 12 \]

\[ M_u = 4.183 \times 10^7 \quad \text{lb - in} \]

\[ \Phi := 0.9 \]

\[ A_{smin} := \frac{(200 \cdot w_{pc} \cdot d_{aver})}{f_y} \]

\[ A_{smin} = 24.36 \quad \text{in}^2 \]

\[ a := 0.25 \cdot d_{aver} \quad \text{(assumed)} \]

\[ A_s := \frac{M_u}{\Phi \cdot f_y \cdot a} \]

\[ A_s = 73.775 \quad \text{in}^2 \quad \text{- governs} \]

\[ a := \frac{(A_s \cdot f_y)}{0.85 \cdot f_c \cdot w_{pc}} \]

\[ a = 8.551 \quad \text{in} \]

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M_n := 40-f_y \left( \frac{d_{aver} - a}{2} \right)

M_n = 9.054 \times 10^7 \quad \text{lb - in}

\Phi \cdot M_n = 8.148 \times 10^7 \quad \text{lb - in}

\Phi \cdot M_n > M_u \quad \text{OK}

\gamma_t := \frac{d_{aver}}{2} \quad \gamma_t = 21 \quad \text{in}

f_r := 7.5 \cdot \sqrt{f_c} \quad f_r = 443.706 \quad \text{psi}

l_g := \frac{b_{pc} \cdot d_{aver}^3}{12} \quad l_g = 2 \times 10^6 \quad \text{in}^4

M_{cr} := \frac{f_r \cdot l_g}{\gamma_t}

M_{cr} = 4.227 \times 10^7 \quad \text{lb - in}

\Phi \cdot M_n = 8.148 \times 10^7 \quad \text{lb - in}

1.2M_{cr} = 5.072 \times 10^7 \quad \text{lb - in}

\Phi \cdot M_n > 1.2M_{cr} \quad \text{OK}

N_L := \frac{A_{s\text{min}}}{a_{\text{bar}8}}

N_L = 30.835 \quad \text{- Number of short bars}

Long Bars use: 30 # 6
Top reinforcement

\( F_{\text{uplift}} := 144.0 \text{ k} \) - Uplift force on a single pile (Fixed pier ; load combination COMD1)

\( n := 5 \) - Number of piles

\( \text{Marm} := 4.75 \text{ ft} \) - Moment arm

\( f_c = 3.5 \times 10^3 \text{ psi} \)

\( d_{\text{aver}} = 42 \text{ in} \) - Average depth of the footing slab (from top to bottom reinforcement)

\( \Phi := 0.9 \) - for flexure

\( h := 46 \text{ in} \) - Thickness of the pile cap

\( b_{pc} := 27.0-12 \text{ in} \) - Length of pile cap \( b_{pc} = 324 \text{ in} \)

\( w_{pc} := 14.5-12 \text{ in} \) - Width of pile cap \( w_{pc} = 174 \text{ in} \)
Provide 34 # 8 bars at 9.75 in (on center) along short direction.

\[ N_{\text{short}} := \frac{A_{\text{smin1}}}{a_{\text{bar8}}} \]

\[ N_{\text{short}} = 33.958 \]

Provide 34 # 8 bars at 9.75 in (on center) along short direction.
\[ A_{g2} := w_{pc} \cdot h \quad A_{g2} = 8.004 \times 10^3 \text{ in}^2 \]

\[ A_{smin2} := 0.0018 \cdot A_{g2} \]

\[ A_{smin2} = 14.407 \text{ in}^2 \]

\[ N_{long} := \frac{A_{smin2}}{a_{bar8}} \quad N_{long} = 18.237 \]

Provide 19 # 8 bars at 9.5 in (on center) along short direction.
PCACOL Output

Piers # 1 & # 3 (fixed). At base for load combination COMB1

Piers # 1 & # 3 (fixed). At base for load combination COMB2
Pier #2 (fixed). At base for load combination COMB1

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Pier #2 (fixed). At base for load combination COMB2
Minimum seat width

For Seismic Performance Category C

\[ L := 236.5 \text{ ft} \] - Length of bridge

\[ H := 23 \text{ ft} \] - Height of the tallest pier

\[ S := 0 \] - Angle of skew

\[ N := (12 + 0.03 \cdot L + 0.12 \cdot H) \cdot (1 + 0.000125 \cdot S^2) \]

\[ N = 21.855 \text{ in} \] - Minimum width of support
APPENDIX G - SOIL PROFILES AT ILLINOIS BRIDGE SITES
Johnson County Bridge Soil Profile

- Boring 55 (N. Abut.)
  Sta. 408+06
  Elev. 379.4 ft.

- Boring 65 (N. Pier)
  Sta. 408+39
  Elev. 379.0 ft.

- Boring 75 (S. Pier)
  Sta. 408+85
  Elev. 381.4 ft.

- Boring 85 (N. Pier)
  Sta. 409+19
  Elev. 381.7 ft.

Approx. Elev. at bottom of Abut. Cap

Fill added after Soil Exploration

Medium stiff to stiff, moist to very moist silty CLAY to stiff to very stiff CLAY (2)

Medium stiff, moist silty CLAY to soft CLAY (1)

Stiff to Hard SANDSTONE with clay binder (4)

Stiff to hard, clay SHALE (3)

Stiff to hard, clay SHALE (3)

Split Spoon REFUSAL

Hard LIMESTONE with occasional shale seams (5)
Madison-Pulaski County Bridge Soil Profile

Boring 1: S. Abut.
Elev. 418.4 ft. (Sta. 117+22)

Boring 2: S. Pier
Elev. 408.1 ft. (Sta. 118+75)

Boring 3: N. Abut.
Elev. 417 ft. (Sta. 120+66)

Boring 4: N. Pier
Elev. 412.5 ft. (Sta. 120+79)

End of Boring

Soft to med. Stiff Gray, Silty Loam with layers of silty Clay

Soft to med. Stiff Gray, Silty Loam with layers of silty Clay

Soft to med. Stiff Gray, Silty Loam with layers of silty Clay

Medium Dense to Dense Gray, Fine to Coarse Sand

Medium Dense to Dense Gray, Fine to Coarse Sand

Medium Dense to Dense Gray, Fine to Coarse Sand

Very Soft

Very Soft

Very Soft

Loose Sand

Loose Sand

Loose Sand

G-5