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

Illinois Transportation Research Center
Illinois Department of Transportation

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Technical Report Documentation Page

1. Report No.
ITRC FR 02-3

2. Government Accession No.

3. Recipient's Catalog No.

4. Title and Subtitle
Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois

5. Report Date
June 2004

6. Performing Organization Code

7. Author(s)
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8. Performing Organization Report No.

9. Performing Organization Name and Address
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10. Work Unit No. (TRAIS)

11. Sponsoring Agency Name and Address
Illinois Transportation Research Center
Southern Illinois University Edwardsville
Engineering Building, Room 3026
Edwardsville, IL 62026-1803

12. Contract or Grant No.
IA-H2, FY 02

13. Type of Report and Period Covered
Final Report
August 2002 through June 2004

14. Sponsoring Agency Code

15. Supplementary Notes

16. 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.

17. Key Words
Highway bridges, Seismic design, LRFD, NCHRP 12-49, AASHTO

18. Distribution Statement
No restrictions. This document is available to the public through the National Technical Information Service (NTIS), Springfield, Virginia 22161.

19. Security Classification (of this report)	20. Security Classification (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	510	

Illinois Transportation Research Center

ITRC Project IA-H2-02

EVALUATION OF COMPREHENSIVE SEISMIC DESIGN OF BRIDGES (LRFD) IN ILLINOIS

Final Report

June 2004

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ACKNOWLEGEMENTS

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.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

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 Unites 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^{o}-17' and a longitude of 89^{o} -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.

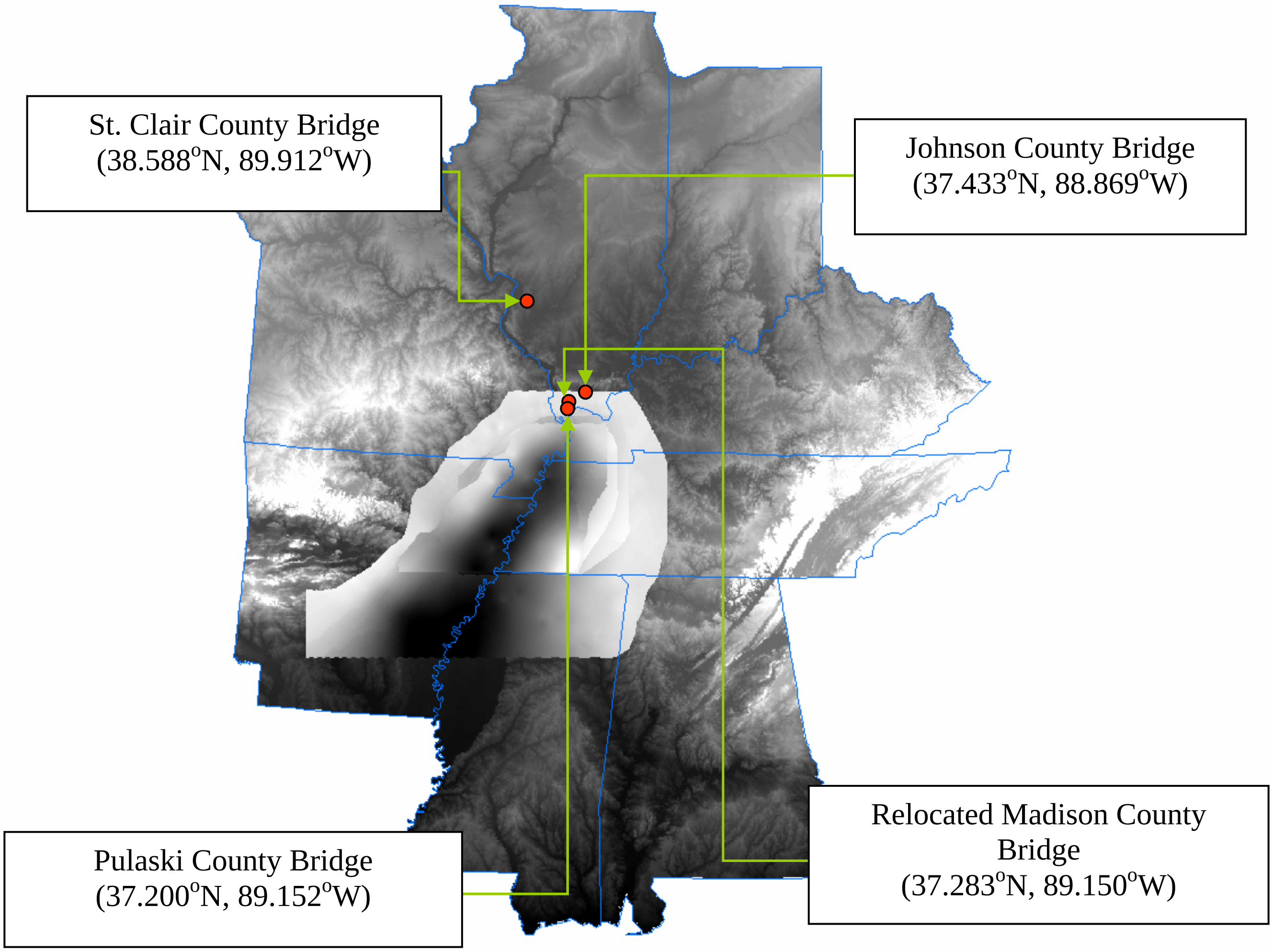


Figure 1.1 – Location of the Southern Illinois Bridge Sites Investigated

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:

- New U.S. Geological Survey (USGS) Map (1996)
- 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 eismic 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.

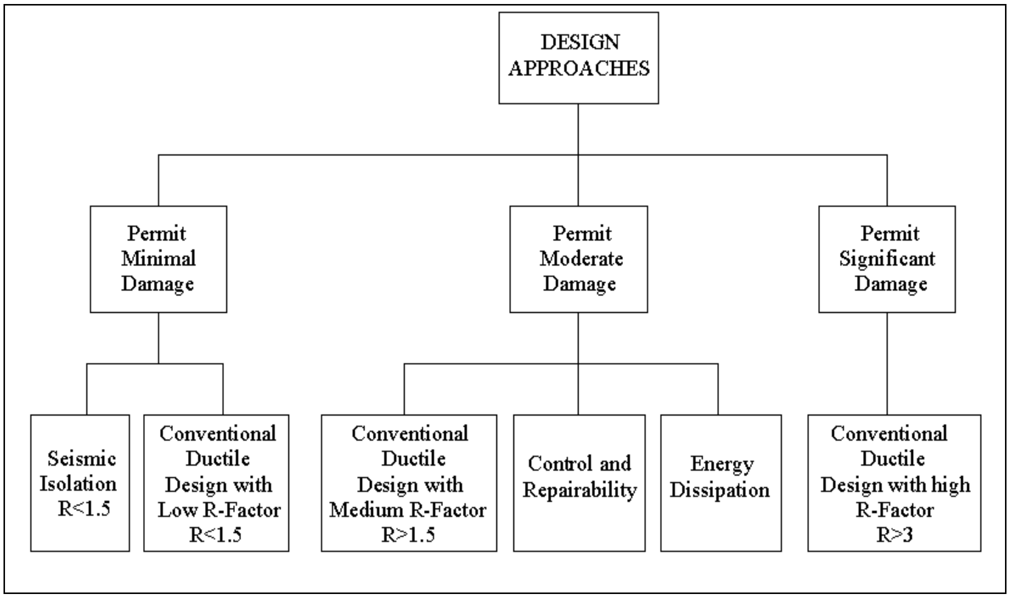


Figure 2.1 – NCHRP Design Approaches (ATC/MCEER 2002)

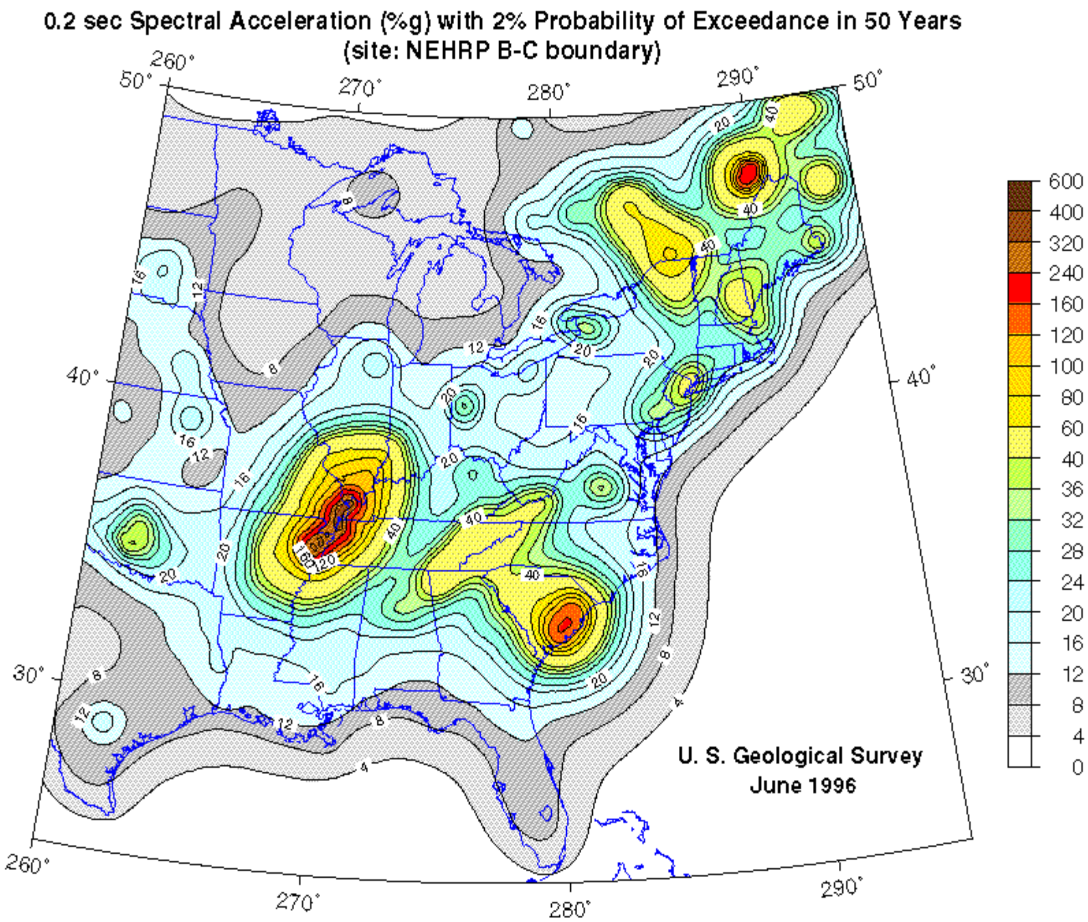


Figure 2.2 – MCE 0.2 Second Spectral Acceleration Map in Central U.S. (USGS 1996)

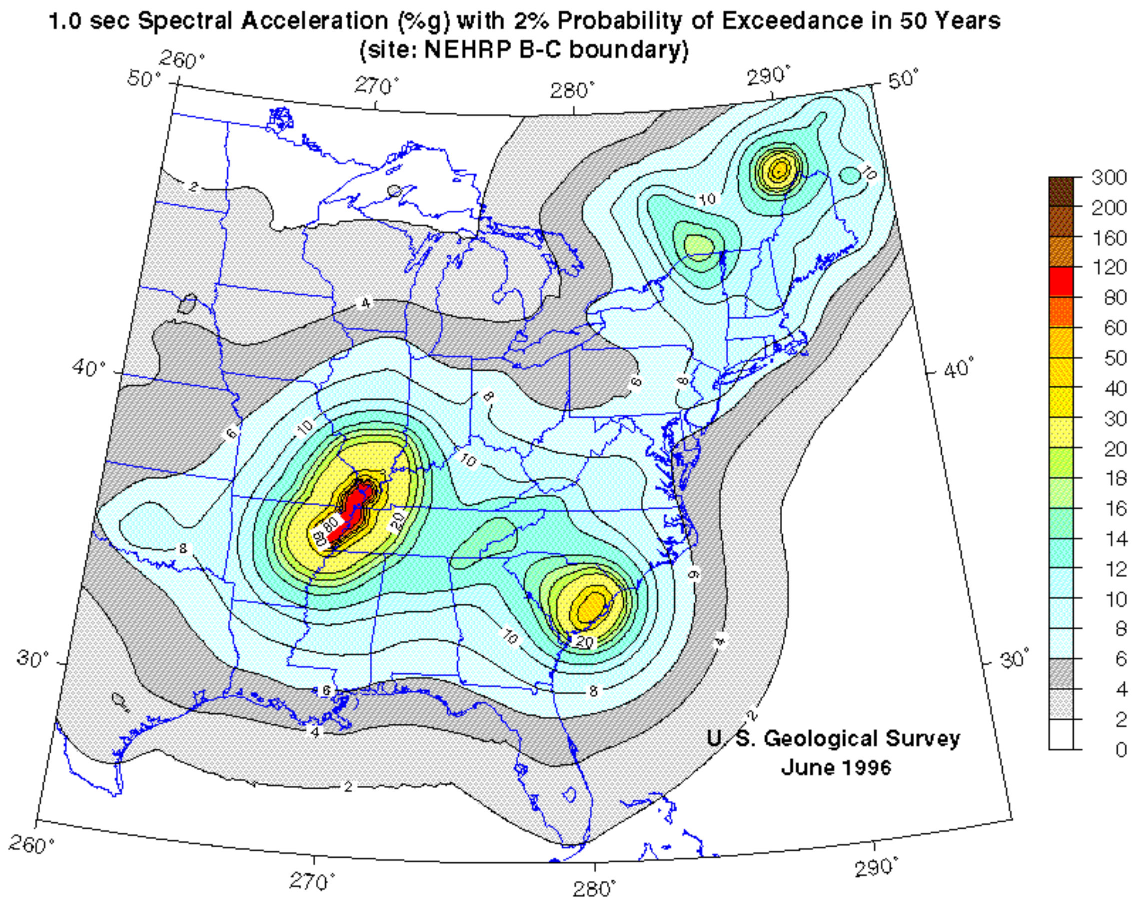


Figure 2.3 – MCE 1.0 Second Spectral Acceleration Map in Central U.S. (USGS 1996)

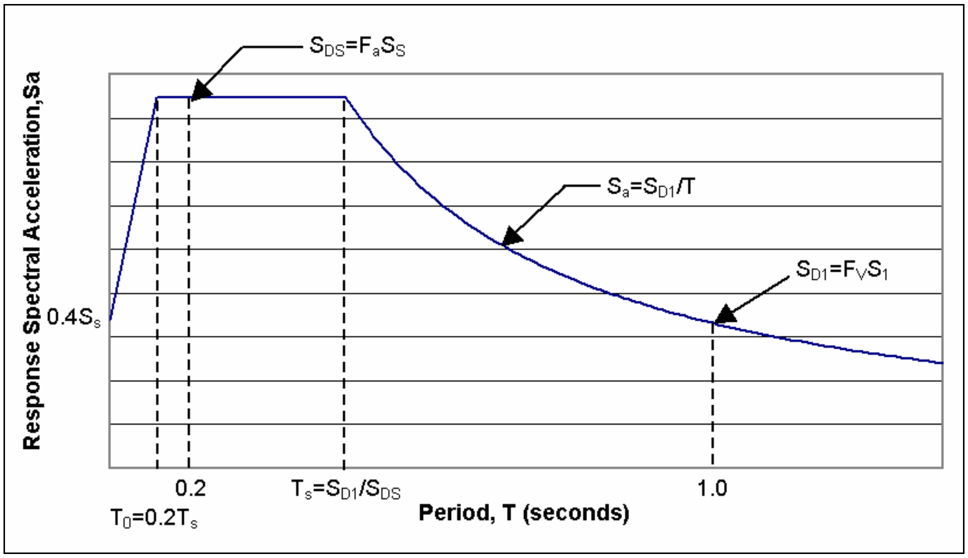


Figure 2.4 – NCHRP Design Spectrum Construction (ATC/MCEER 2002)

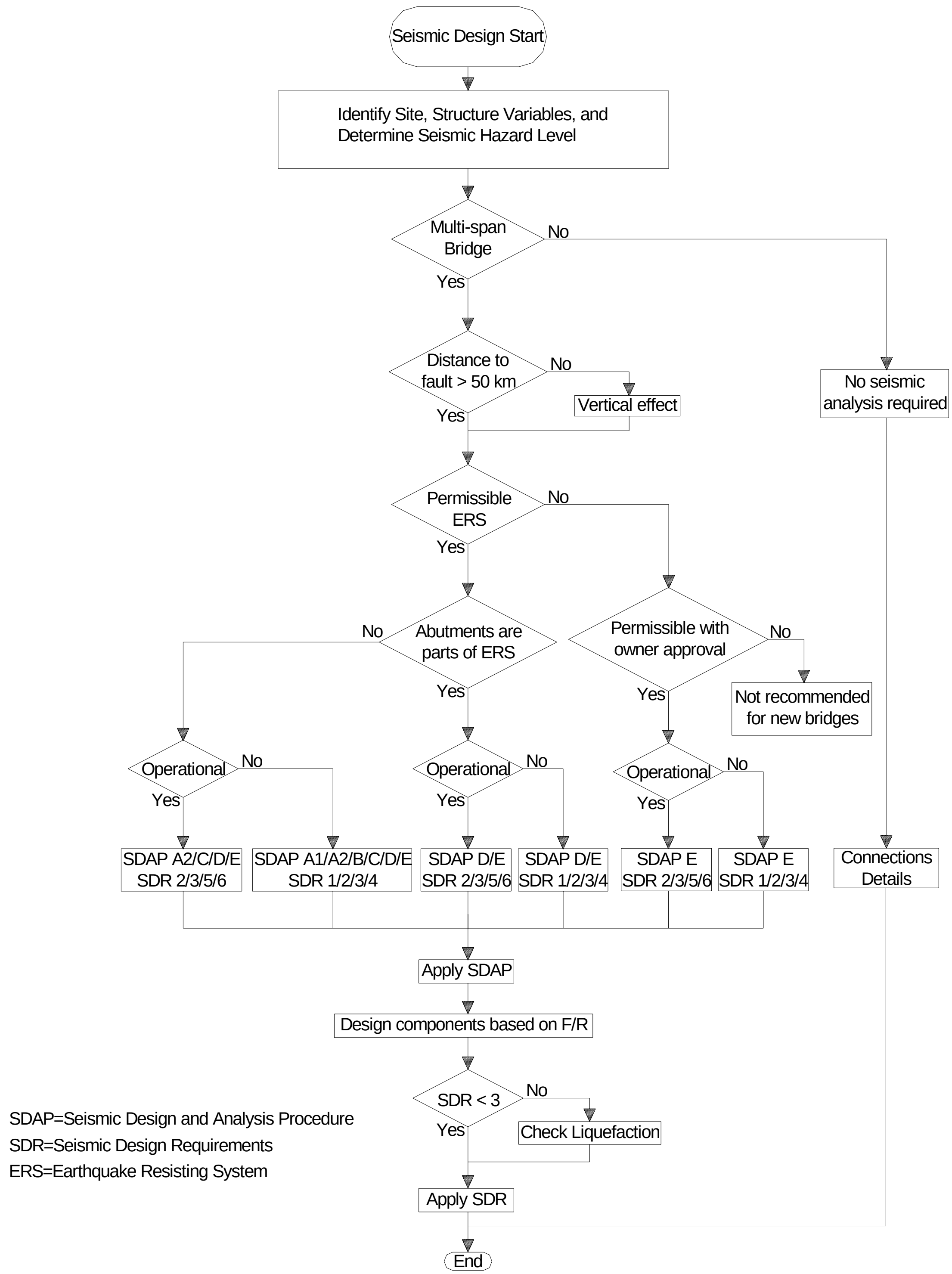


Figure 2.5 – NCHRP Bridge Seismic Design Process

Table 2.1 – NCHRP Performance Objectives (ATC/MCEER 2002)

Probability of Exceedance for Design Earthquake Ground Motions		Performance Objective Level	
		Life Safety	Operational
Rare Earthquake 3% in 75 years/1.5 Median Deterministic	Service	Significant Disruption	Immediate
	Damage	Significant	Minimal
Expected Earthquake 50% in 75 Years	Service	Immediate	Immediate
	Damage	Minimal	Minimal to none

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

Seismic Hazard Level	Value of F_vS_1	Value of F_aS_s
I	$F_vS_1 \le 0.15$	$F_aS_s \le 0.15$
II	$0.15 < F_vS_1 \le 0.25$	$0.15 < F_aS_s \le 0.35$
III	$0.25 < F_vS_1 \le 0.40$	$0.35 < F_aS_s \le 0.60$
IV	$0.40 < F_vS_1$	$0.60 < F_aS_s$

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

Seismic Hazard Level	Life Safety		Operational	
	SDAP	SDR	SDAP	SDR
I	A1	1	A2	2
II	A2	2	C/D/E	3
III	B/C/D/E	3	C/D/E	5
IV	C/D/E	4	C/D/E	6

Table 2.4 – NCHRP Minimum Analysis Requirements

SDAP	A	B	C	D	E
Minimum Analysis Requirement	Not Required	Not Required	CSDA	ERSA	ERSA with DCV

where: CSDA= Capacity Spectrum Design Approach, ERSA= Elastic Response Spectrum Analysis, and DCV= Displacement Capacity Verification

Table 2.5 – NCHRP Response Modification Factors (ATC/MCEER 2002)

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

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.

<u>3.4 Substructure Design Forces</u>

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 that 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 expansion 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.

	AASHTO	NCHRP
Concrete:	$81,450	$141,480
Epoxy Coated Rebars:	$28,500	$68,310
Drilled CIP Piles:	$13,980	$33,860
Total Cost:	**$123,930**	**$243,650**

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.

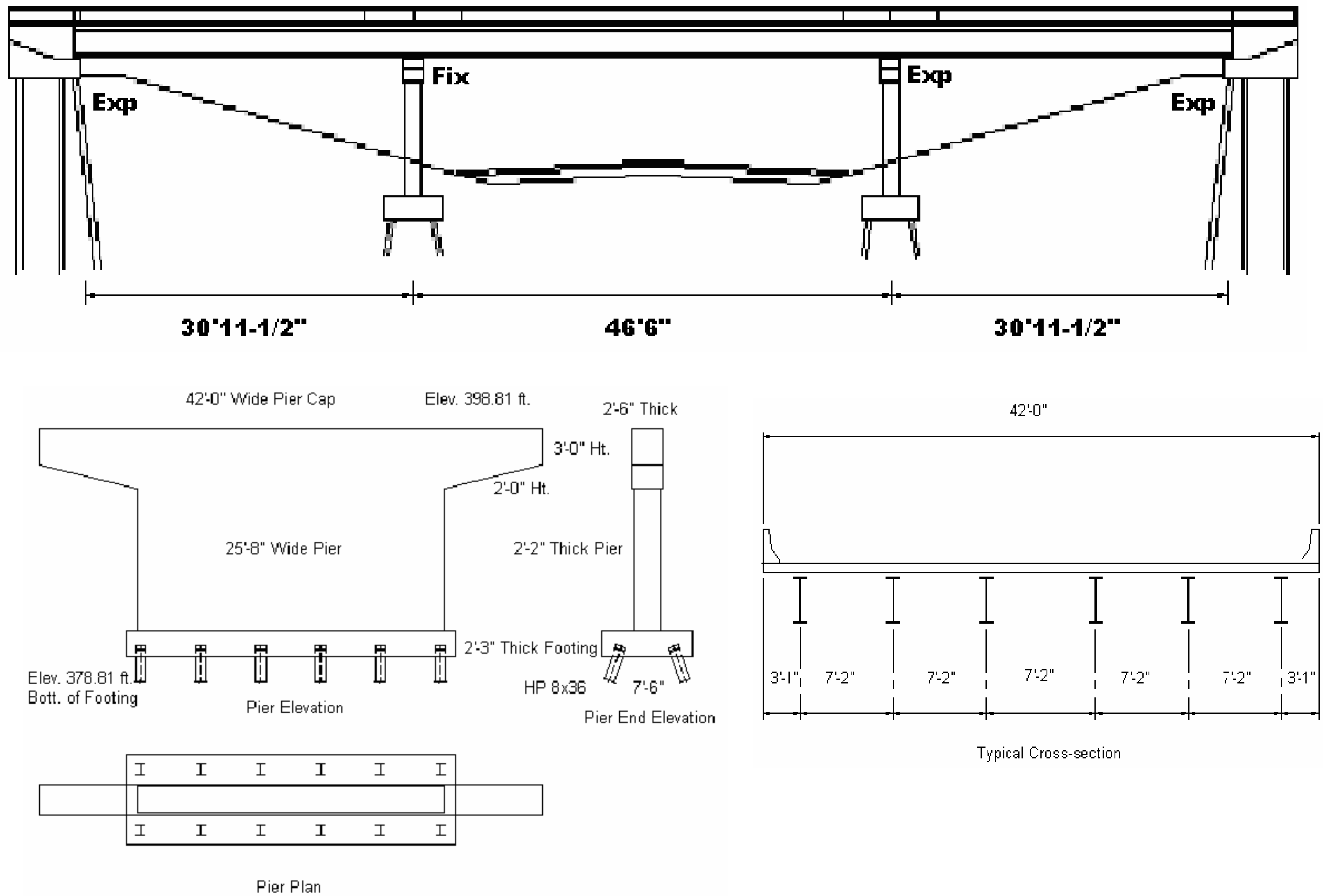


Figure 3.1 – Johnson County Existing Bridge Geometry -- Side View, Pier Elevation, and Superstructure

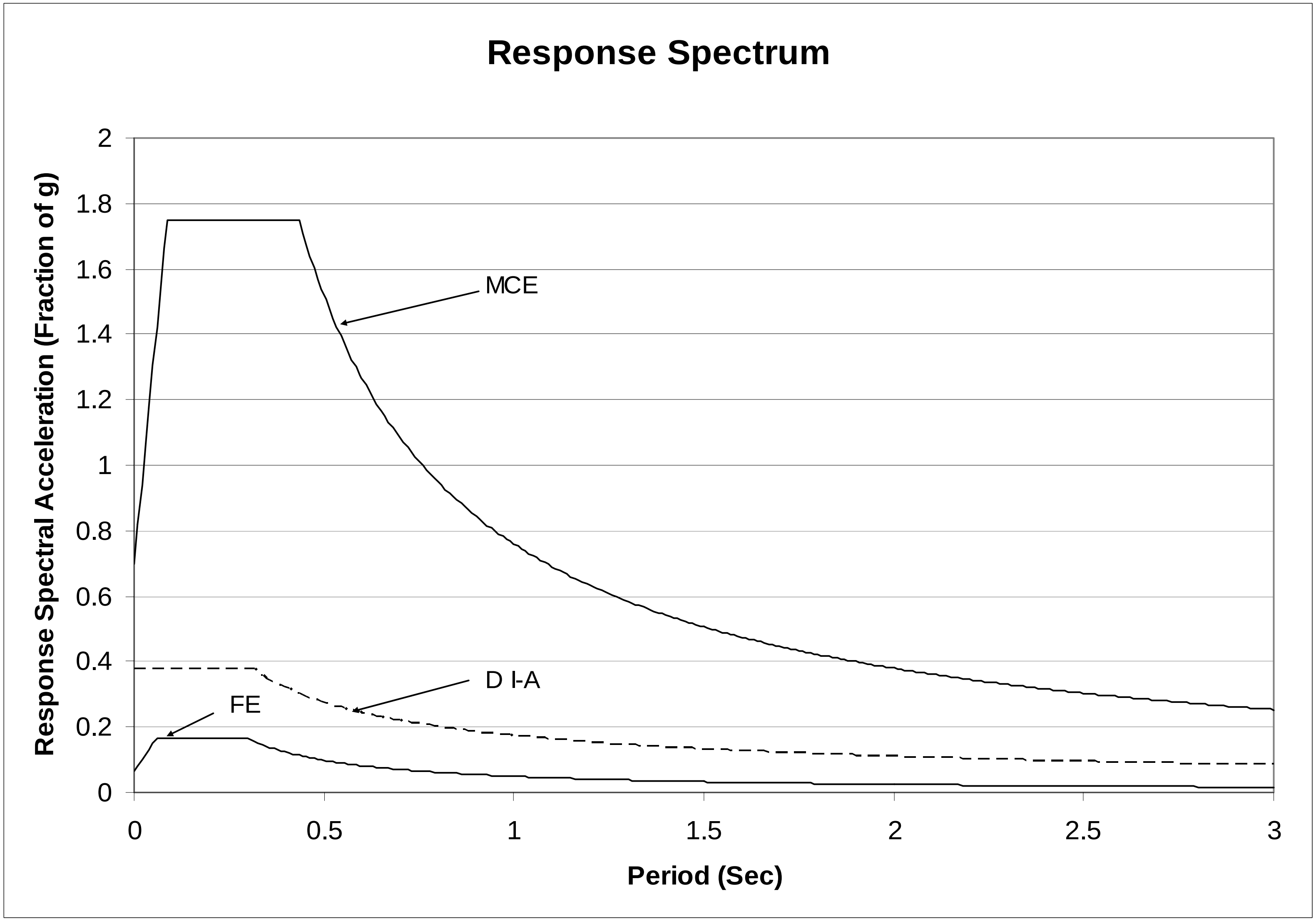
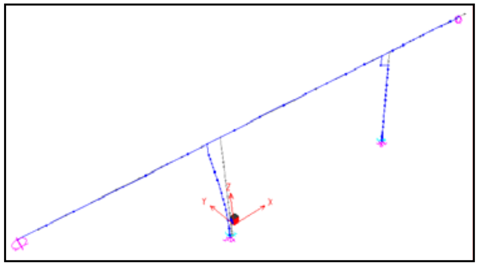
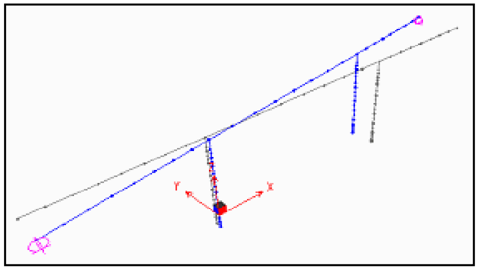


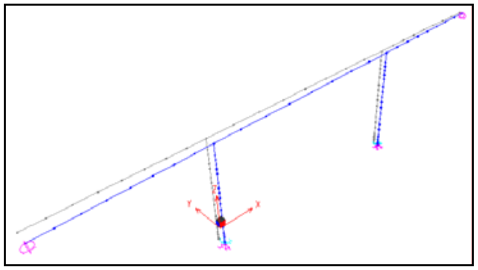
Figure 3.2 – Design Earthquake Response Spectrum for the Johnson County Bridge



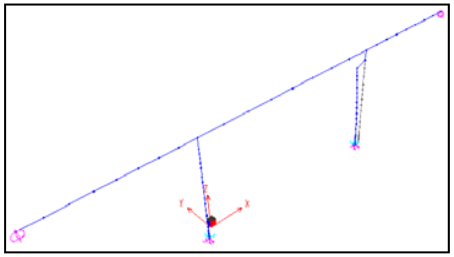
Mode Shape 1 (UX: 69%)



Mode Shape 2 (UY: 23%)



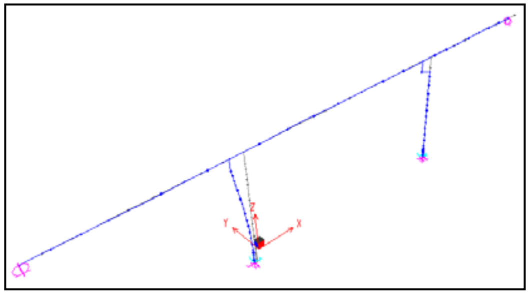
Mode Shape 3 (UY: 77%)



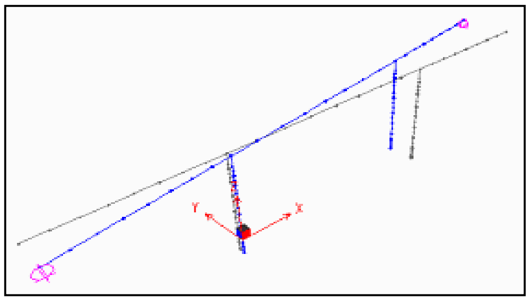
Mode Shape 4 (UX: 18%)

MODAL PARTICIPATING MASS RATIOS				
With Spring for the MCE				
MODE	PERIOD	INDIVIDUAL MODE (%)		
		UX	UY	UZ
1	0.4035	68.93	0.00	0.00
2	0.2898	0.00	22.88	0.00
3	0.2067	0.00	77.00	0.00
4	0.1683	18.24	0.00	0.00
5	0.1292	0.98	0.00	4.36
6	0.1133	6.83	0.00	0.54
7	0.0884	5.02	0.00	0.00
8	0.0687	0.00	0.00	0.25
9	0.0612	0.00	0.00	42.32
10	0.0431	0.00	0.07	0.00
11	0.0392	0.00	0.00	0.45
12	0.0389	0.00	0.00	0.00
13	0.0303	0.00	0.05	0.00
14	0.0289	0.00	0.00	25.51
15	0.0242	0.00	0.00	9.92
16	0.0239	0.00	0.00	9.35
17	0.0210	0.00	0.00	0.81
18	0.0181	0.00	0.00	0.09
19	0.0178	0.00	0.00	0.00
20	0.0171	0.00	0.00	0.00
		100.00	100.00	93.61

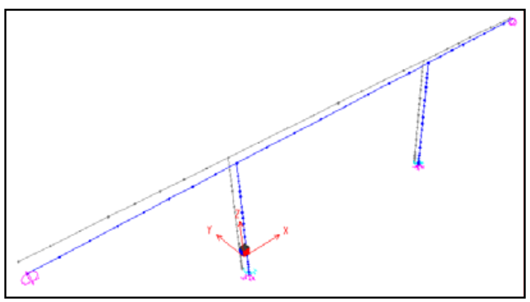
Figure 3.3 – Free Vibration Results (NCHRP MCE Model)



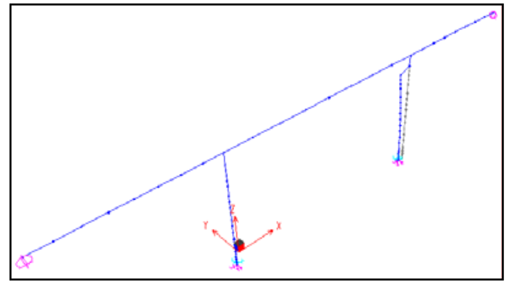
Mode Shape 1 (UX: 69%)



Mode Shape 2 (UY: 23%)



Mode Shape 3 (UY: 77%)



Mode Shape 4 (UX: 18%)

MODAL PARTICIPATING MASS RATIOS				
With Spring for the MCE				
MODE	PERIOD	INDIVIDUAL MODE (%)		
		UX	UY	UZ
1	0.4035	68.93	0.00	0.00
2	0.2898	0.00	22.88	0.00
3	0.2067	0.00	77.00	0.00
4	0.1683	18.24	0.00	0.00
5	0.1292	0.98	0.00	4.36
6	0.1133	6.83	0.00	0.54
7	0.0884	5.02	0.00	0.00
8	0.0687	0.00	0.00	0.25
9	0.0612	0.00	0.00	42.32
10	0.0431	0.00	0.07	0.00
11	0.0392	0.00	0.00	0.45
12	0.0389	0.00	0.00	0.00
13	0.0303	0.00	0.05	0.00
14	0.0289	0.00	0.00	25.51
15	0.0242	0.00	0.00	9.92
16	0.0239	0.00	0.00	9.35
17	0.0210	0.00	0.00	0.81
18	0.0181	0.00	0.00	0.09
19	0.0178	0.00	0.00	0.00
20	0.0171	0.00	0.00	0.00
		100.00	100.00	93.61

Figure 3.4 – Free Vibration Results (AASHTO Model)

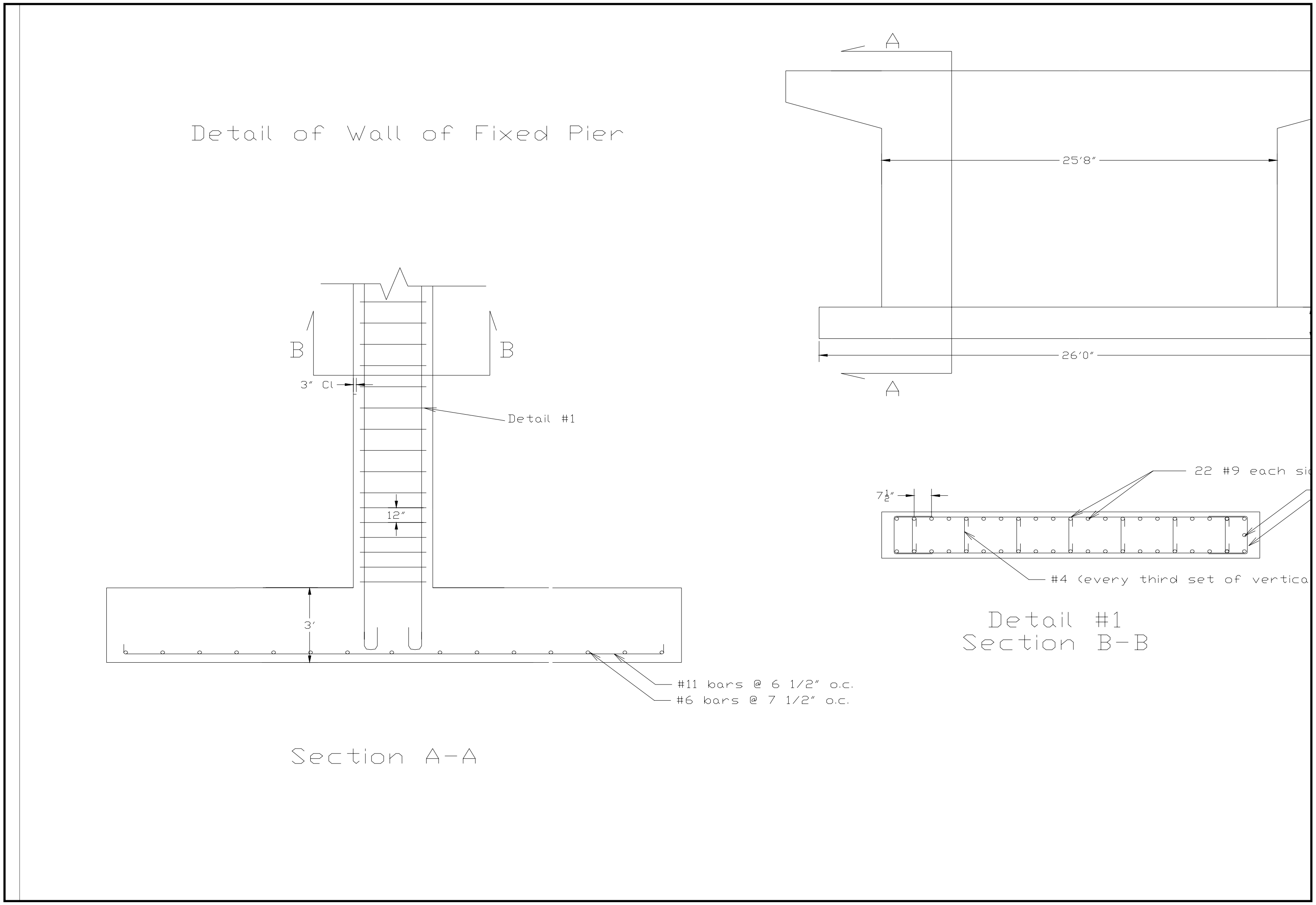


Figure 3.5 – AASHTO Wall Design Details for Fixed Bent

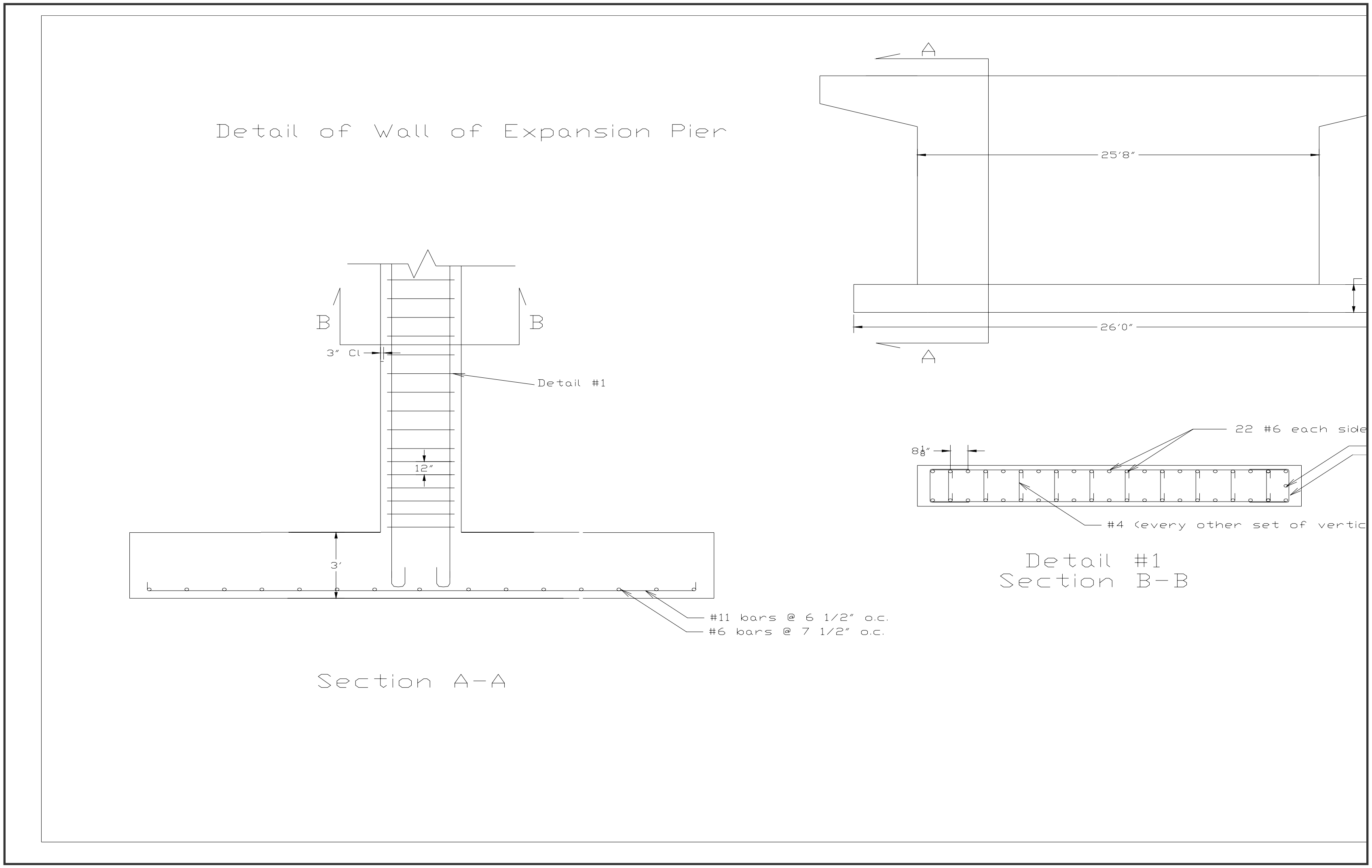


Figure 3.6 – AASHTO Wall Design Details for Expansion Bent

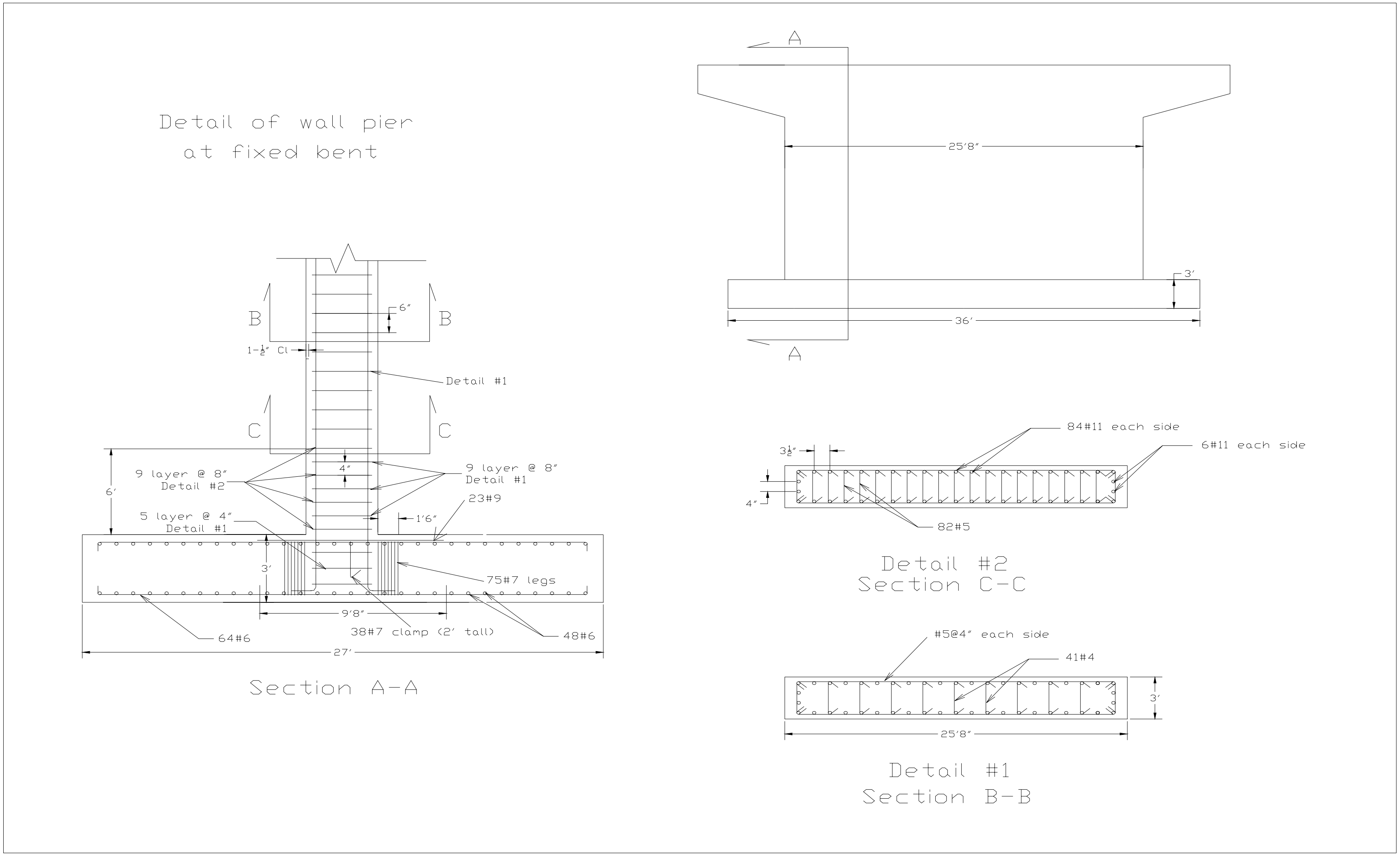


Figure 3.7 – NCHRP Wall Design Details for Fixed Bent

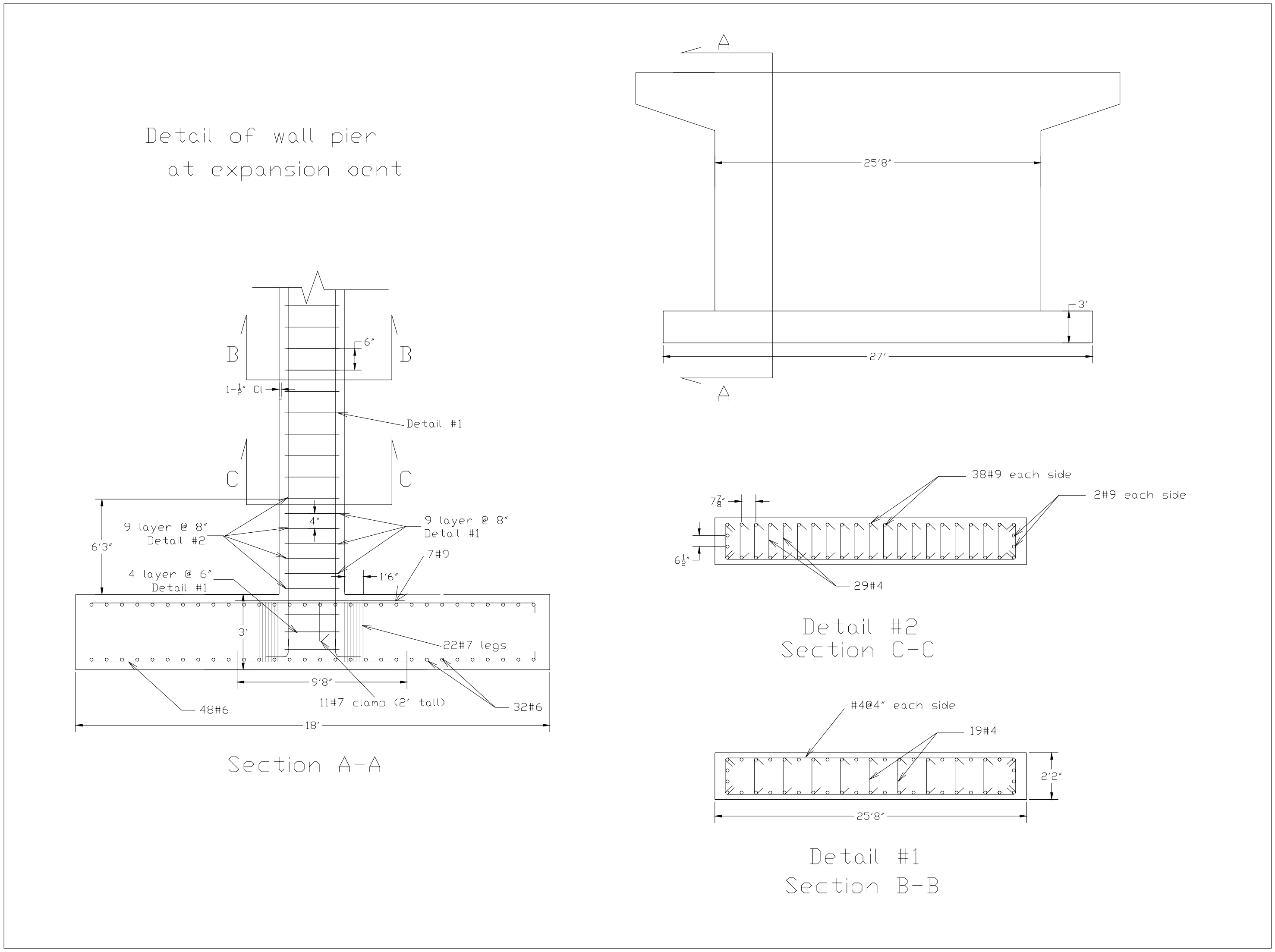


Figure 3.8 – NCHRP Wall Design Details for Expansion Bent

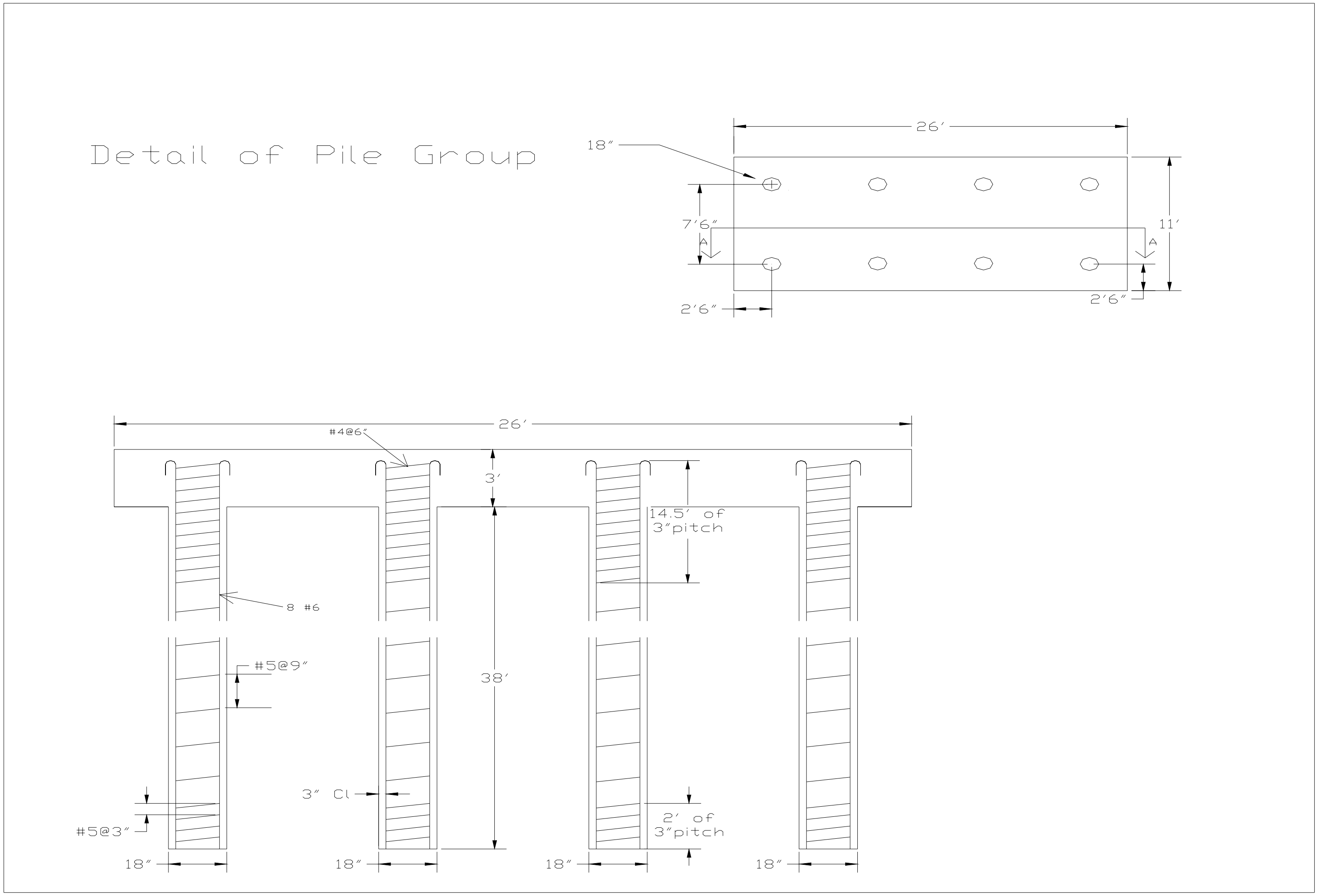


Figure 3.9 – AASHTO Foundation Design Details for Fixed and Expansion Bents

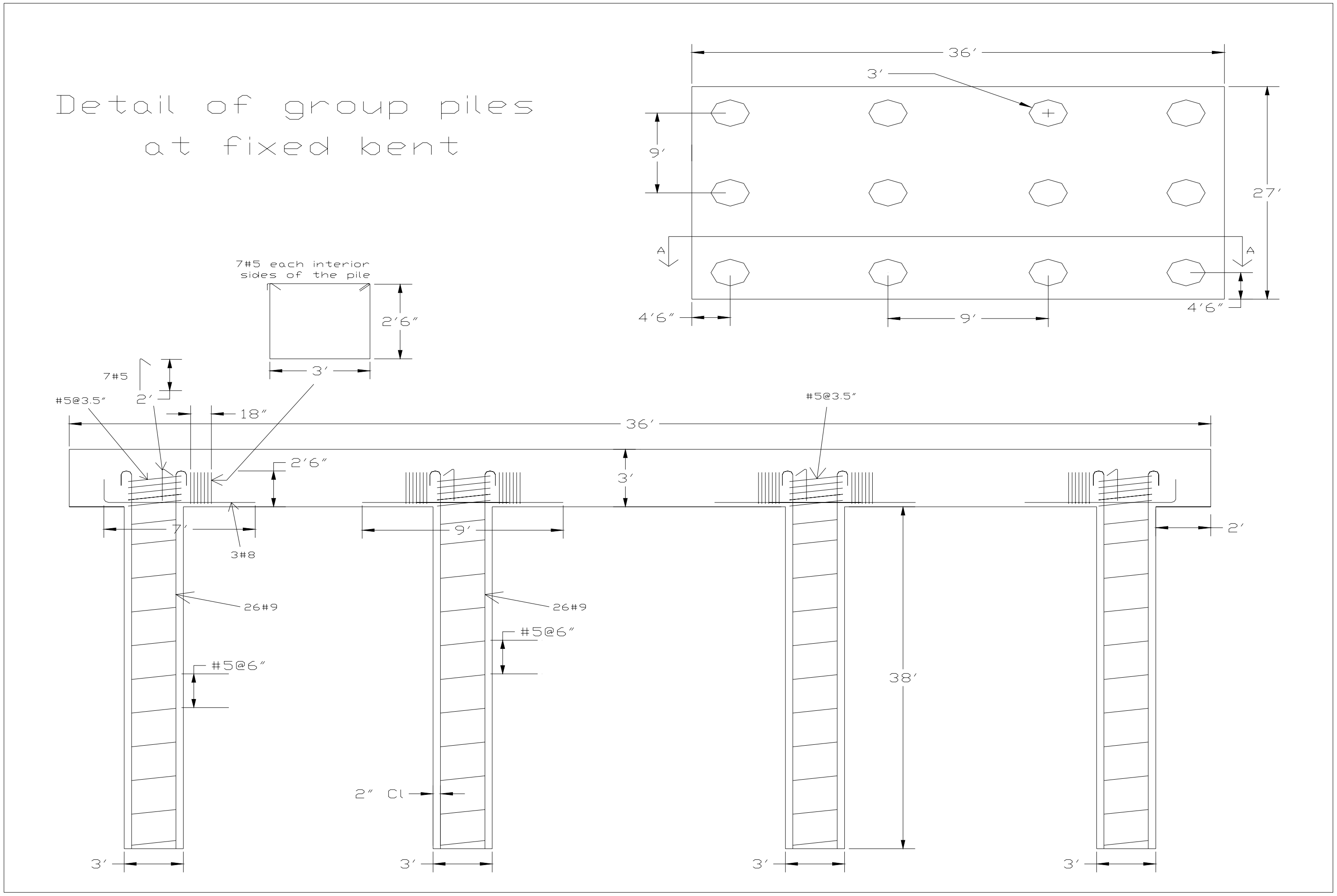


Figure 3.10 – NCHRP Foundation Design Details for Fixed Bent

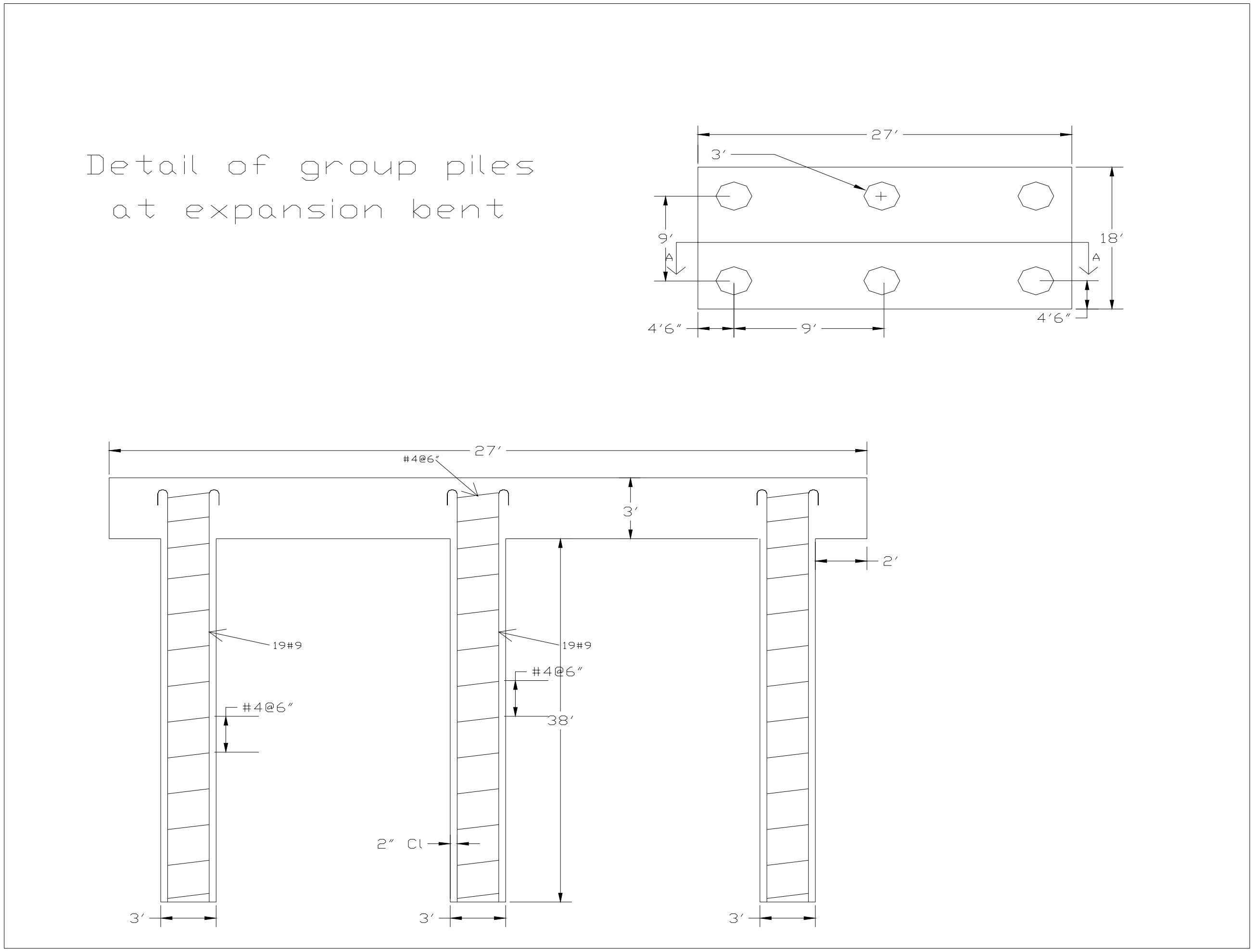


Figure 3.11 – NCHRP Foundation Design Details for Expansion Bent

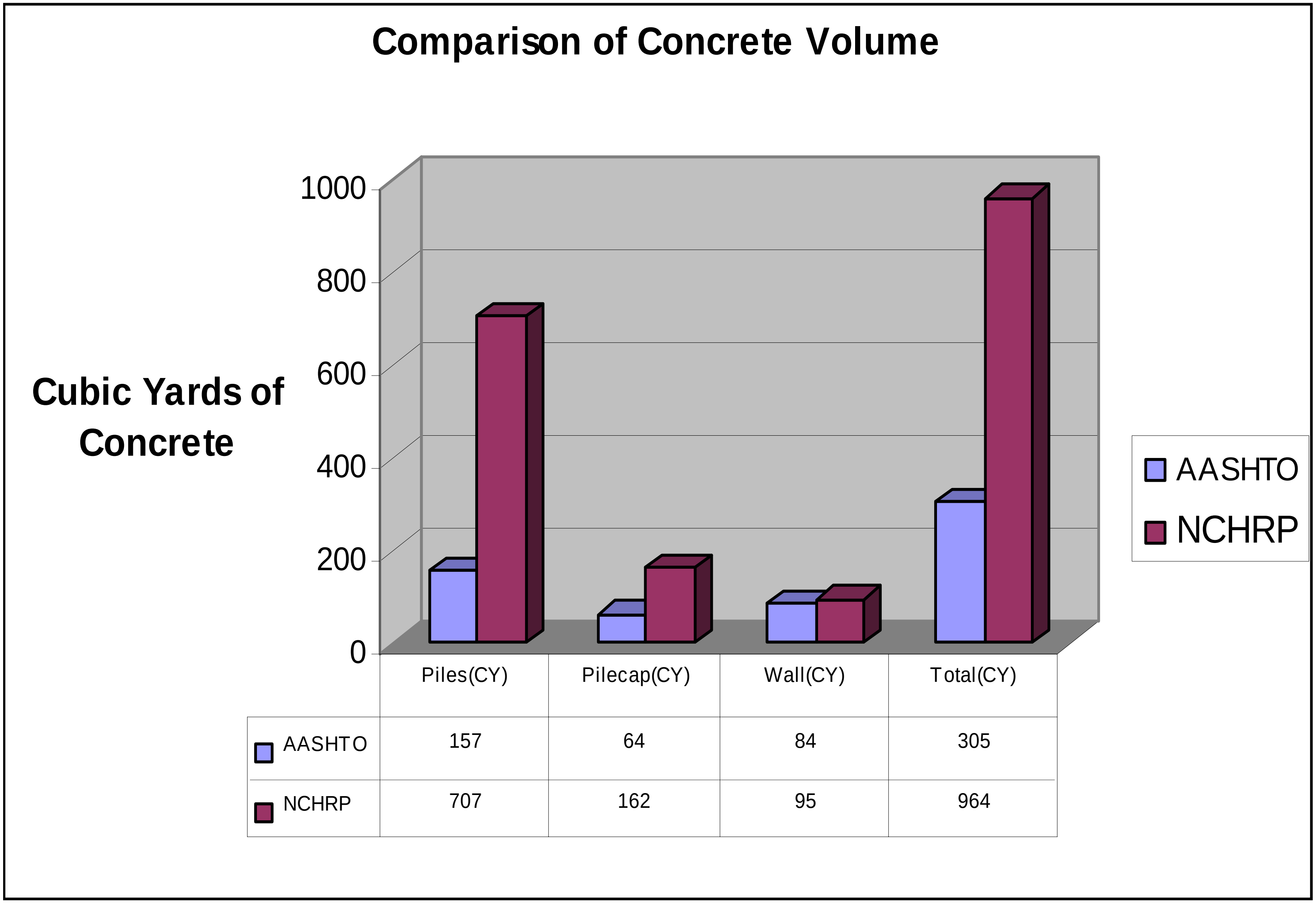


Figure 3.12 – Concrete Volume Comparison (AASHTO vs. NCHRP)

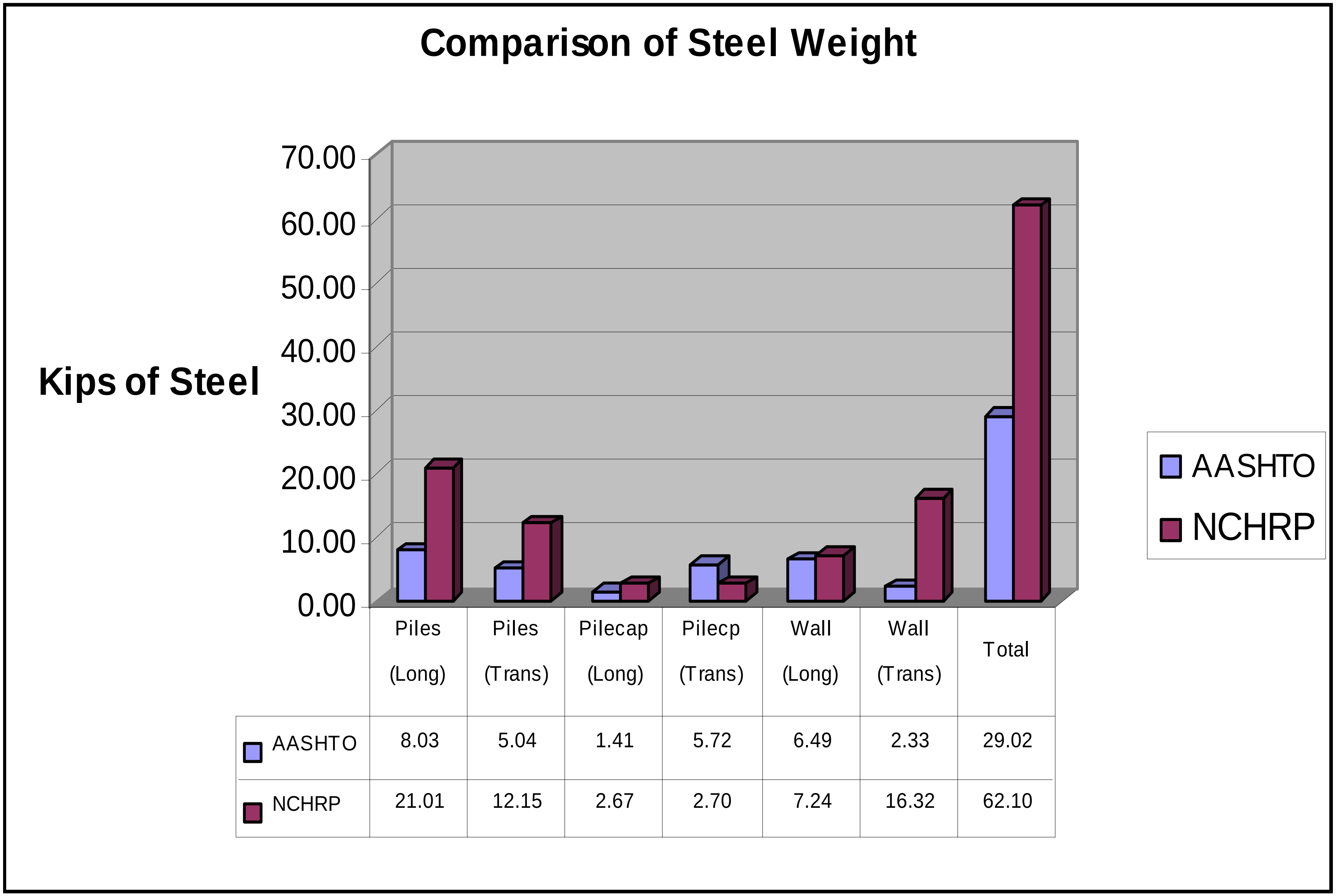


Figure 3.13 – Reinforcing Steel Weight Comparison (AASHTO vs. NCHRP)

Table 3.1 – NCHRP Seismic Design Force and Displacement Result Summary

Displacement		**Longitudinal Direction (in)**	**Transverse Direction (in)**
MCE	Top	3.11	0.95
	Bottom Fix	1.16	0.77
	Bottom Exp	0.25	0.89
FE	Top	0.23	0.01
	Bottom Fix	0.03	0.02
	Bottom Exp	0.09	0.02

MCE		**Final Design Forces & Moments (EQ + DL)**									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	510	1283	0	2660	551	513	1276	1064	0	577
	Bottom			8602	21835				8734	21504	
Exp Pier	Top	98	678	0	1977	510	271	246	791	0	546
	Bottom			1220	12780				5112	3050	

FE		**Final Design Forces & Moments (EQ + DL)**									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	55	11	0	234	537	4	137	94	0	541
	Bottom			940	346				138	2350	
Exp Pier	Top	14	11	0	156	489	4	36	62	0	494
	Bottom			176	522				209	439	

Table 3.2 – AASHTO Seismic Design Force and Displacement Result Summary

Displacement	Longitudinal Direction (in)	Transverse Direction (in)	Max (in)
Bearing of Fixed Pier	1.16924	0.04026	
Top of Fixed Pier	1.10282	0.04102	
Bottom of Fixed Pier Pile Cap	0.18865	0.05776	0.197
Bearing of Expansion Pier	1.17857	0.03341	
Top of Expansion Pier	0.2202	0.03352	
Bottom of Expansion Pier Pile Cap	0.02909	0.03396	0.045

DIVISION 1-A		Forces & Moments - Load Case 1				
		$(1.00*Eq_{trans} + 0.30*Eq_{long}) + 1*DL$				
Support / Location		Shear 2 kips	Shear 3 kip	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips
Fix Pier	Top	32	58	29	268	307
	Bottom	34	27	543	431	469
Exp Pier	Top	1	8	0	445	310
	Bottom	9	28	112	144	481

DIVISION 1-A		Forces & Moments - Load Case 2				
		$(1.00*Eq_{long} + 0.30*Eq_{trans}) + 1*DL$				
Support / Location		Shear 2 kips	Shear 3 kip	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips
Fix Pier	Top	105	17	95	80	306
	Bottom	115	8	1810	129	468
Exp Pier	Top	3	2	0	134	308
	Bottom	30	8	375	43	479

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^{o} 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.

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.

	AASHTO	NCHRP
Concrete:	$69,890	$255,300
Epoxy Coated Rebars:	$31,300	$134,800
HP Piles:	$66,830	$489,000
Total Cost:	**$168,020**	**$879,100**

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.

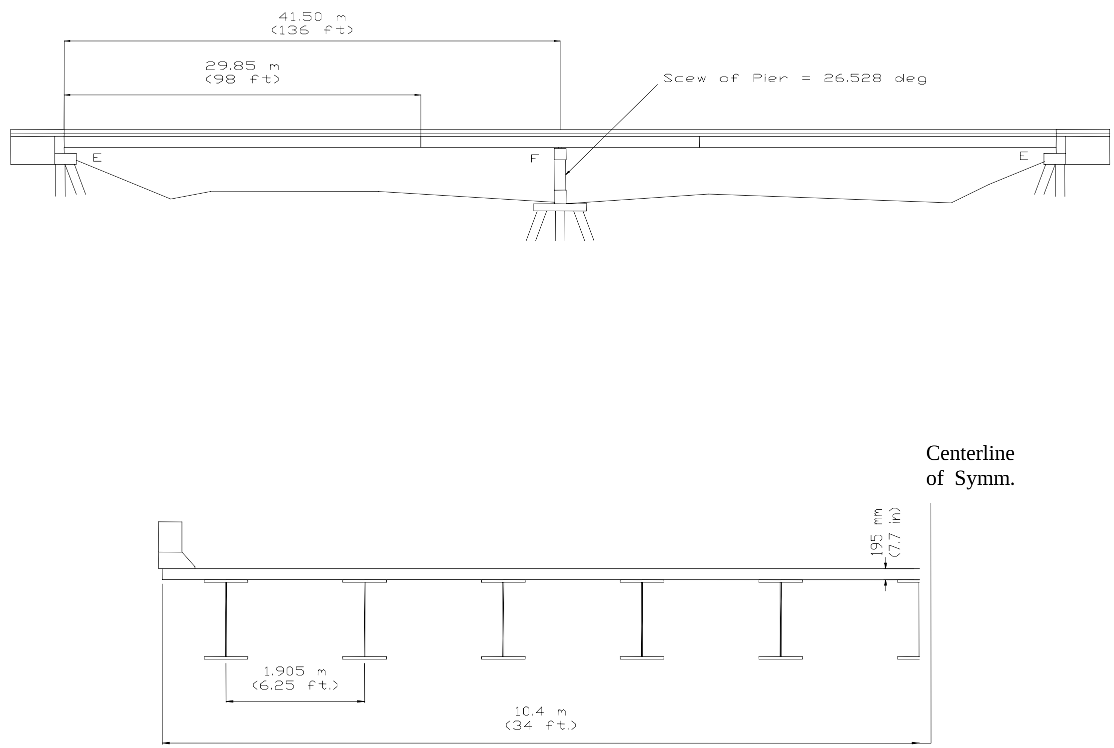


Figure 4.1 – St. Clair County Existing Bridge Geometry -- Side View and Superstructure

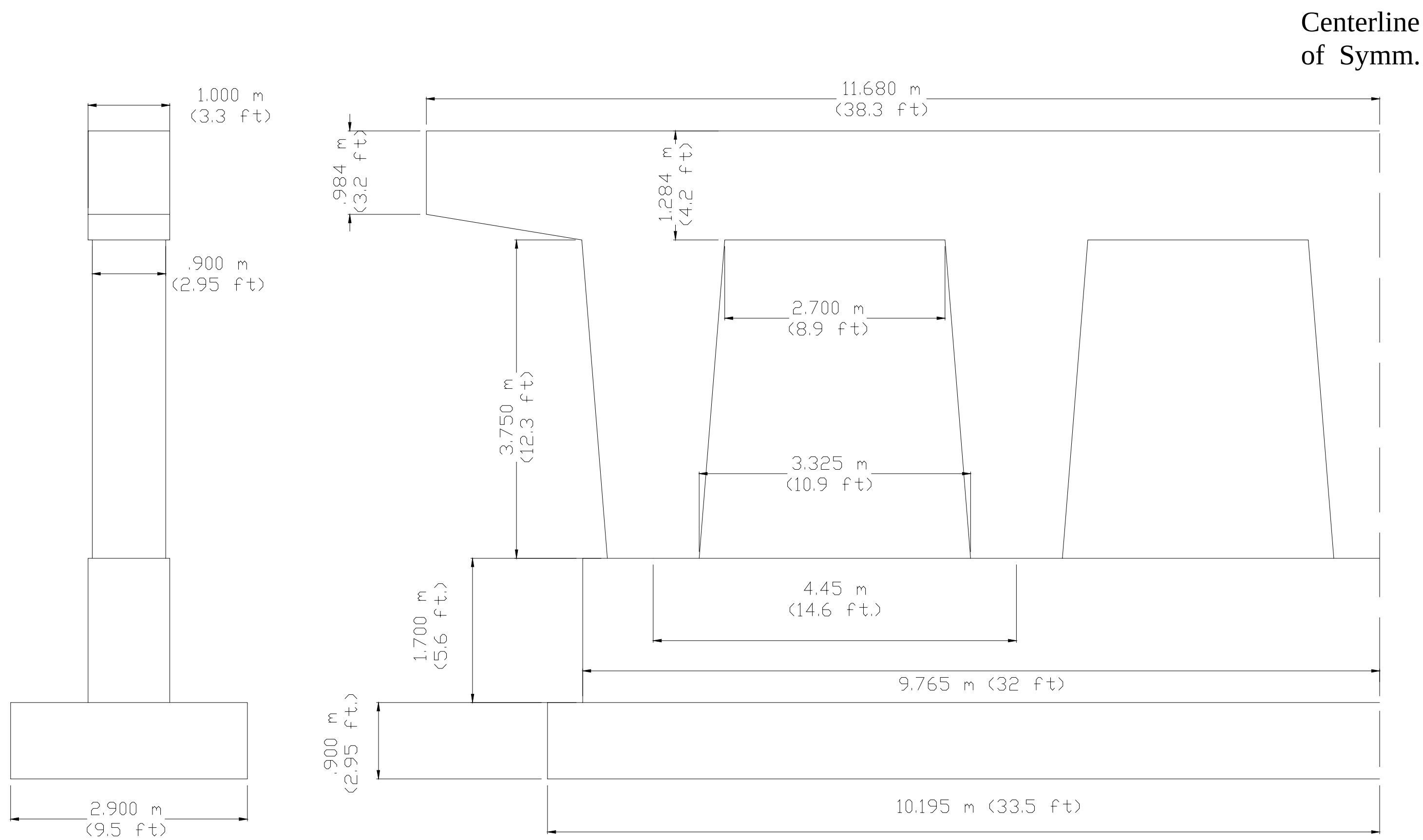


Figure 4.2 – St. Clair County Existing Bridge Geometry – Interior Bent

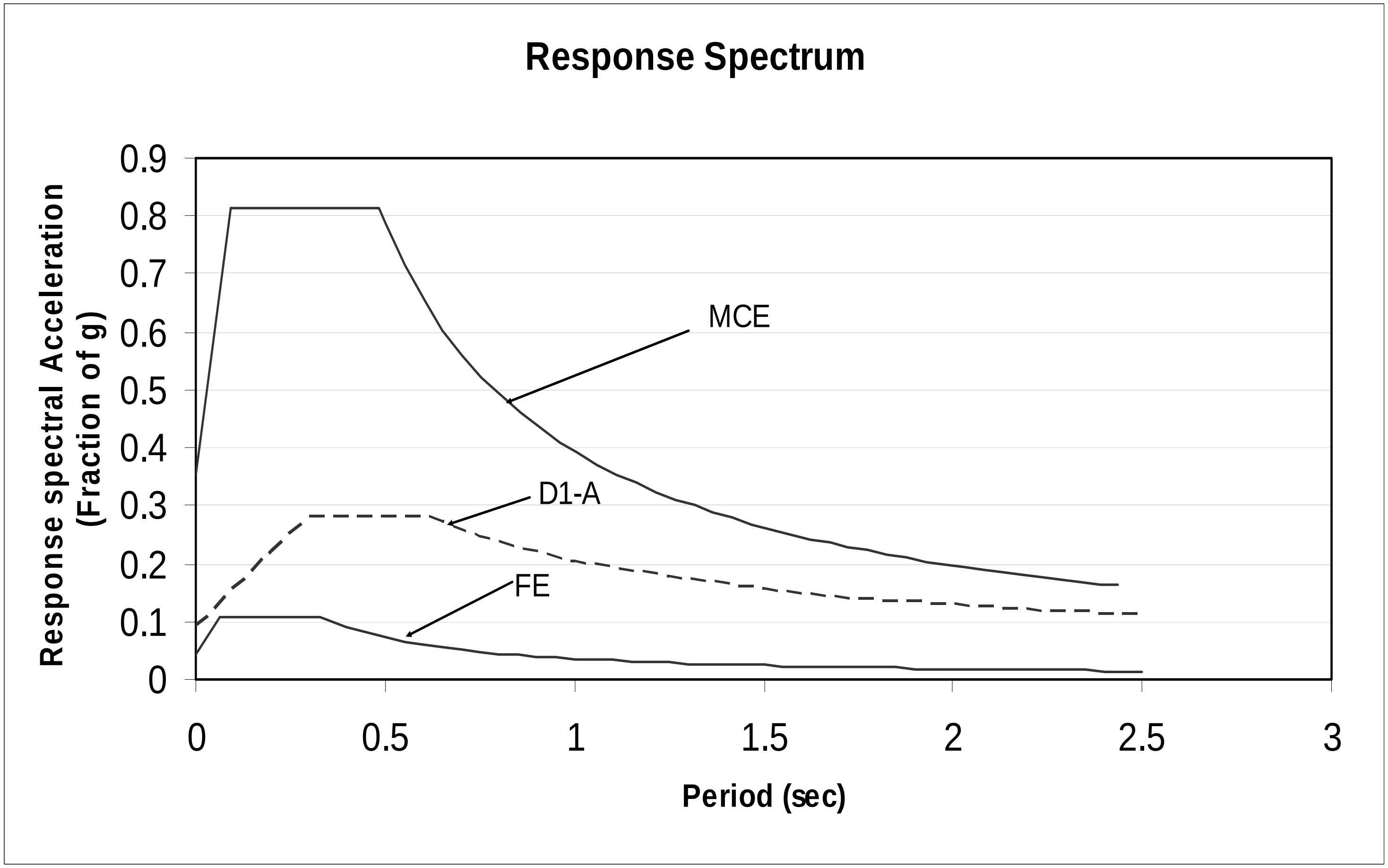
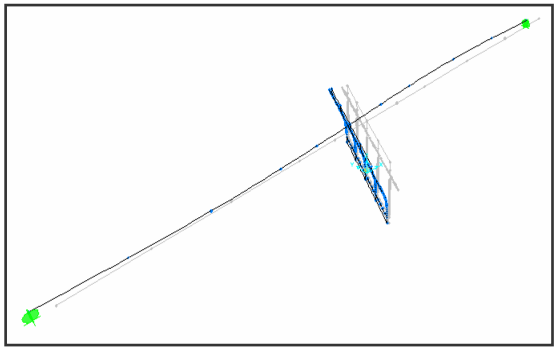
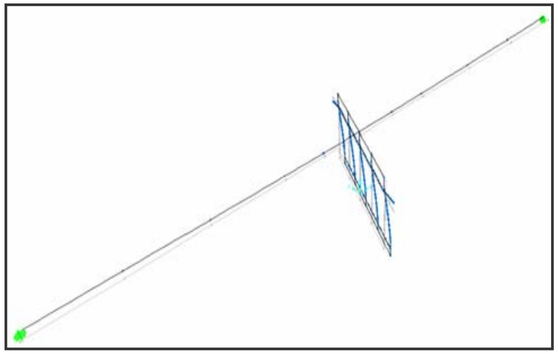


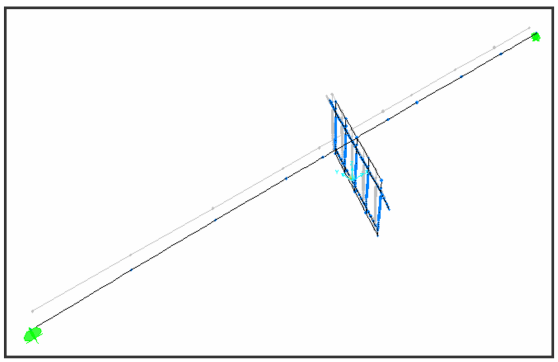
Figure 4.3 – Design Earthquake Response Spectrum for the St. Clair County Bridge



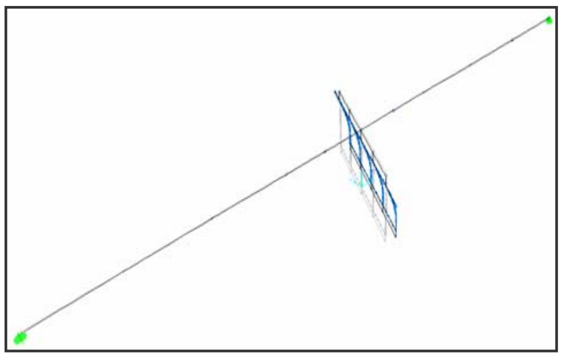
Mode Shape 1



Mode Shape 5



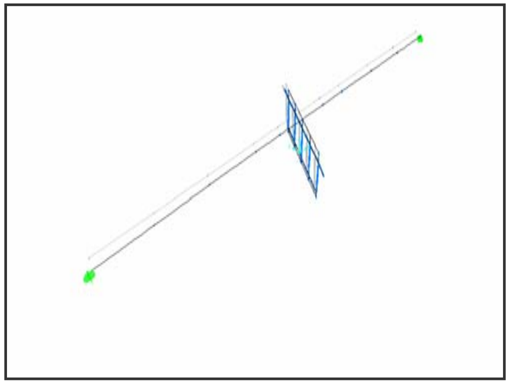
Mode Shape 3



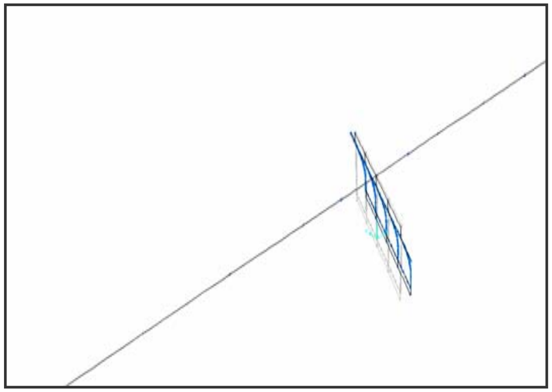
Mode Shape 6

TABLE: Modal Participating Mass Ratios With Spring for MCE				
Mode	Period	Individual Mode (%)		
		UX	UY	UZ
1	0.772195	42.89	11.59	0.00
2	0.596676	1.70	0.65	0.00
3	0.557558	19.43	72.53	0.00
4	0.360047	0.00	0.00	32.08
5	0.233051	0.05	10.19	0.00
6	0.22987	35.92	5.03	0.00
7	0.198011	0.00	0.00	0.00
8	0.152639	0.00	0.00	0.00
9	0.148196	0.00	0.00	0.00
10	0.118068	0.00	0.00	6.12
11	0.1008	0.00	0.00	0.00
12	0.073461	0.00	0.00	52.36
13	0.070734	0.00	0.00	0.00
14	0.0583	0.00	0.00	9.27
15	0.050561	0.00	0.00	0.00
16	0.048321	0.00	0.00	0.00
17	0.030654	0.00	0.00	0.00
18	0.028907	0.00	0.00	0.00
19	0.027706	0.00	0.00	0.00
20	0.025344	0.00	0.00	0.16
		100.00	100.00	99.99

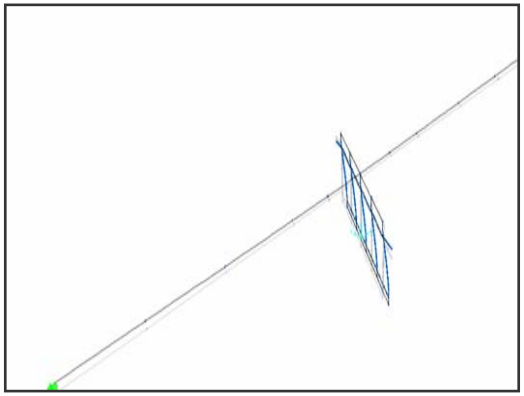
Figure 4.4 – Free Vibration Results (NCHRP MCE Model)



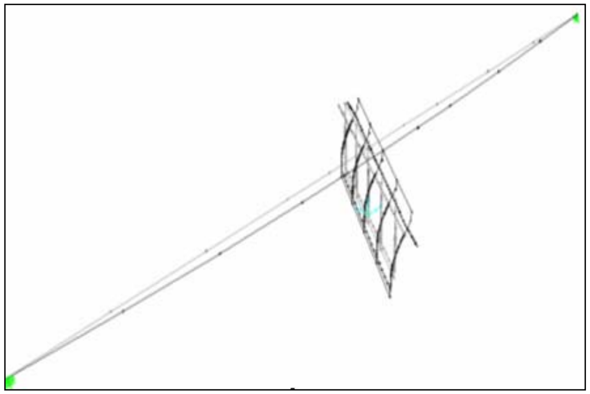
Mode Shape 1



Mode Shape 2



Mode Shape 6



Mode Shape 8

TABLE: Modal Participating Mass Ratios				
Mode Number	Period Sec	UX %	UY %	UZ %
1	1.0392	40.16	0.45	0
2	0.6622	34.43	8.99	0
3	0.6434	0	0	98.66
4	0.6002	2.08	0.07	0
5	0.4894	0	0	0
6	0.3609	16.02	49.67	0
7	0.3214	0	0	0.24
8	0.3168	7.30	30.02	0
9	0.1644	0.01	7.00	0
10	0.1482	0	0	0
11	0.1115	0	0	1.00
12	0.0707	0	0	0
13	0.0613	0	0	0.10
14	0.0506	0	0	0
15	0.0359	0	0	0
16	0.0305	0	0.03	0
17	0.0289	0	0.05	0
18	0.0271	0	2.69	0
19	0.0259	0	0	0
20	0.0246	0	0	0
21	0.0243	0	0.17	0
22	0.0237	0	0.06	0
23	0.0230	0	0	0
24	0.0225	0	0.03	0
25	0.0205	0	0	0
		100.00	99.23	100.00

Figure 4.5 – Free Vibration Results (AASHTO Model)

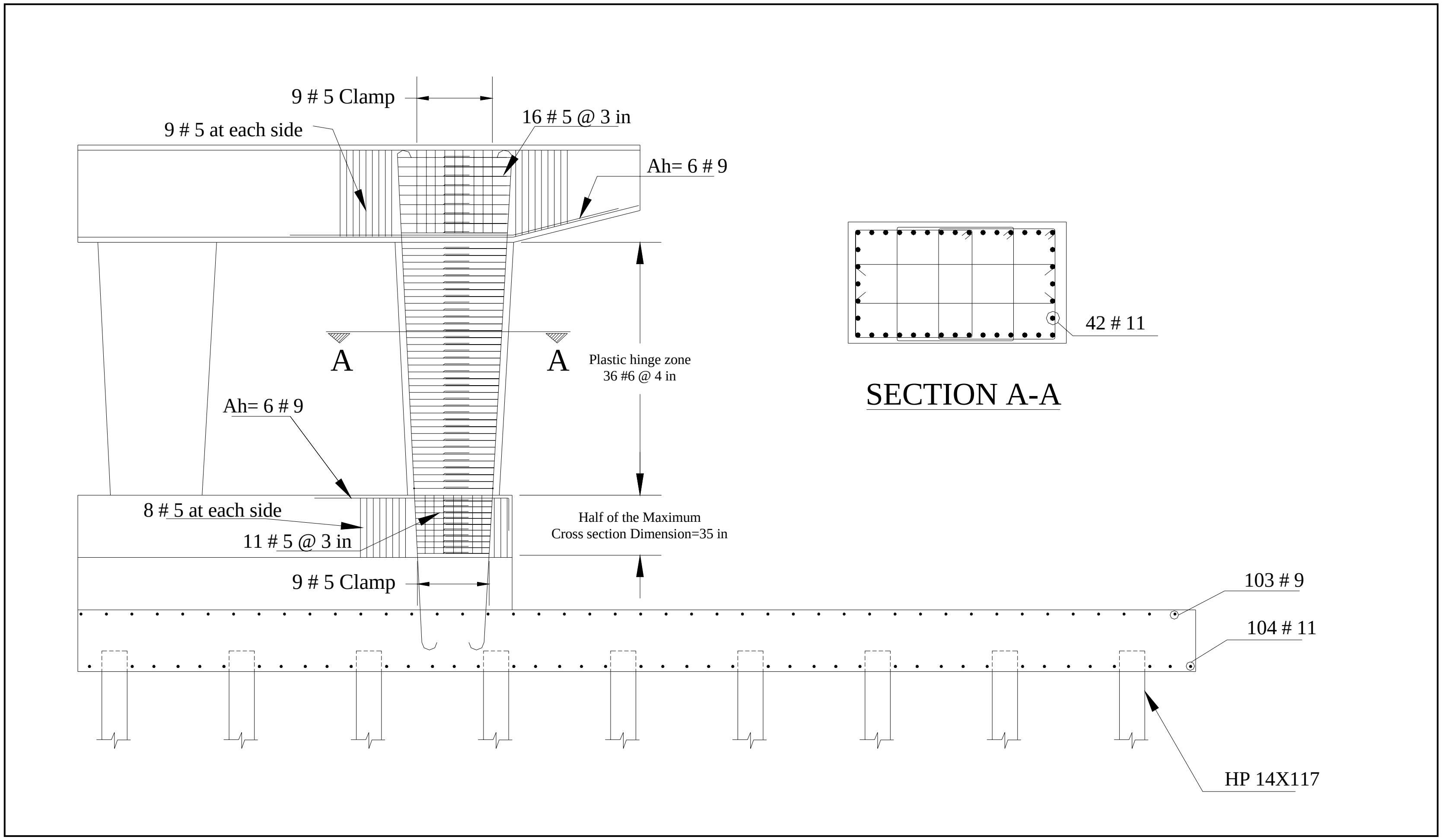


Figure 4.6a – NCHRP Design Details for Interior Bent

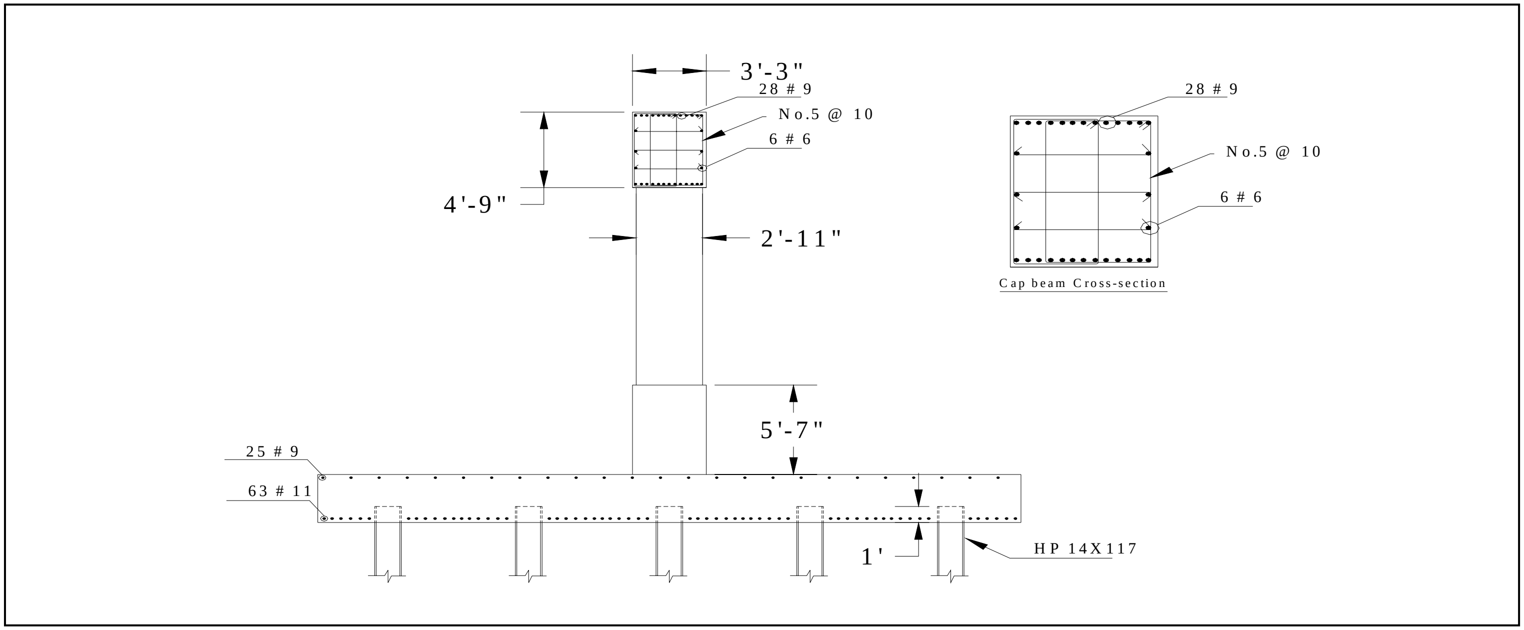


Figure 4.6b – NCHRP Design Details for Interior Bent (cont.)

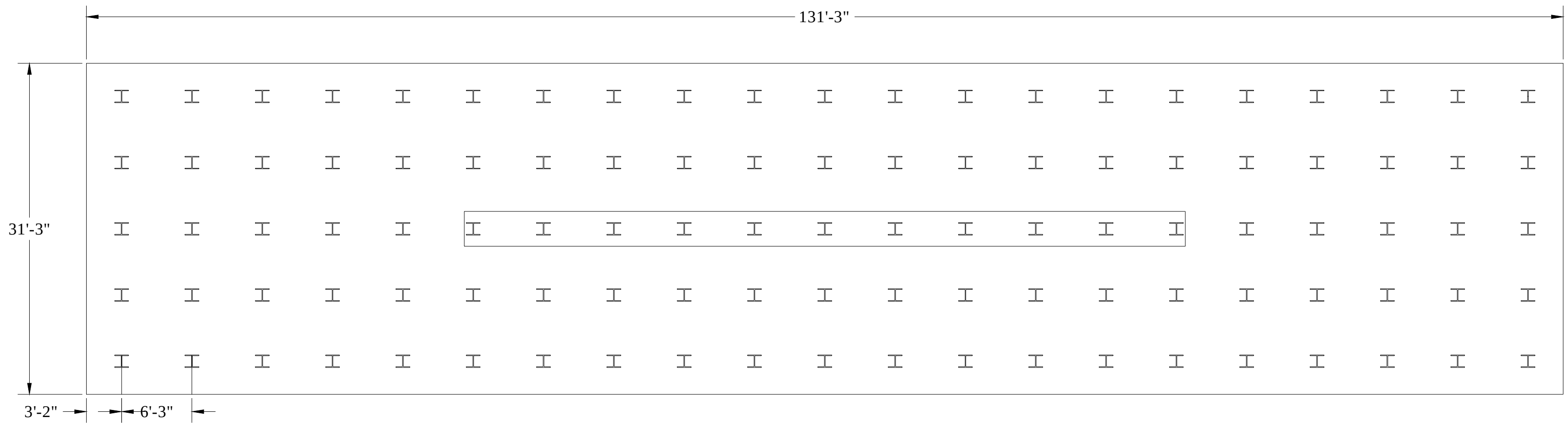


Figure 4.7 – NCHRP Interior Bent Footing Geometry (Piles Used: 105 HP 14x117)

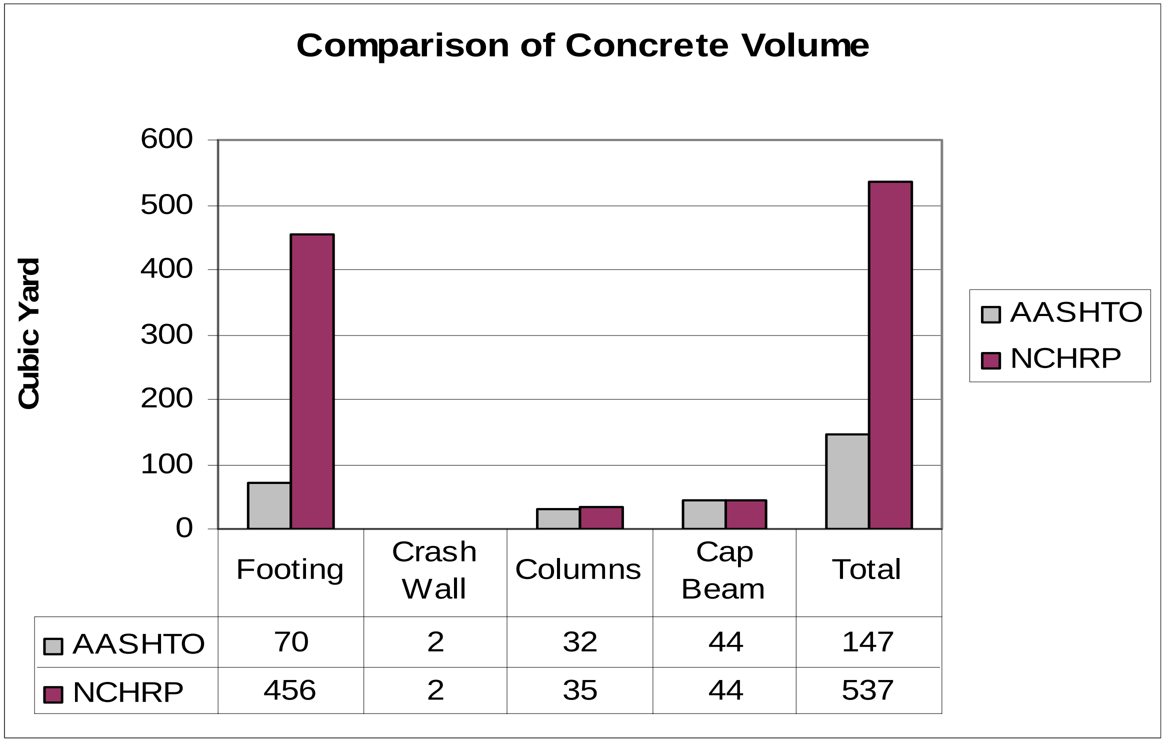


Figure 4.8 – Concrete Volume Comparison (AASHTO vs. NCHRP)

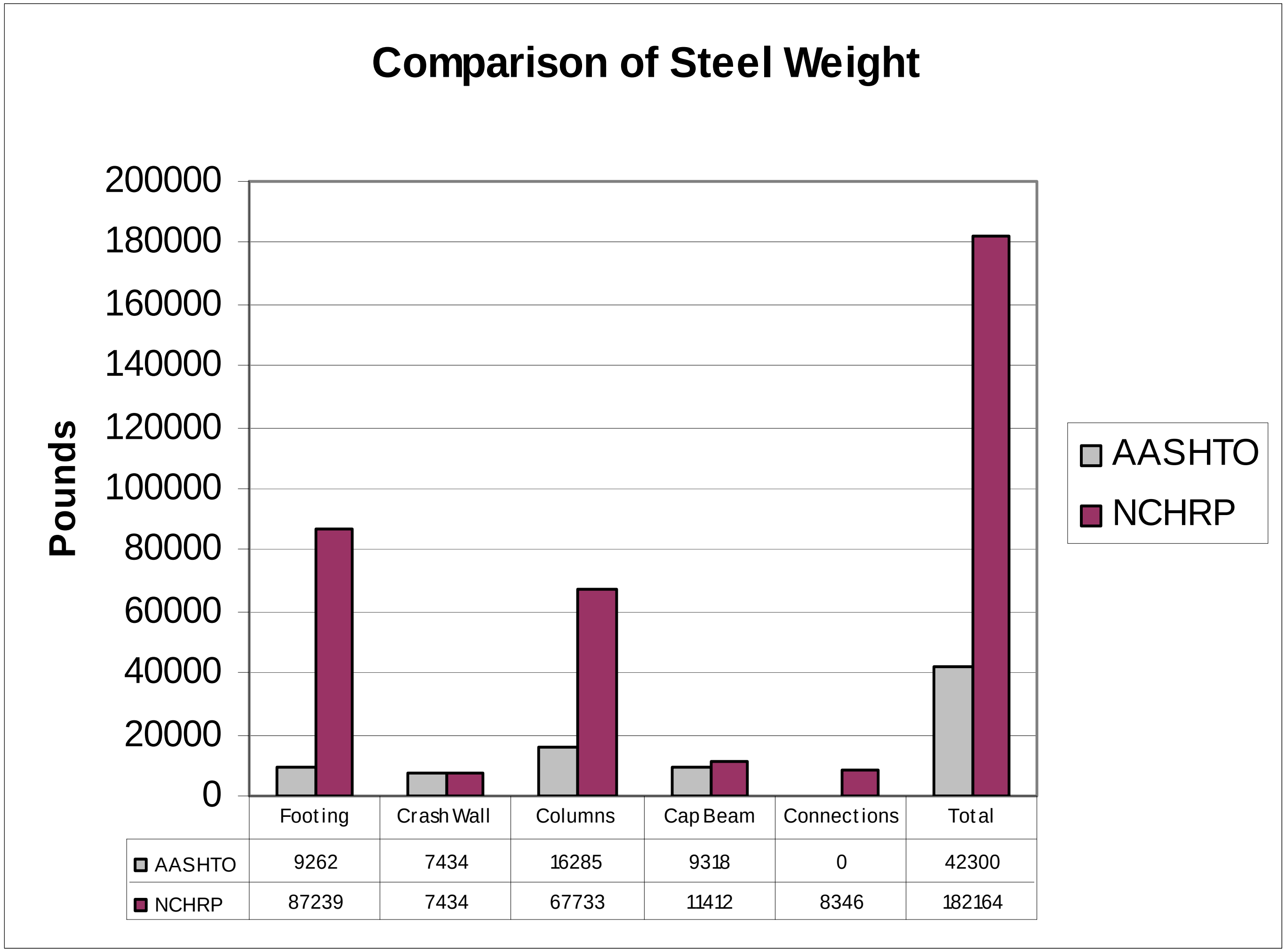


Figure 4.9 – Reinforcing Steel Weight Comparison (AASHTO vs. NCHRP)

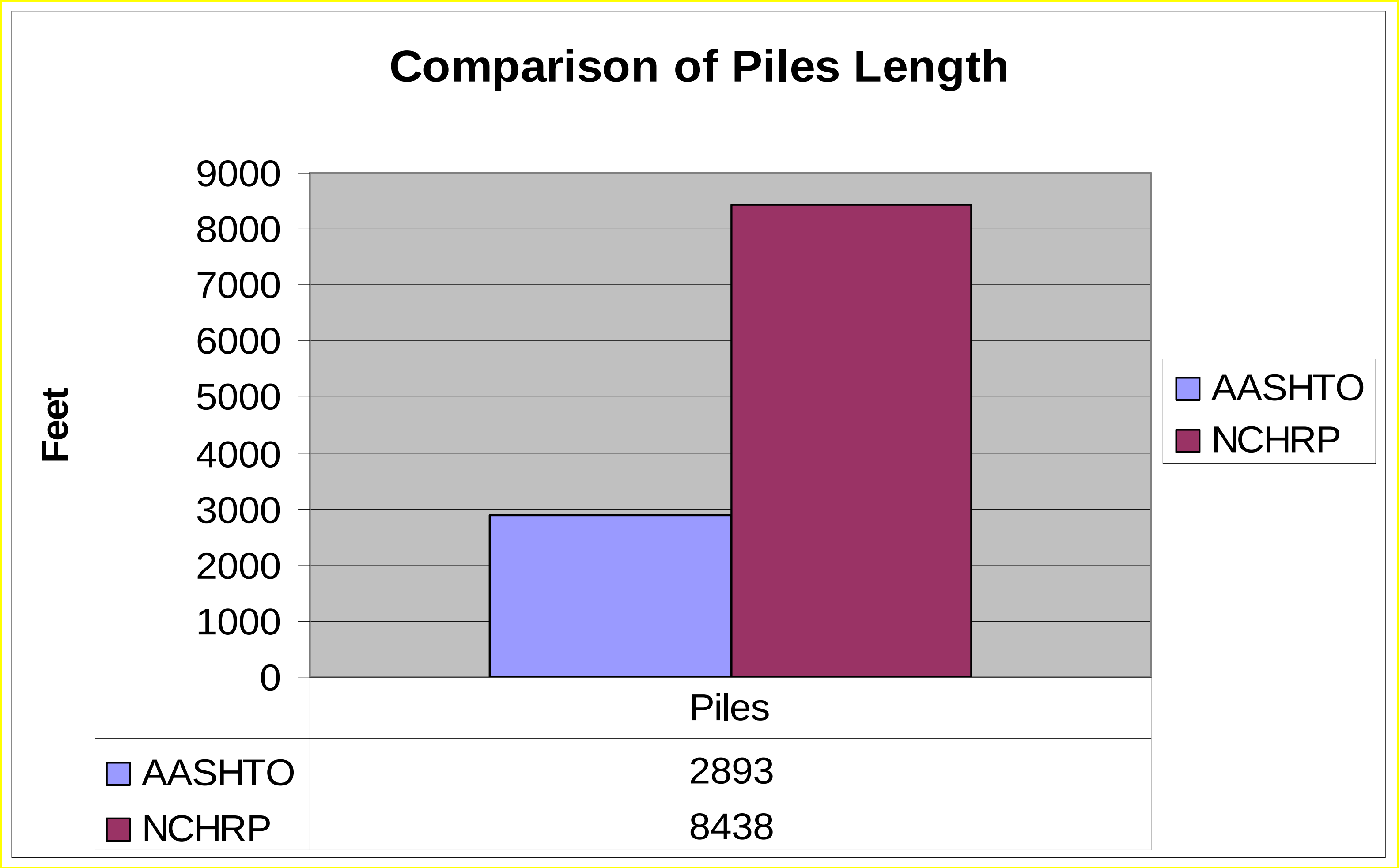


Figure 4.10 – Pile Length Comparison (AASHTO vs. NCHRP)

Table 4.1 – NCHRP Seismic Design Force Result Summary

NCHRP - MCE		Final Design Forces and Moments (EQ+DL) By R=1.5									
		Transvers					Longitudinal				
Location		Shear 2	Shear 3	Moment 2	Moment 3	Axial	Shear 2	Shear 3	Moment 2	Moment 3	Axial
		kips	kips	kip-ft	kip-ft	kips	kips	kips	kip-ft	kip-ft	kips
Exterior Column	top	231	406	3369	996	1072	304	307	2554	1311	936
	bottom	233	419	1715	3859	1100	307	317	1299	5083	964
Middle Column	top	231	527	4893	997	794	304	399	3709	1314	726
	bottom	233	540	1681	3860	822	307	409	1274	5084	754
Central Column	top	231	541	5070	997	517	304	410	3844	1314	517
	bottom	233	553	1335	3716	545	307	420	1011	4895	545

NCHRP - FE		Final Design Forces and Moments (EQ+DL) By R=0.849									
		Transverse					Longitudinal				
Location		Shear 2	Shear 3	Moment 2	Moment 3	Axial	Shear 2	Shear 3	Moment 2	Moment 3	Axial
		kips	kips	kip-ft	kip-ft	kips	kips	kips	kip-ft	kip-ft	kips
Exterior Column	top	18	37	321	77	567	36	44	368	154	574
	bottom	18	40	161	168	595	36	45	187	254	602
Middle Column	top	18	49	465	77	542	36	57	535	154	546
	bottom	18	51	159	309	570	36	58	184	608	574
Central Column	top	18	50	482	77	517	36	59	554	154	517
	bottom	18	53	158	309	545	36	60	183	608	545

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

AASHTO		**Final Design Forces and Moments**									
		Load Combination 1 - (1.0*EQL + 0.3*EQT)+DL					**Load Combination 2 - (0.3*EQL + 1.0*EQT)+DL**				
		Longitudinal (R =3)		**Transverse (R =5)**			**Longitudinal (R =3)**		**Transverse (R =5)**		
Location		**Shear 3 kips**	**Moment 2 kip-ft**	**Shear 2 kips**	**Moment 3 kip-ft**	**Axial kips**	**Shear 3 kips**	**Moment 2 kip-ft**	**Shear 2 kips**	**Moment 3 kip-ft**	**Axial kips**
Exterior Column	**top**	42	347	35	151	335	40	323	19	82	346
	bottom	42	172	35	581	365	40	161	19	312	377
Middle Column	**top**	54	489	35	151	420	50	457	19	82	425
	bottom	54	173	35	581	449	50	163	19	312	454
Central Column	**top**	55	507	35	151	503	52	474	19	82	503
	bottom	55	173	35	581	530	52	163	19	312	530

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.

	AASHTO	NCHRP
Concrete:	$73,790	$359,450
Epoxy Coated Rebars:	$21,700	$121,730
HP Piles:	$69,850	$417,240
Total Cost:	**$165,340**	**$898,420**

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.

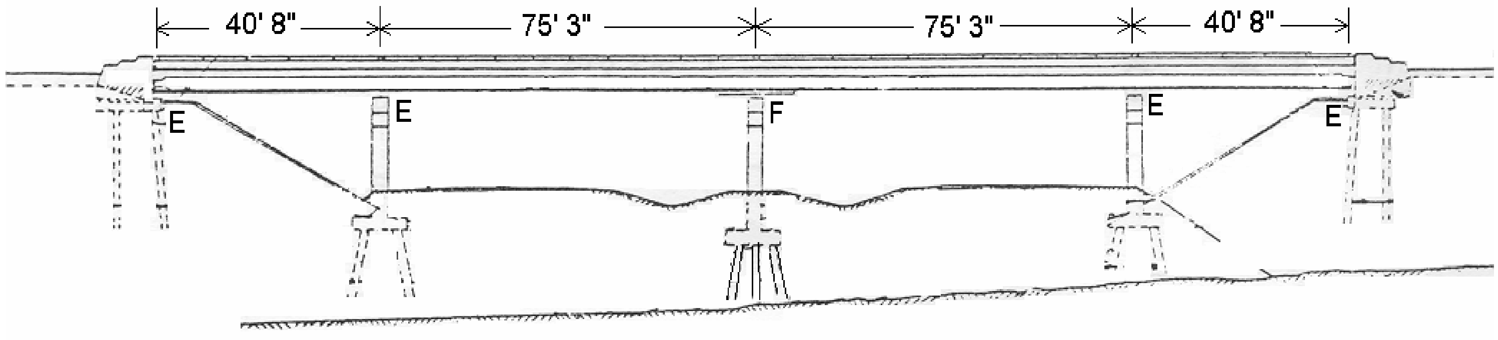


Figure 5.1 – Existing Bridge Geometry – Side View

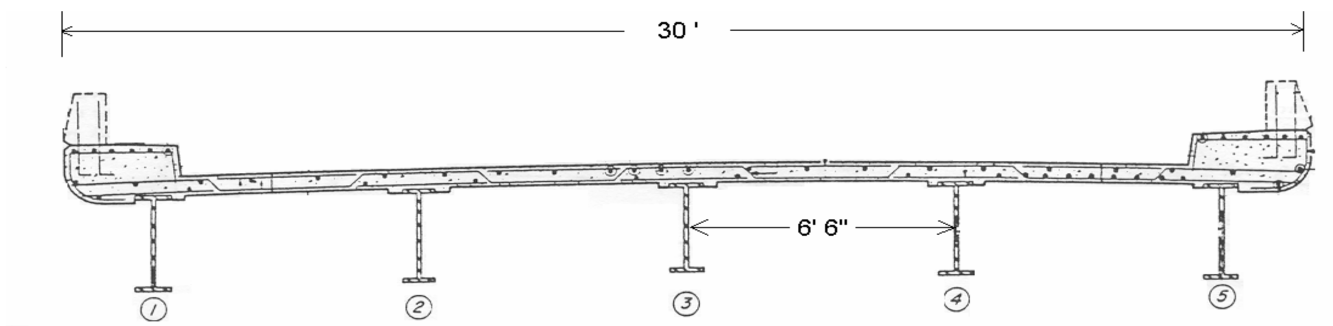


Figure 5.2 – Existing Bridge Geometry – Superstructure

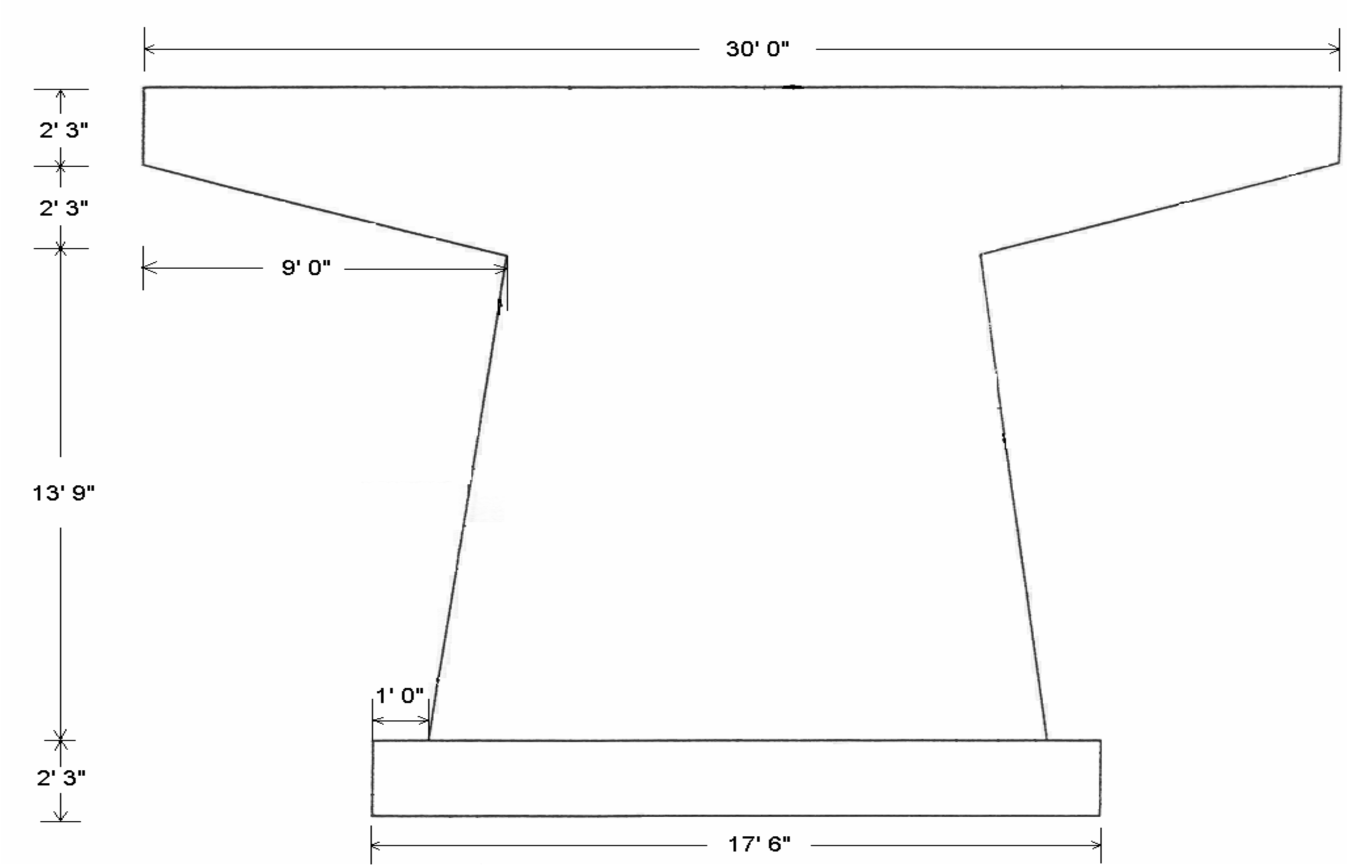


Figure 5.3 – Existing Bridge Geometry – Interior Bent

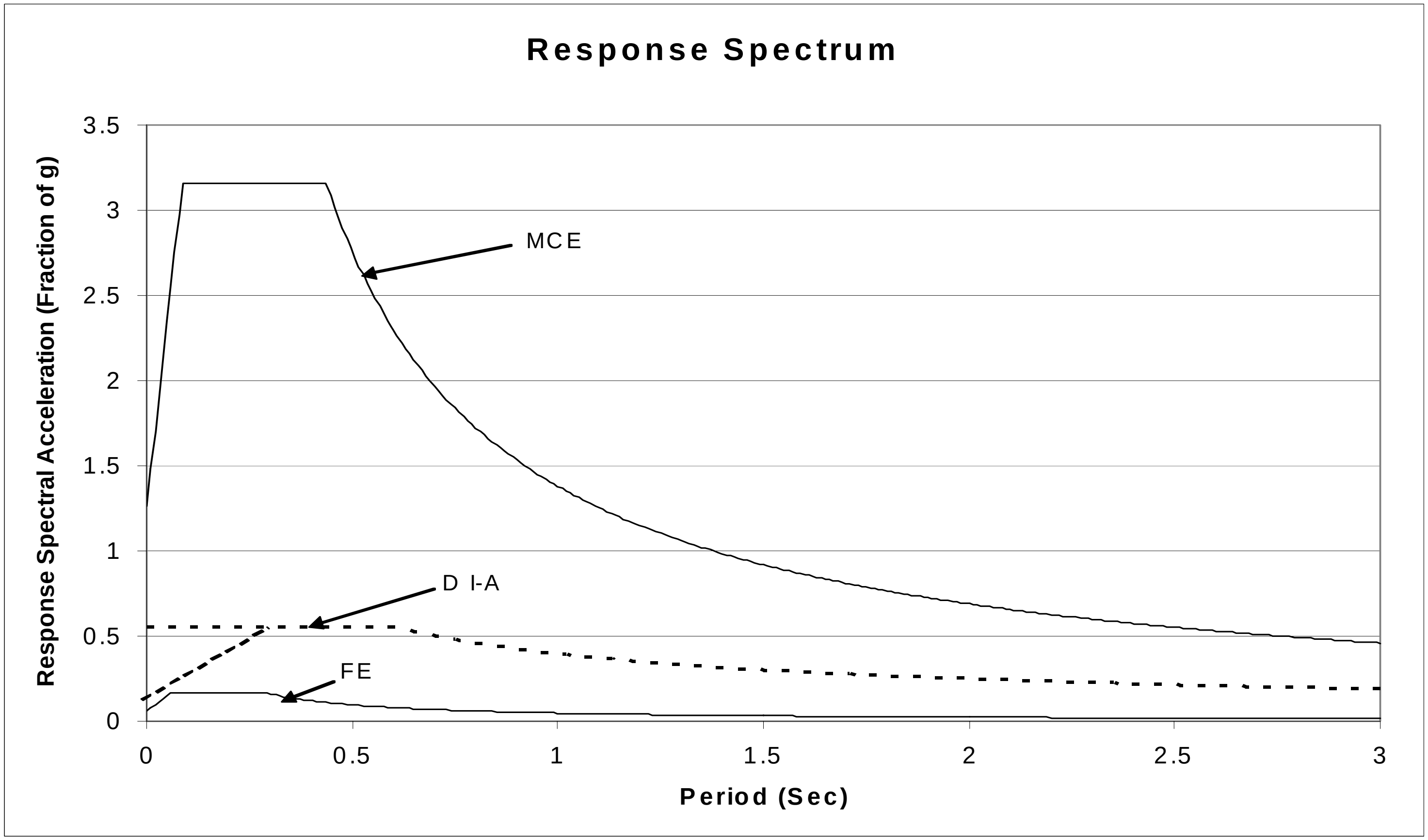
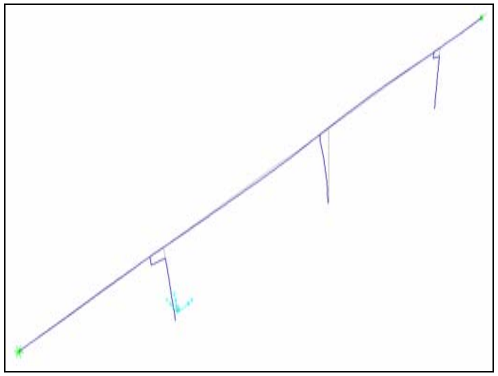
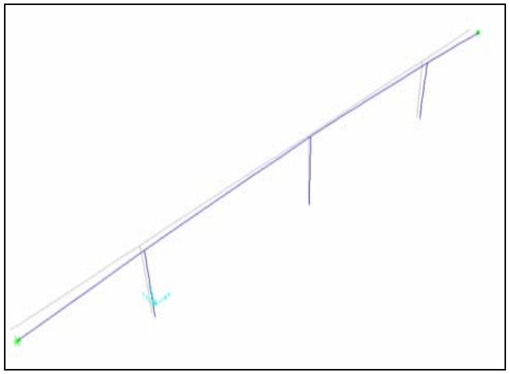


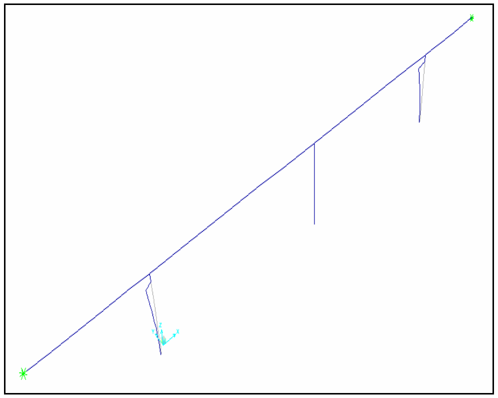
Figure 5.4 – Design Earthquake Response Spectrum for Pulaski County Bridge



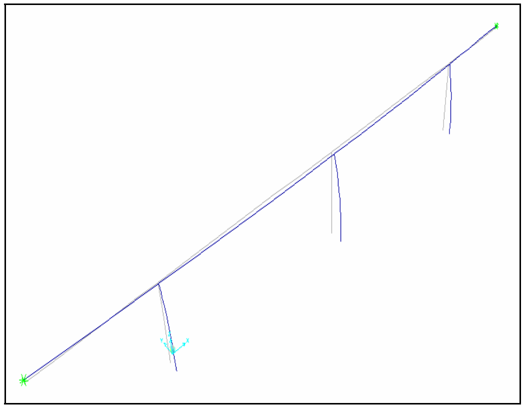
Mode Shape 1- (UX- 46%)



Mode Shape 4- (UY- 67%)



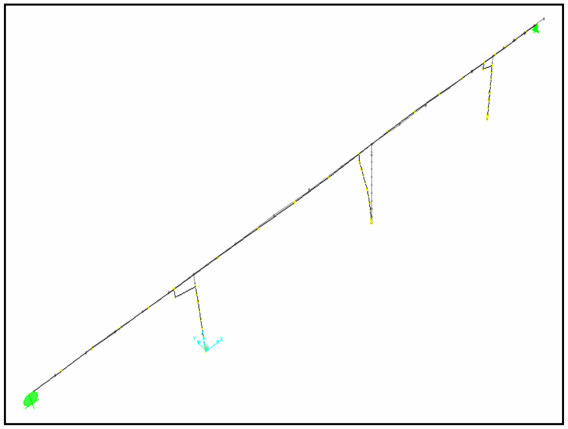
Mode Shape 8- (UX- 17%)



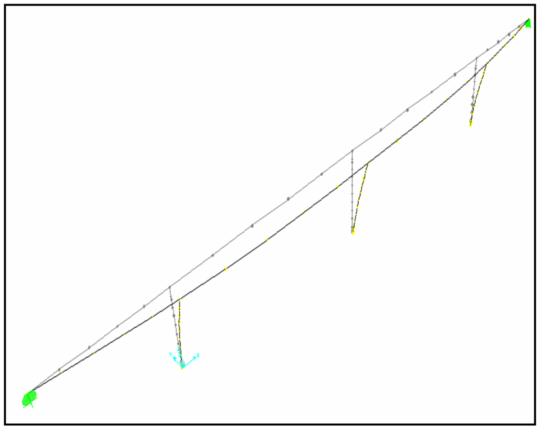
Mode Shape 9- (UY- 21%)

Modal Participating Mass Ratios				
With Spring for MCE				
Mode	**Period**	**INDIVIDUAL MODE(%)**		
		UX	**UY**	**UZ**
1	0.485747	45.94	0.00	0.00
2	0.267381	0.45	0.00	0.00
3	0.221216	0.00	0.00	0.00
4	0.213759	0.00	68.76	0.00
5	0.186965	0.00	0.00	13.77
6	0.175687	0.00	9.61	0.00
7	0.149138	2.64	0.00	0.00
8	0.149138	16.32	0.00	0.00
9	0.14573	0.00	20.79	0.00
10	0.12624	0.00	0.00	0.00
11	0.099286	0.00	0.08	0.00
12	0.096843	18.23	0.00	0.00
13	0.093833	0.00	0.00	0.00
14	0.092676	4.48	0.00	0.00
15	0.092676	11.22	0.00	0.00
16	0.091418	0.69	0.00	0.00
17	0.085216	0.00	0.00	20.20
50	0.021368	0.00	0.00	0.00
		99.96	99.25	33.96

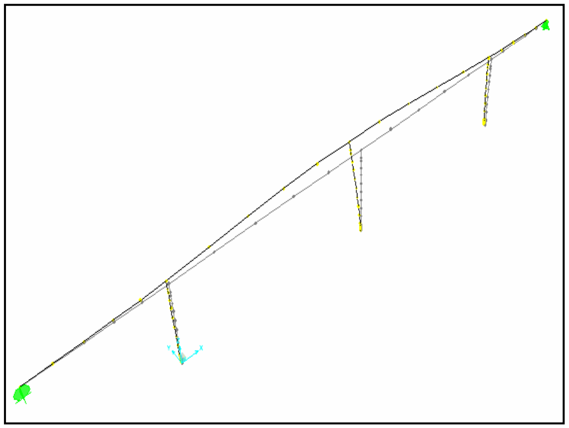
Figure 5.5 – Free Vibration Results (NCHRP MCE Model)



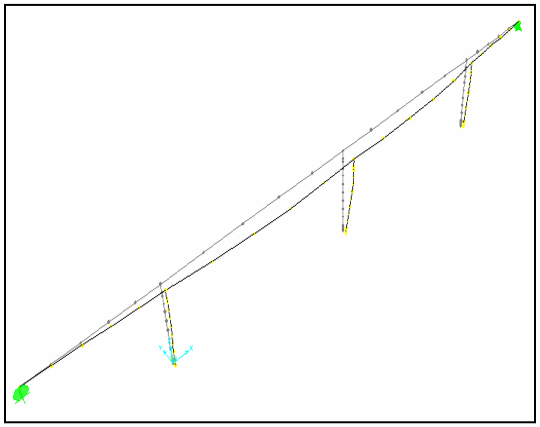
Mode Shape 1 (UX - 68%)



Mode Shape 2 (UY - 17%)



Mode Shape 8 (UY- 13%)



Mode Shape 9 (UY - 43%)

Modal Participating Mass Ratios				
Step	**Period**	**UX**	**UY**	**UZ**
No.	Sec	%	%	%
1	0.947	68.06	0.00	0.00
2	0.293	0.00	17.44	0.00
3	0.270	0.03	0.00	0.00
4	0.226	0.00	0.00	0.00
5	0.189	0.00	0.00	23.80
6	0.179	5.44	0.00	0.00
7	0.158	0.00	12.86	0.00
8	0.114	0.00	0.00	0.00
9	0.102	0.00	41.93	0.00
10	0.093	0.00	0.00	0.00
11	0.088	0.00	0.00	34.78
12	0.076	0.00	2.15	0.00
13	0.075	0.01	0.00	0.00
14	0.068	0.00	0.00	2.41
15	0.058	0.00	6.24	0.00
16	0.057	0.00	3.74	0.00
17	0.048	0.00	0.00	0.00
18	0.046	0.00	0.00	34.45
19	0.045	4.26	0.00	0.00
20	0.044	2.85	0.00	0.00
21	0.044	0.00	4.99	0.00
22	0.039	0.00	0.00	0.00
23	0.037	0.00	0.15	0.00
24	0.036	0.08	0.00	0.00
25	0.035	0.00	4.15	0.00
26	0.035	9.00	0.00	0.00
27	0.031	0.00	2.68	0.00
28	0.031	0.00	0.00	1.87
29	0.028	0.00	0.55	0.00
	Total	89.74	96.87	97.31

Figure 5.6 – Free Vibration Results (AASHTO Model)

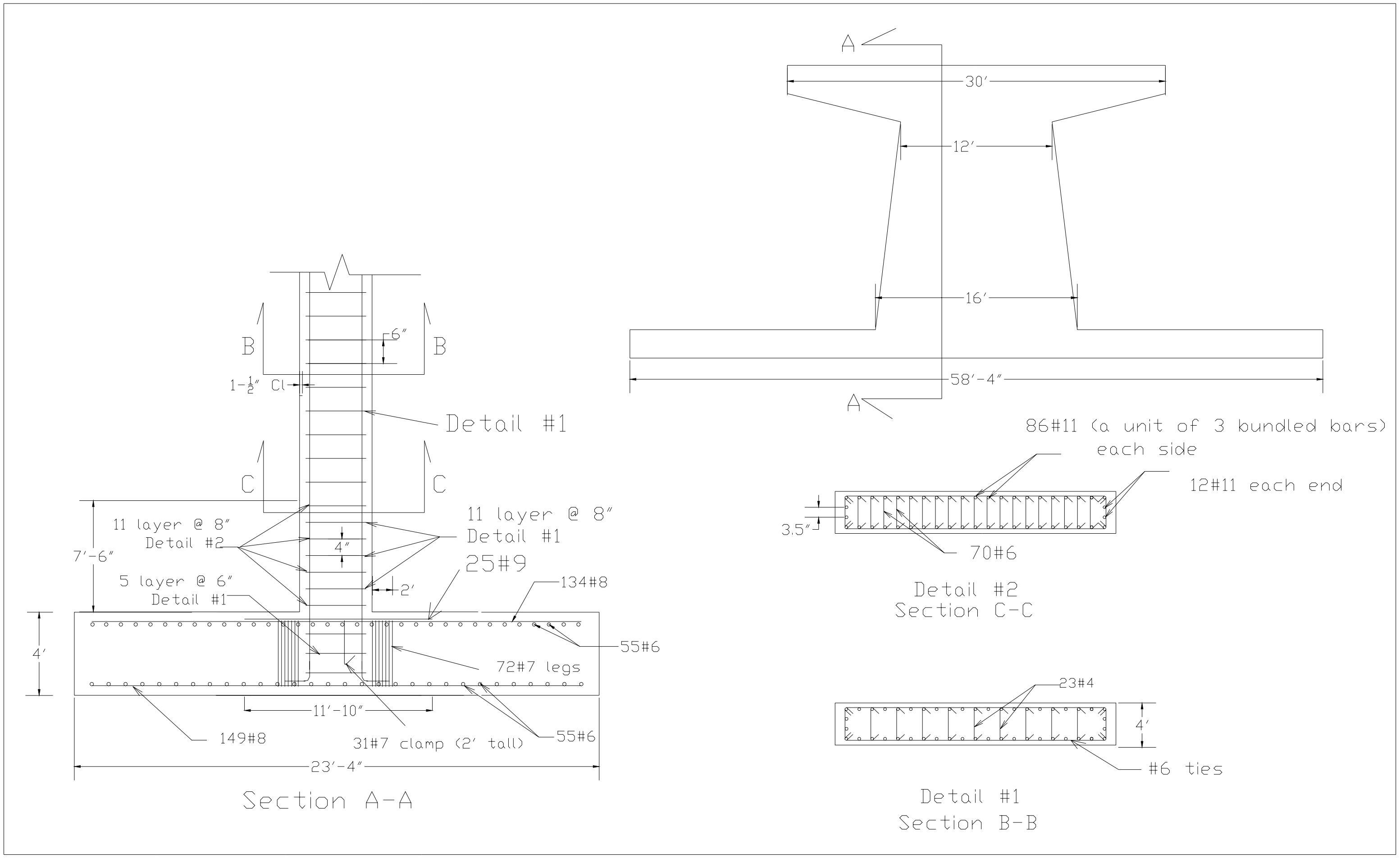


Figure 5.7a – NCHRP Wall Design Details (Bent 2)

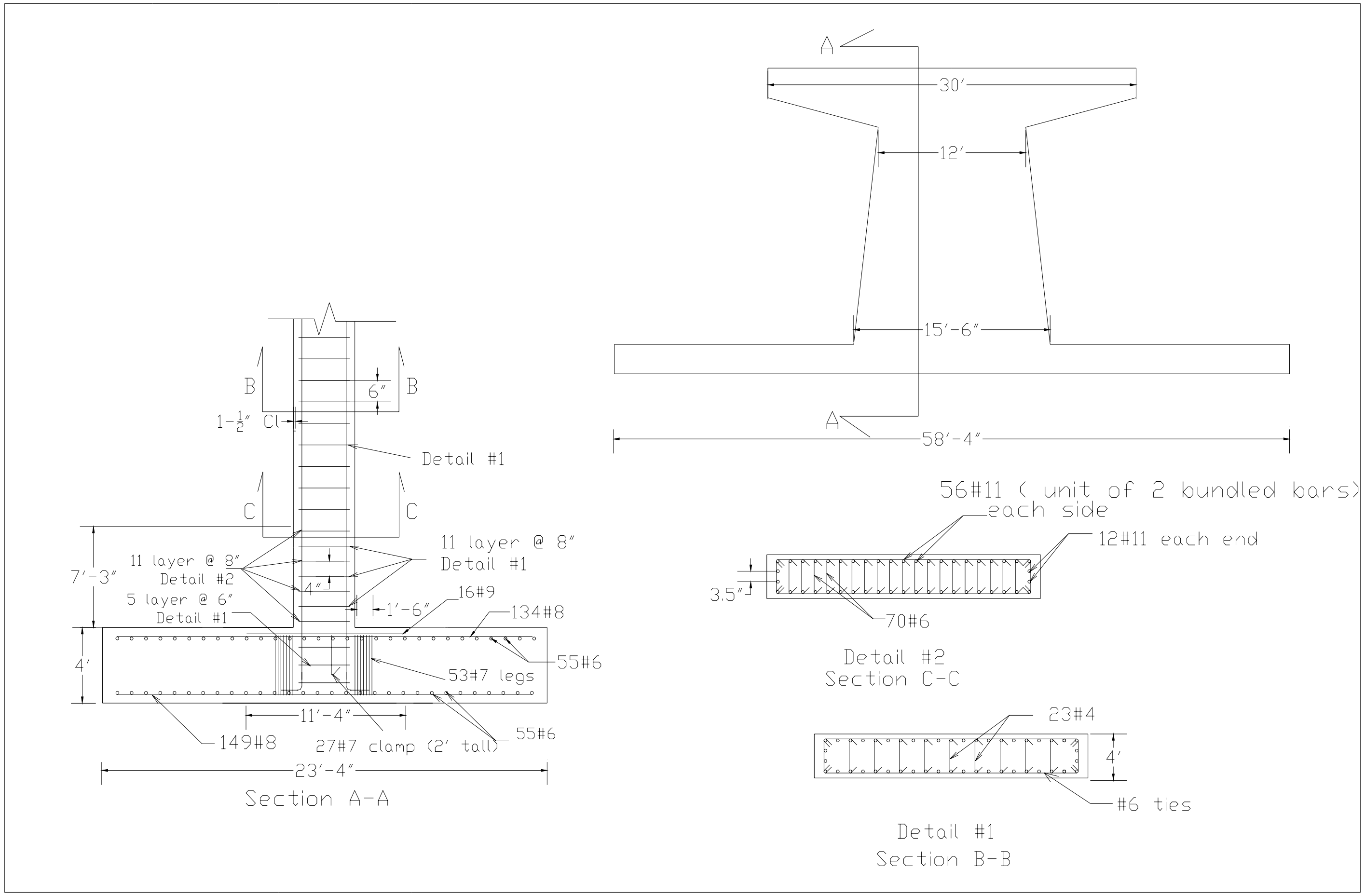
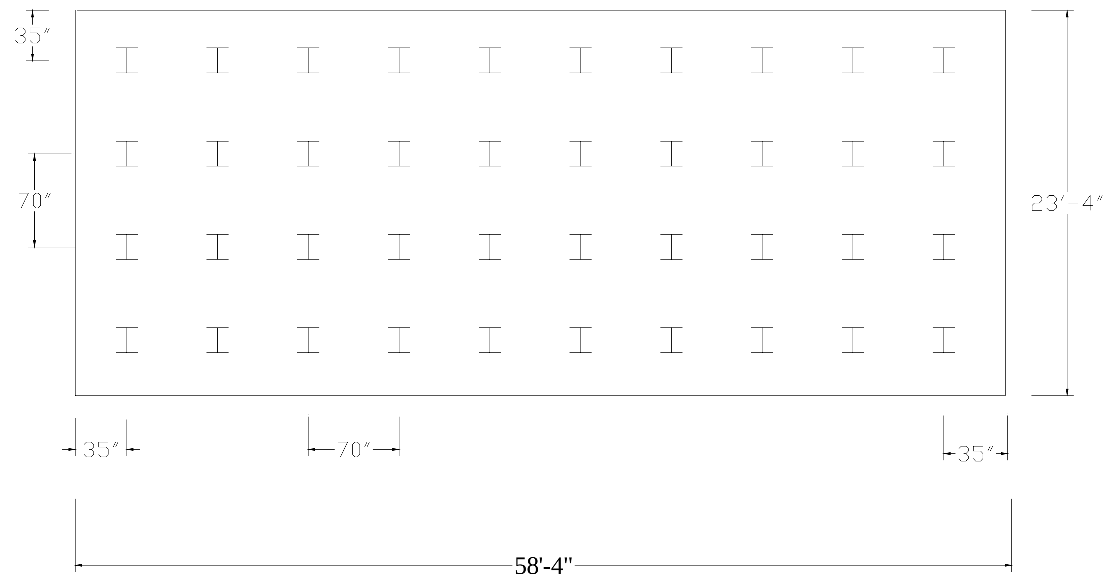


Figure 5.7b – NCHRP Wall Design Details (Bents 1 & 3)



PILE DATA

Type: HP 14X117

Pile Length: 60'

Embedment Length into pile cap: 12"

Figure 5.7c – NCHRP Footing Geometry (All Bents)

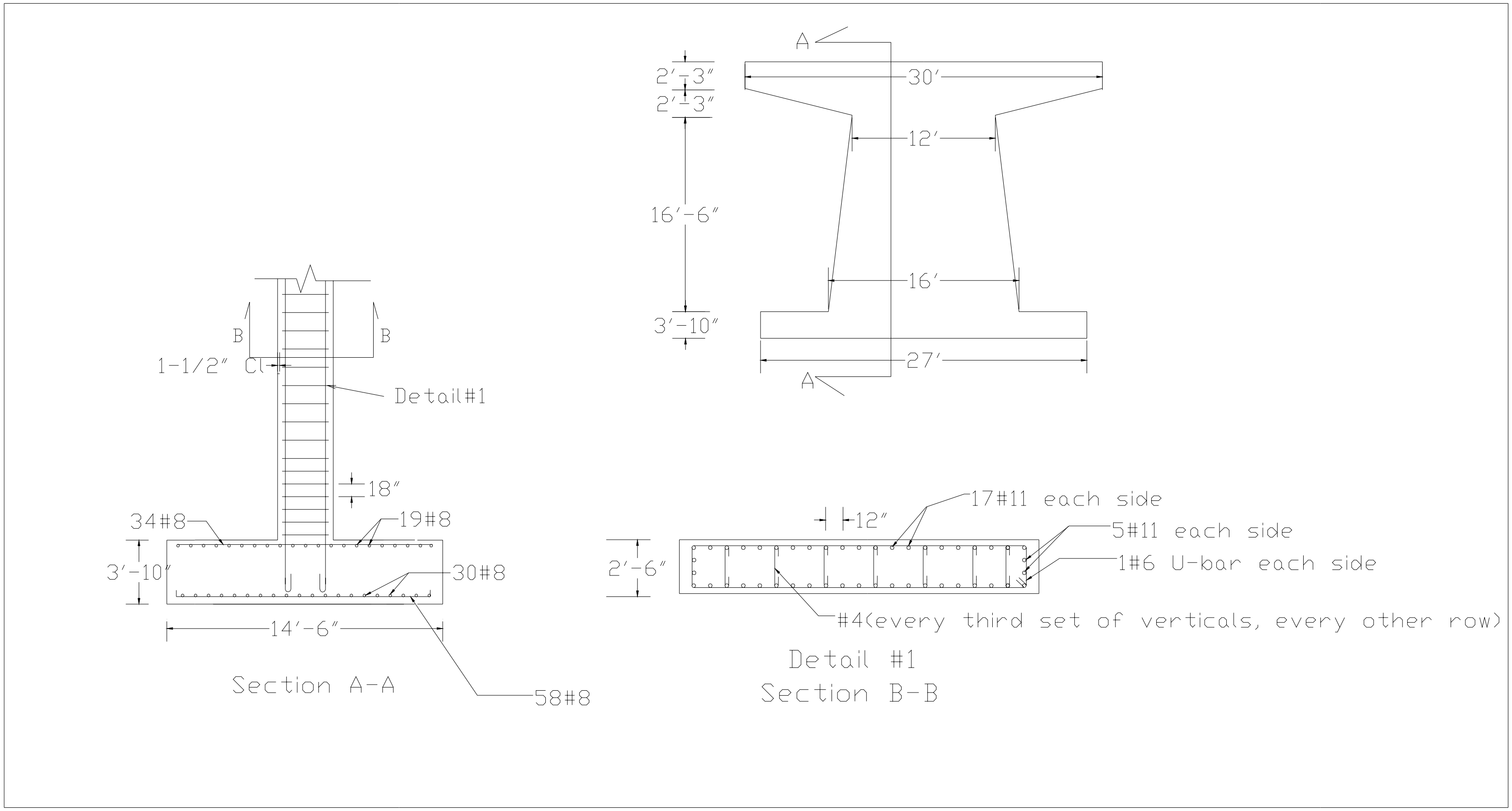


Figure 5.8a – AASHTO Wall Design Details (Fixed Bent)

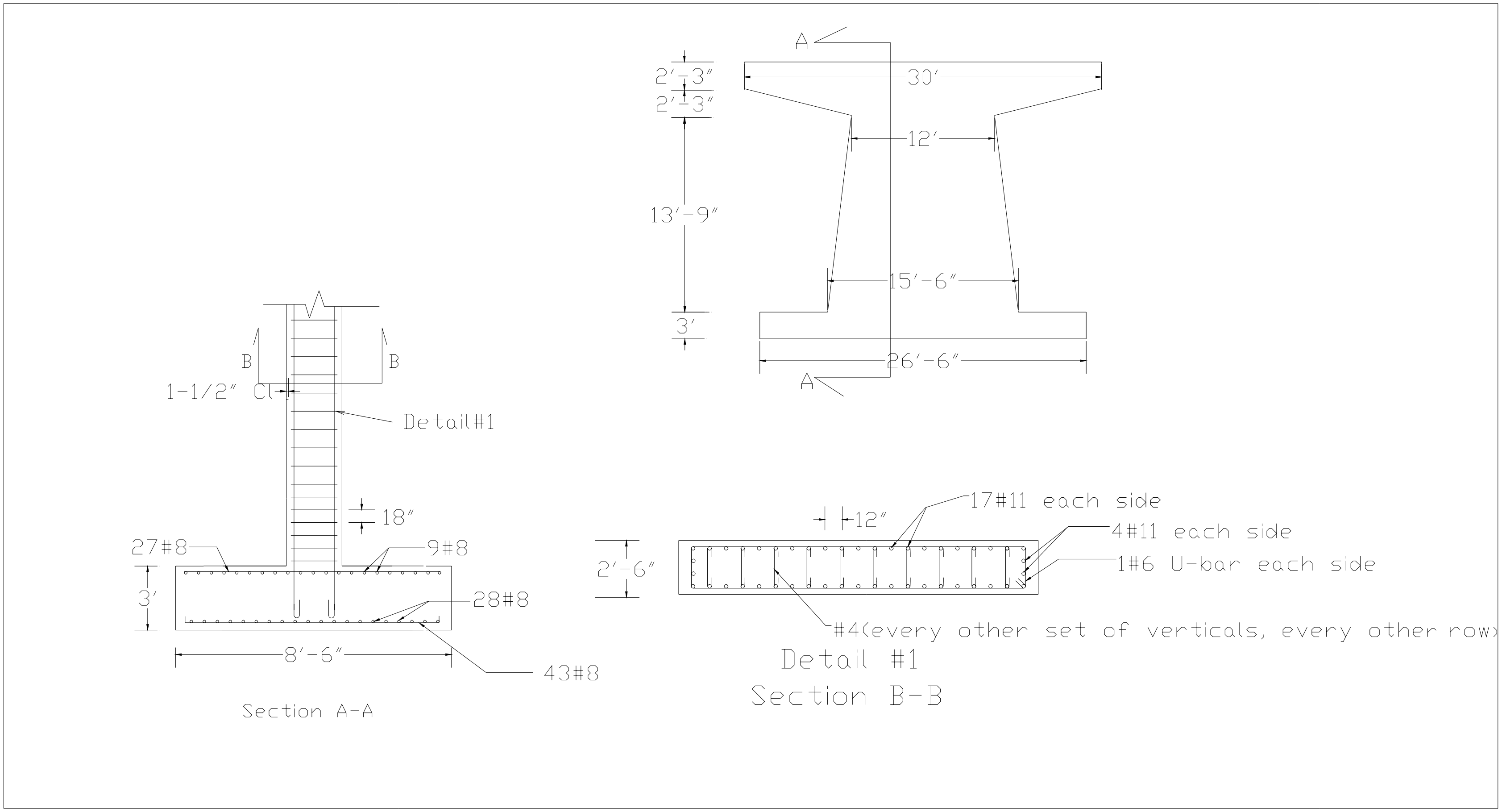
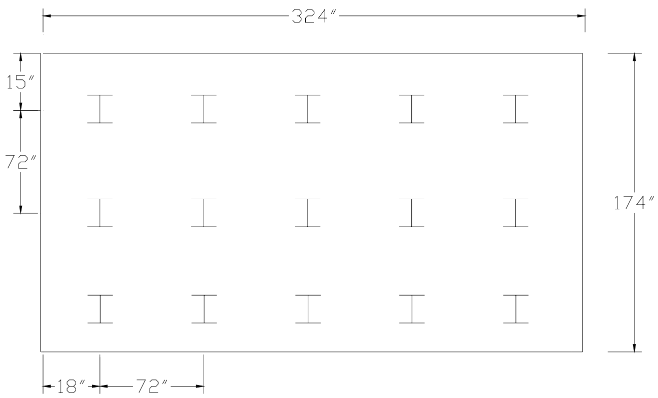
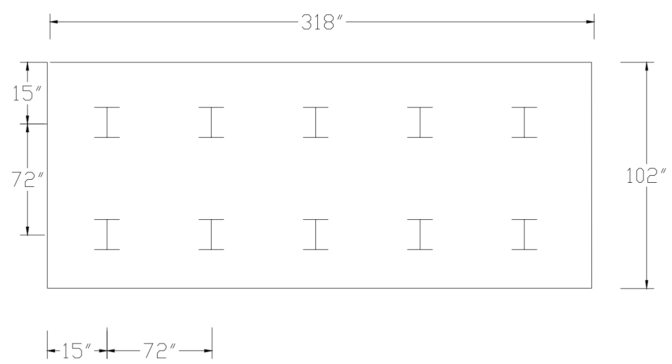


Figure 5.8b – AASHTO Wall Design Details (Expansion Bent)



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)

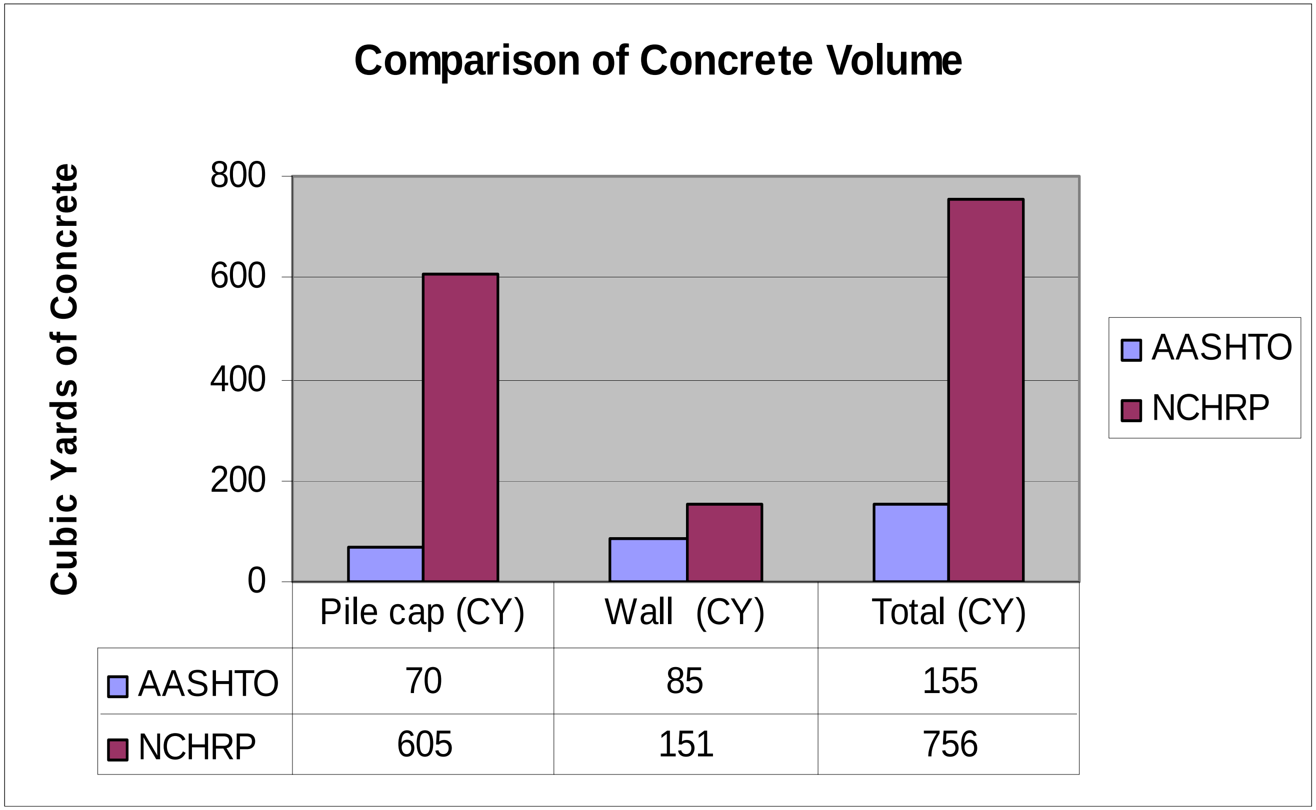


Figure 5.9 – Concrete Volume Comparison (AASHTO vs. NCHRP)

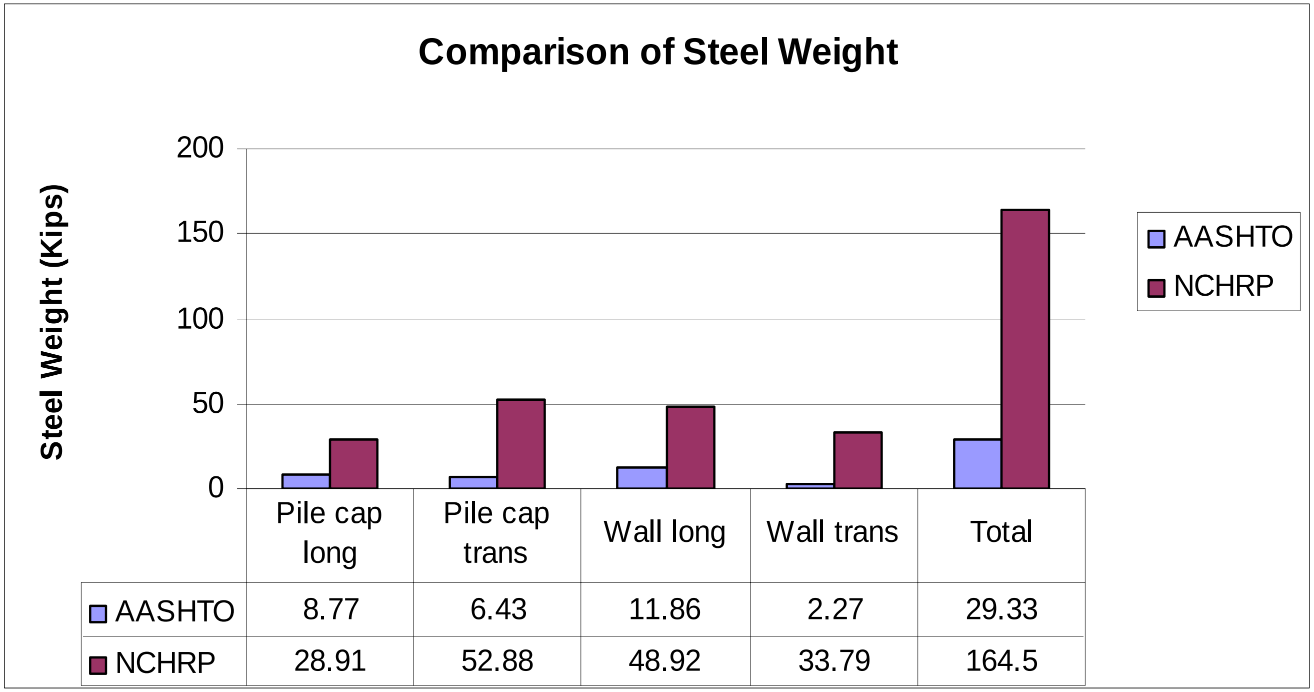


Figure 5.10 – Reinforcing Steel Weight Comparison (AASHTO vs. NCHRP)

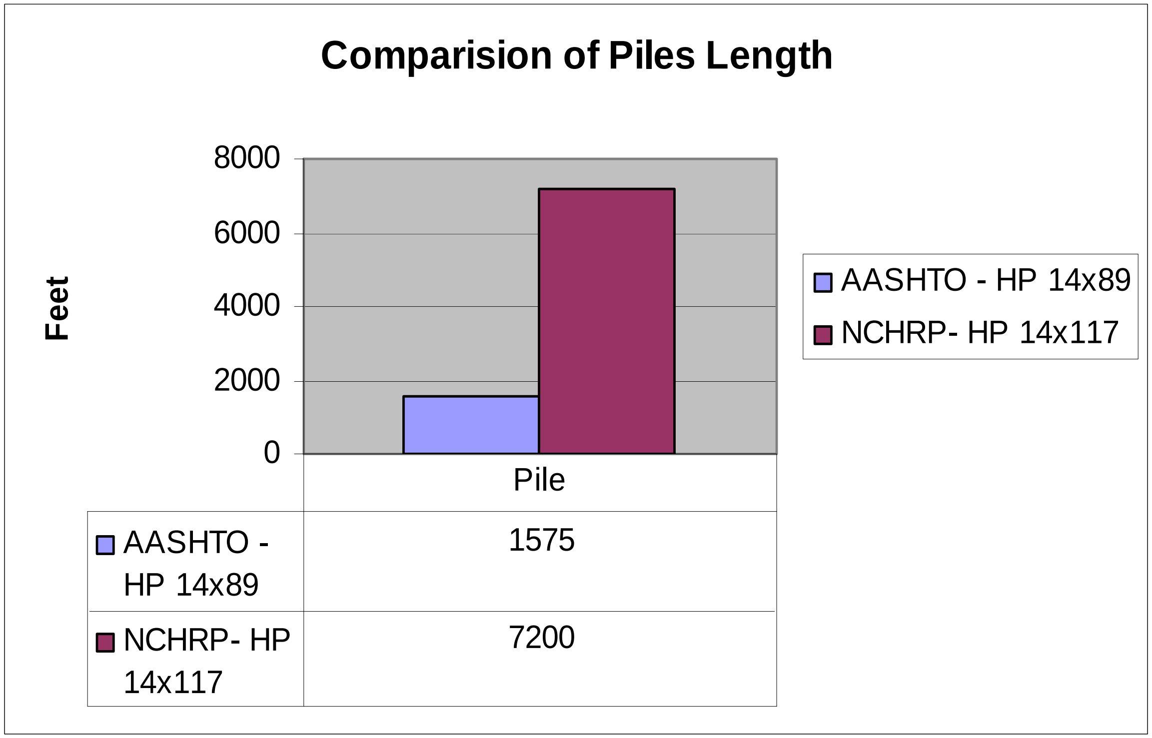


Figure 5.11 – Pile Length Comparison (AASHTO vs. NCHRP)

Table 5.1 – NCHRP Seismic Design Force Result Summary

MCE		**Final Design Forces & Moments {(EQ/ R)+ DL}**									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Pier 2	Top	350	1146	0	28687	519	458	875	11475	0	519
	Bottom			5431	37284				14914	13578	
Pier 1&3	Top	465	1601	0	16116	470	640	1163	6446	0	544
	Bottom			6111	31133				12453	15277	

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 5.2 – AASHTO Seismic Design Force Result Summary

Final Design Forces & Moments			(AASHTO)					
Pier	**Location**	**Joint**	**Load Case**	**Shear 2**	**Shear 3**	**Axial**	**Moment 2**	**Moment 3**
No.				Kip	Kip	Kip	Kip-ft	Kip-ft
			DEAD	0.0	0.0	-352.2	0.0	0.0
1	At "Neck"	**4**	COMD1	21.0	16.4	-335.6	378.2	25.6
Exp			COMD2	6.3	54.8	-347.2	1260.8	7.7
							0.0	0.0
			DEAD	0.0	0.0	-352.2	0.0	0.0
3	At "Neck"	**40**	COMD1	21.0	16.4	-335.6	378.2	25.6
Exp			COMD2	6.3	54.8	-347.2	1260.8	7.7
							0.0	0.0
			DEAD	0.0	0.0	-423.1	0.0	0.0
1	At Bottom	**5**	COMD1	31.7	18.0	-406.4	327.5	204.5
Exp			COMD2	9.5	60.1	-418.1	1091.8	61.4
							0	0
			DEAD	0.0	0.0	-423.1	0.0	0.0
3	At Bottom	**30**	COMD1	31.7	18.0	-406.4	327.5	204.5
Exp			COMD2	9.5	60.1	-418.1	1091.8	61.4
			DEAD	0.0	0.0	427.6	0.0	0.0
2	At "Neck"	**23**	COMD1	480.4	34.8	427.6	317.2	1306.5
Fix			COMD2	144.1	116.1	427.6	1057.5	392.0
							0.0	0.0
2	At Bottom		DEAD	0.0	0.0	-514.8	0.0	0.0
Fix		**14**	COMD1	491.9	39.2	-514.8	517.6	5338.5
			COMD2	147.6	130.7	-514.8	1725.3	1601.5

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 n 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^{o}N, 88.869^{o}W)
- Bridge 2. Located in St. Clair County (38.588^{o}N, 89.912^{o}W)
- Bridge 3. Located in Pulaski County (37.200^{o}N, 89.152^{o}W)
- Bridge 4. Madison County bridge related in Pulaski County (37.283^{o}N, 89.150^{o}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_{max}, corresponding to a very small shear strain (lower than about 3 x 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).

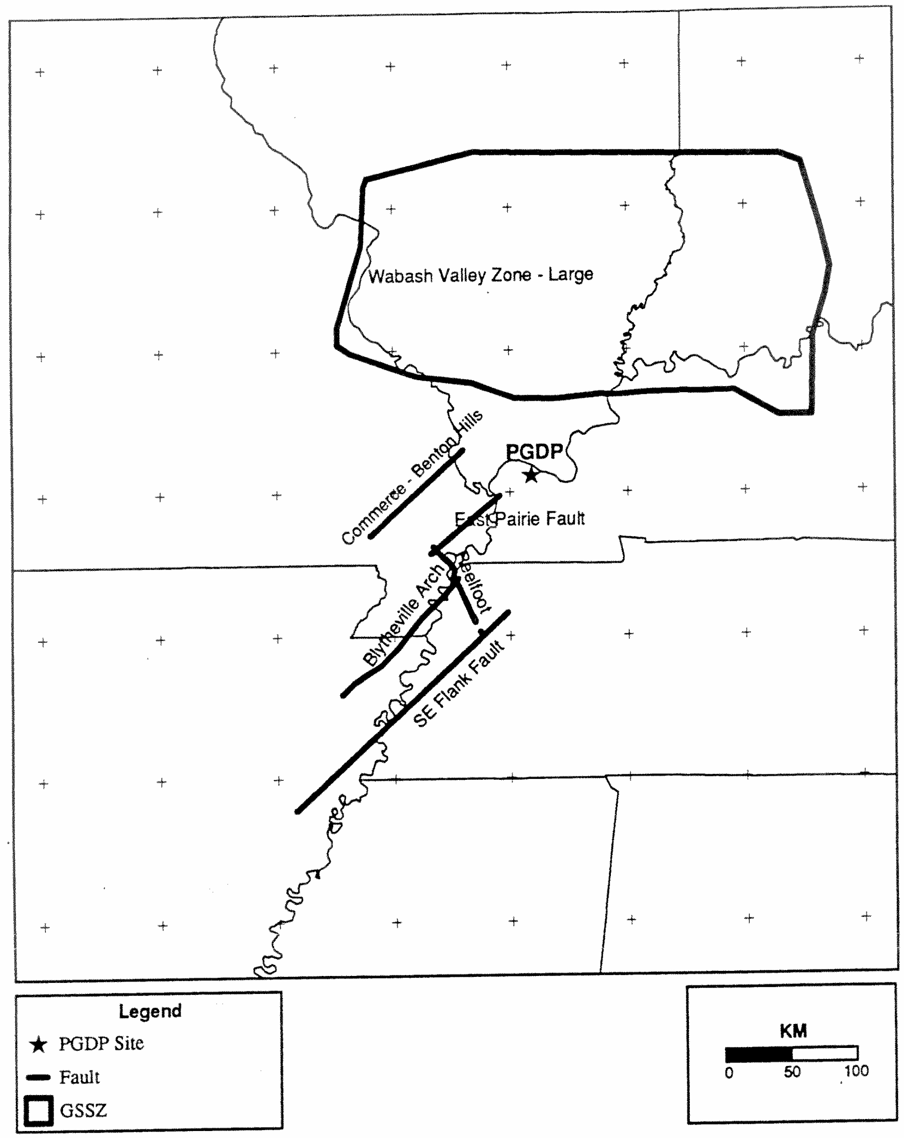


Figure 6.1 – The New Madrid Seismic Zone and Other Seismic Sources Used by Toro Silva (2001)

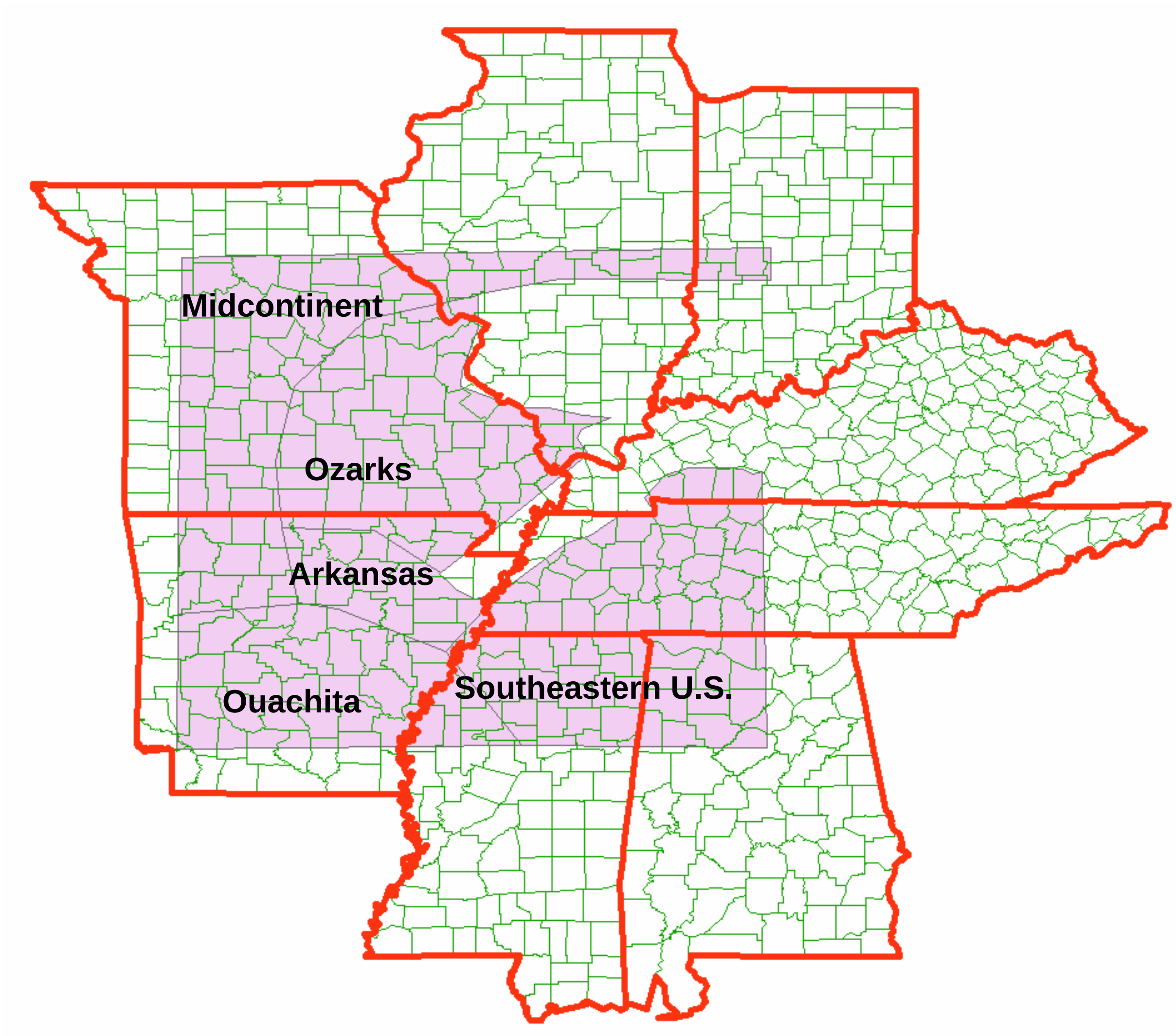


Figure 6.2 – Background Seismic Sources (Adopted from Toro and Silva, 2001).

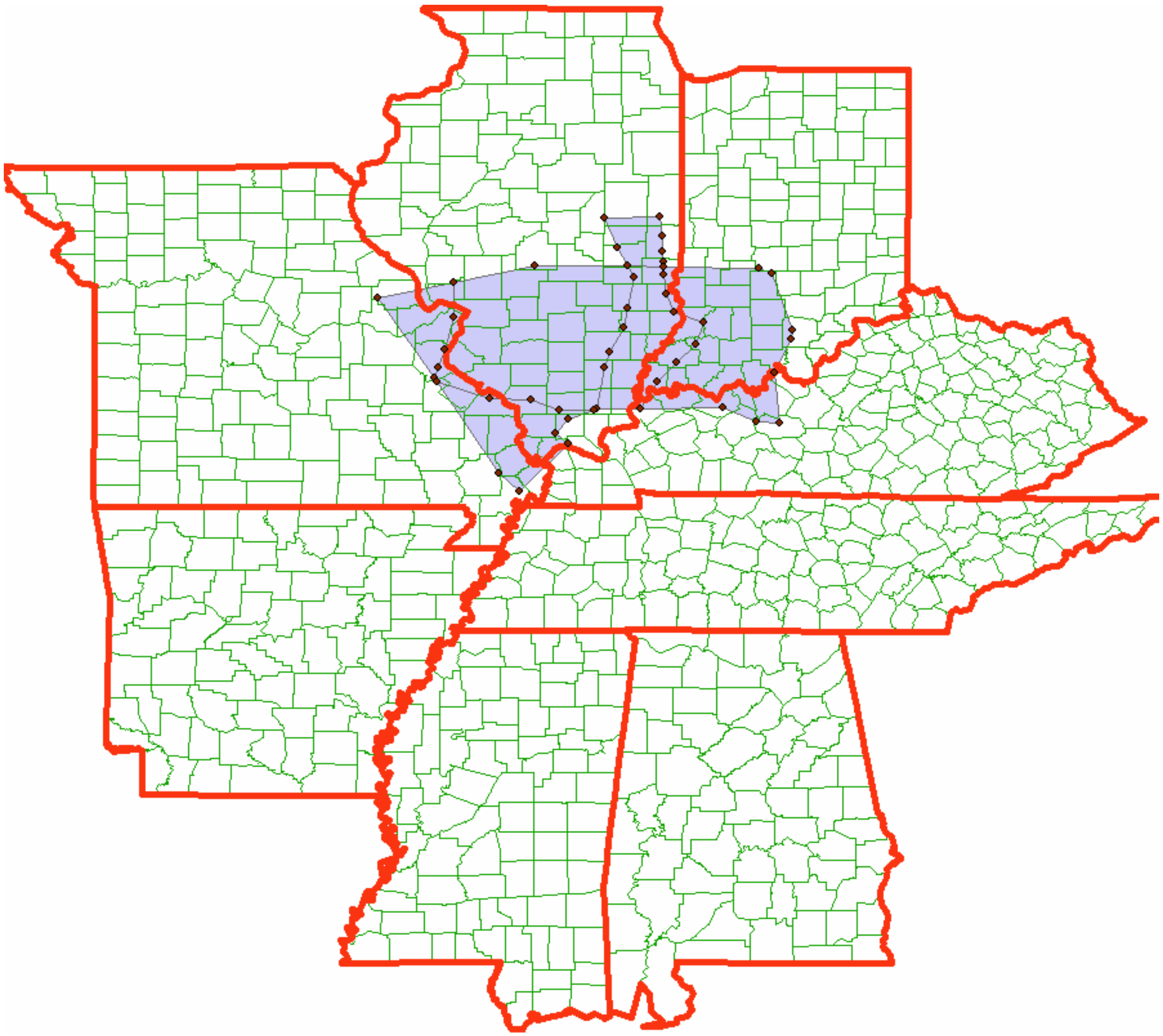


Figure 6.3 – Three Zones Used to Represent Wabash Seismic Zone (Adopted from Toro and Silva, 2001).

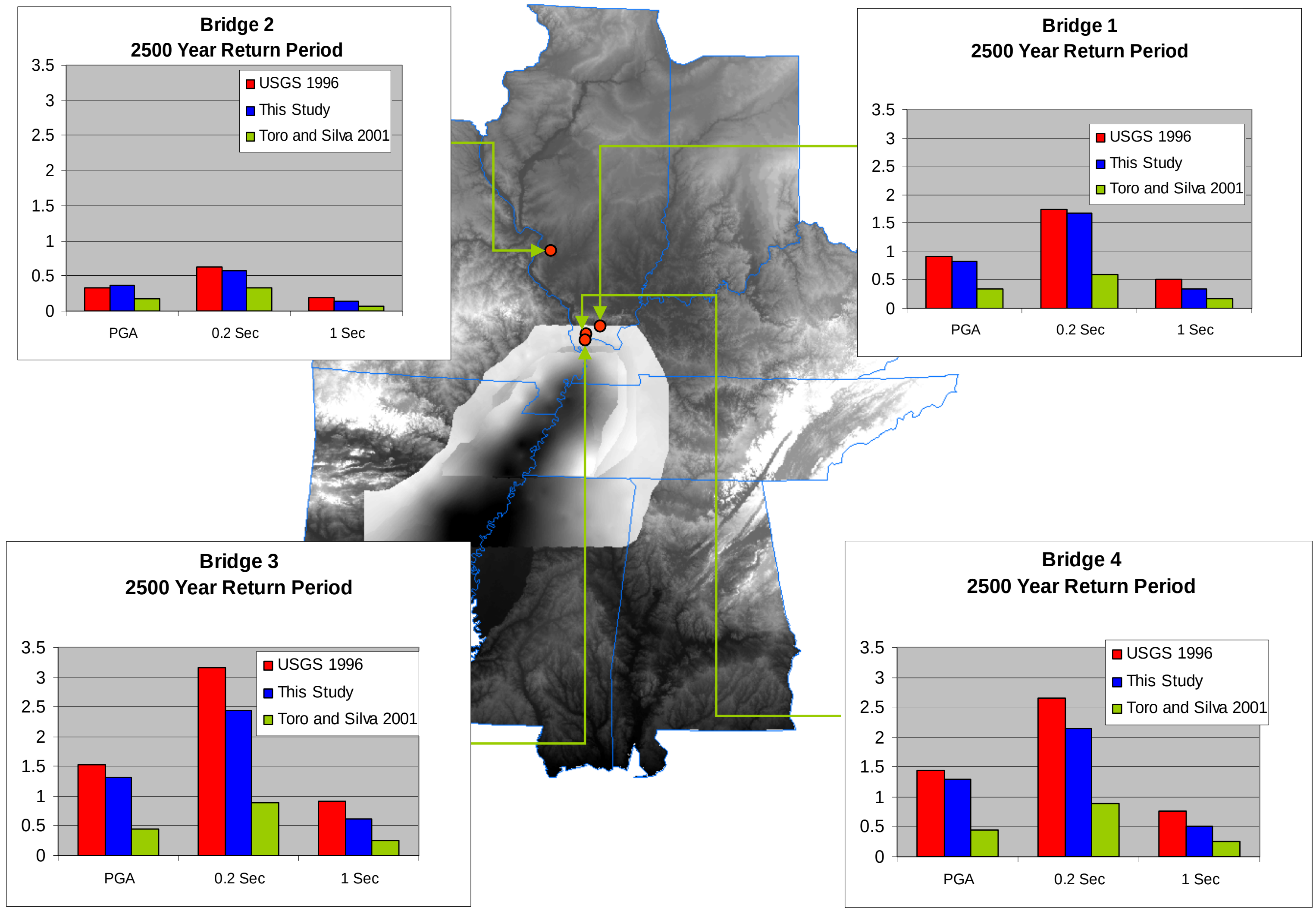


Figure 6.4 – Locations of bridges studied and the corresponding 0.2-second, 1-second spectral accelerations, and PGA comparisons with the USGS 1996 hazard maps, Toro and Silva (2001) for a return period of 2500 years.

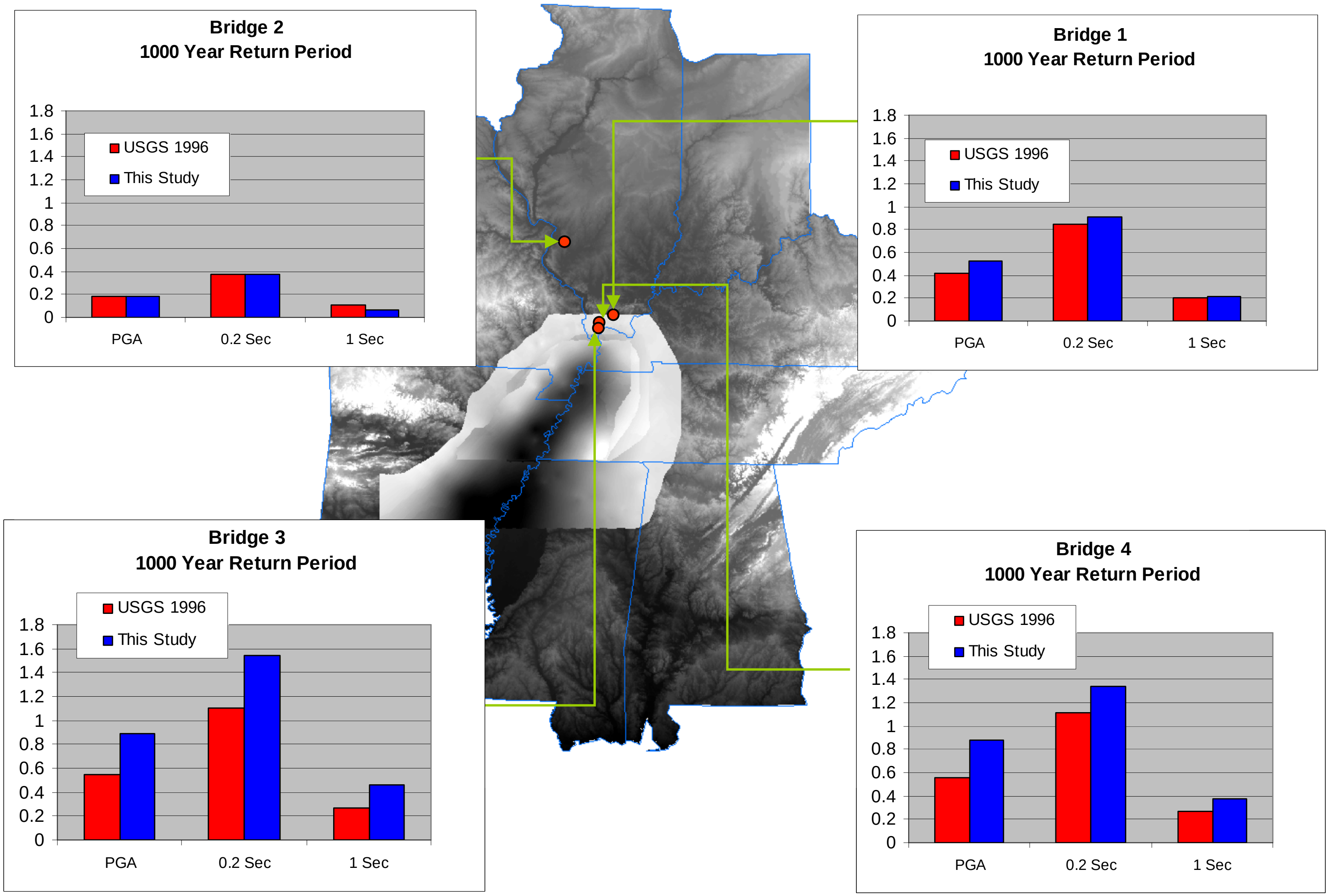


Figure 6.5 – Locations of bridges studied and the corresponding 0.2-second, 1-second spectral accelerations, and PGA comparisons with the USGS 1996 hazard maps for a return period of 1000 years.

Table 6.1 – Summary of Results of the Ground Motion Study at Four Southern Illinois Bridge Sites

	Return Period (Years)	Coordinates		National Hazard Maps			Toro and Silva (2001)			This Study		
		Latitude	**Longitude**	**PGA**	**0.2 sec**	**1 sec**	**PGA**	**0.2 sec**	**1 sec**	**PGA**	**0.2 sec**	**1 sec**
Bridge 1	2500 - USGS 1996	37.433	88.869	0.921	1.750	0.506	0.330	0.600	0.180	0.835	1.685	0.331
	2500 - USGS 2002			0.955	1.766	0.486						
	1000			0.420	0.847	0.203				0.530	0.909	0.210
	Site Coefficient				1.000	1.300					1.390	1.169
	Ground Surface				1.750	0.658					2.342	0.387
Bridge 2	2500 - USGS 1996	38.588	89.912	0.327	0.626	0.192	0.170	0.330	0.075	0.374	0.579	0.148
	2500 - USGS 2002			0.334	0.640	0.181						
	1000			0.184	0.376	0.103				0.177	0.370	0.069
	Site Coefficient				1.337	2.161					0.694	2.865
	Ground Surface				0.837	0.415					0.402	0.424
Bridge 3	2500 - USGS 1996	37.2	89.152	1.533	3.161	0.919	0.450	0.900	0.250	1.320	2.434	0.607
	2500 - USGS 2002			1.737	3.330	1.114						
	1000			0.555	1.102	0.264				0.886	1.542	0.460
	Site Coefficient				1.000	1.500					1.031	2.756
	Ground Surface				3.161	1.379					2.510	1.673
Bridge 4	2500 - USGS 1996	37.283	89.150	1.433	2.655	0.769	0.450	0.900	0.250	1.290	2.153	0.508
	2500 - USGS 2002			1.592	2.930	0.887						
	1000			0.555	1.110	0.264				0.880	1.338	0.377
	Site Coefficient				1.000	1.500					0.688	2.598
	Ground Surface				2.655	1.154					1.480	1.320

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 $Sa_{max\ (MCE)}$ / $Sa_{max\ (AASHTO)}$ and the response modification factor ratio of R_{AASHTO} / R_{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.

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Appendix A -- Detailed Computations for Seismic Analysis and Design of Johnson the County Bridge using Proposed NCHRP Specification

Table of Contents of Appendix-A

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

$S_{DS} := F_a \cdot S_s$

$S_{DS} = 1.75$ Design earthquake response spectral acceleration at short period

$S_{D1} := F_v \cdot S_1$

$S_{D1} = 0.759$ Design earthquake response spectral acceleration at long period

$0.4 \cdot S_{DS} = 0.7$

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

$T_s = 0.434$ Sec

$T_0 := 0.2 \cdot T_s$ Period at the beginning of construction design spectral acceleration plateau

$T_0 = 0.087$ Sec

$F_a \cdot S_s = 1.75$ > 0.6

Seismic Hazard Level IV (Guide Spec. Table 3.7-2)

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

$S_{s\_fe} := 0.1033$ 0.2-second period spectral acceleration

$S_{1\_fe} := 0.0205$ 1-second period spectral acceleration

$F_{a\_fe} := 1.6$ Site coefficient for short period

$F_{v\_fe} := 2.4$ Site coefficient for long period

$S_{DS\_fe} := F_{a\_fe} \cdot S_{s\_fe}$

$S_{DS\_fe} = 0.165$ Design earthquake response spectral acceleration at short period

$S_{D1\_fe} := F_{v\_fe} \cdot S_{1\_fe}$

$S_{D1\_fe} = 0.049$ Design earthquake response spectral acceleration at long period

$0.4 \cdot S_{DS\_fe} = 0.066$

$T_{s\_fe} := \dfrac{S_{D1\_fe}}{S_{DS\_fe}}$ Period at the end of construction design spectral acceleration plateau

$T_{s\_fe} = 0.298$ Sec

$T_{0\_fe} := 0.2 \cdot T_{s\_fe}$ Period at the beginning of construction design spectral acceleration plateau

$T_{0\_fe} = 0.06$ Sec

$F_{a\_fe} \cdot S_{s\_fe} = 0.165$

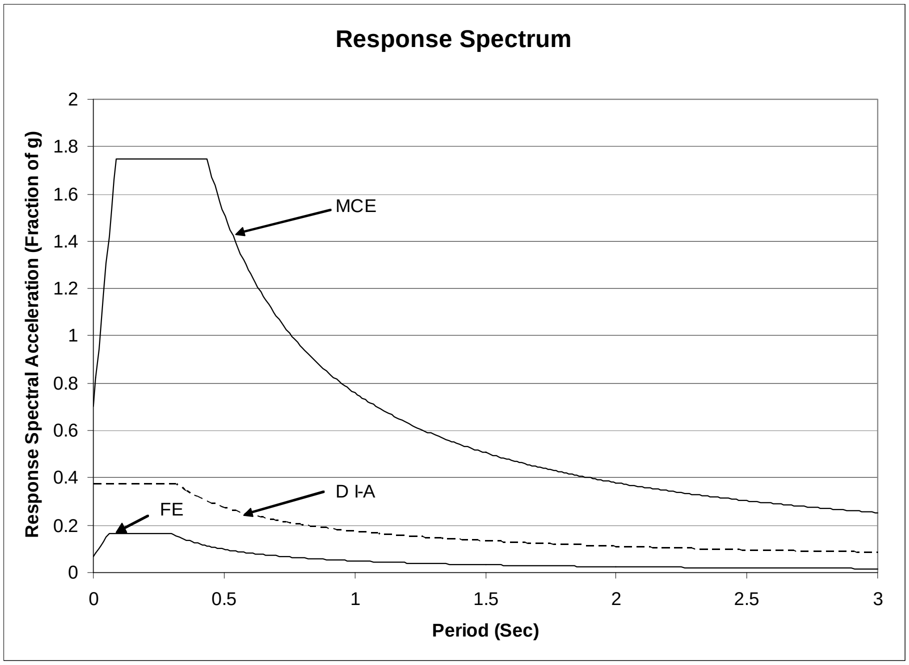

Fig 1. Design response spectrum (MCEER Guide Spec. Fig 3.4.1-1)

Superstructure Section Properties

Rolled beam 27W94

$L_{bridge} := 109.33$ ft — Length of the bridge

$A_{girder} := 27.7$ in^2 — Area of the girders

$d_{girder} := 26.9$ in — Depth of the girders

$I_x := 3270$ in^4

$S_x := 243$ in^3

$I_y := 124$ in^4

$S_y := 24.8$ in^3

6 girder at 7' 2"
Slab thickness = 8"
Hung = 1"

$N := 8$ Number of longitudinal reinforcements

$hung := 1$ in

$b_{slab} := 42 \cdot 12$

$b_{slab} = 504$ in

Effective width of the composite section (AASHTO LRFD 4.6.2.6.2)

$L_1 := 32 \cdot \frac{12}{4}$ 1/4 of the span length

$L_1 = 96$ in

$L_2 := 7.16 \cdot 12$ Girder spacing

$L_2 = 85.92$ in

$L_3 := 8 \cdot 12 + \max(.49, .5 \cdot .745)$

$L_3 = 96.49$ in

$b_{effective} := \min(L_1, L_2, L_3)$

$b_{effective} = 85.92$ in Effective width of the slab in composite section

$h_{slab} := 8$ in Thickness of the slab

$A_{slab} := b_{slab} \cdot \frac{h_{slab}}{N}$

$A_{slab} = 504$ in^2

$I_{x\_slab} := b_{slab} \cdot \frac{h_{slab}^{\;3}}{12 \cdot N}$

$I_{x\_slab} = 2.688 \times 10^3$ in^4

$Y_{slab} := \frac{h_{slab}}{2}$

$Y_{slab} = 4$ in

$$Y_{x\_comp} := \frac{\left[A_{slab} \cdot Y_{slab} + 6A_{girder} \cdot \left(\frac{d_{girder}}{2} + 2 \cdot Y_{slab} + hung\right)\right]}{\left(6A_{girder} + A_{slab}\right)}$$

$Y_{x\_comp} = 8.575$ in From top

$$I_{x\_comp} := 6 \cdot I_x + I_{x\_slab} + 6 \cdot A_{girder} \cdot \left(\frac{d_{girder}}{2} + 2 \cdot Y_{slab} + hung - Y_{x\_comp}\right)^2 + A_{slab} \cdot \left(Y_{slab} - Y_{x\_comp}\right)^2$$

$I_{x\_comp} = 6.485 \times 10^4$ in^4

$$S_{x\_comp} := \frac{I_{x\_comp}}{Y_{x\_comp}}$$

$S_{x\_comp} = 7.563 \times 10^3$ in^4

$A_{comp} := A_{girder} \cdot 6 + A_{slab}$

$A_{comp} = 670.2$ in^2

$$r_x := \sqrt{\frac{I_{x\_comp}}{A_{comp}}}$$

$r_x = 9.837$ in

$$I_{y\_slab} := h_{slab} \cdot \frac{b_{slab}^{\ 3}}{12 \cdot N}$$

$I_{y\_slab} = 1.067 \times 10^7$ in^4

$$I_{y\_comp} := 6 \cdot I_y + 2 \cdot A_{girder} \cdot \left(3.58^2 + 10.74^2 + 17.9^2\right) + I_{y\_slab}$$

$I_{y\_comp} = 1.069 \times 10^7$ in^4

$$S_{y\_comp} := \frac{I_{y\_comp}}{42 \cdot .5 \cdot 12}$$

$S_{y\_comp} = 4.244 \times 10^4$ in^3

$r_y := \sqrt{\dfrac{I_{y\_comp}}{A_{comp}}}$

$r_y = 126.32$ in

$A_{shear} := 0.8 \cdot A_{comp}$

$A_{shear} = 536.16$ in^2

$J := b_{slab} \cdot \dfrac{h_{slab}{}^3}{12 \cdot N} + h_{slab} \cdot \dfrac{b_{slab}{}^3}{12 \cdot N}$

$J = 1.067 \times 10^7$

$S_{x\_plastic} := S_{x\_comp} \cdot 1.5$

$S_{x\_plastic} = 1.134 \times 10^4$ in^3

$S_{y\_plastic} := S_{y\_comp} \cdot 1.5$

$S_{y\_plastic} = 6.366 \times 10^4$ in^3

Mass Calculation

$W_{slab} := 42 \cdot \dfrac{8 \cdot L_{bridge}}{12} \cdot .15$

$W_{slab} = 459.186$ kips

$W_{girders} := \dfrac{94}{1000} \cdot 6 \cdot L_{bridge} \cdot 1.15$

$W_{girders} = 70.911$ kips

$Bar := 5.2 \cdot L_{bridge} \cdot .15$

$Bar = 85.277$ kips

$$FWS := \frac{35}{1000} \cdot 40 \cdot L_{bridge}$$

$FWS = 153.062$ kips   Feature wearing surface

$$W_{total} := W_{slab} + W_{girders} + Bar + FWS$$

$W_{total} = 768.437$ kips

$$w_{total} := \frac{W_{total}}{109.33}$$

$w_{total} = 7.029$ klf

$$W_{pilecap} := 7.5 \cdot 27.5 \cdot 3 \cdot .15$$

$W_{pilecap} = 92.813$ kips

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.

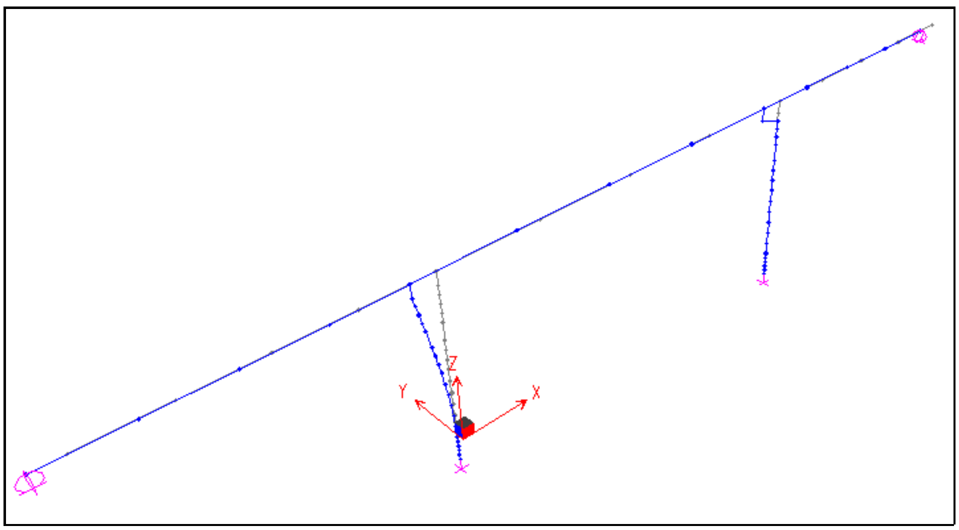

Fig 2. First Mode Shape (T=0.4902 Sec)

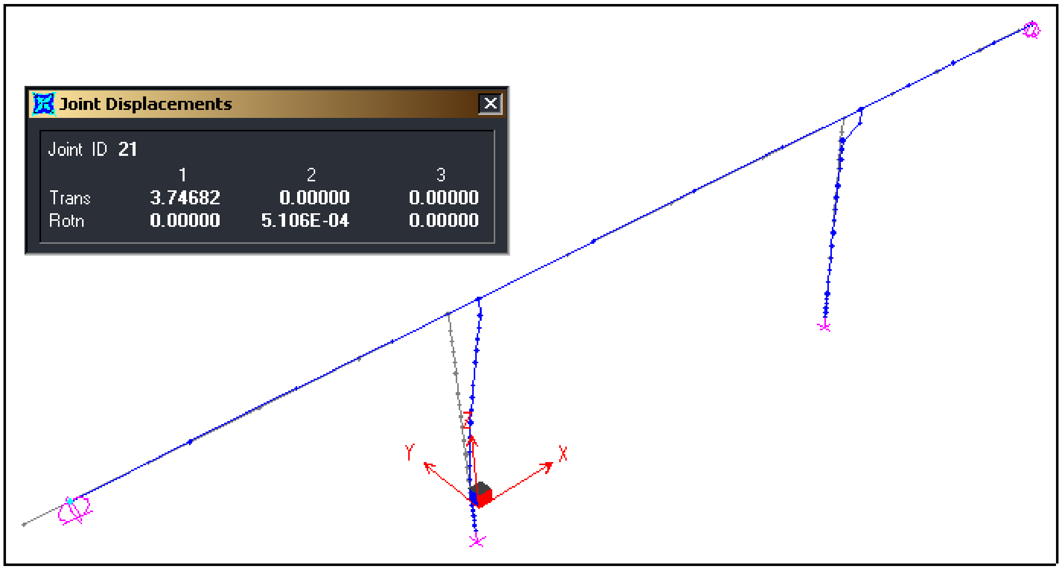

Fig 3. Displacement Diagram for MCE

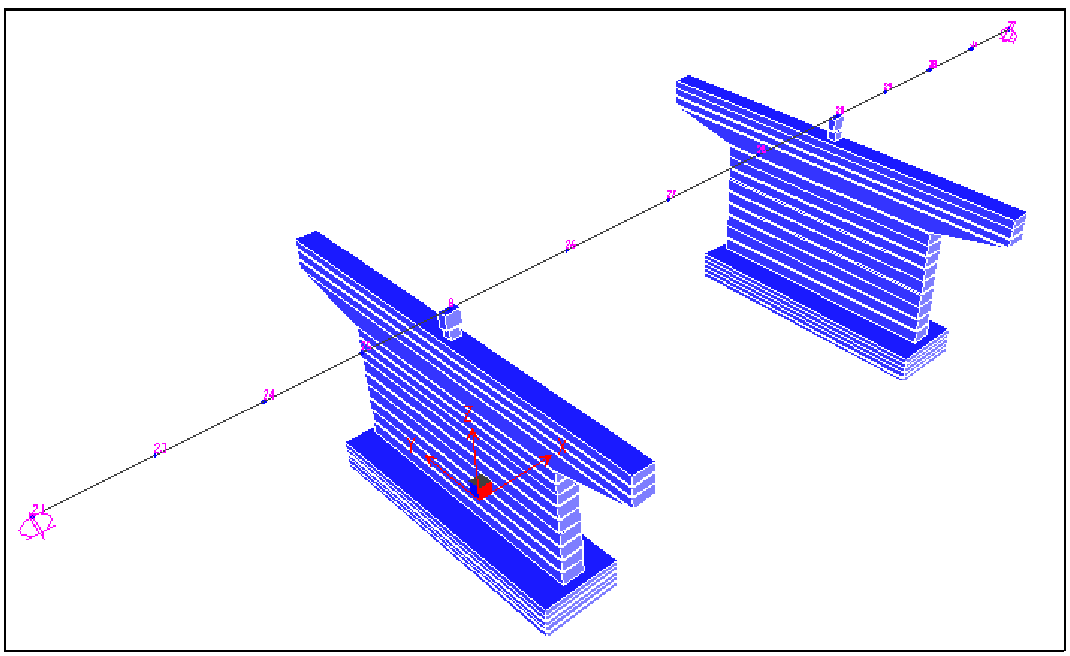

Fig 4. Finite element model of the bridge

Table 1. Periods of the bridge

MODAL PARTICIPATING MASS RATIOS				
With Fix Base				
MODE	PERIOD	INDIVIDUAL MODE (%)		
		UX	UY	UZ
1	0.4902	67.52	0.00	0.00
2	0.1288	0.02	0.00	5.94
3	0.1245	8.80	0.00	0.00
4	0.0703	0.00	0.44	0.00
5	0.0692	0.01	0.00	0.00
6	0.0625	0.00	0.00	44.02
7	0.0555	0.00	62.10	0.00
8	0.0398	0.01	0.00	0.00
9	0.0362	0.00	13.44	0.00
10	0.0262	2.29	0.00	0.02
11	0.0228	0.00	0.00	7.30
12	0.0223	0.62	0.00	0.30
13	0.0219	0.08	0.00	0.03
14	0.0215	0.27	0.00	0.22
15	0.0186	2.60	0.00	0.00
16	0.0184	0.00	0.00	0.00
17	0.0135	0.00	2.54	0.00
18	0.0134	0.00	0.00	0.22
19	0.0132	0.00	0.00	8.24
100	0.0007	0.00	0.53	0.00
		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 claculated based on P-multipliers curves with the coefficients of FHWA.

$L_{pile} := 450$ in Length of the pile

$D := 3$ ft Diameter of the piles

$n := 8$ Ratio of Es/Ec

$N_{fix} := 18$ Number of rebar at each pile (fix pier)

$N_{exp} := 14$ Number of rebar at each pile (expansion pier)

$A_b := 1$ Area of longitudinal rebar

$$A_{axial} := \left(\frac{3.14}{4}\right)\cdot D^2$$

$A_{axial} = 7.065$ ft^2

$$L := \frac{L_{pile}}{12}$$

$L = 37.5$ ft

$E_c := 3600 \cdot 144$ ksf

$E_c = 5.184 \times 10^5$

$$K_{axial} := A_{axial} \cdot \frac{E_c}{L}$$ (Guide Spec. 8.4.3.3)

$K_{axial} = 9.767 \times 10^4$ $\frac{k}{ft}$

Group pile stiffness for expansion bent

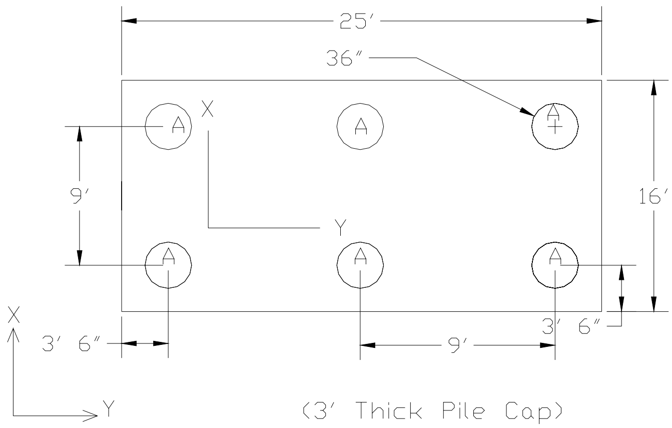

Fig 5. Pile group at the expansion bent

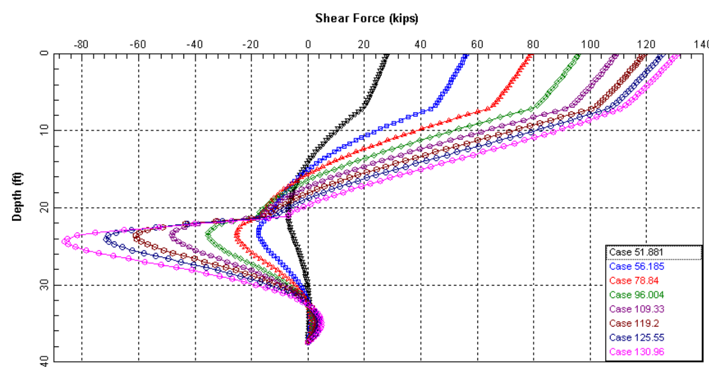

Fig 6. P-multipiler = 0.3 for the third row (shear vs. depth of the pile)

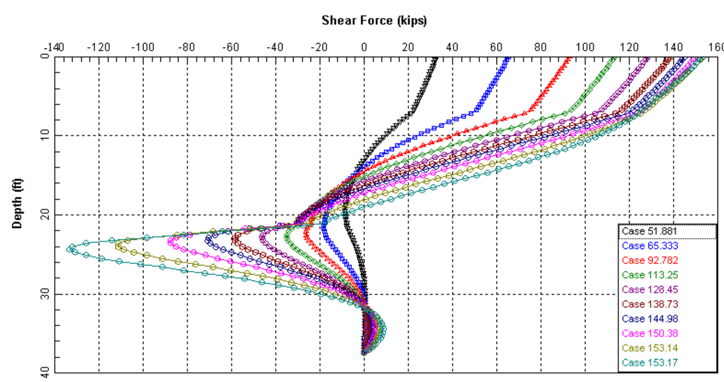

Fig 7. P-multipiler = 0.4 for the second row (shear vs. depth of the pile)

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SHEET NO 12 OF ____ JOB NO 044-0041

SUBJECT FILE

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

TITLE: **Johnson County Bridge**

BY	DATE	CHECKED	DATE
BE	02/18/03		

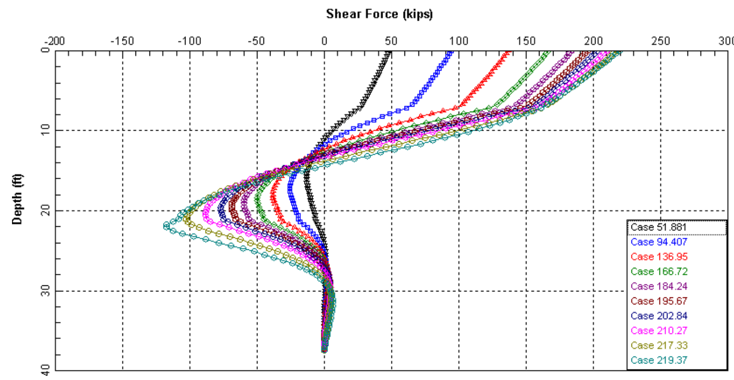

Fig 8. P-multipiler = 0.8 for the leading row (shear vs. depth of the pile)

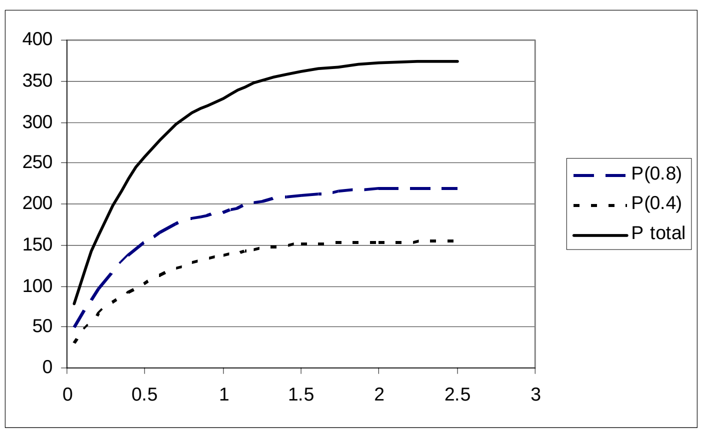

Fig 9. P-multiplier curve for longitudinal direction of bridge (displacement vs. shear)

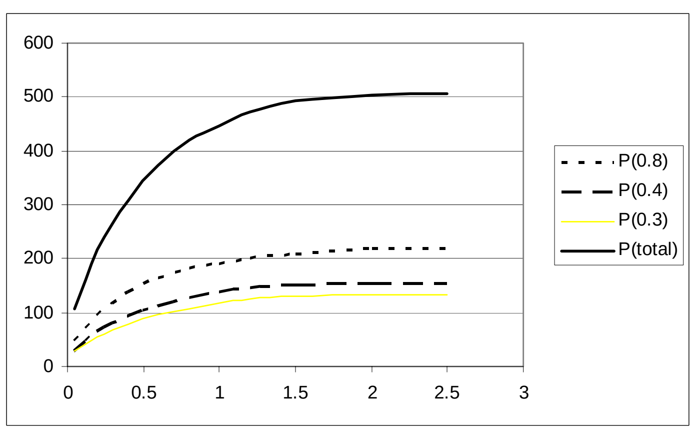

Fig 10. P-multiplier curve for transverse direction of bridge (displacement vs. shear)

$\Delta_x := \frac{.283}{12}$ $\Delta_x = 0.024$ ft Displacement of the pile cap in longitudinal direction

$F_x := 200$ 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

$K_{pilex} := \frac{F_x}{\Delta_x \cdot n_x}$

$K_{pilex} = 4.24 \times 10^3$ $\frac{k}{ft}$

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

$K_{pile} := \frac{(K_{pilex} + K_{piley})}{2}$

$K_{pile} = 3.303 \times 10^3$ $\frac{k}{ft}$

$N_{pile} := 6$ Number of the piles

$D := 3$ ft

$n := 8$

$N_{exp} = 14$ Number of longitudinal reinforcement

$A_b := 1$

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

$A_{axial} = 7.746$ ft^2

$L := \frac{L_{pile}}{12}$

$L = 37.5$ ft

$E_c := 3600 \cdot 144$ ksf

$E_c = 5.184 \times 10^5$

$$K_{axial} := A_{axial} \cdot \frac{E_c}{L}$$

$$K_{axial} = 1.071 \times 10^5 \ \frac{k}{ft}$$

$$K_{x\_total} := N_{pile} \cdot K_{pilex}$$

$$K_{y\_total} := N_{pile} \cdot K_{piley}$$

$$K_{axial} := N_{pile} \cdot K_{axial}$$

$$K_{r\_y} := 6K_{axial} \cdot 4.5^2$$

$$K_{r\_x} := 4K_{axial} \cdot 9^2$$

$$K_{r\_z} := K_{pile} \cdot \left(6 \cdot 4.5^2 + 4 \cdot 9^2\right)$$

$$K_{x\_total} = 2.544 \times 10^4 \ \frac{k}{ft}$$

$$K_{y\_total} = 1.42 \times 10^4 \ \frac{k}{ft}$$

$$K_{axial} = 6.424 \times 10^5 \ \frac{k}{ft}$$

$$K_{r\_x} = 2.082 \times 10^8 \ \frac{k}{ft}$$

$$K_{r\_y} = 7.806 \times 10^7 \ \frac{k}{ft}$$

$$K_{r\_z} = 1.472 \times 10^6 \ \frac{k}{ft}$$

Group pile stiffness for fixed bent

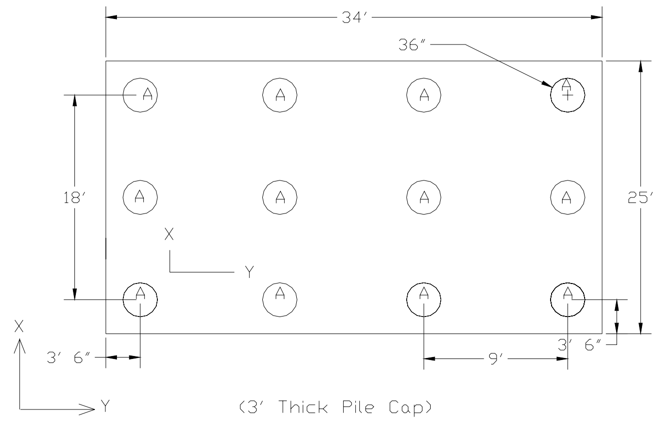

Fig 11. Pile group at the fixed bent

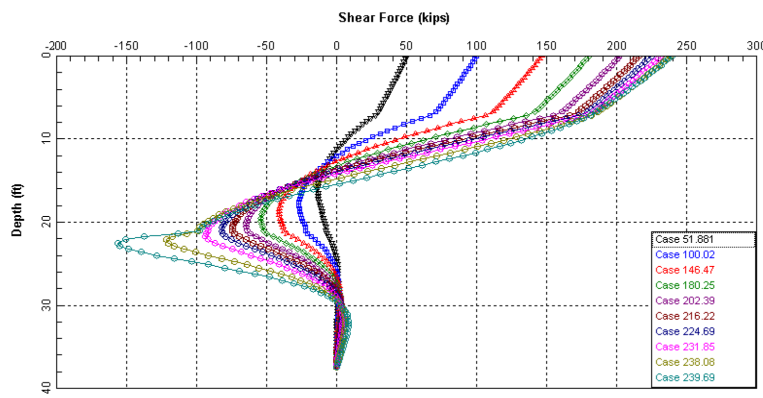

Fig 12.P-multiplier = 0.8 for the leading row (shear vs. depth of the pile)

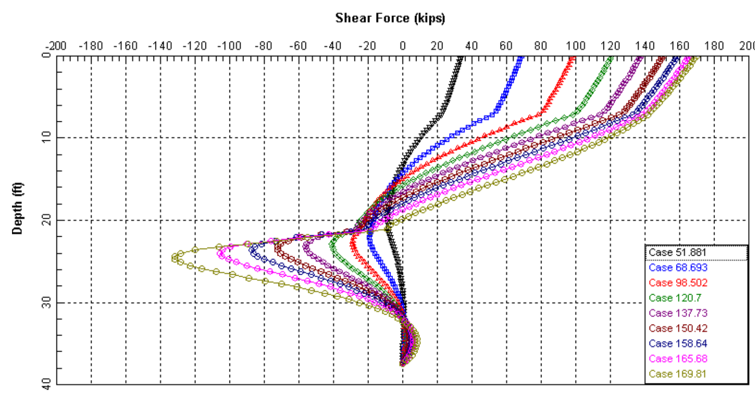

Fig 13. P-multiplier = 0.4 for the second row (shear vs. depth of the pile)

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SHEET NO 16 OF ____			JOB NO 044-0041	
SUBJECT FILE **Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois**				
TITLE: **Johnson County Bridge**	BY **BE**	DATE **02/18/03**	CHECKED	DATE

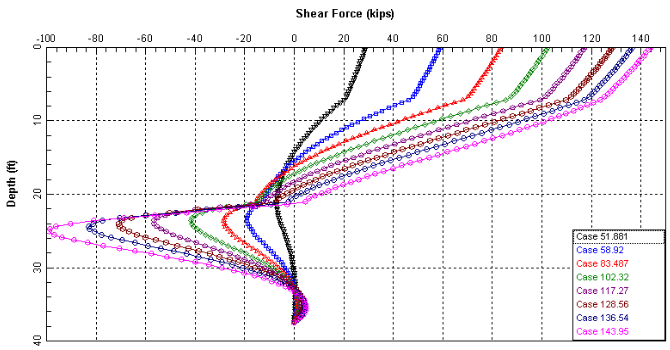

Fig 14. P-multiplier = 0.3 for the third row (shear vs. depth of the pile)

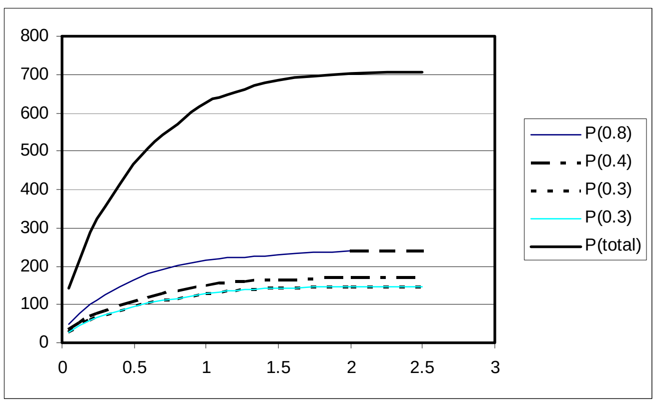

Fig 15. P-multiplier curve for transverse direction of bridge (displacement vs. shear)

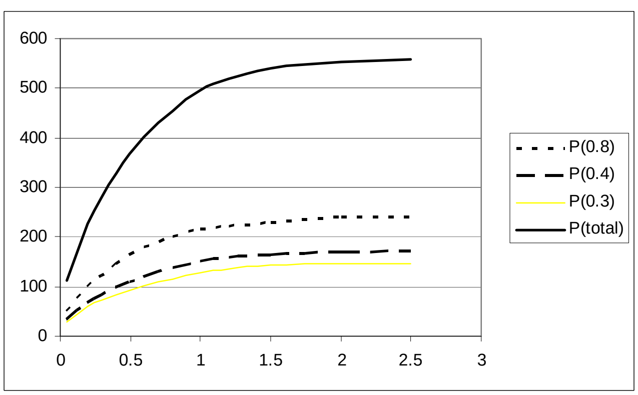

Fig 16. P-multiplier curve for longitudinal direction of bridge (displacement vs. shear)

$K_{pilex} := 1670 \quad \frac{k}{ft}$

From Fig 16 and the 1.27" displacement of the pile cap in longitudinal direction (average of 3 pile)

$K_{piley} := 2380 \quad \frac{k}{ft}$

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

$$K_{pile} := \frac{(K_{pilex} + K_{piley})}{2}$$

$$K_{pile} = 2.025 \times 10^3 \quad \frac{k}{ft}$$

$N_{pile} := 12$

$D := 3$ ft for the piles

$n := 8$

$N_{fix} = 18$

$A_b := 1$

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

$A_{axial} = 7.94 \quad ft^2$

$$L := \frac{L_{pile}}{12}$$

$L = 37.5 \quad ft$

$E_c := 3600\cdot 144 \text{ ksf}$

$E_c = 5.184 \times 10^5$

$$K_{axial} := A_{axial}\cdot\frac{E_c}{L}$$

$$K_{axial} = 1.098 \times 10^{5} \ \frac{k}{ft}$$

$$K_{x\_total} := N_{pile} \cdot K_{pilex}$$

$$K_{y\_total} := N_{pile} \cdot K_{piley}$$

$$K_{axial} := N_{pile} \cdot K_{axial}$$

$$K_{r\_y} := 8K_{axial} \cdot 9^{2}$$

$$K_{r\_x} := 6 \cdot K_{axial} \cdot \left(13.5^{2} + 4.5^{2}\right)$$

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

$$K_{x\_total} = 2.004 \times 10^{4} \ \frac{k}{ft}$$

$$K_{y\_total} = 2.856 \times 10^{4} \ \frac{k}{ft}$$

$$K_{axial} = 1.317 \times 10^{6} \ \frac{k}{ft}$$

$$K_{r\_x} = 1.6 \times 10^{9} \ \frac{k}{ft}$$

$$K_{r\_y} = 8.535 \times 10^{8} \ \frac{k}{ft}$$

$$K_{r\_z} = 3.773 \times 10^{6} \ \frac{k}{ft}$$

Table 2. Periods of the bridge: (a) MCE (b) FE

(a)

MODAL PARTICIPATING MASS RATIOS				
With Spring for the MCE				
MODE	PERIOD	INDIVIDUAL MODE (%)		
		UX	UY	UZ
1	0.4035	68.93	0.00	0.00
2	0.2898	0.00	22.88	0.00
3	0.2067	0.00	77.00	0.00
4	0.1683	18.24	0.00	0.00
5	0.1292	0.98	0.00	4.36
6	0.1133	6.83	0.00	0.54
7	0.0884	5.02	0.00	0.00
8	0.0687	0.00	0.00	0.25
9	0.0612	0.00	0.00	42.32
10	0.0431	0.00	0.07	0.00
11	0.0392	0.00	0.00	0.45
12	0.0389	0.00	0.00	0.00
13	0.0303	0.00	0.05	0.00
14	0.0289	0.00	0.00	25.51
15	0.0242	0.00	0.00	9.92
16	0.0239	0.00	0.00	9.35
17	0.0210	0.00	0.00	0.81
18	0.0181	0.00	0.00	0.09
19	0.0178	0.00	0.00	0.00
20	0.0171	0.00	0.00	0.00
		100.00	100.00	93.61

(b)

MODAL PARTICIPATING MASS RATIOS				
With Spring for the FE				
MODE	PERIOD	INDIVIDUAL MODE (%)		
		UX	UY	UZ
1	0.4035	68.93	0.00	0.00
2	0.1683	18.24	0.00	0.00
3	0.1292	0.98	0.00	4.36
4	0.1133	6.83	0.00	0.54
5	0.1118	0.00	92.45	0.00
6	0.0884	5.02	0.00	0.00
7	0.0687	0.00	0.00	0.25
8	0.0612	0.00	0.00	42.32
9	0.0566	0.00	0.08	0.00
10	0.0392	0.00	0.00	0.45
11	0.0313	0.00	0.02	0.00
12	0.0289	0.00	0.00	25.51
13	0.0242	0.00	0.00	9.92
14	0.0239	0.00	0.00	9.35
15	0.0210	0.00	0.00	0.81
16	0.0205	0.00	0.04	0.00
17	0.0181	0.00	0.00	0.09
18	0.0178	0.00	0.00	0.00
19	0.0167	0.00	0.00	0.26
20	0.0161	0.00	3.90	0.00
		100.00	96.48	93.88

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TITLE: Johnson County Bridge	BY BE	DATE 02/18/03	CHECKED	DATE

MCEER Guide Spec. Table 4.7-1

Base Response Modification Factor (R_B) SDAP D Operational Performance	
Single column	1.5
Wall pier	1.0
Vertical pile	1
All elements for FE	0.9
Superstructure to abutment	0.8
Column and pile to cap beam	0.8

$T_S = 0.434$ Sec

$T := .4035$ Sec

$R_{col} := 1 + (1.5 - 1) \cdot \left(\frac{T}{1.25 \cdot T_S} \right)$ 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

$R_{FE} := 0.9$

Response Modification Factor (R) SDAP D Operational Performance	
Single column	1.37
Wall pier	1.0
Vertical pile	1
All elements for FE	0.9
Superstructure to abutment	0.8
Column and pile to cap beam	0.8

Dead Load	
Location	Axial kips
Fix Pier	534
Exp Pier	486

MCE		Forces & Moments - EQ_{trans}			
		Transverse			
Support / Location		Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	1283	2660	0	0
	Bottom		21677	0	
Exp Pier	Top	678	1977	0	0
	Bottom		12817	0	

MCE		Forces & Moments - EQ_{long}			
		Longitudinal			
Support / Location		Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	1750	0	0	59
	Bottom		0	29504	
Exp Pier	Top	337	0	0	83
	Bottom		0	4185	

MCE		100% + 40% Forces & Moments - EQ									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	700	1283	0	2660	24	513	1750	1064	0	59
	Bottom			11802	21835				8734	29504	
Exp Pier	Top	135	678	0	1977	33	271	337	791	0	83
	Bottom			1674	12780				5112	4185	

MCE		Factored Forces & Moments - EQ by R									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	510	1283	0	2660	17	513	1276	1064	0	43
	Bottom			8602	21835				8734	21504	
Exp Pier	Top	98	678	0	1977	24	271	246	791	0	60
	Bottom			1220	12780				5112	3050	

MCE		Final Design Forces & Moments (EQ + DL)									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	510	1283	0	2660	551	513	1276	1064	0	577
	Bottom			8602	21835				8734	21504	
Exp Pier	Top	98	678	0	1977	510	271	246	791	0	546
	Bottom			1220	12780				5112	3050	

FE		Forces & Moments - EQ_{trans}			
		Transverse			
Support / Location		Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	10	211	0	0
	Bottom		311	0	
Exp Pier	Top	10	140	0	0
	Bottom		470	0	

FE		Forces & Moments - EQ_{long}			
		Longitudinal			
Support / Location		Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	123	0	0	6
	Bottom		0	2115	
Exp Pier	Top	32	0	0	7
	Bottom		0	395	

FE		100% + 40% Forces & Moments - EQ									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	49	10	0	211	2	4	123	84	0	6
	Bottom			846	311				124	2115	
Exp Pier	Top	13	10	0	140	3	4	32	56	0	7
	Bottom			158	470				188	395	

FE		Factored Forces & Moments - EQ by R									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top Bottom	55	11	0 940	234 346	3	4	137	94 138	0 2350	7
Exp Pier	Top Bottom	14	11	0 176	156 522	3	4	36	62 209	0 439	8

FE		Final Design Forces & Moments (EQ + DL)									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top Bottom	55	11	0 940	234 346	537	4	137	94 138	0 2350	541
Exp Pier	Top Bottom	14	11	0 176	156 522	489	4	36	62 209	0 439	494

Displacement		**Longitudinal Direction (in)**	**Transverse Direction (in)**
MCE	Top	3.11	0.95
	Bottom Fix	1.16	0.77
	Bottom Exp	0.25	0.89
FE	Top	0.23	0.01
	Bottom Fix	0.03	0.02
	Bottom Exp	0.09	0.02

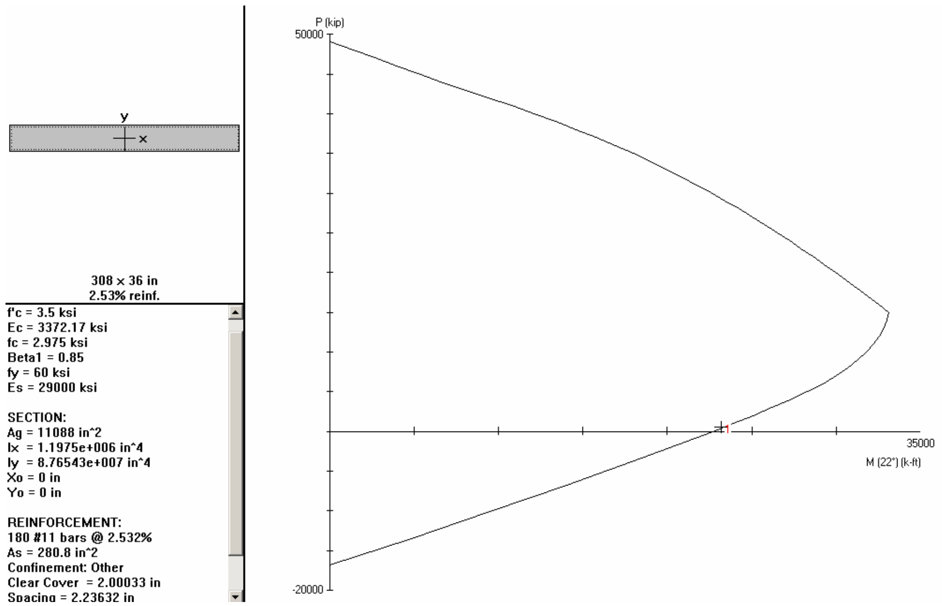

Fig 17: Interaction diagram for the fixed pier

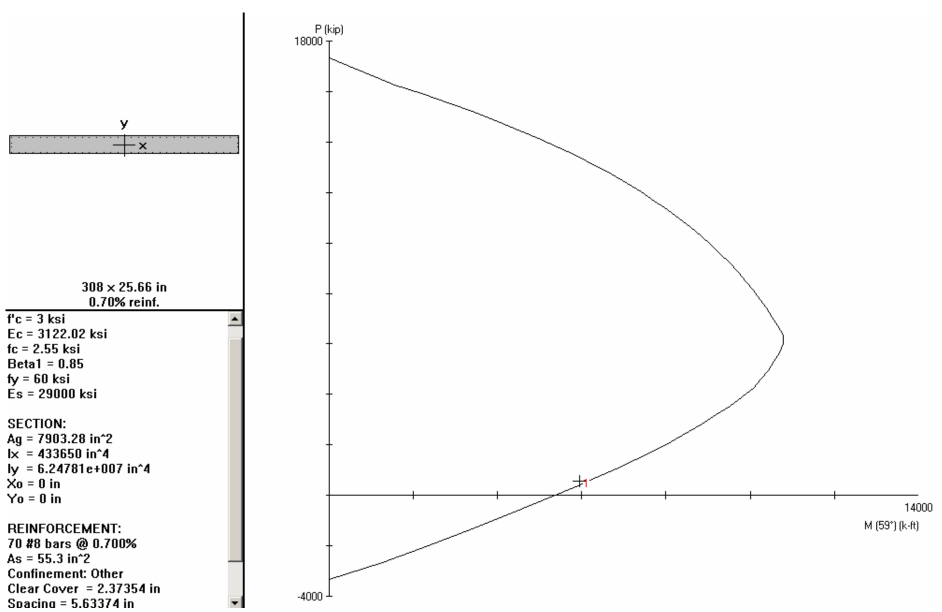

Fig 18: Interaction diagram for the expansion pier

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%)

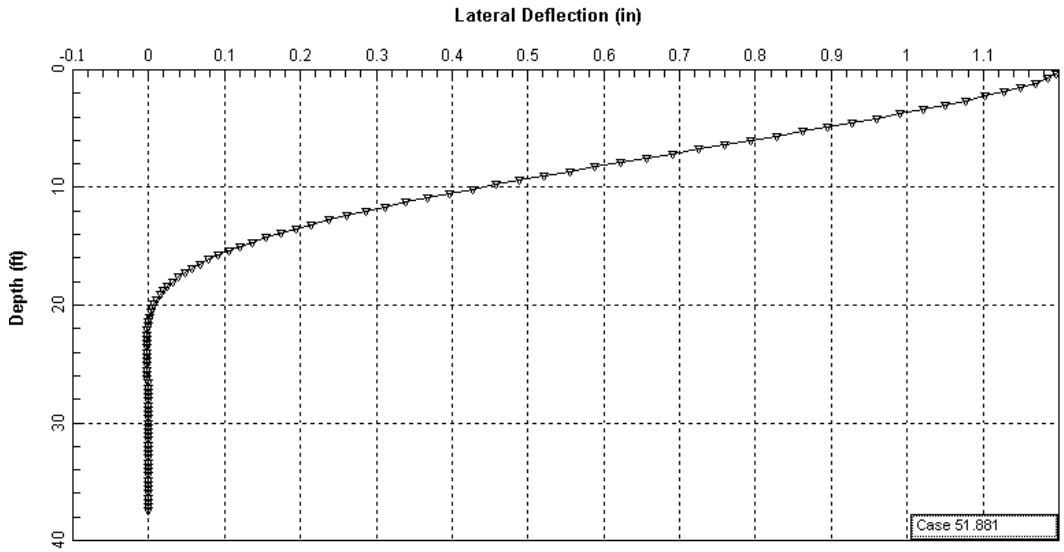

Fig 19. Displacement diagram of the pile

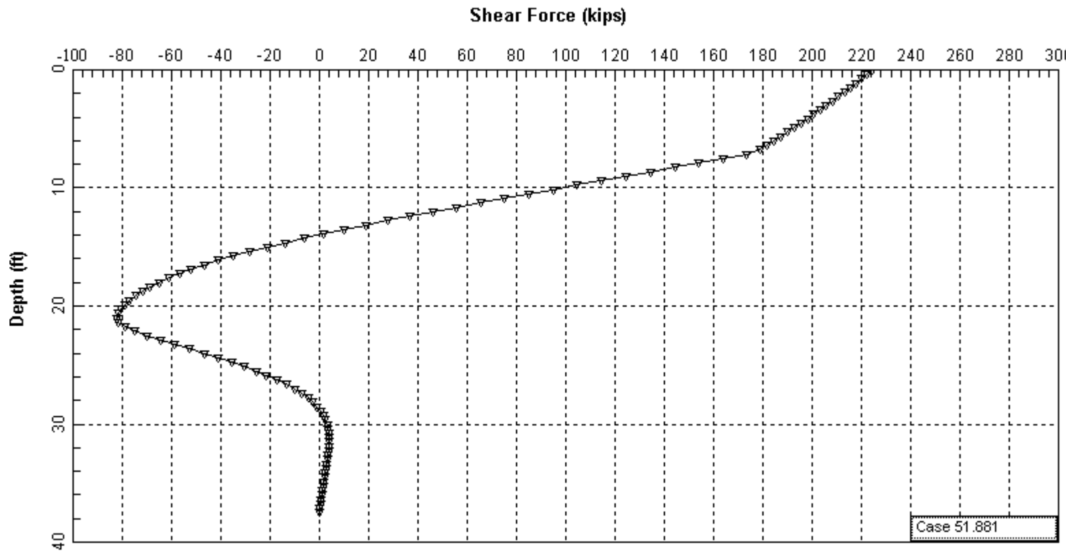

Fig 20. Shear diagram of the pile

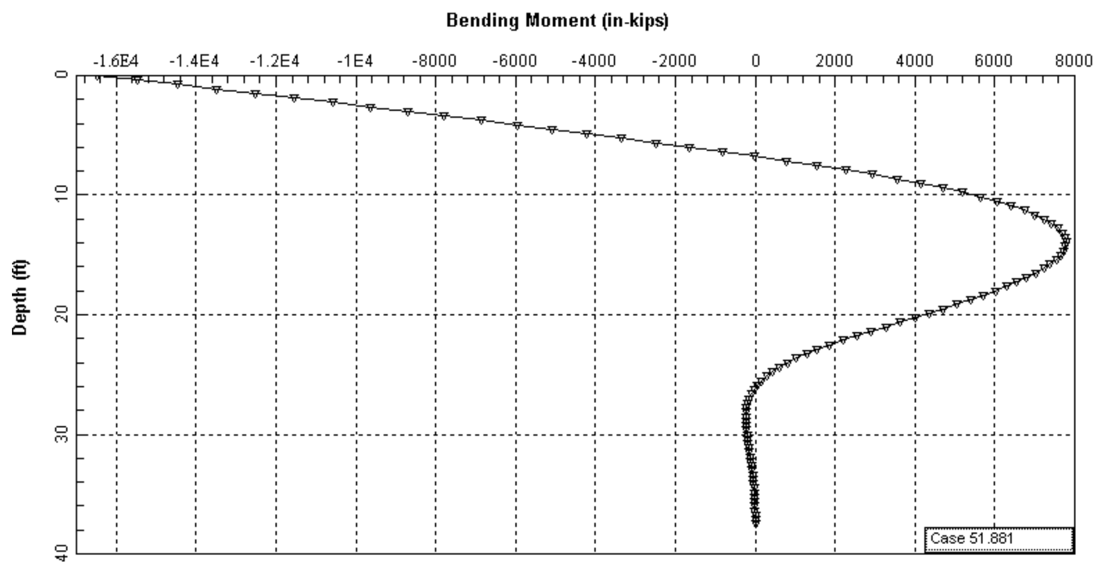

Fig 21. Bending moment diagram of the pile

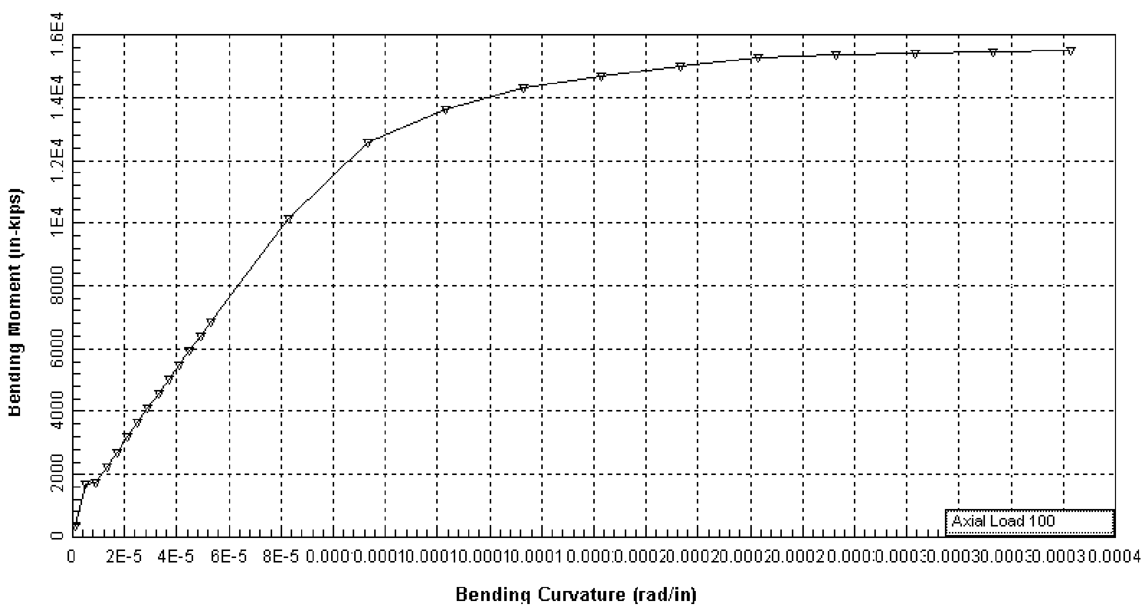

Fig 22. Curvature diagram of the pile (using 18#9)

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.

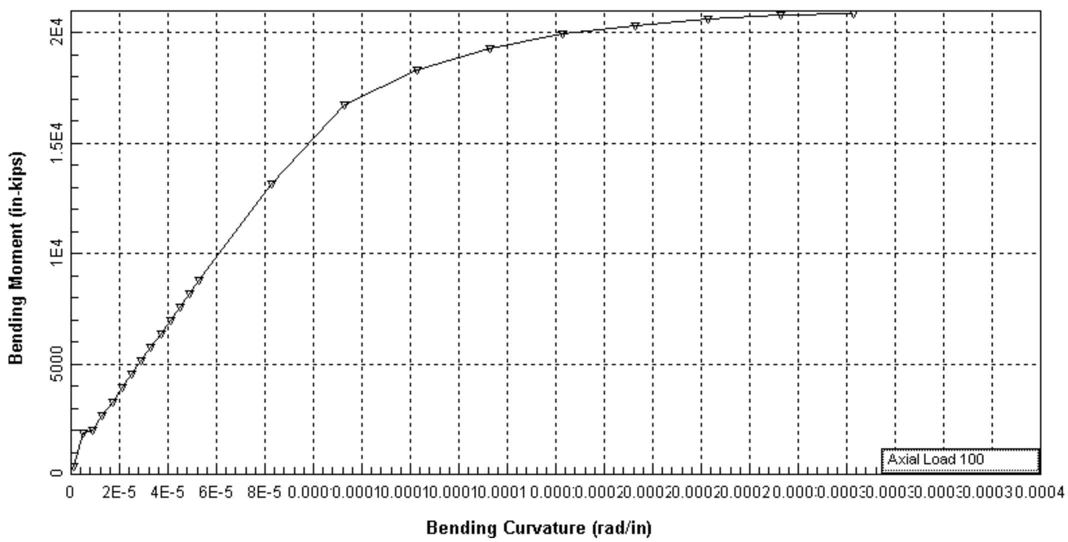

Fig 23. Curvature diagram of the pile (using 18#9)

Figure 23 indicates that by using 26#9 (2.5%) longitudinal reinforcement, pile will remain in elastic range under the MCE.

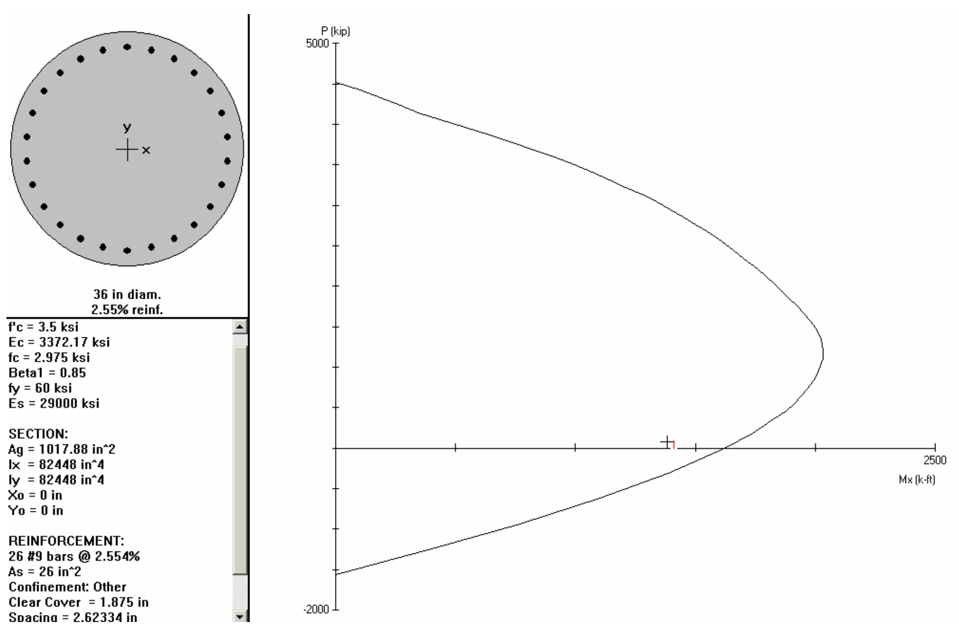

Fig 24 : Interaction diagram of the pile

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

$\Delta := .89$ in displacement of the pile (includes the combination of 100% + 40%)

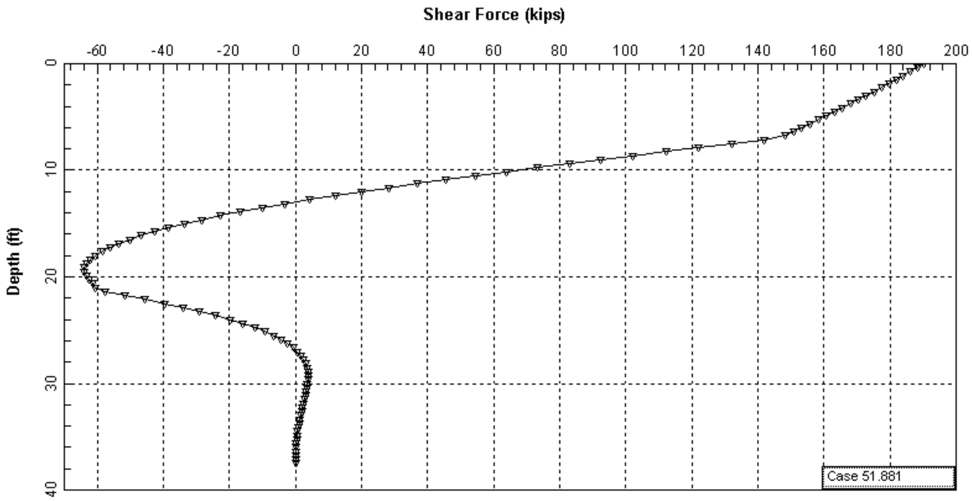

Fig 25. Shear diagram of the pile

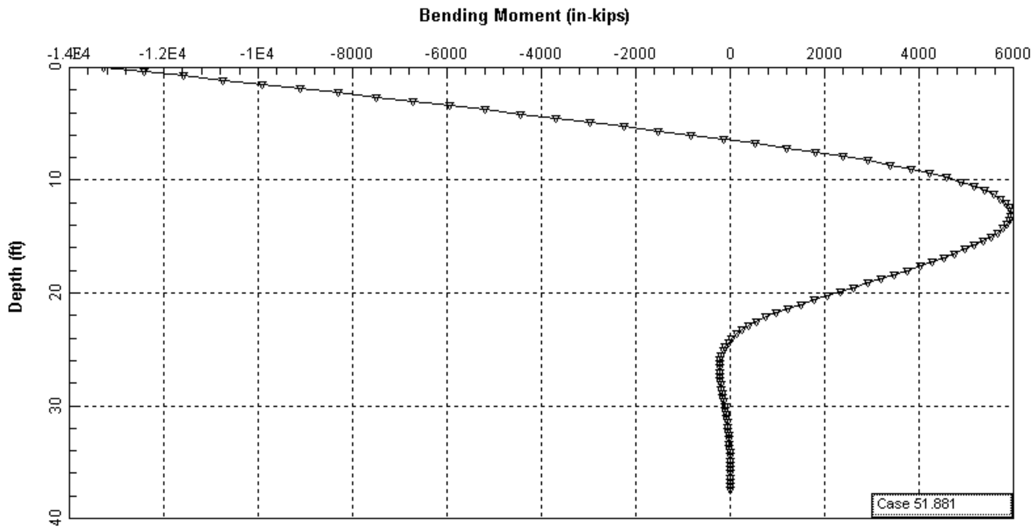

Fig 26. Bending moment diagram of the pile

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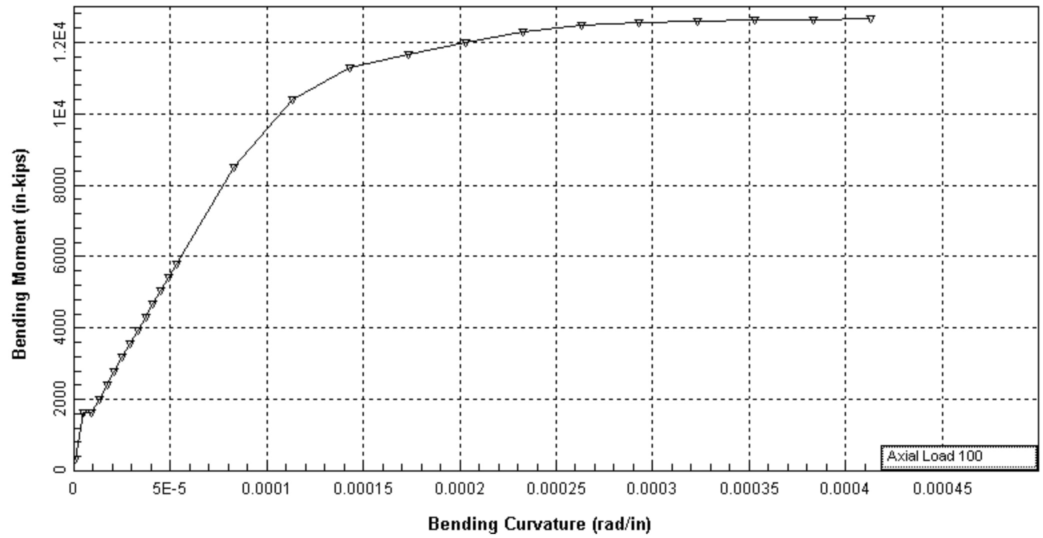


Fig 27. Curvature diagram of the pile using 14 # 9 (1.38%)

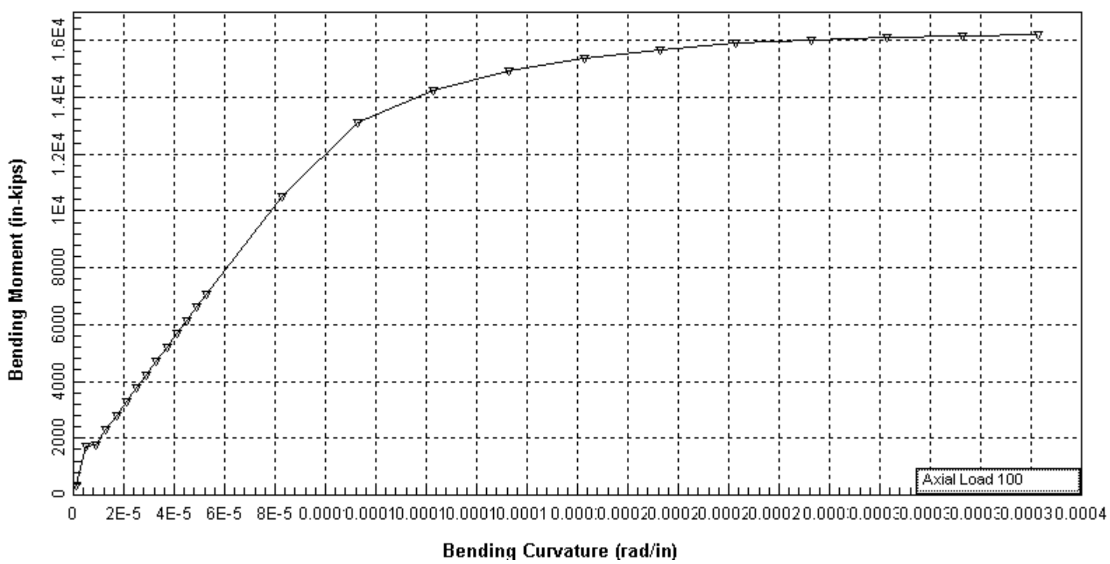


Fig 28. Curvature diagram of the pile using 19 # 9 (1.87%)

Figure 28 shows that by using 14#9 longitudinal reinforcement the pile will remain in elastic range under the MCE.

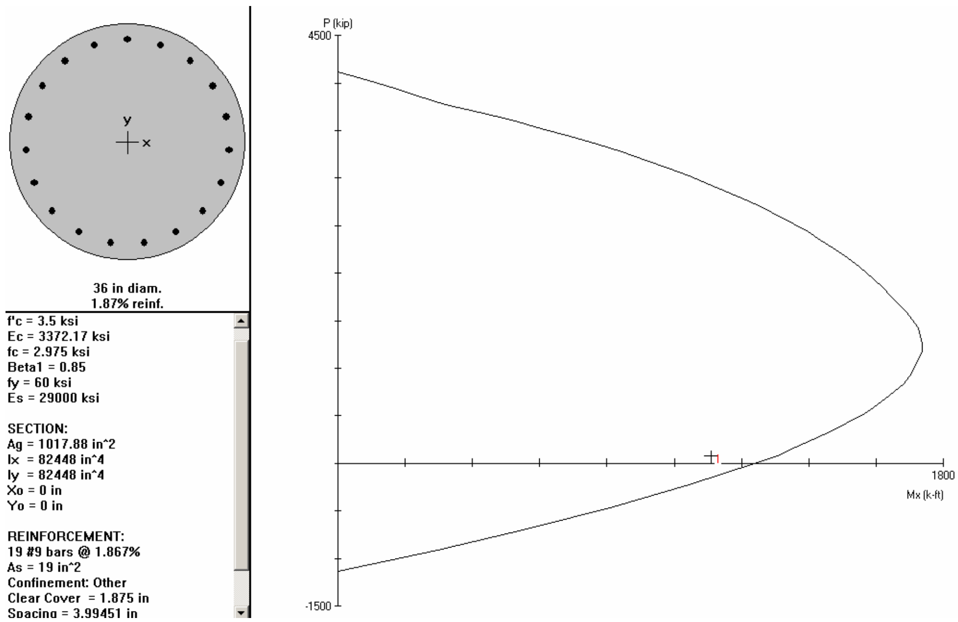


Fig 29 : Interaction diagram of the pile

Pile transverse reinforcement (fix bent)

L=37.5' D=3' 26#9 ρ=2.5%

$R := 0.8$ Response modification factor for the connection (MCEER Guide Spec. Table 4.7-2)

$\varepsilon_y := .00207$

$D := 36$ in Diameter of the pile

$cover := 1.5$ in Clear cover

$d_5 := .625$ in Diameter of spirals

$d_9 := 1.128$ in Diameter of longitudinal reinforments

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

$D_p = 30.622$ in

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

$D_{pp} = 32.375$ in

$L := 37.5\cdot 12$ Length of the pile

$L = 450$ in

$A_b := 1$ in^2 Longitudinal rebar diameter

$\Lambda := 2$ fixed - fixed fixity factor for circle section (MCEER Guide Spec. 8.8.2.3.a)

$P_d := 78$ kips Axial dead load of the pile

$P_{er} := 0$ kips for earthquake effect in longitudinal direction

$A_g := \frac{3.14}{4}\cdot D^2$

$A_g = 1.017 \times 10^3$ in^2

$A_{cc} := \frac{3.14}{4} \cdot D_{pp}^{\ 2}$ see Design Example 8 design step 8.2.3

$A_{cc} = 822.79$ in^2

$K_{shape} := .32$ for circle section MCEER Guide Spec. 8.8.2.3.a

$V := 225$ kips Shear of the pile

$f_c := 3500$ psi f'c of concrete

$f_{yh} := 60$ ksi for steel

$A_v := .8 A_g$ Shear area of concrete

$A_v = 813.888$ in^2

$OS := 1.5$ Over strength factor

$\phi := .85$ for shear

$\alpha := 25$ degree MCEER Guide Spec. 8.8.5.2

$\tan\alpha := .466$

$H_c := 36$ in thickness of the pile cap

$N := 26$ number of rebars in pile

$\rho_t := N \cdot \frac{A_b}{A_g}$

$\rho_t = 0.026$

Method 2: Explicit Approach MCEER Guide Spec. 8.8.2.3

$L = 450$ in

$D_p = 30.622$

$D_p = 30.622$ n

$\frac{D_p}{L} = 0.068$

$\tan\alpha = 0.466$

$P_e := .8P_d$ Effect dead load with the effect of uplift

$V_p := \frac{\Lambda}{2} \cdot P_e \cdot \frac{D_p}{L}$ The contribution due to arch action

$V_p = 4.246$ kips

$\phi = 0.85$

$V_c := 2\left[\sqrt{(f_c)}\right] \cdot \frac{A_{cc}}{1000}$

$V_c = 97.354$ kips

$V_u := \frac{V}{R}$

$V_u = 281.25$ kips

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

$V_{s\_rq} = 229.282$ kips

$K_{shape} = 0.32$

$f_{yh} = 60$ ksi

$f_{su} := 1.5 \cdot f_{yh}$

$f_{su} = 90$ ksi

$A_v = 813.888$ in^2

$\rho_t = 0.026$

$s := 6$ in < 6 Maximum spacing (NCHRP 5.10.6.2) OK

$A_{sh} := .31$ in^2 #5

$$\rho_{v\_pr} := 2\frac{A_{sh}}{s \cdot D_{pp}}$$

$\rho_{v\_pr} = 3.192 \times 10^{-3}$

$$\tan\theta := \left(1.6 \cdot \rho_{v\_pr} \cdot \frac{A_v}{\Lambda \cdot \rho_t \cdot A_g}\right)^{.25}$$

$\tan\theta = 0.532$

$\tan\theta := \max(\tan\theta, .466)$

$\tan\theta = 0.532$

$$V_{s\_pr} := \frac{3.14}{2} \cdot \frac{A_{sh}}{s} \cdot f_{yh} \cdot \frac{D_{pp}}{\tan\theta}$$

$V_{s\_pr} = 296.342$ kips

$V_{s\_rq} = 229.282$ kips OK

Method 1: Implicit Shear Detailing Approach MCEER Guide Spec.8.8.2.3

$s = 6$ in

$A_{sh} = 0.31$ in^2

$$\rho_{v\_rq} := K_{shape} \cdot \Lambda \cdot \frac{\rho_t}{\phi} \cdot \frac{f_{su}}{f_{yh}} \cdot \frac{A_g}{A_v} \cdot \tan\alpha \cdot \tan\theta$$

$$\rho_{v\_rq} = 8.94 \times 10^{-3}$$

$$\rho_{v\_rq\_nonplastic} := \rho_{v\_rq} - 2\frac{\sqrt{f_c}}{1000 f_{yh}}$$

$$\rho_{v\_rq\_nonplastic} = 6.968 \times 10^{-3}$$

$$\rho_{v\_pr} = 3.192 \times 10^{-3}$$

$$A_{sh\_method1} := .31$$

$$s_{method1} := 2 \cdot \frac{A_{sh\_method1}}{D_{pp} \cdot \rho_{v\_rq\_nonplastic}}$$

$s_{method1} = 2.748$ in < 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' D=3' 19#9 ρ=1.87%

$d_4 := .5$ in

$d_9 := 1.128$ in

$$D_p := D - 2\cdot\left(cover + d_4 + \frac{d_9}{2}\right)$$

$D_p = 30.872$ in

$$D_{pp} := D - 2\left(cover + \frac{d_4}{2}\right)$$

$D_{pp} = 32.5$ in

$V := 190$ kips

$N := 19$ number of rebars in pile

$$\rho_t := N\cdot\frac{A_b}{A_g}$$

$\rho_t = 0.019$

Method 2: Explicit Approach

$V_u := \dfrac{V}{R}$

$V_u = 237.5$ kips

$V_{s\_rq} := \left[\dfrac{V_u}{\phi} - \left(V_p + V_c\right)\right]$

$V_{s\_rq} = 177.812$ kips

$\rho_t = 0.019$

$s := 6$ in < 6 Maximum spacing (NCHRP 5.10.6.2) OK

$A_{sh} := .2$ in^2 #4

$\rho_{v\_pr} := 2\dfrac{A_{sh}}{s \cdot D_{pp}}$

$\rho_{v\_pr} = 2.051 \times 10^{-3}$

$\tan\theta := \left(1.6 \cdot \rho_{v\_pr} \cdot \dfrac{A_v}{\Lambda \cdot \rho_t \cdot A_g}\right)^{.25}$

$\tan\theta = 0.515$

$\tan\theta := \max(\tan\theta, .466)$

$\tan\theta = 0.515$

$V_{s\_pr} := \dfrac{3.14}{2} \cdot \dfrac{A_{sh}}{s} \cdot f_{yh} \cdot \dfrac{D_{pp}}{\tan\theta}$

$V_{s\_pr} = 198.19$ kips

$V_{s\_rq} = 177.812$ kips OK

Method 1: Implicit Shear Detailing Approach

$s = 6$ in

$A_{sh} = 0.2$ in^2 #4

$$\rho_{v\_rq} := K_{shape} \cdot \Lambda \cdot \frac{\rho_t}{\phi} \cdot \frac{f_{su}}{f_{yh}} \cdot \frac{A_g}{A_v} \cdot \tan\alpha \cdot \tan\theta$$

$$\rho_{v\_rq} = 6.326 \times 10^{-3}$$

$$\rho_{v\_rq\_nonplastic} := \rho_{v\_rq} - 2\frac{\sqrt{f_c}}{1000 f_{yh}}$$

$$\rho_{v\_rq\_nonplastic} = 4.354 \times 10^{-3}$$

$$\rho_{v\_pr} = 2.051 \times 10^{-3}$$

$$A_{sh\_method1} := .2$$

$$s_{method1} := 2 \cdot \frac{A_{sh\_method1}}{D_{pp} \cdot \rho_{v\_rq\_nonplastic}}$$

$s_{method1} = 2.826$ in < Method 2, use Method 2

$$\frac{\rho_{v\_pr}}{\rho_{v\_rq\_nonplastic}} = 0.471$$

Summery of transverse reinforcement
Use #4@6" spiral

Joint transverse reinforcement for the expansion bent

L=37.5' D=3' 19#9 ρ=1.87%

$H_c = 36$ in

$R = 0.8$

$\varepsilon_y = 2.07 \times 10^{-3}$

$D = 36$ in

$\rho_t = 0.019$

$L = 450$

$A_b = 1$ in^2 Longitudinal rebar diameter

$\Lambda = 2$ fixed - fixed

$P_d = 78$ kips

$D_p = 30.872$ in

$D_{pp} = 32.5$ in

$\phi = 0.85$

$A_g = 1.017 \times 10^3$ in^2

$A_v = 813.888$ in^2

$f_{yh} = 60$ ksi

$f_{su} := 1.5 \cdot f_{yh}$

$f_c = 3.5 \times 10^3$ psi

$P_{max} := P_d + P_{er}$

$P_{max} = 78$ kips

$A_{sc} := N \cdot A_b$

$A_{sc} = 19$ in^2 Longitudinal reinforcement in pile

$K_{shape} = 0.32$

$M_n := 1110$ $k - ft$

Explicit approach for joint design

$f_h := 0$

$P_{max} = 78$ kips

$M_p := OS \cdot M_n$

$OS = 1.5$

$M_n = 1.11 \times 10^3$ k − ft

$M_p = 1.665 \times 10^3$ k − ft

$b_b := 12.5 \cdot 12$ in

$D = 36$ in

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

$L_{mid\_dept\_jt} = 72$ in

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

$A_{mid} = 1.08 \times 10^4$ in^2

$f_v := \dfrac{P_{max}}{A_{mid}}$

$f_v = 7.222 \times 10^{-3}$ ksi

$h_b := H_c$

$h_b = 36$ in

$h_c := D$

$h_c = 36$ in

$b_{je} := \sqrt{2} \cdot D$

$b_{je} = 50.912$ in

$$v_{hv} := \frac{M_n \cdot 12}{h_b \cdot h_c \cdot b_{je}}$$

$v_{hv} = 0.202$ ksi

$$p_t := \frac{(f_h + f_v)}{2} - \sqrt{\left[\frac{(f_h - f_v)}{2}\right]^2 + v_{hv}^2}$$

$p_t = -0.198$

$$p_{t\_max} := \frac{3.5}{1000} \cdot \sqrt{f_c}$$

$p_{t\_max} = 0.207$ OK, No extra reinforcement is necessary

$$p_c := \frac{(f_h + f_v)}{2} + \sqrt{\left[\frac{(f_h - f_v)}{2}\right]^2 + v_{hv}^2}$$

$p_c = 0.206$

$$p_{c\_max} := \frac{.25}{1000} \cdot f_c$$

$p_{c\_max} = 0.875$ OK, No extra reinforcement is necessary

$$\rho_{s\_min} := \frac{3.5}{1000} \cdot \frac{\sqrt{f_c}}{f_{yh}}$$

$\rho_{s\_min} = 3.451 \times 10^{-3}$ OK

$A_{sh} := .2$

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

$s_{max} = 7.133$ $\boxed{\text{Use \#4@6"}}$

Joint transverse reinforcement for the fixed bent

L=37.5' D=3' 26#9 ρ=2.5%

$H_c = 36$ in

$R = 0.8$

$\varepsilon_y = 2.07 \times 10^{-3}$

$D = 36$ in

$\rho_t := .0255$

$L = 450$

$A_b = 1$ in^2 Longitudinal rebar diameter

$\Lambda = 2$ fixed - fixed

$P_d = 78$ kips

$D_p = 30.872$ in

$D_{pp} = 32.5$ in

$\phi = 0.85$

$A_g = 1.017 \times 10^3$ in^2

$A_v = 813.888$ in^2

$f_{yh} = 60$ ksi

$f_{su} := 1.5 \cdot f_{yh}$

$f_c = 3.5 \times 10^3$ psi

$P_{max} := P_d + P_{er}$

$P_{max} = 78$ kips

$N := 26$

$A_{sc} := N \cdot A_b$

$A_{sc} = 26$ in^2 Longitudinal reinforcement in pile

$K_{shape} = 0.32$

$M_n := 1340$ $k - ft$

Explicit approach for joint design

$f_h := 0$

$P_{max} = 78$ kips

$M_p := OS \cdot M_n$

$OS = 1.5$

$M_n = 1.34 \times 10^3$ $k - ft$

$M_p = 2.01 \times 10^3$ $k - ft$

$b_b := 12.5 \cdot 12$ in

$D = 36$ in

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

$L_{mid\_dept\_jt} = 72$ in

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

$A_{mid} = 1.08 \times 10^4$ in^2

$$f_v := \frac{P_{max}}{A_{mid}}$$

$f_v = 7.222 \times 10^{-3}$ ksi

$h_b := H_c$

$h_b = 36$ in

$h_c := D$

$h_c = 36$ in

$b_{je} := \sqrt{2} \cdot D$

$b_{je} = 50.912$ in

$$v_{hv} := \frac{M_n \cdot 12}{h_b \cdot h_c \cdot b_{je}}$$

$v_{hv} = 0.244$ ksi

$$p_t := \frac{(f_h + f_v)}{2} - \sqrt{\left[\frac{(f_h - f_v)}{2}\right]^2 + v_{hv}^{\;2}}$$

$p_t = -0.24$

$$p_{t\_max} := \frac{3.5}{1000} \cdot \sqrt{f_c}$$

$p_{t\_max} = 0.207$ Extra reinforcement is required

$$p_c := \frac{(f_h + f_v)}{2} + \sqrt{\left[\frac{(f_h - f_v)}{2}\right]^2 + v_{hv}^{\;2}}$$

$p_c = 0.247$

$$p_{c\_max} := \frac{.25}{1000} \cdot f_c$$

$p_{c\_max} = 0.875$ OK

$A_{jv} := .16A_{sc}$ Vertical reinforcement (Stirrups) (Guide Spec. 8.8.4.3.2)

$A_{jv} = 4.16$ in^2

Use #5

$\frac{A_{jv}}{.31} = 13.419$ $\boxed{\text{7\#5 vertical legs in (development length) each side}}$

$A_{clamp} := .08 \cdot A_{sc}$ Vertical reinforcement (Clamping reinforcement) (Guide Spec. 8.8.4.3.2)

$A_{clamp} = 2.08$

$\frac{A_{clamp}}{.31} = 6.71$ $\boxed{\text{7\#5 in 36" of the joint}}$

$l_{d8} := .63 \cdot 1 \cdot \frac{f_{yh}}{\sqrt{\frac{f_c}{1000}}}$ Development length for #9 (AASHTO LRFD 5.11.2.2.1-1)

$0.3 \cdot 1.128 \cdot f_{yh} = 20.304$ in

$l_{d8} = 20.205$ in

$A_h := 0.08 \cdot \frac{A_{sc}}{.79}$ Horizontal reinforcement (Guide Spec. 8.8.4.3.3)

$A_h = 2.633$ $\boxed{\text{3\#8 in 40" length each side}}$

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

$l_{d9} = 45.221$ n

$.4 \cdot 1.41 \cdot f_{yh} = 33.84$

$\rho_s := .4 \frac{A_{sc}}{l_{d9}^{\,2}}$ Hoops (Guide Spec. 8.8.4.3.4)

$\rho_s = 5.086 \times 10^{-3}$

$\rho_{s\_min} := \frac{3.5}{1000} \cdot \frac{\sqrt{f_c}}{f_{yh}}$ Minimum required horizontal reinforcement (Guide Spec. 8.8.4.2.

$\rho_{s\_min} = 3.451 \times 10^{-3}$ OK

$A_{sh} := .31$ in^2

$s = 6$ in

$s_{max} := \frac{2 \cdot A_{sh}}{D_{pp} \cdot \rho_s}$

$s_{max} = 3.751$ $\boxed{\text{Use \#5@3.5}}$

Deep beam design (Source: Reinforcement concrete, Nawy, 3rd edition, section 6.9)

Shear design (fixed pier)

$f_c := 3500$ psi

$f_y := 60000$ psi

$h := 25.66$ ft

$b := 3$ ft

$d := .9 \cdot h$

$d = 23.094$ ft

$l := 17.75$ ft

$\frac{l}{d} = 0.769$ <5 so design antology applys

$\Phi := 0.85$ for shear

$V := 1276$ kips

$M := 21504$ k – ft

$\Phi \cdot \left(8 \cdot \sqrt{f_c} \cdot b \cdot d \cdot \frac{144}{1000}\right) = 4.014 \times 10^3$ kips ΦVn

$A_1 := \frac{M}{V \cdot d}$

$A_1 = 0.73$

$A_2 := 3.5 - 2.5 \cdot \frac{M}{V \cdot d}$

$A_2 = 1.676$ A2 < 3.5 OK

$\rho := .0253$

$$V_c := A_2 \cdot \left(1.9 \cdot \sqrt{f_c} + 2500 \cdot \rho \cdot \frac{V \cdot d}{M}\right) \cdot b \cdot d \cdot \frac{144}{1000}$$

$V_c = 3.328 \times 10^3$ kips

$$V_{cmax} := 6 \cdot \sqrt{f_c} \cdot b \cdot d \cdot \frac{144}{1000}$$

$V_{cmax} = 3.541 \times 10^3$ < Vc (controls)

$V_c := V_{cmax}$

$V_c = 3.541 \times 10^3$ kips

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

$V_s = -1.509 \times 10^3$ Shear reinforcement is not required

Flexure design for deep beam

$\Phi := 1$

$M = 2.15 \times 10^{4}$ k − ft

$\frac{l}{h} = 0.692$ < 1 so jd=0.6l

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

$j = 0.461$

$j \cdot d = 10.65$ ft

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

$A_s = 134.61$ in^2 < 274.56 in2 OK

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

$A_{smin} = 33.255$ in^2 < 274.56 in^2 OK

Deep beam design (Source: Reinforcement concrete, Nawy, 3rd edition, section 6.9)

Shear design (expansion pier)

$f_c := 3500$ psi

$f_y := 60000$ psi

$h := 25.66$ ft

$b := 3$ ft

$d := .9 \cdot h$

$d = 23.094$ ft

$l := 17.75$ ft

$\frac{l}{d} = 0.769$ <1 so design antology applys

$\Phi := 0.85$ for shear

$V := 678$ kips

$M := 12780$ k – ft

$\Phi \cdot \left(8 \cdot \sqrt{f_c} \cdot b \cdot d \cdot \frac{144}{1000}\right) = 4.014 \times 10^3$ kips ΦVn

$A_1 := \frac{M}{V \cdot d}$

$A_1 = 0.816$

$A_2 := 3.5 - 2.5 \cdot \frac{M}{V \cdot d}$

$A_2 = 1.459$ A2 < 3.5 OK

$\rho := .0062$

$$V_c := A_2 \cdot \left(1.9 \cdot \sqrt{f_c} + 2500 \cdot \rho \cdot \frac{V \cdot d}{M}\right) \cdot b \cdot d \cdot \frac{144}{1000}$$

$V_c = 1.913 \times 10^3$ kips

$$V_{cmax} := 6 \cdot \sqrt{f_c} \cdot b \cdot d \cdot \frac{144}{1000}$$

$V_{cmax} = 3.541 \times 10^3$ > Vc so use Vc

$V_c = 1.913 \times 10^3$ kips

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

$V_s = -828.574$ Shear reinforcement is not required

Flexure design for deep beam

$\Phi := 1$

$M = 1.278 \times 10^4$ k − ft

$l = 17.75$ ft

$\frac{l}{h} = 0.692$ < 1 so jd=0.6l

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

$j = 0.461$

$j \cdot d = 10.65$ ft

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

$A_s = 80$ in^2 > 50.56 in^2, so we use 80#9 (1% steel, As=80.01 in^2)

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

$A_{smin} = 33.255$ in^2 < 80 in^2 OK

For the fixed bent

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

$h := 36$ in   thickness of the wall

1-

$D := h - 3$ in   Controls but last check should be done after design

$D = 33$ in

2-

$H := 201$ in

$\frac{1}{6} \cdot H = 33.5$ in

3-

18 in

4-

$\varepsilon_y := 0.00207$

$d_b := 1.41$ in   for #11

$M := 21504$ k − ft

$V := 1276$ k

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

$N := 180$ number of longitudinal reinforcement

$R := 0.8$ R-factor for joint (table 3.1-2)

$L := 201$ in Height of the pier

$A_b := 1.56$ 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.

$P_d := 534$ kips Axial dead load in the pier

$P_e := 59$ kips Axial earthquake force

$M_{topL} := 0$ k – ft

$M_{botL} := M$

$OS := 1.5$ Over strength factor

$M_{p\_topL} := OS \cdot M_{topL}$

$M_{p\_topL} = 0$ k – ft

$M_{p\_botL} := OS \cdot M_{botL}$

$M_{p\_botL} = 3.226 \times 10^4$ k – ft

$$V_{uL} := \frac{\left(M_{p\_topL} + M_{p\_botL}\right) \cdot 12}{L \cdot R}$$

$V_{uL} = 2.407 \times 10^3$ kips

$V_{u\_analysis} := \frac{V}{R}$ kips

$V_u := \max\left(V_{uL}, V_{u\_analysis}\right)$

$V_u = 2.407 \times 10^3$ kips

$f_c := 3500$ psi

$b_w := 308$ in

$d := D$ in

$h := 36$ in

$V_c := .6\left[\sqrt{(f_c)}\right]\cdot b_w\cdot\frac{d}{1000}$ Shear resistance in the end regions (plastic hinge zone)

$V_c = 360.786$ kips

$L = 201$ in

$D_p := 36 - 5$ Width of the column core

$D_p = 31$ in

$\frac{D_p}{L} = 0.154$

$V_p := \frac{\Lambda}{2}\cdot P_e\cdot\frac{D_p}{L}$

$V_p = 4.55$ kips Contribution due to arch action

$\phi := 0.85$ For shear

$V_u = 2.407 \times 10^3$ kips

$V_s := \frac{V_u}{\phi} - (V_p + V_c)$

$V_s = 2.467 \times 10^3$ kips

$K_{shape} := .5$ For wall in weak axis (MCEER 8.8.2.3)

$f_{yh} := 60$ ksi For the transverse and longitudinal steel

$f_{su} := 1.5\cdot f_{yh}$ Ultimate tensile stress of the longitudinal reinforcement

$f_{su} = 90$ ksi

$A_v := b_w \cdot d$ Shear area

$A_v = 1.016 \times 10^4$ in^2

$\rho_t := .0248$ Longitudinal reinforcement ratio

$s := 4$ in < 4" OK (MCEER 8.8.2.4) for plastic hinge zone

$A_{sh} := .2$ #4 Area of transverse reinforcement (ties)

$legs := 17$ Number of legs

$$\rho_{v\_pr} := legs \cdot \frac{A_{sh}}{b_w \cdot s}$$

$\rho_{v\_pr} = 2.76 \times 10^{-3}$

$A_g := b_w \cdot h$

$A_g = 1.109 \times 10^4$ in^2

$$\tan\theta := \left(1.6 \cdot \rho_{v\_pr} \cdot \frac{A_v}{\Lambda \cdot \rho_t \cdot A_g}\right)^{.25}$$

$\tan\theta = 0.636$

$D_{pp} := h - 4$ Width of the perimeter transverse direction

$$A_{vs} := s \cdot \frac{V_s}{f_{yh} \cdot D_{pp}} \cdot \tan\theta$$

$\frac{A_{vs}}{legs} = 0.192$ in^2 < Ash OK

Method 1: Implicit Shear Detailing Approach (Guide Spec. 8.8.2.3)

$$\rho_v := K_{shape} \cdot 2\Lambda \cdot \frac{\rho_t}{\phi} \cdot \frac{f_{su}}{f_{yh}} \cdot \frac{A_g}{A_v} \cdot \frac{D_p}{L} \cdot \tan\theta$$ For fixed - fixed case of transverse direction

$\rho_v = 4.68 \times 10^{-3}$

$A_{vs} := \rho_v \cdot b_w \cdot s$

$legs\_new := 30$

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

Outside the plastic hinge zone
Method 2: Explicit Approach
8.8.2.3

$legs\_outplastic := 14$

$V_c := 2 \cdot \sqrt{f_c} \cdot b_w \cdot \frac{d}{1000}$ Shear resistance of concrete out side of the plastic hinge zone

$V_c = 1.203 \times 10^3$ kips

$V_p = 4.55$

$V_s := \frac{V_u}{\phi} - (V_p + V_c)$

$V_s = 1.625 \times 10^3$ kips

$s_m := 6$ in Maximum spacing (MCEER 8.8.2.6)

$A_{shm} := .2$ #4

$\rho_{v\_pr} := legs\_outplastic \cdot \frac{A_{shm}}{b_w \cdot s_m}$

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

$A_g = 1.109 \times 10^4$

$\tan\theta := \left(1.6 \cdot \rho_{v\_pr} \cdot \frac{A_v}{\Lambda \cdot \rho_t \cdot A_g}\right)^{.25}$

$\tan\theta = 0.547$

$D_{pp} = 32$ in

$A_{vs} := s_m \cdot \frac{V_s}{f_{yh} \cdot D_{pp}} \cdot \tan\theta$

$$\frac{A_{vs}}{legs\_outplastic} = 0.198 \qquad in^2 \qquad < \text{Ash OK}$$

Method 1: Implicit Shear Detailing Approach

$legs\_new := 26$

$f_{yh} = 60$

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

$\rho_{vstar} = 2.708 \times 10^{-3}$

$$\rho_{pr} := legs\_new \cdot \frac{A_{shm}}{b_w \cdot s_m}$$

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

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

$legs = 17$

$f_{yh} = 60$ ksi

$U_{sf} := 15.95$ ksi Strain energy capacity

$A_{sh} = 0.2$ in^2

$A_c := (b_w - 6) \cdot (h - 6)$

$A_c = 9.06 \times 10^3$ in^2

$s := 4$ in < 4" OK (MCEER 8.8.2.4)

$$\rho_{s1} := legs \cdot \frac{A_{sh}}{D_{pp} \cdot s}$$

$\rho_{s1} = 0.027$

$\rho_t = 0.025$

$A_g = 1.109 \times 10^4$

$f_c = 3.5 \times 10^3$

$P_e := P_d + P_e$

$P_e = 593$ kips Factored axial load include seismic effect

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

$\rho_{s2} = 5.895 \times 10^{-3}$ OK

$D_p = 31$

$legs = 17$

$legs\_pmax := 17$

$\frac{legs\_pmax \cdot A_{sh}}{s \cdot D_p} = 0.027$ Include the area of total legs in both direction of cross section (Guide Spec. 8.8.2.4)

$s_{max} := legs\_pmax \cdot \frac{A_{sh}}{D_{pp} \cdot \rho_{s2}}$

$s_{max} = 18.024$ in OK

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

$\rho_s = 0.027$

$s = 4$ in

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

$P_d = 534$ kips

$P_{eminL} := P_d \cdot .8$ kips For the uplift effect

$P_{eminL} = 427.2$ kips

$M_{p\_top} := 0$ k − ft

$M_{botL} = 2.15 \times 10^4$ k − ft

$OS = 1.5$

$P_{eL} := P_d + OS \cdot (P_{eminL} - P_d)$

$P_{eL} = 373.8$ kips

$M_{p\_bot} := OS \cdot M_{botL}$

$$V_u := \frac{M_{p\_bot} \cdot 12}{L \cdot R}$$

$V_u = 2.407 \times 10^3$ ips

$A_v = 1.016 \times 10^4$

$$V_c := 0.6 \cdot \sqrt{f_c} \cdot \frac{A_v}{1000}$$

$V_c = 360.786$ kips

$$V_p := \Lambda \cdot P_{eL} \cdot \frac{D_p}{L \cdot 2}$$

$V_p = 28.825$ kips

$$V_s := \frac{V_u}{\phi} - V_c - V_p$$

$V_s = 2.442 \times 10^3$

$s := 4$ in

$A_{sh} = 0.2$ in^2

$$\rho_v := legs \cdot \frac{A_{sh}}{b_w \cdot s}$$

$\rho_v = 2.76 \times 10^{-3}$

$\tan\theta := \left[1.6 \cdot \rho_v \cdot \frac{A_v}{(\Lambda \cdot \rho_t \cdot A_g)}\right]^{.25}$

$\tan\theta = 0.636$

$\tan\alpha := \frac{D_p}{L}$

$\tan\alpha = 0.154$ OK

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

$\frac{A_s}{legs} = 0.19$ OK < Ash OK

Anti buckling steel
(Guide Spec. 8.8.2.5)

$legs\_antibuckling := 82$

$d_b = 1.41$ in Diameter of longitudinal rebars

$s_{new} := 6 \cdot d_b$ Maximum spacing of ties

$s_{new} = 8.46$ in Required spacing of anti-buckling

$s = 4$ in

$A_{sh} := .31$ #5

$A_b := 1.56 \cdot N$

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

$\frac{A_{bh}}{legs\_antibuckling} = 0.308$

$s_{max} := .25 \cdot 60$

$s_{max} = 15$ OK

Final check for the plastic hinge zone
(Guide Spec. 4.9.1)

$\tan\theta = 0.636$

$$L_p := \frac{(h-4)}{12}\cdot\left(\frac{1}{\tan\theta} + .5\cdot\tan\theta\right)$$

$L_p = 5.043$ ft

$V_u = 2.407 \times 10^3$

$M_{p\_botL} = 3.226 \times 10^4$ k − ft

$V_{uL} = 2.407 \times 10^3$ kips

$$\frac{M_{p\_botL}}{V_u}\cdot\left[1 - .85\cdot\frac{\left(\frac{M_{p\_botL}}{1.5}\right)}{M_{p\_botL}}\right] = 5.807 \quad \text{ft} \quad \text{controls}$$

$$\frac{1.5}{12}\cdot\left(4400\cdot\varepsilon_y\cdot d_b + .08\cdot\frac{12M_{p\_botL}}{V_u}\right) = 3.213$$

Summery 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

$\rho_{hmin} := .0025$

$A_{sh\_rq} := \rho_{hmin}\cdot(b_w - 4)\cdot h$

$A_{sh\_rq} = 27.36$ in^2

$A_{sh} = 0.31$ #5

$$N\_h := \frac{A_{sh\_rq}}{A_{sh}}$$

$N\_h = 88.258$ Number of horizontal reinforcement

$s_h := 4$ in

$$V_r := \frac{3}{1000}\sqrt{f_c}\left(b_w - 3\right)\cdot h$$

$V_r = 1.949 \times 10^3$ kips

$$V_n := \phi\cdot\left(.756\cdot\sqrt{f_c} + \rho_{hmin}\cdot f_{yh}\cdot 1000\right)\cdot\left(b_w - 3\right)\cdot\frac{h}{1000}$$

$V_n = 1.817 \times 10^3$ kips

$$V := \min\left(V_n, V_r\right)$$

$V = 1.817 \times 10^3$ kips

$V_{rq} := 1379$ 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

$h := 26$ in   thickness of the wall

1-

$D := h - 3$ in   controls but last check should be done after design

$D = 23$ in

2-

$H := 213$ in

$\frac{1}{6} \cdot H = 35.5$ in

3-

18 in

4-

$\varepsilon_y := 0.00207$

$d_b := 1$ in   for #8

$M := 3050$ k − ft

$V := 271$ k

$1.5\left(0.08 \cdot M \cdot \frac{12}{V} + 4400 \cdot \varepsilon_y \cdot d_b\right) = 29.869$ in   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)

$N := 80$ number of longitudinal reinforcement

$R := 0.8$ R-factor for joint (table 3.1-2)

$L := 213$ in Height of the pier

$A_b := 1$ 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.

$P_d := 487$ kips Axial dead load in the pier

$P_e := 60$ kips Axial earthquake force

$M_{topL} := 0$ k – ft

$M_{botL} := M$

$OS := 1.5$ Over strength factor

$M_{p\_topL} := OS \cdot M_{topL}$

$M_{p\_topL} = 0$ k – ft

$M_{p\_botL} := OS \cdot M_{botL}$

$M_{p\_botL} = 4.575 \times 10^3$ k – ft

$$V_{uL} := \frac{(M_{p\_topL} + M_{p\_botL}) \cdot 12}{L \cdot R}$$

$V_{uL} = 322.183$ kips

$V_{u\_analysis} := \frac{V}{R}$ kips

$V_u := \max(V_{uL}, V_{u\_analysis})$

$V_u = 338.75$ kips

$f_c := 3500$ psi

$b_w := 308$ in

$d := D$

$d = 23$ in

$h = 26$ in

$V_c := .6\left[\sqrt{(f_c)}\right]\cdot b_w\cdot\frac{d}{1000}$ Shear resistance in the end regions (plastic hinge zone)

$V_c = 251.457$ kips

$L = 213$ in

$D_p := h - 5$ Width of the column core

$D_p = 21$ in

$\frac{D_p}{L} = 0.099$

$V_p := \frac{\Lambda}{2}\cdot P_e\cdot\frac{D_p}{L}$

$V_p = 2.958$ kips Contribution due to arch action

$\phi := 0.85$ For shear

$V_u = 338.75$ kips

$V_s := \frac{V_u}{\phi} - (V_p + V_c)$

$V_s = 144.115$ kips

$K_{shape} := .5$ For wall in weak axis (MCEER 8.8.2.3)

$f_{yh} := 60$ ksi For the transverse and longitudinal steel

$f_{su} := 1.5\cdot f_{yh}$ Ultimate tensile stress of the longitudinal reinforcement

$f_{su} = 90$ ksi

$A_v := b_w \cdot d$ Shear area

$A_v = 7.084 \times 10^3$ in^2

$\rho_t := .0101$ Longitudinal reinforcement ratio

$s := 4$ in < 4" OK (MCEER 8.8.2.4) for plastic hinge zone

$A_{sh} := .11$ #3 Area of transverse reinforcement (ties)

$legs := 3$ Number of legs

$$\rho_{v\_pr} := legs \cdot \frac{A_{sh}}{b_w \cdot s}$$

$\rho_{v\_pr} = 2.679 \times 10^{-4}$

$A_g := b_w \cdot h$

$A_g = 8.008 \times 10^3$ in^2

$$\tan\theta := \left(1.6 \cdot \rho_{v\_pr} \cdot \frac{A_v}{\Lambda \cdot \rho_t \cdot A_g}\right)^{.25}$$

$\tan\theta = 0.44$

$D_{pp} := h - 4$ Width of the perimeter transverse direction

$$A_{vs} := s \cdot \frac{V_s}{f_{yh} \cdot D_{pp}} \cdot \tan\theta$$

$\frac{A_{vs}}{legs} = 0.064$ in^2 < Ash OK

Method 1: Implicit Shear Detailing Approach (Guide Spec. 8.8.2.3)

$$\rho_v := K_{shape} \cdot 2\Lambda \cdot \frac{\rho_t}{\phi} \cdot \frac{f_{su}}{f_{yh}} \cdot \frac{A_g}{A_v} \cdot \frac{D_p}{L} \cdot \tan\theta$$ For fixed - fixed case of transverse direction

$\rho_v = 8.744 \times 10^{-4}$

$A_{vs} := \rho_v \cdot b_w \cdot s$

$legs\_new := 10$

$\frac{A_{vs}}{legs\_new} = 0.108$ in^2 < Ash provided OK

Outside the plastic hinge zone
Method 2: Explicit Approach
8.8.2.3

$legs\_outplastic := 2$

$V_c := 2 \cdot \sqrt{f_c} \cdot b_w \cdot \frac{d}{1000}$ Shear resistance of concrete out side of the plastic hinge zone

$V_c = 838.19$ kips

$V_p = 2.958$

$V_s := \frac{V_u}{\phi} - (V_p + V_c)$

$V_s = -442.619$ kips

$s_m := 6$ in Maximum spacing (MCEER 8.8.2.6)

$A_{shm} := .11$ #3

$\rho_{v\_pr} := legs\_outplastic \cdot \frac{A_{shm}}{b_w \cdot s_m}$

$\rho_{v\_pr} = 1.19 \times 10^{-4}$

$A_g = 8.008 \times 10^3$

$\tan\theta := \left(1.6 \cdot \rho_{v\_pr} \cdot \frac{A_v}{\Lambda \cdot \rho_t \cdot A_g}\right)^{.25}$

$\tan\theta = 0.359$

$D_{pp} = 22$ in

$A_{vs} := s_m \cdot \frac{V_s}{f_{yh} \cdot D_{pp}} \cdot \tan\theta$

$\frac{A_{vs}}{legs\_outplastic} = -0.362$ in^2 No reinforcement is required

Method 1: Implicit Shear Detailing Approach

$legs\_new := 2$

$f_{yh} = 60$

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

$\rho_{vstar} = -1.098 \times 10^{-3}$ No reinforcement is required

$\rho_{pr} := legs\_new \cdot \frac{A_{shm}}{b_w \cdot s_m}$

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

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

$legs = 3$

$f_{yh} = 60$ ksi

$U_{sf} := 15.95$ ksi Strain energy capacity

$A_{sh} = 0.11$ in^2

$A_c := (b_w - 6) \cdot (h - 6)$

$A_c = 6.04 \times 10^3$ in^2

$s := 4$ in < 4" OK (MCEER 8.8.2.4)

$\rho_{s1} := legs \cdot \frac{A_{sh}}{D_{pp} \cdot s}$

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

$\rho_t = 0.01$

$A_g = 8.008 \times 10^3$

$f_c = 3.5 \times 10^3$

$P_e := P_d + P_e$

$P_e = 547$ kips Factored axial load include seismic effect

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

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

$D_p = 21$

$legs = 3$

$legs\_pmax := 17$

$\frac{legs\_pmax \cdot A_{sh}}{s \cdot D_p} = 0.022$ Include the area of total legs in both direction of cross section (Guide S

$s_{max} := legs\_pmax \cdot \frac{A_{sh}}{D_{pp} \cdot \rho_{s2}}$

$s_{max} = -2.272 \times 10^3$ ı 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

$P_{eminL} := P_d \cdot .8$ kips

$P_{eminL} = 389.6$ kips

$M_{p\_top} := 0$ k − ft

$M_{botL} = 3.05 \times 10^3$ k − ft

$OS = 1.5$

$P_{eL} := P_d + OS \cdot (P_{eminL} - P_d)$

$P_{eL} = 340.9$ kips

$M_{p\_bot} := OS \cdot M_{botL}$

$$V_u := \frac{M_{p\_bot} \cdot 12}{L \cdot R}$$

$V_u = 322.183$ kips

$A_v = 7.084 \times 10^3$

$$V_c := 0.6 \cdot \sqrt{f_c} \cdot \frac{A_v}{1000}$$

$V_c = 251.457$ kips

$$V_p := \Lambda \cdot P_{eL} \cdot \frac{D_p}{L \cdot 2}$$

$V_p = 16.805$ kips

$$V_s := \frac{V_u}{\phi} - V_c - V_p$$

$V_s = 110.777$

$s := 4$ in

$A_{sh} = 0.11$ in^2

$$\rho_v := legs \cdot \frac{A_{sh}}{b_w \cdot s}$$

$\rho_v = 2.679 \times 10^{-4}$

$$\tan\theta := \left[1.6\cdot\rho_v\cdot\frac{A_v}{(\Lambda\cdot\rho_t\cdot A_g)}\right]^{.25}$$

$\tan\theta = 0.44$

$$\tan\alpha := \frac{D_p}{L}$$

$\tan\alpha = 0.099$ OK

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

$\frac{A_s}{legs} = 0.049$ OK < Ash OK

Anti buckling steel
(Guide Spec. 8.8.2.5)

legs_antibuckling := 29

$d_b := 1.128$ in Diameter of longitudinal rebars

$s_{new} := 6\cdot d_b$ Maximum spacing of ties

$s_{new} = 6.768$ in Required spacing for antibuckling reinforcement

$s = 4$ in

$A_{sh} := .2$ #4

$A_b := 0.79\cdot N$

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

$$\frac{A_{bh}}{legs\_antibuckling} = 0.196$$

$s_{max} := .25\cdot 60$

$s_{max} = 15$ OK

Final check for the plastic hinge zone
(Guide Spec. 4.9.1)

$\tan\theta = 0.44$

$$L_p := \frac{(h-4)}{12}\cdot\left(\frac{1}{\tan\theta} + .5\cdot\tan\theta\right)$$

$L_p = 4.569$ ft

$M_{p\_botL} = 4.575 \times 10^3$ k – ft

$V_{uL} = 322.183$ kips

$$\frac{M_{p\_botL}}{V_u}\cdot\left[1 - .85\cdot\frac{\left(\frac{M_{p\_botL}}{1.5}\right)}{M_{p\_botL}}\right] = 6.153$$ ft controls

$$\frac{1.5}{12}\cdot\left(4400\cdot\varepsilon_y\cdot d_b + .08\cdot\frac{12M_{p\_botL}}{V_u}\right) = 2.988$$

Summery 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

$\rho_{hmin} := .0025$

$A_{sh\_rq} := \rho_{hmin}\cdot(b_w - 4)\cdot h$

$A_{sh\_rq} = 19.76$ in^2

$A_{sh} = 0.2$

$$N\_h := \frac{A_{sh\_rq}}{A_{sh}}$$

$N\_h = 98.8$ Number of horizontal reinforcement

$s_h := 4$ in

$$V_r := \frac{3}{1000}\sqrt{f_c}\left(b_w - 4\right)\cdot h$$

$V_r = 1.403 \times 10^3$ kips

$$V_n := \phi\cdot\left(.756\cdot\sqrt{f_c} + \rho_{hmin}\cdot f_{yh}\cdot 1000\right)\cdot\left(b_w - 4\right)\cdot\frac{h}{1000}$$

$V_n = 1.308 \times 10^3$ kips

$$V := \min\left(V_n, V_r\right)$$

$V = 1.308 \times 10^3$ kips

$V_{rq} := 811$ 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)

$N := 180$ number of longitudinal reinforcement

$b_w := 36$ in Height of pile cap

$H_c := 36$ in Height of the joint

$h := 36$ in Dimension of the column

$D := h - 3$ in

$D = 33$ in

$R := 0.8$ R-factor for joint (table 3.1-2)

$\varepsilon_y := .00207$ For the longitudinal steel

$L := 201$ in Height of the pier

$A_b := 1.56$ in^2 Area of rebars for longitudinal rebar

$P_d := 534$ kips Axial dead load in the pier

$P_e := 59$ kips Axial earthquake force

$M_{botL} := 21504$ $k - ft$

$OS := 1.5$ Over strength factor

$M_{p\_botL} := OS \cdot M_{botL}$

$M_{p\_botL} = 3.226 \times 10^4$ $k - ft$

$f_c := 3500$ psi

$b_w := 308$ in

$d := D$ in

$L = 201$ in

$D_p := 36 - 4$ Width of the column core

$D_p = 32$ in

$\phi := 0.85$ For shear

$f_{yh} := 60$ ksi For the transverse and longitudinal steel

$f_{su} := 1.5 \cdot f_{yh}$ Ultimate tensile stress of the longitudinal reinforcement

$f_{su} = 90$ ksi

$A_v := b_w \cdot d$ Shear area

$A_v = 1.016 \times 10^4$ in^2

$\rho_t := .0253$ Longitudinal reinforcement ratio

$s := 4$ in < 4" OK (MCEER 8.8.2.4) for plastic hinge zone

$A_{sh} := .2$ #4 Area of transverse reinforcement (ties)

$legs := 16$ Number of legs

$$\rho_{v\_pr} := legs \cdot \frac{A_{sh}}{b_w \cdot s}$$

$\rho_{v\_pr} = 2.597 \times 10^{-3}$

$A_g := b_w \cdot h$

$A_g = 1.109 \times 10^4$ in^2

$L = 201$ in Length of the column

$A_b := 1.56$ in^2 For longitudinal rebar

$D_p = 32$ in Width of the column core

$\phi := 0.85$ For shear

$A_v := D_p^{\,2}$ Shear area Av=bw*d

$A_g := D^2$

$f_{yh} := 60$ ksi

$f_{su} := 1.5 \cdot f_{yh}$

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

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

$A_{sc} = 280.8$ 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

$P := P_d + P_e$

$P = 593$ kips Maximum axial force include earthquake effect

$M_{p\_botL} = 3.226 \times 10^4$ k – ft From push over analysis

$b_b := 25 \cdot 12$ in Width of pile cap

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

$L_{mid\_dept\_jt} = 69$ in

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

$A_{mid} = 2.07 \times 10^4$ in^2

$f_v := \dfrac{P_e}{A_{mid}}$ Average axial stress in the vertical direction

$f_v = 2.85 \times 10^{-3}$ ksi

$h_b := H_c$ Joint depth

$h_b = 36$ in

$h_c := D$ Dimension of column

$h_c = 33$ in

$b_{je} := 2h_c$ effective joint width for shear stress calculations

$b_{je} = 66$ in

$v_{hv} := \frac{M_{p\_botL} \cdot 12}{h_b \cdot h_c \cdot b_{je}}$ Joint shear stress

$v_{hv} = 4.937$ 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

$p_t = -4.935$

$p_{t\_max} := \frac{3.5}{1000} \cdot \sqrt{f_c}$ 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

$p_c = 4.938$

$p_{c\_max} := \frac{.25}{1000} \cdot f_c$ Maximum compression stress

$p_{c\_max} = 0.875$ OK

$A_{jv} := .16 A_{sc}$ Vertical reinforcement (Stirrups) (Guide Spec. 8.8.4.3.2)

$A_{jv} = 44.928$ in^2

Use #7

$\frac{A_{jv}}{.6} = 74.88$ 75#7 vertical legs in (development length) each side

$A_{clamp} := .08 \cdot A_{sc}$ Vertical reinforcement (Clamping reinforcement) (Guide Spec. 8.8.4.3.2)

$A_{clamp} = 22.464$

$\frac{A_{clamp}}{.6} = 37.44$ 38#7 in 36" of the joint

$l_{d9} := .63 \cdot 1.128 \cdot \dfrac{f_{yh}}{\sqrt{\dfrac{f_c}{1000}}}$ Development length for #9 (AASHTO LRFD 5.11.2.2.1-1)

$0.3 \cdot 1.128 \cdot f_{yh} = 20.304$ in

$l_{d9} = 22.791$ in

$A_h := 0.08 \cdot A_{sc}$ Horizontal reinforcement (Guide Spec. 8.8.4.3.3)

$A_h = 22.464$ 23#9 in 40" length each side

$l_{d11} := 1.25 \cdot 1.56 \cdot \dfrac{f_{yh}}{\sqrt{\dfrac{f_c}{1000}}}$

$l_{d11} = 62.539$

$.4 \cdot 1.41 \cdot f_{yh} = 33.84$

$\rho_s := .4 \dfrac{A_{sc}}{{l_{d11}}^2}$ Hoops (Guide Spec. 8.8.4.3.4)

$\rho_s = 0.029$

$\rho_{s\_min} := \dfrac{3.5}{1000} \cdot \dfrac{\sqrt{f_c}}{f_{yh}}$ Minimum required horizontal reinforcement (Guide Spec. 8.8.4.2.2)

$\rho_{s\_min} = 3.451 \times 10^{-3}$ OK

$A_{sh} := .2$ in^2

$s = 4$ in

$legs := 41$

$s_{max} := legs \cdot \dfrac{A_{sh}}{D_{pp} \cdot \rho_s}$

$s_{max} = 9.211$ 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

$b_w := 36$ in Height of pile cap

$H_c := 36$ in Height of the joint

$h := 26$ in Dimension of the column

$D := h - 3$ in

$D = 23$ in

$R := 0.8$ R-factor for joint (table 3.1-2)

$\varepsilon_y := .00207$ For the longitudinal steel

$L := 213$ in Height of the pier

$A_b := 1$ 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.

$P_d := 486$ kips Axial dead load in the pier

$P_e := 60$ kips Axial earthquake force

$M_{botL} := 3050$ $k - ft$

$OS := 1.5$ Over strength factor

$M_{p\_botL} := OS \cdot M_{botL}$

$M_{p\_botL} = 4.575 \times 10^3$ $k - ft$

$f_c := 3500$ psi

$b_w := 308$ in

$d := D$ in

$L = 213$ in

$D_p := h - 4$ Width of the column core

$D_p = 22$ in

$\phi := 0.85$ For shear

$f_{yh} := 60$ ksi For the transverse and longitudinal steel

$f_{su} := 1.5 \cdot f_{yh}$ Ultimate tensile stress of the longitudinal reinforcement

$f_{su} = 90$ ksi

$A_v := b_w \cdot d$ Shear area

$A_v = 7.084 \times 10^3$ in^2

$\rho_t := .0101$ Longitudinal reinforcement ratio

$s := 4$ in < 4" OK (MCEER 8.8.2.4) for plastic hinge zone

$A_{sh} := .2$ #4 Area of transverse reinforcement (ties)

$legs := 16$ Number of legs

$$\rho_{v\_pr} := legs \cdot \frac{A_{sh}}{b_w \cdot s}$$

$\rho_{v\_pr} = 2.597 \times 10^{-3}$

$A_g := b_w \cdot h$

$A_g = 8.008 \times 10^3$ in^2

$L = 213$ in Length of the column

$A_b = 1$ in^2 For longitudinal rebar

$D_p = 22$ in Width of the column core

$\phi := 0.85$ For shear

$A_v := D_p^{\,2}$ Shear area Av=bw*d

$A_g := D^2$

$f_{yh} := 60$ 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

$A_{sc} = 80$ 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

$P := P_d + P_e$

$P = 546$ kips Maximum axial force include earthquake effect

$M_{p\_botL} = 4.575 \times 10^3$ k – ft From push over analysis

$b_b := 25 \cdot 12$ in Width of pile cap

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

$L_{mid\_dept\_jt} = 59$ in

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

$A_{mid} = 1.77 \times 10^4$ in^2

$f_v := \dfrac{P_e}{A_{mid}}$ Average axial stress in the vertical direction

$f_v = 3.39 \times 10^{-3}$ ksi

$h_b := H_c$ Joint depth

$h_b = 36$ in

$h_c := D$ Dimension of column

$h_c = 23$ in

$b_{je} := 2h_c$ effective joint width for shear stress calculations

$b_{je} = 46$ in

$$v_{hv} := \frac{M_{p\_botL} \cdot 12}{h_b \cdot h_c \cdot b_{je}}$$ Joint shear stress

$v_{hv} = 1.441$ 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

$p_t = -1.44$

$$p_{t\_max} := \frac{3.5}{1000} \cdot \sqrt{f_c}$$ 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

$p_c = 1.443$

$$p_{c\_max} := \frac{.25}{1000} \cdot f_c$$ Maximum compression stress

$p_{c\_max} = 0.875$ OK

$A_{jv} := .16A_{sc}$ Vertical reinforcement (Stirrups) (Guide Spec. 8.8.4.3.2)

$A_{jv} = 12.8$ in^2

Use #7

$\frac{A_{jv}}{.6} = 21.333$ $\boxed{\text{22\#7 vertical legs in (development length) each side}}$

$A_{clamp} := .08 \cdot A_{sc}$ Vertical reinforcement (Clamping reinforcement) (Guide Spec. 8.8.4.3.2)

$A_{clamp} = 6.4$

$\frac{A_{clamp}}{.6} = 10.667$ $\boxed{\text{11\#7 in 36" of the joint}}$

$l_{d9} := .63 \cdot 1.128 \cdot \dfrac{f_{yh}}{\sqrt{\dfrac{f_c}{1000}}}$ Development length for #9 (AASHTO LRFD 5.11.2.2.1-1)

$0.3 \cdot 1.128 \cdot f_{yh} = 20.304$ in

$l_{d9} = 22.791$ in

$A_h := 0.08 \cdot A_{sc}$ Horizontal reinforcement (Guide Spec. 8.8.4.3.3)

$A_h = 6.4$ $\boxed{\text{7\#9 in 40" length each side}}$

$l_{d11} := 1.25 \cdot 1.56 \cdot \dfrac{f_{yh}}{\sqrt{\dfrac{f_c}{1000}}}$

$l_{d11} = 62.539$

$.4 \cdot 1.41 \cdot f_{yh} = 33.84$

$\rho_s := .4 \dfrac{A_{sc}}{l_{d11}^{\,2}}$ Hoops (Guide Spec. 8.8.4.3.4)

$\rho_s = 8.182 \times 10^{-3}$

$\rho_{s\_min} := \dfrac{3.5}{1000} \cdot \dfrac{\sqrt{f_c}}{f_{yh}}$ Minimum required horizontal reinforcement (Guide Spec. 8.8.4.2.2)

$\rho_{s\_min} = 3.451 \times 10^{-3}$ OK

$A_{sh} := .11$ in^2

$s := 6$ in

$legs := 19$

$s_{max} := legs \cdot \dfrac{A_{sh}}{D_{pp} \cdot \rho_s}$

$s_{max} = 12.164$ $\boxed{\text{Use \#3@6" with 19 legs}}$

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

$C := \frac{V}{W}$

$C = 3.208$

$\Delta_{limit} := .25 \cdot C \cdot H$

$\Delta_{limit} = 13.431$ ft

$\Delta := .28$ ft < Δlimit OK

Expansion Pier

$W := 400$ kips Average axial force at column

$V := 678$ kips

$H := \frac{201}{12}$

$H = 16.75$ ft

$C := \frac{V}{W}$

$C = 1.695$

$\Delta_{limit} := .25 \cdot C \cdot H$

$\Delta_{limit} = 7.098$ ft

$\Delta := .28$ ft < Δlimit OK

Minimum seat requirement

$L := 109.33 \cdot .3048$

$L = 33.324$ m Length of bridge

$H := \frac{213}{12} \cdot .3048$

$H = 5.41$ m Heigth of the tallest pier

$B := 42 \cdot .3048$

$B = 12.802$ m Width of superstructure

$F_v := 1.5$

$S_1 := .759$

$\alpha := 0$

$$N := \left[.1 + .0017 \cdot L + .007 \cdot H + .05 \cdot \sqrt{H} \cdot \sqrt{1 + \left(2 \cdot \frac{B}{L}\right)^2} \right] \cdot \left(1 + 1.25 \cdot F_v \cdot S_1\right)$$

$N = 0.827$ m

$N := \frac{N}{.3048}$

$N = 2.712$ ft 33" Minimum seat width

Pile cap design procedure for Johnson County bridge

$Fc := 4000$ psi

$Fy := 60000$ psi

Moments and Forces acting on piles:

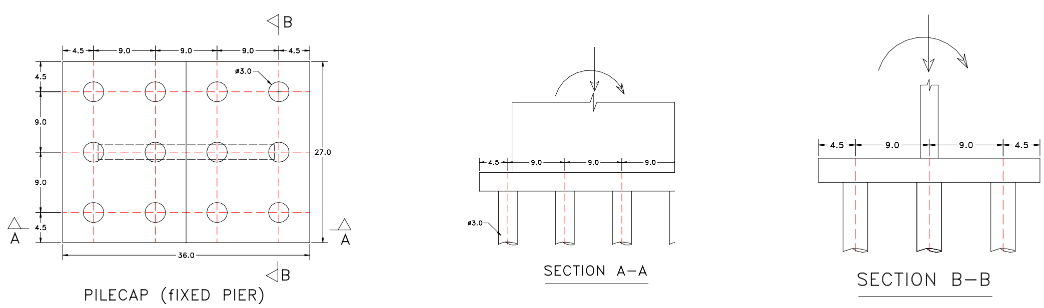
		Axial (kips)	M (kip-ft) Trans (case1)	M (kip-ft) Long (case2)
Fixed	**Long.**	1014	8602	21504
Pier	**Trans.**	988	21835	8734
Exp.	**Long.**	764	1220	3050
Pier	**Trans.**	728	12780	5112

Dimensions of Fixed pier

$W := 2.17$ Width of Wall (ft)

$L := 25.67$ Length of wall (ft)

$T := 3$ Thickness of pile cap (ft)



$s := 9$

$m1 := 3$ Number of rows

$n1 := 4$ Number of piles in the rows

$\Sigma d1 := \frac{s^2}{12} \cdot n1 \cdot \left(n1^2 - 1\right) \cdot m1$ Sum of the squares of the distances to each pile from the center or gravity of piles

$\Sigma d1 = 1.215 \times 10^3$ ft^2 $\Sigma d1 := 6 \cdot 13.5^2 + 6 \cdot 4.5^2$

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SUBJECT FILE

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

TITLE:	BY	DATE	CHECKED	DATE
Johnson County Bridge	NA	03/11/03		

$n2 := 3$ Number of rows

$m2 := 4$ Number of piles in the rows

$$\Sigma d2 := \frac{s^2}{12} \cdot n2 \cdot \left(n2^2 - 1\right) \cdot m2 \qquad \Sigma d2 := 8 \cdot 9^2$$

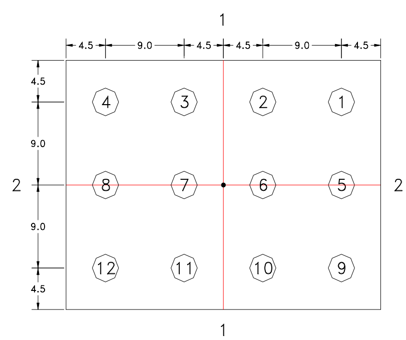
$\Sigma d2 = 648$ ft^2 Or

$M1 := 21504$ kip − ft

$M2 := 8734$ kip − ft

$V := 1014$ kip

Piles Coordinates:



$x1 := 13.5$	$x5 := 9$	$x9 := 13.5$
$y1 := 9$	$y5 := 0$	$y9 := -9$
$x2 := 4.5$	$x6 := 4.5$	$x10 := 4.5$
$y2 := 9$	$y6 := 0$	$y10 := -9$
$:= -4.5$	$x7 := -4.5$	$x11 := -4.5$
$:= 9$	$y7 := 0$	$y11 := -9$
$x4 := -13.5$	$x8 := -13.5$	$x12 := -13.5$
$y4 := 9$	$y8 := 0$	$y12 := -9$

$A := 81$ ft^2 Area for each pile

$\delta := 0.12$ $\frac{kip}{ft^3}$ Soil unit weight

$h := 2$ ft soil hight

$p := A \cdot \delta \cdot h$

$p = 19.44$ kips Soil pressure for each pile

$n := 12$ Number of piles in the group

$$P1 := \frac{V}{n} + M1\cdot\frac{x1}{\Sigma d1} + M2\cdot\frac{y1}{\Sigma d2} + p$$

$P1 = 464.179$ kips

$$P2 := \frac{V}{n} + M1\cdot\frac{x2}{\Sigma d1} + M2\cdot\frac{y2}{\Sigma d2} + p$$

$P2 = 304.89$ kips

$$P3 := \frac{V}{n} + M1\cdot\frac{x3}{\Sigma d1} + M2\cdot\frac{y3}{\Sigma d2} + p$$

$P3 = 145.601$ kips

$$P4 := \frac{V}{n} + M1\cdot\frac{x4}{\Sigma d1} + M2\cdot\frac{y4}{\Sigma d2} + p$$

$P4 = -13.688$ kips

$$P5 := \frac{V}{n} + M1\cdot\frac{x5}{\Sigma d1} + M2\cdot\frac{y5}{\Sigma d2} + p$$

$P5 = 263.229$ kips

$$P6 := \frac{V}{n} + M1\cdot\frac{x6}{\Sigma d1} + M2\cdot\frac{y6}{\Sigma d2} + p$$

$P6 = 183.584$ kips

$$P7 := \frac{V}{n} + M1\cdot\frac{x7}{\Sigma d1} + M2\cdot\frac{y7}{\Sigma d2} + p$$

$P7 = 24.296$ kips

$$P8 := \frac{V}{n} + M1\cdot\frac{x8}{\Sigma d1} + M2\cdot\frac{y8}{\Sigma d2} + p$$

$P8 = -134.993$ kips

$$P9 := \frac{V}{n} + M1 \cdot \frac{x9}{\Sigma d1} + M2 \cdot \frac{y9}{\Sigma d2} + p$$

$P9 = 221.568$ kips

$$P10 := \frac{V}{n} + M1 \cdot \frac{x10}{\Sigma d1} + M2 \cdot \frac{y10}{\Sigma d2} + p$$

$P10 = 62.279$ kips

$$P11 := \frac{V}{n} + M1 \cdot \frac{x11}{\Sigma d1} + M2 \cdot \frac{y11}{\Sigma d2} + p$$

$P11 = -97.01$ kips

$$P12 := \frac{V}{n} + M1 \cdot \frac{x12}{\Sigma d1} + M2 \cdot \frac{y12}{\Sigma d2} + p$$

$P12 = -256.299$ kips

Pile punching shear check:

$Vu := P1$

$Vu = 464.179$ Kips

$d := 33$ in

$b := 69\pi$ in

$$Vc := 4 \cdot \sqrt{Fc} \cdot b \cdot \frac{d}{1000}$$

$Vc = 1.81 \times 10^{3}$ kips

$\phi := 0.85$

$\frac{Vu}{\phi} = 546.093$ $\frac{Vu}{\phi} < Vc$ OK

One way shear Check:

$V1 := P1 + P2 + P3 + P4$

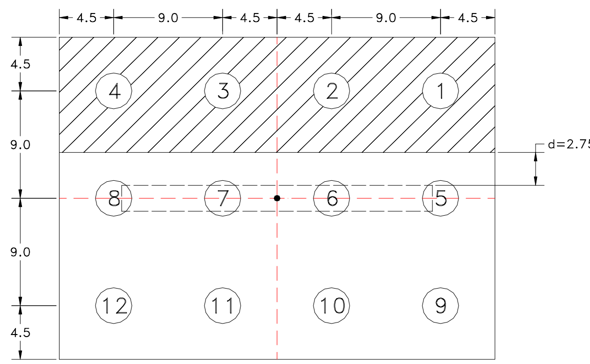
$V1 = 900.982$ Kips

$b2 := 432$ in

$Vc2 := 4 \cdot \sqrt{Fc} \cdot b2 \cdot \frac{d}{1000}$

$Vc2 = 3.607 \times 10^{3}$

$\frac{V1}{\phi} = 1.06 \times 10^{3}$ $\frac{V1}{\phi} < Vc2$ OK



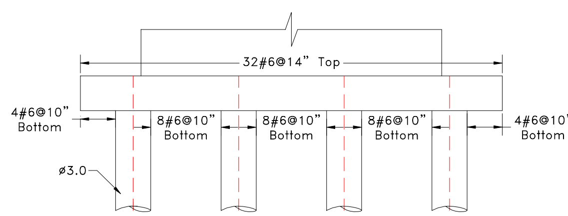
Flexure bending design: Only in one direction

$Mu := (P1 + P2 + P3 + P4) \cdot 1000 \cdot 7.92$

$Mu = 7.136 \times 10^{6}$ $\Phi := 0.9$

$As := \frac{Mu}{\Phi \cdot Fy \cdot 0.9 \cdot d}$

$As = 4.449$ in^{2}



Reinforcement

$Asmin := 0.0018 \cdot 36 \cdot 432$

$Asmin = 27.994$

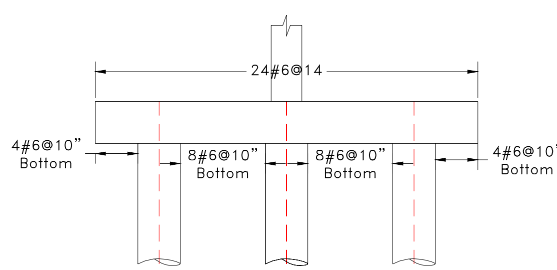
In long Direction : 64 # 6

$Asmin := 0.0018 \cdot 36 \cdot 324$

$Asmin = 20.995$

In short direction: 48 # 6

Minimum Reinforcement will apply in both directio
Asmin is divided in two part for top and bottom of p



Reinforcement

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SUBJECT FILE

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TITLE:	BY	DATE	CHECKED	DATE
Johnson County Bridge	NA	03/11/03		

Dimensions of Exp. pier

$W := 2.17$ Width of Wall (ft)

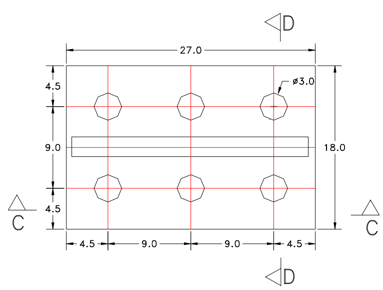
$L := 25.67$ Length of wall (ft)

$T := 3$ Thickness of pile cap (ft)

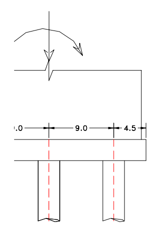
$w := 18$ Pile cap width (ft)

$l := 27$ Pile cap Length (ft)

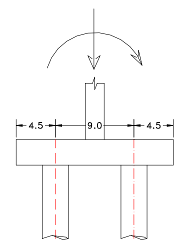
		Axial (kips)	M (kip-ft) Trans (case1)	M (kip-ft) Long (case2)
Fixed	Long.	1014	8602	21504
Pier	Trans.	988	21835	8734
Exp.	Long.	764	1220	3050
Pier	Trans.	728	12780	5112



PILECAP (EXP PIER)



:TION C–C



SECTION D–D

$m1 := 2 \quad n := 6$

$n1 := 3$

$\Sigma d1 := \frac{s^2}{12} \cdot n1 \cdot \left(n1^2 - 1\right) \cdot m1$ $\qquad \Sigma d1 := 4 \cdot 9^2$

$\Sigma d1 = 324$ $\qquad ft^2$

$m2 := 3$

$n2 := 2$

$\Sigma d2 := \frac{s^2}{12} \cdot n2 \cdot \left(n2^2 - 1\right) \cdot m2$ $\qquad \Sigma d2 := 6 \cdot 4.5^2$

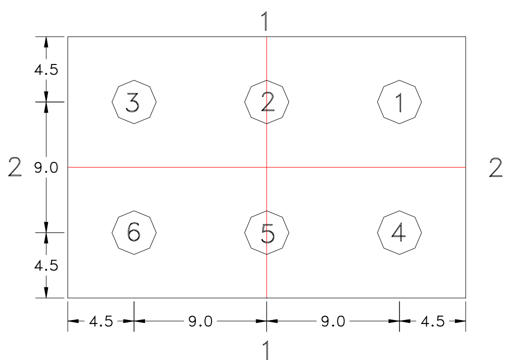
$\Sigma d2 = 121.5$ $\qquad ft^2$

$M1 := 12780$ kip − ft

$M2 := 1220$ kip − ft

$V := 764$ kip

Piles Coordinates:



$x1 := 9$ $x4 := 9$

$y1 := 4.5$ $y4 := -4.5$

$x2 := 0$ $x5 := 0$

$y2 := 4.5$ $y5 := -4.5$

$x3 := -9$ $x6 := -9$

$y3 := 4.5$ $y6 := -4.5$

$$P1 := \frac{V}{n} + M1 \cdot \frac{x1}{\Sigma d1} + M2 \cdot \frac{y1}{\Sigma d2} + p$$

$P1 = 546.959$ kips

$$P2 := \frac{V}{n} + M1 \cdot \frac{x2}{\Sigma d1} + M2 \cdot \frac{y2}{\Sigma d2} + p$$

$P2 = 191.959$ kips

$$P3 := \frac{V}{n} + M1 \cdot \frac{x3}{\Sigma d1} + M2 \cdot \frac{y3}{\Sigma d2} + p$$

$P3 = -163.041$ kips

$$P4 := \frac{V}{n} + M1 \cdot \frac{x4}{\Sigma d1} + M2 \cdot \frac{y4}{\Sigma d2} + p$$

$P4 = 456.588$ kips

$$P5 := \frac{V}{n} + M1 \cdot \frac{x5}{\Sigma d1} + M2 \cdot \frac{y5}{\Sigma d2} + p$$

$P5 = 101.588$ kips

$$P6 := \frac{V}{n} + M1 \cdot \frac{x6}{\Sigma d1} + M2 \cdot \frac{y6}{\Sigma d2} + p$$

$P6 = -253.412$ kips

Pile punching shear check:

$Vu := P1$

$Vu = 546.959$ Kips

$d := 33$ in

$b := 69\pi$ in

$$Vc := 4 \cdot \sqrt{Fc} \cdot b \cdot \frac{d}{1000}$$

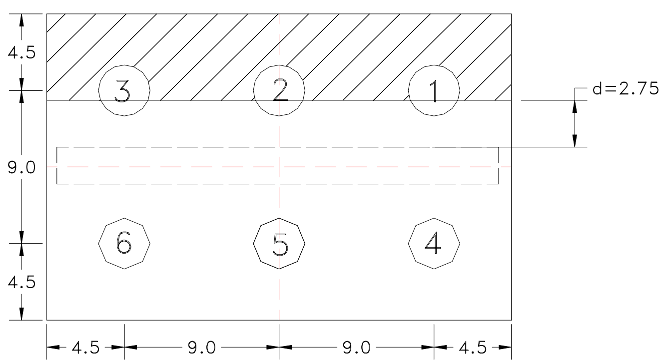
$Vc = 1.81 \times 10^{3}$ kips

$\phi := 0.85$

$\frac{Vu}{\phi} = 643.481$ $\frac{Vu}{\phi} < Vc$ OK

$d := 45$

One way shear Check:



$V1 := (P1 + P2 + P3) \cdot 0.74$

$V1 = 426.148$ Kips

$b2 := 324$ in

$$Vc2 := 4 \cdot \sqrt{Fc} \cdot b2 \cdot \frac{d}{1000}$$

$Vc2 = 3.688 \times 10^{3}$

$\frac{V1}{\phi} = 501.35$ $\frac{V1}{\phi} < Vc2$ OK

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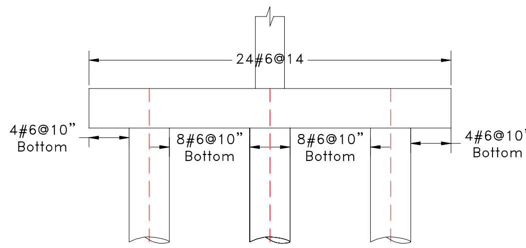
Flexure bending design: Only in one directio

$$Mu := (P1 + P2 + P3)\cdot 1000\cdot 3.32$$

$$Mu = 1.912 \times 10^{6}$$

$$As := \frac{Mu}{\Phi\cdot Fy\cdot 0.9\cdot d} \qquad \Phi := 0.9$$

$$As = 0.874 \quad in^{2}$$



Reinforcement

$$Asmin := 0.0018\cdot 36\cdot 324$$

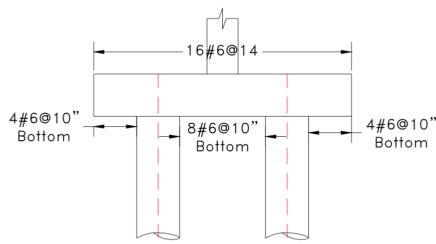
$$Asmin = 20.995$$

In Long Direction : 48 # 6

$$Asmin := 0.0018\cdot 36\cdot 216$$

$$Asmin = 13.997$$

In short Direction : 32 # 6



SECTION D–D

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

$qu1 := \frac{38 \cdot 2000}{144}$ psi $\quad qu1 = 527.778$

$qu2 := \frac{50 \cdot 2000}{144}$ psi $\quad qu2 = 694.444$

$f1 := 2.5 \cdot \sqrt{qu1}$ $\quad f1 = 57.434$ psi Less than $.15 \cdot qu1 = 79.167$ psi

$f2 := 2.5 \cdot \sqrt{qu2}$ $\quad f2 = 65.881$ psi less than $.158qu2 = 109.722$ psi

$Qu1 := \frac{\pi \cdot Ds \cdot L1 \cdot f1 \cdot 144}{1000}$ $\quad Qu1 = 311.788$ kips

$Qu2 := \frac{\pi \cdot Ds \cdot L2 \cdot f2 \cdot 144}{1000}$ $\quad Qu2 = 2.146 \times 10^3$ kips

$Wpile := .150 \cdot \left(\frac{\pi}{4} \cdot Ds^2\right) \cdot L$ $\quad Wpile = 39.761$ kips

$Qallow := 1.33 \cdot \frac{(Qu1 + Qu2 - Wpile)}{2}$

kips

$Qallow = 1.608 \times 10^3$

$Qallowuplift := 1.33 \cdot \frac{(0.7Qu1 + 0.7Qu2 + Wpile)}{2}$

$Qallowuplift = 1.17 \times 10^3$ kips

Total settlement of Drilled shaft:

$Ac := \frac{\pi}{4} \cdot Ds^2$ $\quad Ac = 7.069 \quad ft^2$

$Ec := 144 \cdot 57 \cdot \sqrt{3500}$ $\quad Ec = 4.856 \times 10^5 \quad ksf$

$Ecore1 := 144 \cdot 57 \cdot \sqrt{695}$ $\quad Ecore1 = 2.164 \times 10^5 \quad ksf$

$Ecore2 := 144 \cdot 57 \cdot \sqrt{625}$ $\quad Ecore2 = 2.052 \times 10^5 \quad ksf$

$Emass1 := 0.2 \cdot Ecore1$ $\quad Emass1 = 4.328 \times 10^4 \quad ksf$

$Emass2 := 0.4 \cdot Ecore2$ $\quad Emass2 = 8.208 \times 10^4 \quad ksf$

$\frac{Ec}{Emass1} = 11.22$ $\quad \frac{Ec}{Emass2} = 5.916$ $\quad If := 0.4$

$$s1 := \left[\left[\frac{(Qu1 \cdot L1)}{(Ac \cdot Ec)}\right] + \left[\frac{(Qu1 \cdot If)}{(Ds \cdot Emass1)}\right]\right] \cdot 12 \qquad s1 = 0.016 \quad in$$

$$s2 := \left[\left[\frac{(Qu2 \cdot L2)}{(Ac \cdot Ec)}\right] + \left[\frac{(Qu2 \cdot If)}{(Ds \cdot Emass2)}\right]\right] \cdot 12 \qquad s2 = 0.222 \quad in$$

$S := s1 + s2$

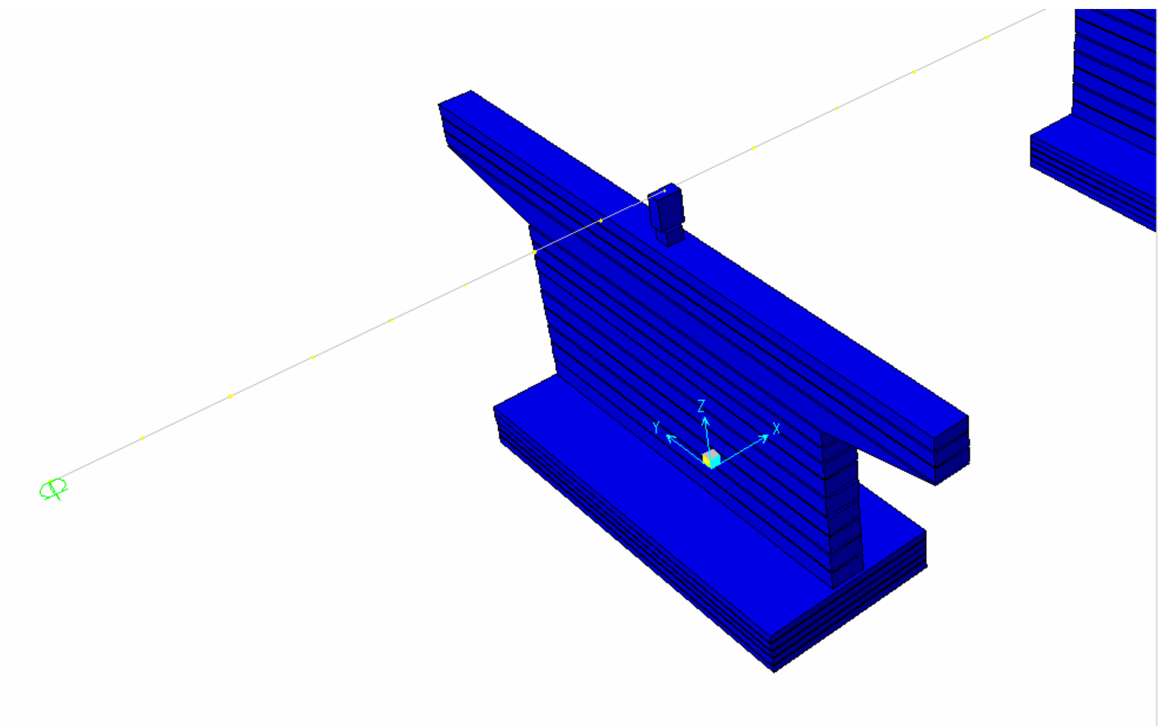
$S = 0.238$ in Total settlement

Appendix B -- Detailed Computations for Seismic Analysis and Design of the Johnson County Bridge Using AASHTO Specifications

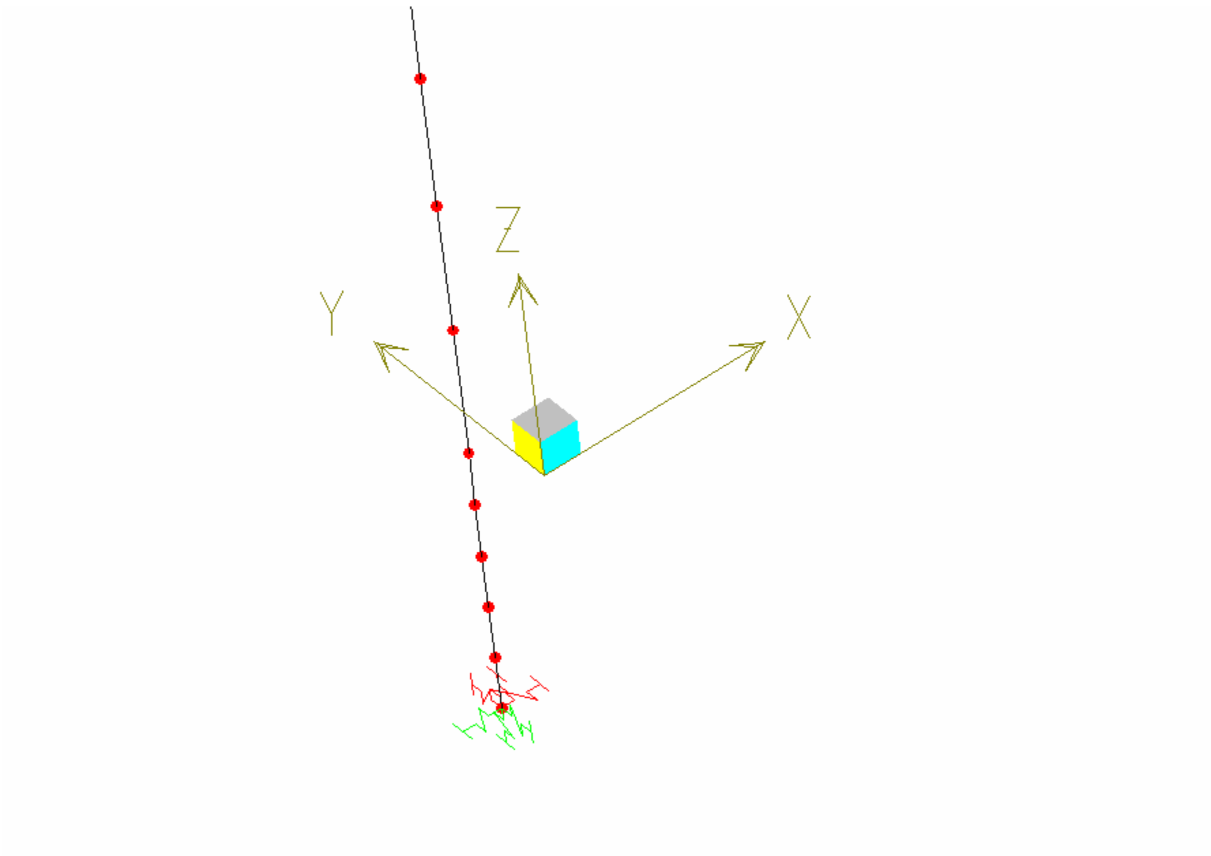
Table of contents of Appendix- B **Page**

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.

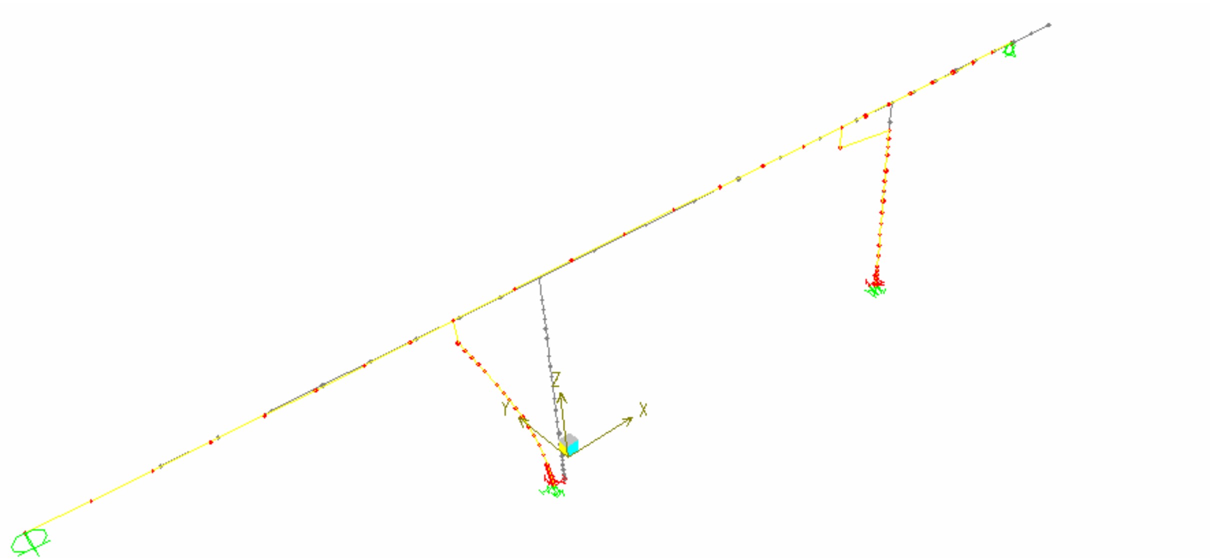
View of SAP2000 Model (with concrete extrusions shown):



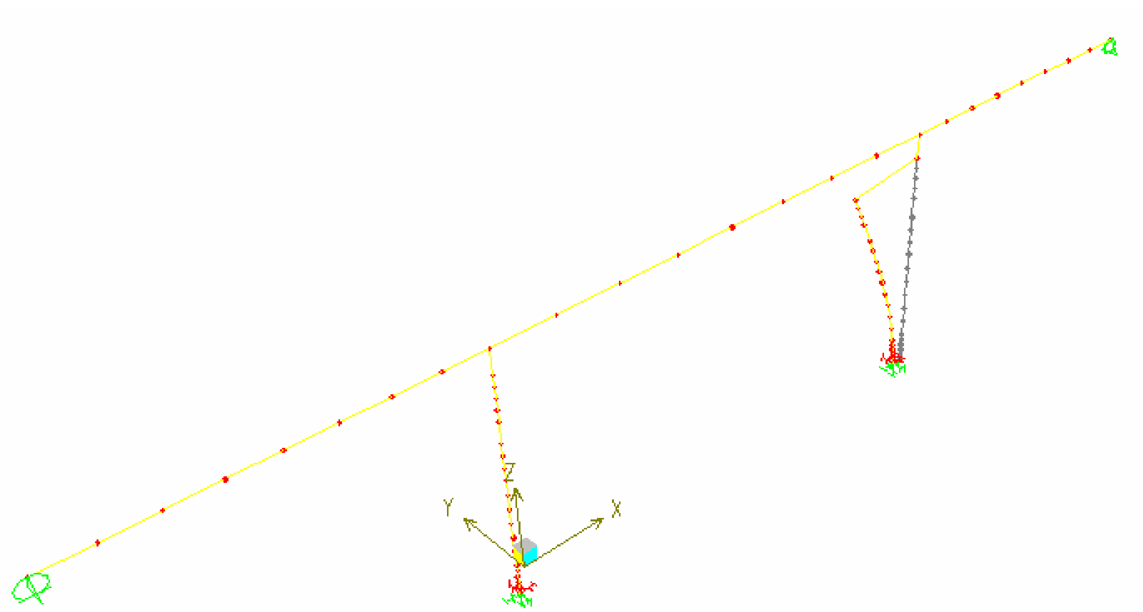
Enlarged View of Translational and Rotational Springs:



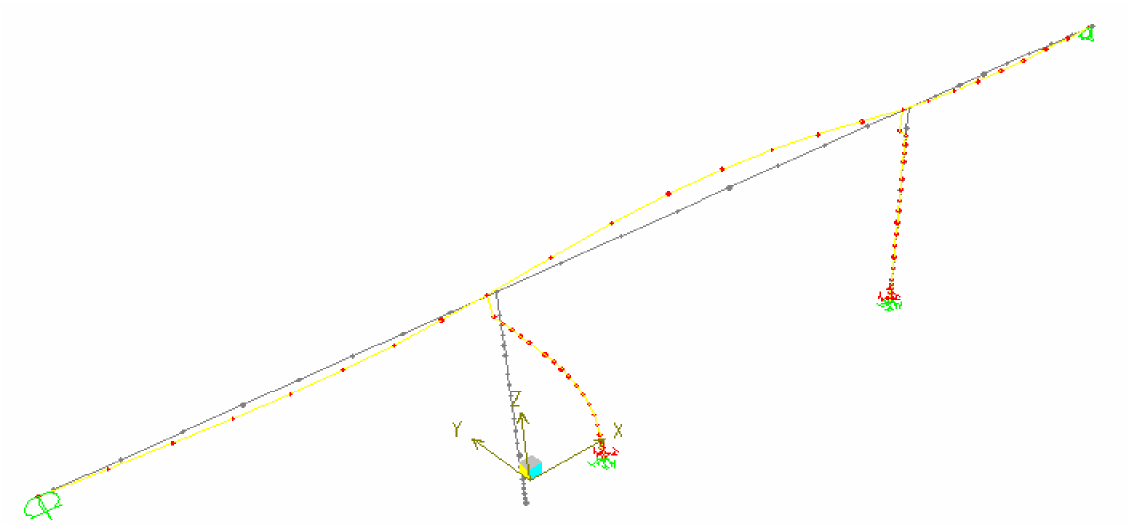
Mode Shape 1 – Period = .6981 seconds



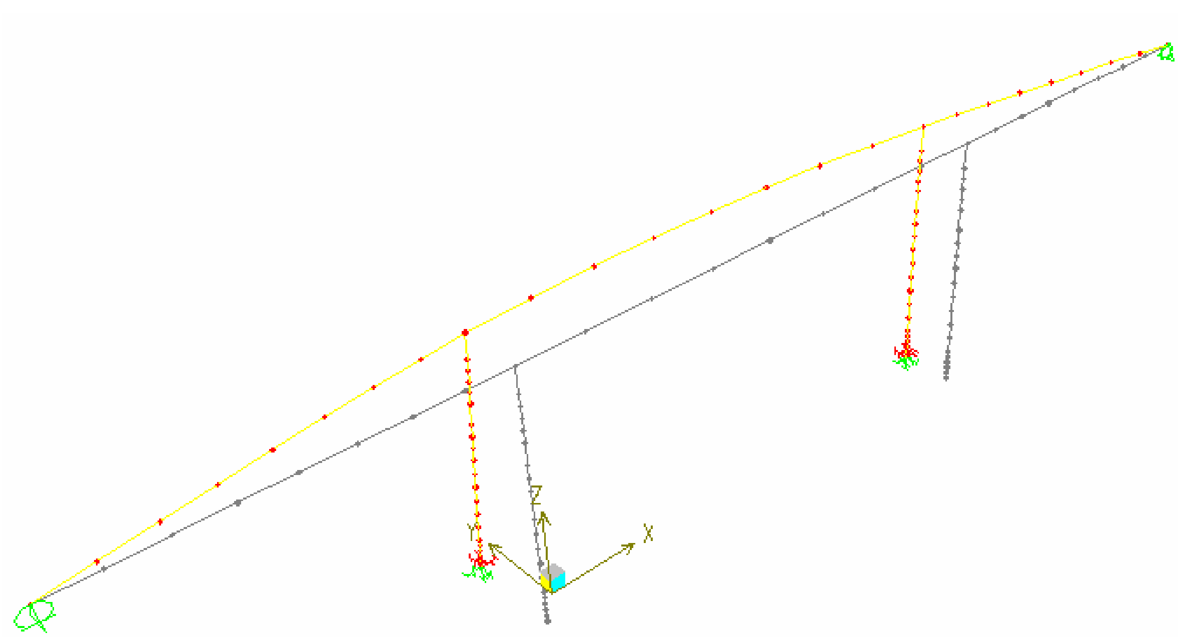
Mode Shape 2 – Period = .1915 seconds



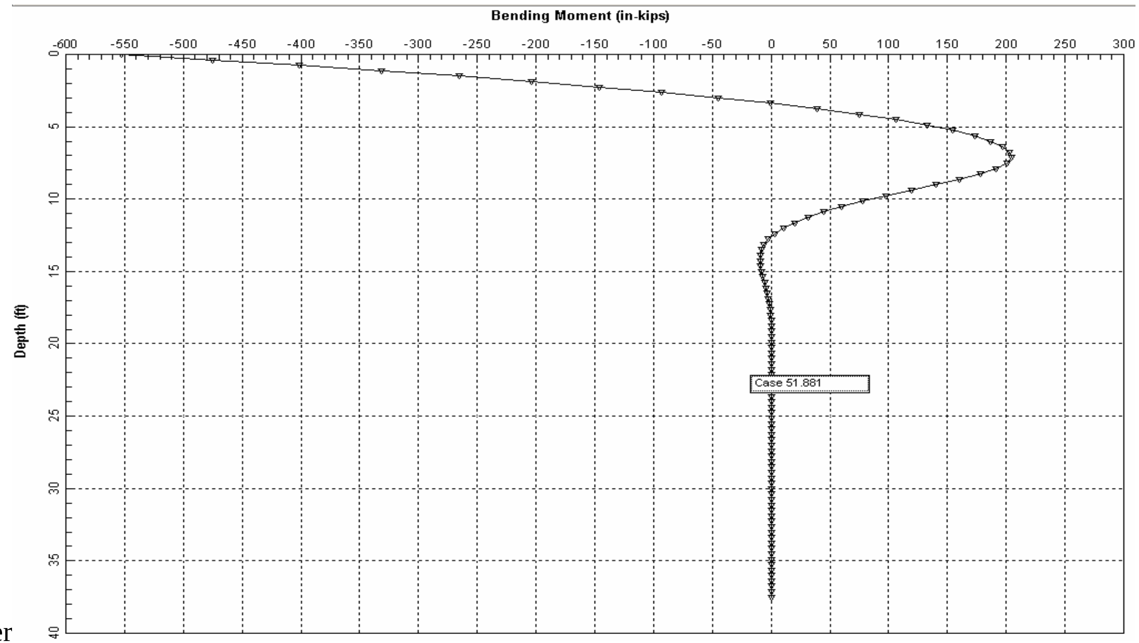
Mode Shape 3 – Period = .1416 seconds



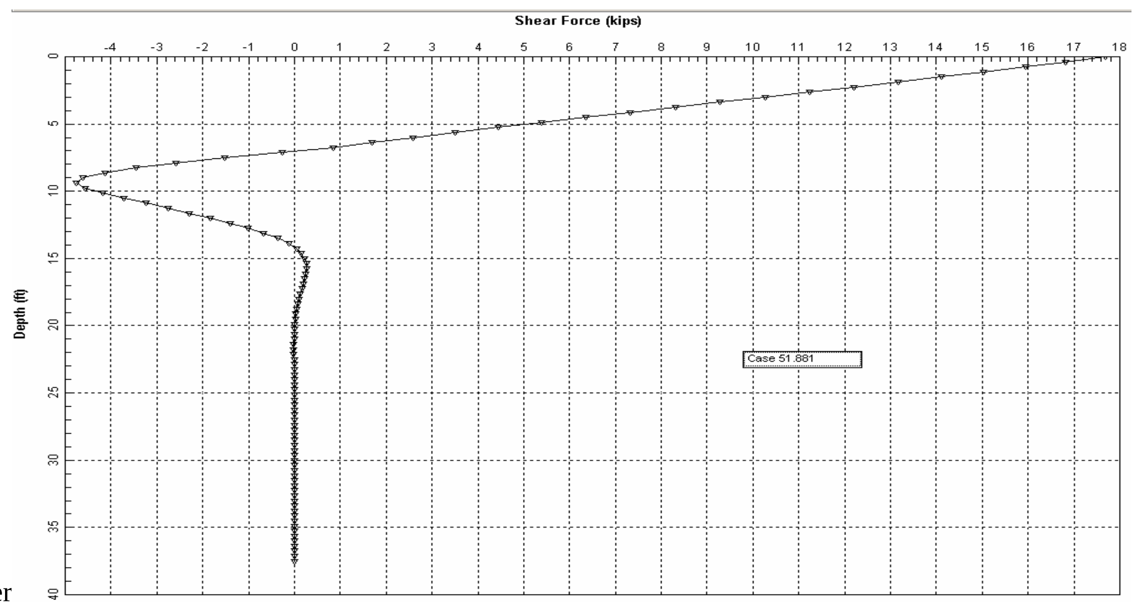
Mode Shape 5 – Period = .1073 seconds



SOUTHERN ILLINOIS UNIVERSITY EDWARDSVILLE **Civil Engineering** www.siue.edu/civil SIUE, Edwardsville, IL 62025-1800 (618)650-2533 Fax: (618)650-2555	SUBJECT FILE: **Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois**	BY: **J.B.**
		CHECKED: **W.N.M.**
TITLE: **Johnson County Bridge**	**AASHTO** **Standard Specifications**	DATE: **6/24/2004**
		Sheet No: **7** of 32



Exp. Pier



Exp. Pier

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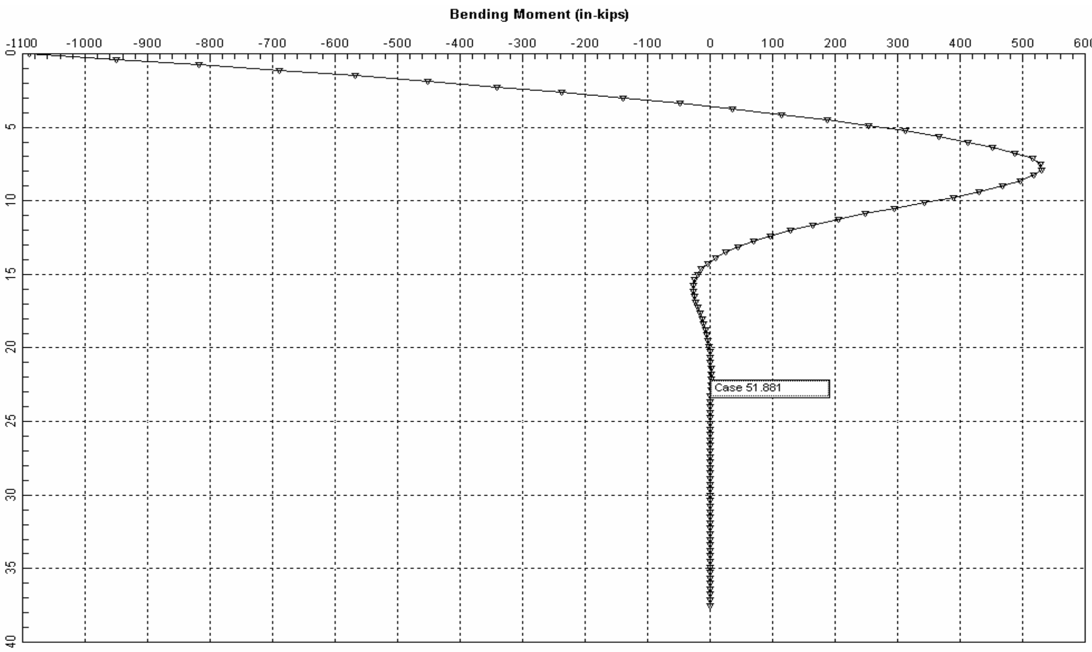
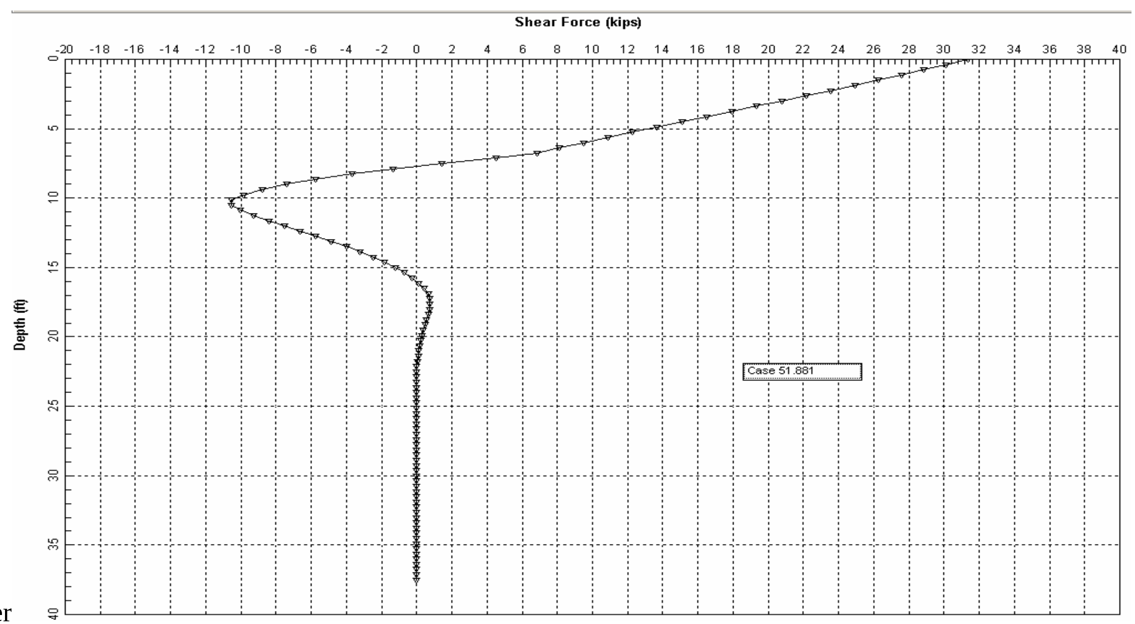


Fixed Pier



Fixed Pier

Piles Springs For Fixed Pier

Pile Information for L-Pile input:

$\phi := 18$ in $\quad \Delta := .205$ in

clearcover $:= 3$ in (to spiral edge)

6 #8 bars for vertical reinforcement

Results from L-Pile: $M_u := \frac{1150}{12}$ $\quad M_u = 96$ k − ft

Spring Stiffness: $L_{pile} := \frac{450}{12}$ $\quad L_{pile} = 37.5$ ft

$D := 18$ in

$A_{pile} := \frac{\left[\left(\frac{\pi}{4}\right)\cdot D^2\right]}{144} \quad A_{pile} = 1.767$ ft^2

$E_c := \left(57\cdot\sqrt{3500}\right)\cdot 144 \quad E_c = 485591.8$ ksf

$k_{axial} := A_{pile}\cdot\frac{E_c}{L_{pile}} \quad k_{axial} = 2.288\times 10^4 \ \frac{k}{ft}$

Using LPile:

shear $:= 31.5$ kips

$\Delta := .205$ in

$k_{lateral} := \frac{12shear}{\Delta} \quad k_{lateral} = 1.844\times 10^3 \ \frac{k}{ft}$

Lateral Springs:

K.x = K.y => $K_x := 8\cdot k_{lateral} \quad K_x = 1.475\times 10^4 \ \frac{k}{ft}$

$K_{axial} := 8\cdot k_{axial} \quad K_{axial} = 1.831\times 10^5 \ \frac{k}{ft}$

Rotational Springs:

$K_{rx} := (k_{axial})\cdot[8\cdot(3.75^2)]$ $\quad K_{rx} = 2.574\times10^6$

$K_{ry} := (k_{axial})\cdot[[4\cdot(3.75^2)] + [4\cdot(11.25^2)]]$ $\quad K_{ry} = 1.287\times10^7$

$K_{rz} := (k_{lateral})\cdot[4\cdot(11.859^2) + 2(5.303^2)]$ $\quad K_{rz} = 1.141\times10^6$

Piles Springs For Expansion Pier

Pile Information for L-Pile input:

$\phi := 18$ in $\quad \Delta := .205$ in

$clearcover := 3$ in (to spiral edge)

6 #8 bars for vertical reinforcement

Results from L-Pile: $M_u := \frac{550}{12}$ $\quad M_u = 46$ k – ft

Spring Stiffness: $L_{pile} := \frac{450}{12}$ $\quad L_{pile} = 37.5$ ft

$D := 18$ in

$A_{pile} := \frac{\left[\left(\frac{\pi}{4}\right)\cdot D^2\right]}{144}$ $\quad A_{pile} = 1.767$ ft^2

$E_c := (57\cdot\sqrt{3500})\cdot 144$ $\quad E_c = 485591.8$ ksf

$k_{axial} := A_{pile}\cdot\frac{E_c}{L_{pile}}$ $\quad k_{axial} = 2.288\times10^4$ $\frac{k}{ft}$

Using LPile:

$shear := 17.75$ kips

$\Delta := .05$ in

$k_{lateral} := \frac{12shear}{\Delta}$ $\quad k_{lateral} = 4.26\times10^3$ $\frac{k}{ft}$

Lateral Springs:

K.x = K.y => $K_x := 8 \cdot k_{lateral}$ $K_x = 3.408 \times 10^4 \ \frac{k}{ft}$

$K_{axial} := 8 \cdot k_{axial}$ $K_{axial} = 1.831 \times 10^5 \ \frac{k}{ft}$

Rotational Springs:

$K_{rx} := (k_{axial}) \cdot \left[8 \cdot \left(3.75^2\right)\right]$ $K_{rx} = 2.574 \times 10^6$

$K_{ry} := (k_{axial}) \cdot \left[\left[4 \cdot \left(3.75^2\right)\right] + \left[4 \cdot \left(11.25^2\right)\right]\right]$ $K_{ry} = 1.287 \times 10^7$

$K_{rz} := (k_{lateral}) \cdot \left[4 \cdot \left(11.859^2\right) + 2\left(5.303^2\right)\right]$ $K_{rz} = 2.636 \times 10^6$

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MODAL PARTICIPATING MASS RATIOS				
With Springs for Division 1-A				
MODE	PERIOD	INDIVIDUAL MODE (%)		
		UX	UY	UZ
1	0.6981	66.18	0.00	0.00
2	0.1915	13.23	0.00	0.00
3	0.1416	12.81	0.00	0.37
4	0.1304	0.48	0.00	10.35
5	0.1073	0.00	87.50	0.00
6	0.0741	0.00	0.00	15.84
7	0.0719	0.00	0.00	61.08
8	0.0714	7.24	0.00	0.00
9	0.0666	0.00	4.01	0.00
10	0.0559	0.00	0.00	0.01
11	0.0530	0.00	0.27	0.00
12	0.0439	0.00	0.00	9.79
13	0.0326	0.00	0.00	0.06
14	0.0306	0.01	0.00	0.00
15	0.0263	0.00	0.01	0.00
16	0.0229	0.00	0.00	0.00
17	0.0217	0.05	0.00	0.00
18	0.0207	0.00	0.00	0.76
19	0.0183	0.00	0.00	0.00
20	0.0166	0.00	4.06	0.00
21	0.0162	0.00	0.00	0.96
22	0.0118	0.00	0.00	0.00
23	0.0117	0.00	0.00	0.00
24	0.0115	0.00	0.00	0.00
25	0.0108	0.00	0.00	0.00
		100.00	95.85	99.22

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AASHTO
Standard Specifications

DIVISION 1-A		**Forces & Moments - EQ_{trans}**			
		Transverse			
Support / Location		Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	115	535	0	0
	Bottom	54	862	0	0
Exp Pier	Top	15	890	0	0
	Bottom	56	288	0	0

DIVISION 1-A		**Forces & Moments - EQ_{long}**			
		Longitudinal			
Support / Location		Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	210	0	190	7
	Bottom	229	0	3619	8
Exp Pier	Top	5	0	0	11
	Bottom	60	0	749	11

Dead Load	
Location	Axial kips
Fix Pier	306
	468
Exp Pier	308
	479

DIVISION 1-A		**Forces & Moments - Eq_{trans} / R**			
		Transverse			
Support / Location		Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	57.5	268	0	0
	Bottom	27	431	0	0
Exp Pier	Top	7.5	445	0	0
	Bottom	28	144	0	0

DIVISION 1-A		**Forces & Moments - Eq_{long} / R**			
		Longitudinal			
Support / Location		Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	105	0	95	4
	Bottom	115	0	1810	4
Exp Pier	Top	3	0	0	6
	Bottom	30	0	375	6

DIVISION 1-A		Forces & Moments - Load Case 1 $(1.00{*}Eq_{trans} + 0.30{*}Eq_{long}) + 1{*}DL$				
Support / Location		Shear 2 kips	Shear 3 kip	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips
Fix Pier	Top	32	58	29	268	307
	Bottom	34	27	543	431	469
Exp Pier	Top	1	8	0	445	310
	Bottom	9	28	112	144	481

DIVISION 1-A		Forces & Moments - Load Case 2 $(1.00{*}Eq_{long} + 0.30{*}Eq_{trans}) + 1{*}DL$				
Support / Location		Shear 2 kips	Shear 3 kip	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips
Fix Pier	Top	105	17	95	80	306
	Bottom	115	8	1810	129	468
Exp Pier	Top	3	2	0	134	308
	Bottom	30	8	375	43	479

Displacement	Longitudinal Direction (in)	Transverse Direction (in)	Max (in)
Bearing of Fixed Pier	1.16924	0.04026	
Top of Fixed Pier	1.10282	0.04102	
Bottom of Fixed Pier Pile Cap	0.18865	0.05776	0.197
Bearing of Expansion Pier	1.17857	0.03341	
Top of Expansion Pier	0.2202	0.03352	
Bottom of Expansion Pier Pile Cap	0.02909	0.03396	0.045

Axial Capacity (Side Resistance) of Limestone and Shale:

$B_r := 1.5$ ft $\quad L_1 := 5$ ft (Shale)

$L_2 := 11.5$ ft (Hard Limestone w/ occasional shale seams)

$C_{o1} := 3.8 \cdot \frac{2000}{144}$ $\quad C_{o1} = 52.778$ psi

(tons / ft^2)(1 ft^2 / 144 in^2)(2000 lb / ton) = psi

$C_{o2} := 50 \cdot \frac{2000}{144}$ $\quad C_{o2} = 694.444$ psi

From Figure 4.6.5.3.1A:

$q_{sr1} := 0$ $\quad q_{sr2} := 75$ psi

$Q_{sr1} := \pi \cdot B_r \cdot L_1 \cdot 0 \cdot \frac{144}{1000}$ $\quad Q_{sr1} = 0$ kips

$Q_{sr2} := \pi \cdot B_r \cdot L_2 \cdot q_{sr2} \cdot \frac{144}{1000}$ $\quad Q_{sr2} = 585$ kips

$W_{pile} := .150 \cdot \left(\frac{\pi}{4} \cdot B_r^{\,2}\right) \cdot 37.5$ $\quad W_{pile} = 9.9$ kips

Note: 1.33 increase in allowable load; Factor of Safety = 2.5 (AASHTO 4.6.5.4)

$$Q_{allow} := 1.33 \cdot \frac{\left(Q_{sr2} - W_{pile}\right)}{2.5}$$

$Q_{allow} = 306$ kips (Allowable Downward Force)

$$Q_{allowup} := 1.33 \cdot \frac{\left(.7 \cdot Q_{sr2} + W_{pile}\right)}{2.5}$$

$Q_{allowup} = 223$ kips (Allowable Uplift Force)

*Note: Q.ult = Q.s + Q.t - W.self (Compression) and .7*Q.s + W (uplift);*
Q.allow = Q.ult / FS

Total Settlement of a Drilled Shaft (4.6.5.5.2):

$Q := 240\text{kips}$

$I_{\rho s} := .55$

$B_r = 1.5$ ft

$E_m := 4 \cdot 10^6 \cdot \frac{144}{1000}$

$E_m = 5.76 \times 10^5$ ksf

$A := \frac{\pi}{4} \cdot 1.5^2$ $A = 1.767$ ft^2

$D_r := 37.5\text{ft}$

$E_c := 57000 \cdot \sqrt{3500} \cdot \frac{144}{1000}$

$E_c = 485592$ ksf

$\rho_S := Q \cdot \left[\left(\frac{I_{\rho s}}{B_r \cdot E_m} \right) + \left(\frac{D_r}{A \cdot E_c} \right) \right]$ $\rho_S = 0.01064$ ft $12 \cdot \rho_S = 0.128$ in

Value of E(m) is approximate

Under the ultmiate 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):

$P_w := 712$ kips

$M_{eql} := 4260$ k – ft $\quad M_{eqt} := 251$ k – ft

$d1 := 3.75 \quad d2 := 3.75 \quad d3 := 11.25$

(distant from center of pile group to center of closest pile center)

$\Sigma d1 := 8\cdot\left(d1^2\right) \qquad \Sigma d1 = 112.5 \quad ft^2$ (weak axis)

$\Sigma d2 := 4\cdot\left(d2^2\right) + 4\cdot\left(d3^2\right) \qquad \Sigma d2 = 562.5 \quad ft^2$ (strong axis)

(Sum of squares of the distances to each pile from the center of the pile group)

Piles Loads:

$$F_{pile1} := \left(\frac{P_w}{8}\right) + \left(M_{eql}\cdot\frac{d1}{\Sigma d1}\right) + \left(M_{eqt}\cdot\frac{d3}{\Sigma d2}\right) \qquad F_{pile1} = 236 \text{ kips}$$

$$F_{pile4} := \left(\frac{P_w}{8}\right) + \left(M_{eql}\cdot\frac{d1}{\Sigma d1}\right) - \left(M_{eqt}\cdot\frac{d3}{\Sigma d2}\right) \qquad F_{pile4} = 226 \text{ kips}$$

$$F_{pile5} := \left(\frac{P_w}{8}\right) - \left(M_{eql}\cdot\frac{d1}{\Sigma d1}\right) + \left(M_{eqt}\cdot\frac{d3}{\Sigma d2}\right) \qquad F_{pile5} = -48 \text{ kips}$$

$$F_{pile8} := \left(\frac{P_w}{8}\right) - \left(M_{eql}\cdot\frac{d1}{\Sigma d1}\right) - \left(M_{eqt}\cdot\frac{d3}{\Sigma d2}\right) \qquad F_{pile8} = -58 \text{ kips}$$

$$F_{piledeadload} := .150\cdot\left(\frac{\pi}{4}\cdot 1.5^2\right)\cdot 37.5 \qquad F_{piledeadload} = 10 \text{ kips}$$

*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):

$P_W := 619$ kips

$M_{eql} := 942$ k – ft $M_{eqt} := 41$ k – ft

$d1 := 3.75$ $d2 := 3.75$ $d3 := 11.25$

(distant from center of pile group to center of closest pile center)

$\Sigma d1 := 8 \cdot \left(d1^2\right)$ $\Sigma d1 = 112.5$ ft^2 (weak axis)

$\Sigma d2 := 4 \cdot \left(d2^2\right) + 4 \cdot \left(d3^2\right)$ $\Sigma d2 = 562.5$ ft^2 (strong axis)

(Sum of squares of the distances to each pile from the center of the pile group)

Piles Loads:

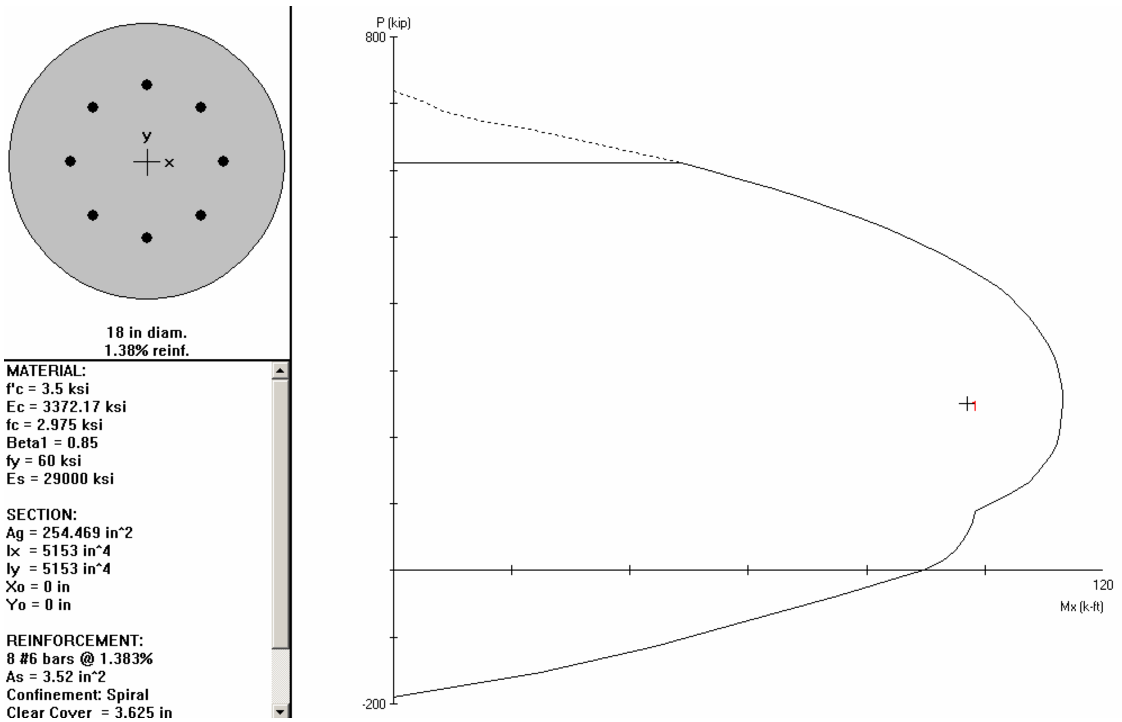
$$F_{pile1} := \left(\frac{P_W}{8}\right) + \left(M_{eql} \cdot \frac{d1}{\Sigma d1}\right) + \left(M_{eqt} \cdot \frac{d3}{\Sigma d2}\right) \qquad F_{pile1} = 110 \text{ kips}$$

$$F_{pile4} := \left(\frac{P_W}{8}\right) + \left(M_{eql} \cdot \frac{d1}{\Sigma d1}\right) - \left(M_{eqt} \cdot \frac{d3}{\Sigma d2}\right) \qquad F_{pile4} = 108 \text{ kips}$$

$$F_{pile5} := \left(\frac{P_W}{8}\right) - \left(M_{eql} \cdot \frac{d1}{\Sigma d1}\right) + \left(M_{eqt} \cdot \frac{d3}{\Sigma d2}\right) \qquad F_{pile5} = 47 \text{ kips}$$

$$F_{pile8} := \left(\frac{P_W}{8}\right) - \left(M_{eql} \cdot \frac{d1}{\Sigma d1}\right) - \left(M_{eqt} \cdot \frac{d3}{\Sigma d2}\right) \qquad F_{pile8} = 45 \text{ kips}$$

PCACOL Output. Piles



Important Quantities:

$f_c := 3.5$ ksi $\quad E_c := 1820 \cdot \left(\sqrt{f_c}\right)$ $\quad E_c = 3.405 \times 10^3$ ksi $\quad f_r := .24 \cdot \left(\sqrt{f_c}\right)$ $\quad f_r = 0.449$ ksi

$f_s := 60$ ksi $\quad E_s := 29000$ ksi $\quad n := \frac{E_s}{E_c}$ $\quad n = 8.517$

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

$L := 46.5 \quad H := 17.75 \quad S := 0$

$N := (8 + 0.02 \cdot L + 0.08 \cdot H) \cdot \left(1 + 0.000125 \cdot S^2\right)$ $\quad N = 10.35$ This is the Required Length of Seat per side in Inches

Available Seat Width for entire expansion Pier (Pier 2 on each bridge) equals 26 inches

6.6.2 Special Requirments when designed as Columns (N/A if designed as pier)

Can use 7.6.3 as a suggested minimum for vertical and horizantal reinforcement of .25%.

Wall Design for Seismic:

From SAP2000 Model (fixed pier forces only):

$P_1 := 472$ Axial Load on pier for LC1

$M_{w1} := 1810$ Moment about strong axis (bending in weak direction) of wall for LC1

$M_{s1} := 129$ Moment about weak axis (bending in strong direction) of wall for LC1

$P_2 := 470$ Axial Load on pier for LC2

$M_{w2} := 543$ Moment about strong axis of wall for LC2

$M_{s2} := 431$ Moment about weak axis of wall for LC2

Division 1: Section 8 8.16 Strength Design Method

8.16.1.2.2

Calculation of the ϕ factor:

$f_c = 3.5$ $A_g := 308 \cdot 26$ $A_g = 8.008 \times 10^3$

$P_u := 473$ $\phi P_n := P_u$ $\phi := .9 - 2 \cdot \frac{\phi P_n}{f_c \cdot A_g}$ $\phi = 0.87$

8.16.5 Slenderness Effects in Compression Members

$r := .3 \cdot 26$ $r = 7.8$ in 8.16.5.2.2

$k := 2$ 8.16.5.2.3

$l_u := (3 + 2 + 11) \cdot 12 + 9$ $l_u = 201$ in

$\text{slenderness} := k \cdot \frac{l_u}{r}$ $\text{slenderness} = 51.538$ **Moment Magnifier is required.**

8.16.5.2.7 $\beta_d := 1$ $I_s := \left(.79 \cdot 9^2\right) \cdot 50$ $I_s = 3.2 \times 10^3$

$I_g := \frac{1}{12} \cdot 308 \cdot 26^3$ $I_g = 4.511 \times 10^5$

$$EI := \frac{\left[\frac{\left(E_c \cdot I_g\right)}{5} + E_s \cdot I_s\right]}{1 + \beta_d}$$ $EI = 199994062$

$$P_c := \frac{\left(\pi^2 \cdot EI\right)}{\left(2 \cdot l_u\right)^2}$$ $P_c = 12214$

$\phi P_c := \phi \cdot P_c$ $\phi P_c = 10581$

$P_u := 473.5$

$$\delta_b := \frac{1}{1 - \frac{P_u}{\phi P_c}}$$ $\delta_b = 1.05$

$\frac{l_u}{1500} = 0.13$ *In definition of M.2s; so it has appreciable sidesway since:*

Largest Deflection according to SAP2000 model is around 1.17".

$$\delta_s := \frac{1}{1 - \frac{P_u}{\phi P_c}} \qquad \delta_s = 1.047$$

Since the moment due to gravity load (m.2b) is zero, δ.b moment magnifier does not apply to this pier.

	Fix Pier:	Expansion pier:
$M_{cw1} := \delta_s \cdot M_{w1}$	$M_{cw1} = 1.895 \times 10^3$	$1.05 \cdot 374 = 393$
$M_{cs1} := \delta_s \cdot M_{s1}$	$M_{cs1} = 135.043$	$1.05 \cdot 43 = 45.15$
$M_{cw2} := \delta_s \cdot M_{w2}$	$M_{cw2} = 568.439$	$1.05 \cdot 112 = 117.6$
$M_{cs2} := \delta_s \cdot M_{s2}$	$M_{cs2} = 451.192$	$1.05 \cdot 144 = 151.2$

Minimum Steel Required:

Using Division 1-A (7.6.3): Piers must have a ρ(min) = .0025 for both ρ.h and ρ.n

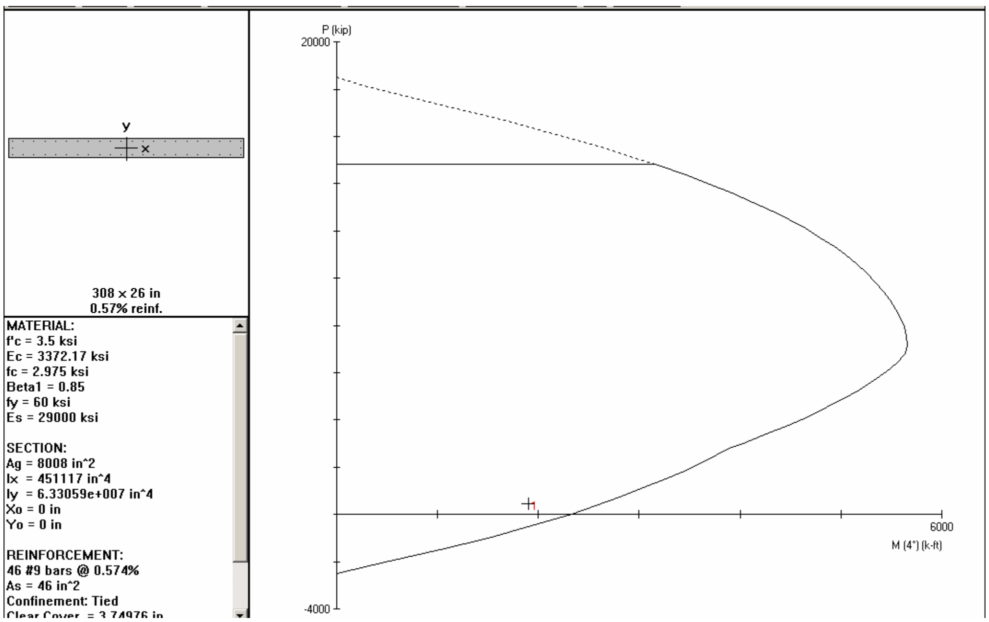
$(.0025)(308 \cdot 26) = 20.02 \ \text{in}^2$ *(minimum amount of vertical steel)*

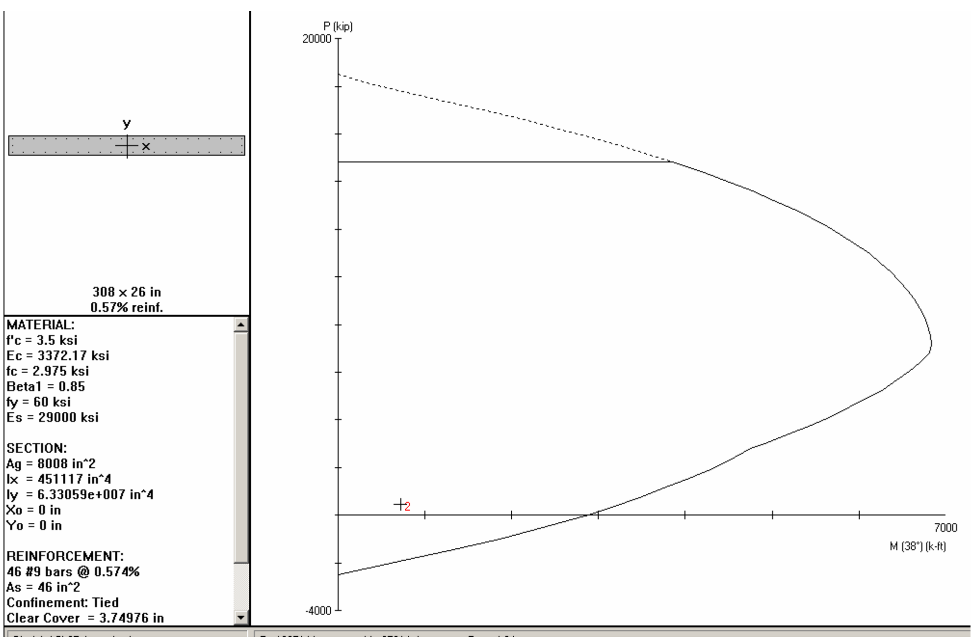
Using PCACOL:

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.

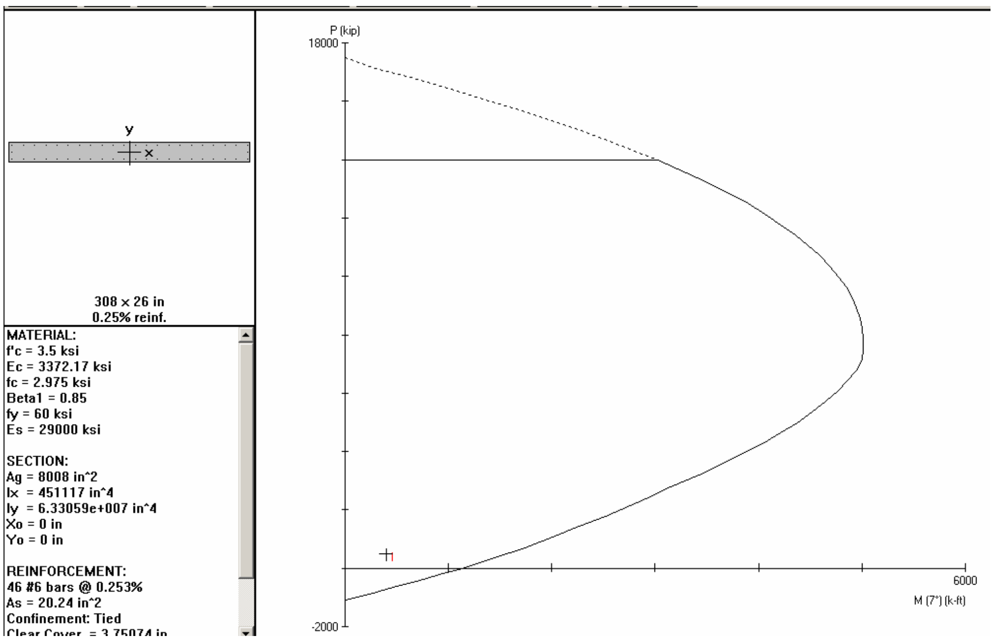
Expansion: vertical reinforcement is 46 #6 for .25%. 22 bars spaced evenly along each long face, with one additional vertical bar placed in the center on each short side.

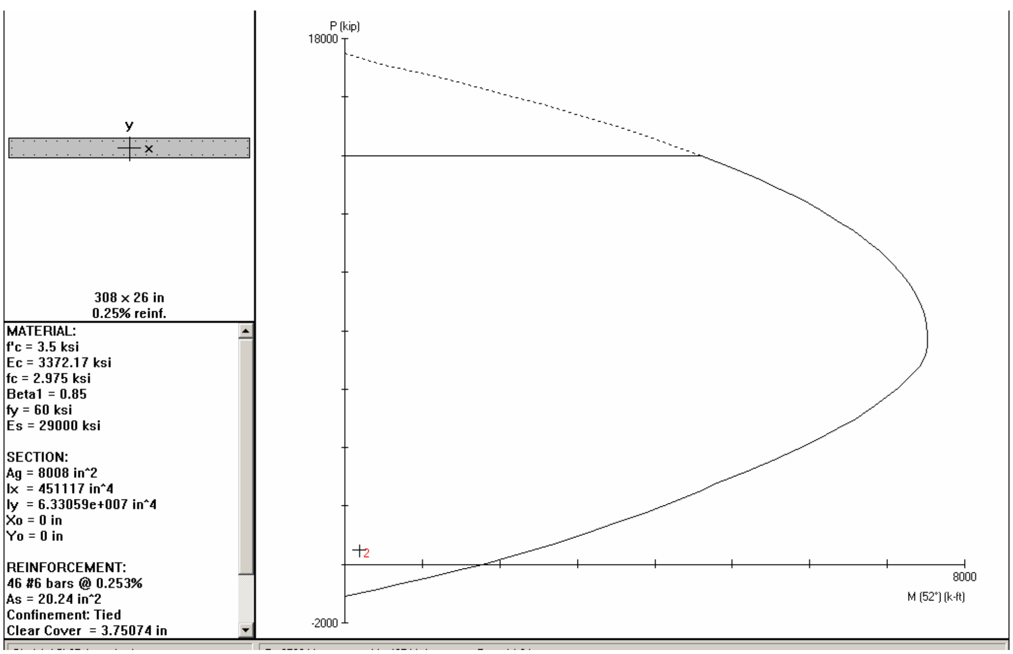
Clear cover of 3" was used for each wall, measured to side of transverse rebars, assuming #6 bars were the appropriate size for transverse reinforcement.





Johnson County Bridge – Fixed Pier





Johnson County Bridge – Expansion Pier

8.16.2.3.1 - Transverse Reinforcement:

$\rho_h := .0025 \qquad A_h := \rho_h \cdot 12 \cdot 26 \qquad A_h = 0.78 \quad in^2$

$A_b := .79 \qquad s := 12 \cdot 2 \cdot \frac{A_b}{A_h} \qquad s = 24.308$ Spacing for #8 bars.

$A_b := .60 \qquad s := 12 \cdot 2 \cdot \frac{A_b}{A_h} \qquad s = 18.462$ Spacing for #7 bars.

$A_b := .44 \qquad s := 12 \cdot 2 \cdot \frac{A_b}{A_h} \qquad s = 13.538$ **Spacing for #6 bars; Use these at 12" o.c.**

$A_b := .31 \qquad s := 12 \cdot 2 \cdot \frac{A_b}{A_h} \qquad s = 9.538$ Spacing for #5 bars.

$$A_h := 12 \cdot 2 \cdot \frac{.44}{12} \qquad A_h = 0.88 \qquad \rho_h := \frac{A_h}{12 \cdot 26} \qquad \rho_h = 0.0028$$

8.18.2.3.4: *spacing for "cross ties" can not be more than 24"*

Cross ties will restraint 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: $l_d := 1.7 \cdot \left(.04 \cdot .44 \cdot \frac{60000}{\sqrt{3500}} \right) \quad l_d = 30.344$ *Use 31" for development length of U's*

8.27.3: *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 $f_c := 3500$ $\rho_h := .003$ $f_y := 60000$

R = 1.0

Div 1A (7.6.3): $v_c := 2 \cdot \sqrt{f_c}$ $v_c = 118$ psi

Fix Pier: $V_{w1} := 105$ Weak direction shear for LC1

$V_{s1} := 8$ Strong direction shear for LC1

$V_{w2} := 31.5$ Weak direction shear for LC2

$V_{s2} := 27$ Strong direction shear for LC2

$V_{R1} := \sqrt{V_{w1}^2 + V_{s1}^2}$ $V_{R1} = 105$ kip

$V_{R2} := \sqrt{V_{w2}^2 + V_{s2}^2}$ $V_{R2} = 41$ 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.*

$v_r := \frac{V_{R1} \cdot 1000}{308 \cdot 26}$ $v_r = 13$ psi

applied shear stress at wall/pilecap connection

$\phi v_c := .85 v_c$ $\phi v_c = 101$ 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:

$\Delta_1 := .189 \quad \Delta_2 := .058 \quad \Delta_{max} := \sqrt{\left(\Delta_1{}^2\right) + \left(\Delta_2{}^2\right)} \quad \Delta_{max} = 0.198$ inches

From LPile: $v_{max} := 31.5$

Shear Stress: $v_{stress} := \dfrac{v_{max}}{(18 \cdot 14.125)} \cdot \dfrac{1000}{1.33} \quad v_{stress} = 93.153$ psi

$v_c := \left(.95 \cdot \sqrt{3500}\right) \quad v_c = 56.203$ psi

The Pile requires shear reinforcement since V.stress is more than V.c

(8-7): $f_{s1} := \left(v_{stress} - v_c\right) \cdot 18 \cdot \dfrac{9}{.31} \quad f_{s1} = 19310$ psi

The limit for f.s = 24000 psi; so 9" pitch on a #5 spiral will work.

$v_{noshear} := \left(.5 \cdot v_c\right) \cdot \dfrac{(18 \cdot 14.125)}{1000} \quad v_{noshear} = 7$ 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:

$M := 97$ 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 ρ = .00138; minimum is .5% (D1-A, 6.4.2(C)).

Required Pitch Spacing with #5 bars:

(8.18.2.2):

$a_s := .31 \qquad d_b := .625 \qquad D_c := 18 - (3 \cdot 2) \qquad D_c = 12$

$A_g := \left(\frac{1}{4} \cdot \pi \cdot 18^2\right) \qquad A_c := \left(\frac{1}{4} \cdot \pi \cdot D_c^{\,2}\right) \qquad f_c := 3500 \qquad f_y := 60000$

(minimum) $\rho_s := .45 \cdot \left[\left(\frac{A_g}{A_c}\right) - 1\right] \cdot \frac{f_c}{f_y} \qquad \rho_s = 0.0328$

$S_{max} := \frac{\left[4 \cdot a_s \cdot \left(D_c - d_b\right)\right]}{\left(D_c^{\,2} \cdot \rho_s\right)} \qquad S_{max} = 2.99$ *Use #5 bars at pitch of 3".*

(Above equation comes from Nawy page 347; it uses that fact that ρ.s is actually the volume of the spiral divided by the volume of the core concrete.)

Use a #5 sprial 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: $48 \cdot d_b = 30$ in (controls)

- or - 12 in

Use a 30" lap splice to connect two pieces of spiral together.

(8.24): Development Lengths:

for flexure: effective depth = 14.125" (controls)
15 * d.b = 11.25"

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.

Pile cap: *Moment:*

$M_u := 3465$ k – ft

$b_{eff} := (.8 \cdot 2.67) + 3.75 \quad b_{eff} = 5.89$ ft

(IDOT Bridge Manual 3.7.5)

(actual spacing = 7.5'; so 5.89' controls)

$$R := \frac{(M_u \cdot 12000)}{(.9 \cdot 26 \cdot 12 \cdot 32^2)} \qquad R = 145 \text{ psi}$$

$$\omega := .0025 \cdot \left(\frac{60000}{3500}\right) \qquad R_d := \omega \cdot 3500 \cdot (1 - .59 \cdot \omega) \qquad R_d = 146.207 \text{ psi}$$

$A_s := (.0025 \cdot 12 \cdot 32) \quad A_s = 0.96 \ \frac{in^2}{ft}$ of footing (transverse steel)

Transverse rebar for footing is chosen to be #6 bars @ 5.5" o.c.

8.17.1.1:

$f_r := 7.5 \cdot \sqrt{3500} \quad f_r = 444$ psi $\qquad I_g := 12 \cdot (12 \cdot b_{eff}) \cdot 36^3 \quad I_g = 39544879 \ in^4$

$y_t := 18$ in

$$M_{cr} := \frac{(f_r \cdot I_g)}{(y_t)} \qquad (1.2 \cdot M_{cr}) \cdot \frac{1}{12000} = 97479 \text{ k – ft}$$

$$R_{cr} := \frac{(1.2 \cdot M_{cr})}{(.9 \cdot 26 \cdot 12 \cdot 32^2)} \qquad R_{cr} = 4068 \text{ psi}$$

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.

$A_s := (.0025 \cdot 12 \cdot 32) \cdot 1.33 \qquad A_s = 1.277 \ \frac{in^2}{ft}$

Use # 6 bars at 4" o.c. for transverse reinforcement

$A_{sl} := (.0018 \cdot 12 \cdot 32) \qquad A_{sl} = 0.691 \ \frac{in^2}{ft}$

Use #6 bars at 7.5" o.c. for longitudinal reinforcement

Shear: **One-Way** (8.15.5.6):

(4.4.11.3): d = 32" d.p = 18"

Location of critical section with respect to pile center: $l_{cs} := (3.75 \cdot 12) - 13 - 32$ $l_{cs} = 0.0$ in

Pile center is located directly over critical section; thus according to 4.4.11.3.2: we use interpolation and find that 1/2 of reaction is used in this shear check.

(8.15.5) & (8.15.5.6): $v := \dfrac{(.5 \cdot 236 \cdot 1000)}{\left(12 \cdot b_{eff} \cdot 32\right)} \cdot \dfrac{1}{1.33}$ $v = 39$ psi

$v_c := \left(.9 \cdot \sqrt{3500}\right)$ $v_c = 53$ psi

Pile cap works in one-way shear.

Two-Way (8.15.5.6):

$b_o := \pi \cdot 2 \cdot (9 + 16)$ $b_o = 157$ in

$v_{2way} := \dfrac{(236 \cdot 1000)}{\left(b_o \cdot 32\right)} \cdot \dfrac{1}{1.33}$ $v_{2way} = 35$ psi

$v_c = 53$ psi

Pile cap works in two-way shear.

Pullout of Piles in Tension:

$A_s := 8 \cdot .44 \quad A_s = 3.52$ $\quad$ in^2

$f_y := 24000$

$FS := 2$ (Also: 33% increase in allowable stresses)

$$\frac{1.33\left(A_s \cdot f_y\right) \cdot \frac{1}{1000}}{FS} = 56$$ kips (Allowable Uplift of a Single Pile)

$F_{uplift} := \frac{58}{.8}$ (Divide Ultimate Load by R = .8 for connections)

$F_{uplift} = 73$ kips (Largest Uplift Force on a Single Pile)

$A_{s2} := 11 \cdot .44 \quad A_{s2} = 4.84$ $\quad$ in^2

$$1.33 \cdot \frac{\left(A_{s2} \cdot f_y\right)}{FS} \cdot \frac{1}{1000} = 77$$ kips

We need 3 (or 4) #6 bar dowels of appropriate length to resist pullout failure

8.25.1:

$l_d := .04 \cdot .44 \cdot \frac{60000}{\sqrt{3500}} \quad l_d = 17.85$ in

8.25.3.3:

$.75 \cdot l_d = 13.387$ *Use a development length of 13.5" into the main part of the pile for the #6 dowels*

8.29.2:

$l_{hb} := 1200 \cdot \frac{.750}{\sqrt{3500}} \quad l_{hb} = 15.213$ in

8.29.3.2: $.7 \cdot l_{hb} = 10.649$ *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: $v_{max} := \frac{31.5}{.8}$ (Use R factor of .8 for connections)

Shear Stress: $v_{stress} := \frac{v_{max}}{(18 \cdot 14.125)} \cdot \frac{1000}{1.33}$ $v_{stress} = 116.442$ psi

(Use 33% increase in allowable stresses)

(8-7): $f_s := \left(v_{stress}\right) \cdot 18 \cdot \frac{3}{.31}$ $f_s = 20283$ psi

The limit for f(s) = 24000 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 ever 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

Table of contents of Appendix-C

Pertinent Information about Materials used, etc. :

$pconv := \left(1.45 \times 10^{-4}\right)$		**This is the N/m^2 to psi conversion factor.**	
$feet := \frac{1}{.3048}$	$feet = 3.281$	**This is the meter to feet conversion.**	
$inch := feet \cdot 12$	$inch = 39.37$	**This will convert meters to inches.**	
$f_c := 24 \cdot \left(10^6\right) \cdot pconv$	$f_c = 3480.000$		
$f_y := 400 \cdot \left(10^6\right) \cdot pconv$	$f_y = 5.8 \times 10^4$	(Reinforcement)(psi)	
$F_{y1} := 345\left(10^6\right) \cdot pconv$	$F_{y1} = 5.003 \times 10^4$	(Structural) (psi)	
$F_{y2} := 250 \cdot \left(10^6\right) \cdot pconv$	$F_{y2} = 3.625 \times 10^4$	(Structural) (psi)	
$E_c := 57000 \cdot \left(\sqrt{f_c}\right)$	$E_c = 3.363 \times 10^6$	**Modulus of Elasticity of concrete.**	
$E_s := 29000000$	$E_s = 2.9 \times 10^7$	**Modulus of Elasticity of steel**.	
$n := \frac{E_s}{E_c}$	$n = 8.624$	**Modular Ratio**	
$boltD := 24 \cdot \left(10^{-3}\right) \cdot inch$	$boltD = 0.945$	**Bolt Diameter (in.)**	
$Skew := 26 + \left(\frac{31}{60}\right) + \left(\frac{40}{3600}\right)$	$Skew = 26.528$	**Skew Angle (degrees)**	

Calculations for Moment of Inertia for Superstructure

$L_{eff} := 41.50 \cdot inch$	$L_{eff} = 1.634 \times 10^3$	Effective length of a span (in)
$t_{avg} := .195 \cdot inch$	$t_{avg} = 7.677$	Slab thickness (in)
$t_w := .014 \cdot inch$	$t_w = 0.551$	Web thickness (in)
$b_{f1} := .300 \cdot inch$	$b_{f1} = 11.811$	Top flange width outerline (in)
$b_{f2} := .600 \cdot inch$	$b_{f2} = 23.622$	Top flange width innerline (in)
$gspace := 1.905 \cdot inch$	$gspace = 75$	Girderline spacing (in)

1. Effective Flange Width of interior maybe be taken as least of:

$efw_1 := .25 \cdot L_{eff}$	$efw_1 = 408.465$	
$efw_2 := (12 \cdot t_{avg}) + t_w$	$efw_2 = 92.677$	
$efw_3 := (12t_{avg}) + (.5 \cdot b_{f1})$	$efw_3 = 98.031$	(outer)
$efw_{3b} := (12 \cdot t_{avg}) + (.5 \cdot b_{f2})$	$efw_{3b} = 103.937$	(inner)
$efw_4 := gspace$	$efw_4 = 75$	**This controls**

$EFW_{inside} := gspace$ $\qquad EFW_{inside} = 75$

2. Effective Flange Width of exterior maybe be taken as .5 EFW plus least of:

$efw_5 := \frac{1}{8} \cdot L_{eff}$	$efw_5 = 204.232$	
$efw_6 := (6 \cdot t_{avg}) + (.5 \cdot t_w)$	$efw_6 = 46.339$	
$efw_7 := (6 \cdot t_{avg}) + (.25 \cdot b_{f1})$	$efw_7 = 49.016$	
$efw_8 := (6 \cdot t_{avg}) + (.25 \cdot b_{f2})$	$efw_8 = 51.969$	
$ohw := .925 \cdot inch$	$ohw = 36.417$	**This controls**

$EFW_{outside} := (.5 \cdot EFW_{inside}) + ohw$ $\qquad EFW_{outside} = 73.917$

We can use all of the slab for moment of inertia.

For Moment of Inertia Calculations of the superstructure.

$I_{yy1} := 36.2084$ $\qquad I_{zz1} := 3676.5792$

$I_{yy2} := 61.4485$ $\qquad I_{zz2} := 5343.1700$

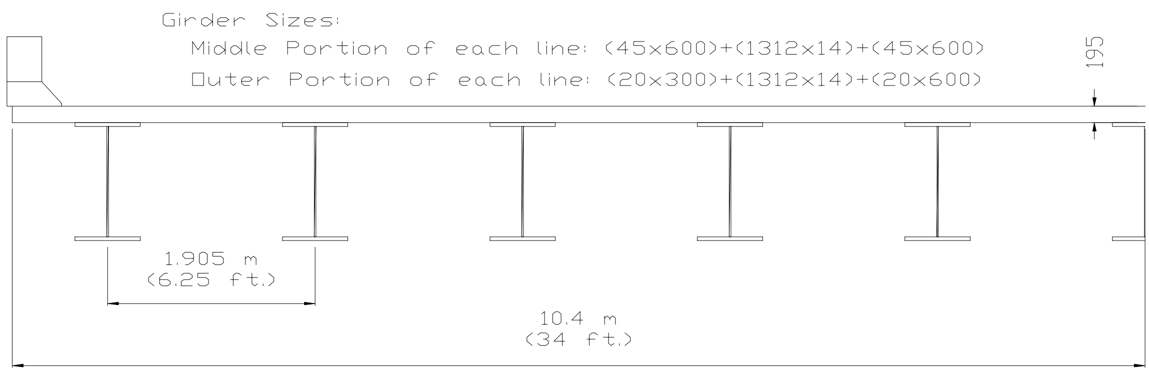


Fig.1. Cross-section of superstructure

Torsional Properties:

We assume only the deck will resist torsion.

$b_{deck} := 20.8 \cdot feet$ $\quad b_{deck} = 68.241 \quad$ (ft)

$h_{deck} := .195 \cdot feet$ $\quad h_{deck} = 0.64 \quad$ (ft)

$J := \left(b_{deck}\right) \cdot \frac{\left(h_{deck}\right)^3}{3}$ $\quad J = 5.956 \quad \left(ft^4\right)$

Cross-Sectional Area of Superstructure:

$b_{slab} := 20.8 \cdot \frac{feet}{n}$

$b_{slab} = 7.913$

$h_{slab} := .195 \cdot feet$

$h_{slab} = 0.64$ $\quad area_{slab} := h_{slab} \cdot b_{slab} \quad area_{slab} = 5.062$

$area_{girder1} := 11\cdot[(.02\cdot.3) + (.02\cdot.6) + (1.372\cdot.014)]\cdot feet\cdot feet$ $\quad area_{girder1} = 4.406$

$area_{girder2} := 11\cdot[(.045\cdot.6) + (.045\cdot.6) + (1.372\cdot.014)]\cdot feet\cdot feet$ $\quad area_{girder2} = 8.668$

$area_{super1} := (area_{slab} + area_{girder1})$ $\quad area_{super1} = 9.468 \quad ft^2$

$area_{super2} := (area_{slab} + area_{girder2})$ $\quad area_{super2} = 13.73 \quad ft^2$

$area_{barrier} := [(.2\cdot.535) + (.065\cdot.535) + (.320\cdot.330) + (.5\cdot.180\cdot.330)]\cdot feet\cdot feet$ $\quad area_{barrier} = 2.982$

$area_{median} := \left[.150\cdot\left[\frac{(1.3 + 1.236)}{2}\right]\right]\cdot feet\cdot feet$ $\quad area_{median} = 2.047$

<u>Loads :</u>

Dead Loads of the superstructure

$w_{barrier} := 2\cdot area_{barrier}\cdot 150$ $\quad w_{barrier} = 895 \quad plf$

$w_{slab} := (19.05\cdot feet)\cdot(.195\cdot feet)\cdot 150$ $\quad w_{slab} = 5998 \quad plf$

$w_{girder1} := 11[(1.372\cdot feet)\cdot(.014\cdot feet) + (.020\cdot feet)\cdot(.900\cdot feet)]\cdot 490$

$w_{girder2} := 11[(1.372\cdot feet)(.014\cdot feet) + (.045\cdot feet)\cdot(1.200\cdot feet)]\cdot 490$

$w_{girder1} = 2159 \quad plf$

$w_{girder2} = 4247 \quad plf$

$w_{median} := area_{median}\cdot .190$ $\quad w_{median} = 0.389 \quad plf$

Interior Cross-frames: 2 - L4x4x5/16 @ 2546 mm
1 - L4x4x5/16 @ 1904 mm
weight(lb.) per linear foot = 8.16

10 spaces underneath bridge 13 cross frames per length total number = 130

$w_{cframe} := 13 \cdot 10 \cdot [(2.546 + 2.546 + 1.904) \cdot feet] \cdot \frac{8.16}{272}$ $\quad w_{cframe} = 89.516$ plf

Future Wearing Surface was 2.4 kN/m^2 or about 50 psf.

$w_{bridge} := (3 + 4.2 + .3 + 3.9 + 4.2 + 3) \cdot feet$ $\quad w_{bridge} = 61.024$ ft

$w_{fws} := 2.4 \cdot 1000 \cdot pconv \cdot 144 \cdot w_{bridge}$ $\quad w_{fws} = 3.058 \times 10^3$ f

Adding the individual pieces together we get:

$w_{outer} := w_{barrier} + w_{median} + w_{cframe} + w_{slab} + w_{girder1} + w_{fws}$ $\quad w_{outer} = 12199$ plf

$w_{inner} := w_{barrier} + w_{median} + w_{cframe} + w_{slab} + w_{girder2} + w_{fws}$ $\quad w_{inner} = 14288$ 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.

$s := 1.5$ AASHTO Table 3.5.1

- Design Response Spectrum Development - MCE

Using USGS website to find probalistic ground motion values, in %g, at nearest grid point area (2% PE in 50 year):

$S_s := 0.6261940$ 0.2-second period spectral acceleration

$S_1 := 0.196946$ 1-second period spectral acceleration

$F_a := 1.3$ Site coefficient for short period

$F_v := 2.0$ Site coefficient for long period

$S_{DS} := F_a \cdot S_s$

$S_{DS} = 0.814$ Design earthquake response spectral acceleration at short period

$S_{D1} := F_v \cdot S_1$

$S_{D1} = 0.394$ Design earthquake response spectral acceleration at long period

$0.4 \cdot S_{DS} = 0.326$

$T_S := \dfrac{S_{D1}}{S_{DS}}$ Period at the end of construction design spectral acceleration plateau

$T_S = 0.484$ Sec

$T_0 := 0.2 \cdot T_S$ Period at the beginning of construction design spectral acceleration plateau

$T_0 = 0.097$ Sec

$F_a \cdot S_s = 0.814$ 0.6<Fa.Ss

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 probalistic ground motion values, in %g, at nearest grid point area (2% PE in 50 year):

$S_{s\_fe} := 0.06763$ 0.2-second period spectral acceleration

$S_{1\_fe} := 0.01485$ 1-second period spectral acceleration

$F_{a\_fe} := 1.6$ Site coefficient for short period

$F_{v\_fe} := 2.4$ Site coefficient for long period

$S_{DS\_fe} := F_{a\_fe} \cdot S_{s\_fe}$

$S_{DS\_fe} = 0.108$ Design earthquake response spectral acceleration at short period

$S_{D1\_fe} := F_{v\_fe} \cdot S_{1\_fe}$

$S_{D1\_fe} = 0.036$ Design earthquake response spectral acceleration at long period

$0.4 \cdot S_{DS\_fe} = 0.043$

$T_{S\_fe} := \dfrac{S_{D1\_fe}}{S_{DS\_fe}}$ Period at the end of construction design spectral acceleration plateau

$T_{S\_fe} = 0.329\ \mathrm{Sec}$

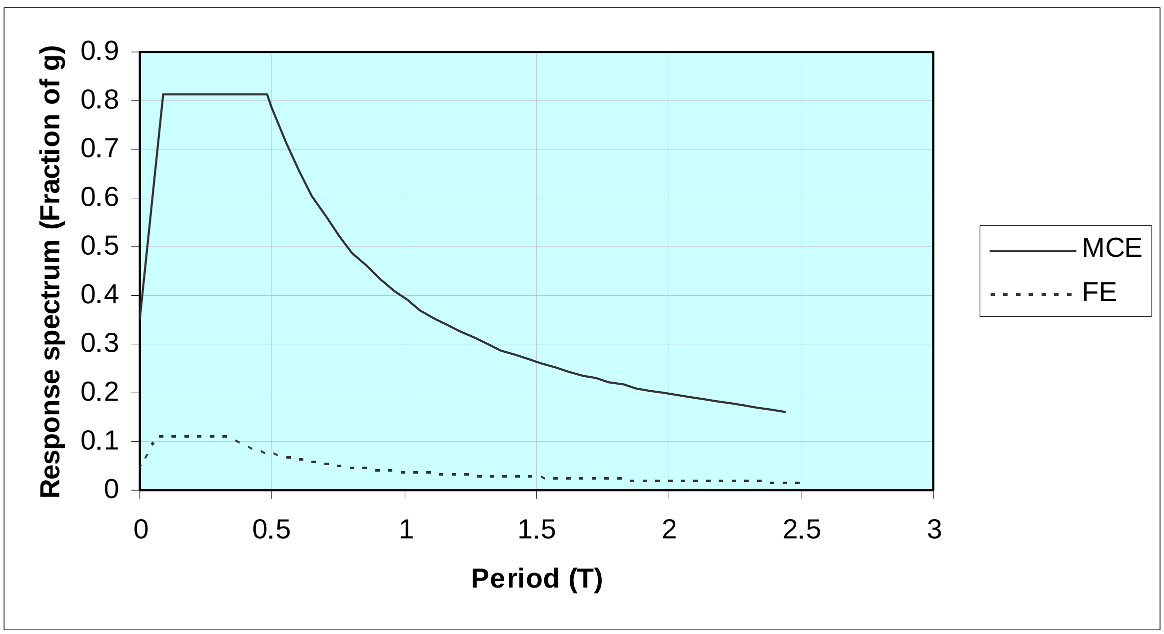
$T_{0\_fe} := 0.2 \cdot T_{S\_fe}$ Period at the beginning of construction design spectral acceleration plateau

$T_{0\_fe} = 0.066\ \mathrm{Sec}$

$F_{a\_fe} \cdot S_{s\_fe} = 0.108$ $\qquad F_{a\_fe} \cdot S_{s\_fe} \le 0.15$

$F_{v\_fe} \cdot S_{1\_fe} = 0.036$ $\qquad F_{v\_fe} \cdot S_{1\_fe} \le 0.15$

Seismic Hazard Level I (Guide Spec. Table 3.7-1)



Graph 1. response Spectrum (MCE and FE)

Response Modification Factor for MCE:

$T_S = 0.484$

$T := 0.7720$ Period of vibration of Bridge

$R_B := 1.5$ Base Response Modification Factor for substructure [MCEER Giude Spec. (Table 4.7.1)]

$R_{col} := 1 + (R_B - 1)\frac{T}{1.25 \cdot T_S}$ Response Modification Factor for substructure [MCEER Giude Spec.(4.7.1)]

$R_{col} = 1.638 < R_B$

So $R_{col} := 1.5$

Response Modification Factor for FE:

$T_S := 0.329$

$T := 0.62169$ Period of vibration of Bridge

$R_B := 0.9$ Base Response Modification Factor for substructure [MCEER Giude Spec. (Table 4.7.1)]

$R_{col} := 1 + (R_B - 1)\frac{T}{1.25 \cdot T_S}$ Response Modification Factor for substructure [MCEER Giude Spec.(4.7.1)]

$R_{col} = 0.849 < R_B$ ok

So $R_{col} := 0.849$

SAP 2000 model:

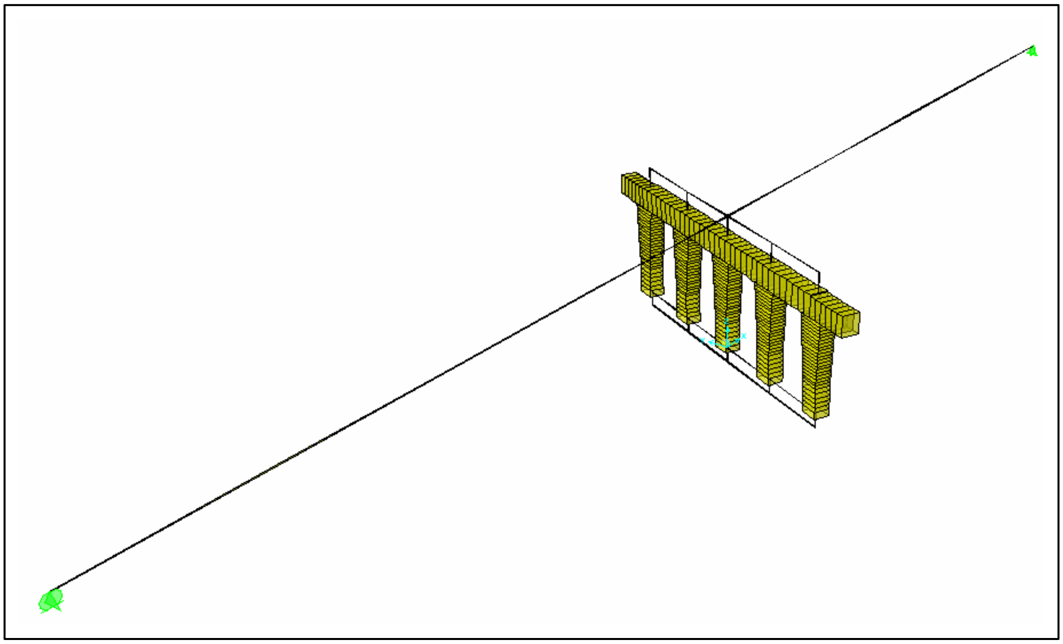


Fig.2.SAP modeling of Saint Clair Bridge with section extrution.

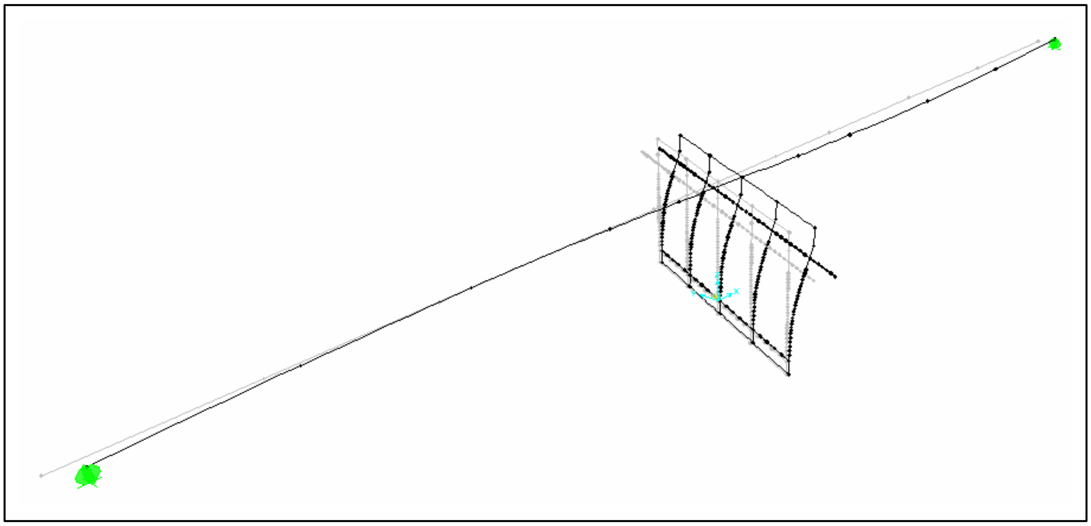


Fig.3.First Mode shape, longitudinal direction (T=0.77220 Sec)

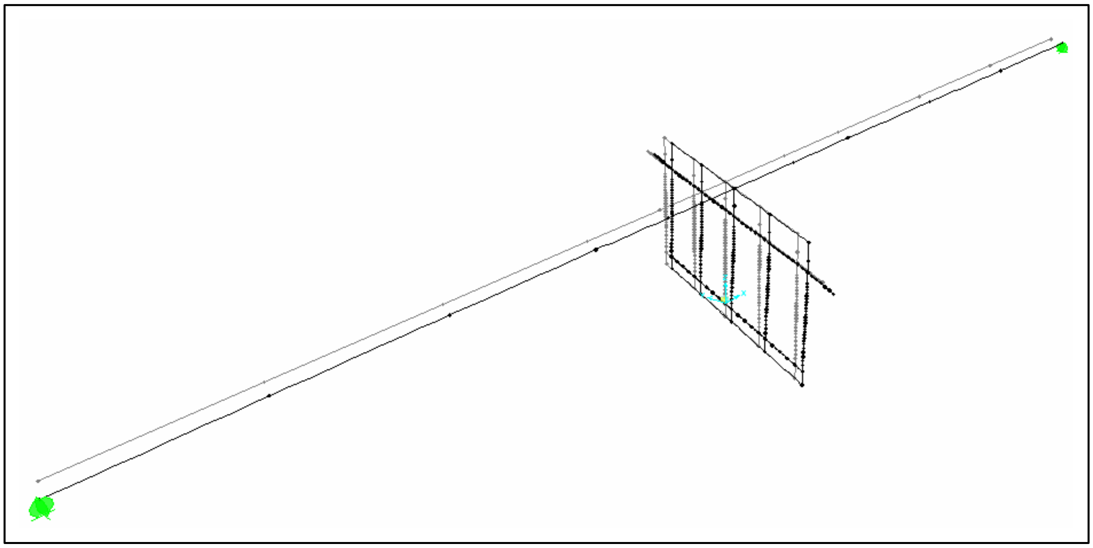


Fig.4.Third Mode shape, Transvers direction (T=0.55756 sec)

TABLE: Modal Participating Mass Ratios									
OutputCase	**StepType**	**StepNum**	**Period**	**UX**	**UY**	**UZ**	**UX**	**UY**	**UZ**
Text	Text	Unitless	Sec	Unitless	Unitless	Unitless	Percentage	Unitless	Unitless
EIGENMODES	Mode	1	0.772195	0.4289	0.1159	1.62E-18	42.89	11.59	0.00
EIGENMODES	Mode	2	0.596676	0.017	0.0066	3.524E-19	1.70	0.66	0.00
EIGENMODES	Mode	3	0.557558	0.1943	0.7253	9.831E-17	19.43	72.53	0.00
EIGENMODES	Mode	4	0.360047	3.623E-17	2.49E-16	0.3208	0.00	0.00	32.08
EIGENMODES	Mode	5	0.233051	0.000544	0.1019	3.783E-18	0.05	10.19	0.00
EIGENMODES	Mode	6	0.22987	0.3592	0.0503	4.816E-19	35.92	5.03	0.00
EIGENMODES	Mode	7	0.198011	2.469E-16	9.885E-17	1.47E-16	0.00	0.00	0.00
EIGENMODES	Mode	8	0.152639	7.672E-20	4.402E-20	6.566E-19	0.00	0.00	0.00
EIGENMODES	Mode	9	0.148196	0.00002595	0.000006588	3.132E-16	0.00	0.00	0.00
EIGENMODES	Mode	10	0.118068	1.01E-17	1.422E-17	0.0612	0.00	0.00	6.12
EIGENMODES	Mode	11	0.1008	0.000000403	0.00004337	7.727E-15	0.00	0.00	0.00
EIGENMODES	Mode	12	0.073461	1.26E-15	6.781E-16	0.5236	0.00	0.00	52.36
EIGENMODES	Mode	13	0.070734	1.663E-07	3.788E-08	5.893E-13	0.00	0.00	0.00
EIGENMODES	Mode	14	0.0583	1.18E-16	2.771E-17	0.0927	0.00	0.00	9.27
EIGENMODES	Mode	15	0.050561	3.263E-16	1.444E-16	0.00002029	0.00	0.00	0.00
EIGENMODES	Mode	16	0.048321	4.589E-12	1.788E-08	2.25E-12	0.00	0.00	0.00
EIGENMODES	Mode	17	0.030654	0.000000766	1.412E-07	3.277E-11	0.00	0.00	0.00
EIGENMODES	Mode	18	0.028907	1.235E-09	1.008E-09	8.313E-14	0.00	0.00	0.00
EIGENMODES	Mode	19	0.027706	9.583E-16	7.176E-16	6.679E-13	0.00	0.00	0.00
EIGENMODES	Mode	20	0.025344	1.781E-17	4.69E-16	0.0016	0.00	0.00	0.16
							100.00	100.01	99.99

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 $.9\cdot\phi\cdot F_y\cdot A_g$

Tension $.9\cdot\phi\cdot F_y\cdot A_n$

Compressive resistance
AASHTO LRFD, 6.9.2.1&2

$\phi_c := 0.9$ AASHTO LRFD 6.5.4.2

Use HP 14X117 for the NCHRP design
A=34.4 in^2
Ixx=1220 in^4 Zxx=194 in^3
Iyy=443 in^4 Zyy=91.4 in^3
Depth=14.2 in
Width=14.9 in

$E := \frac{29000000\cdot 144}{1000}$ ksf

$N := 105$ 21X5 (75" in long and 75" spacing in transverse direction)

$K_{long\_single} := 45\cdot\frac{12}{.6}$

$K_{long\_single} = 900 \quad \frac{k}{ft}$

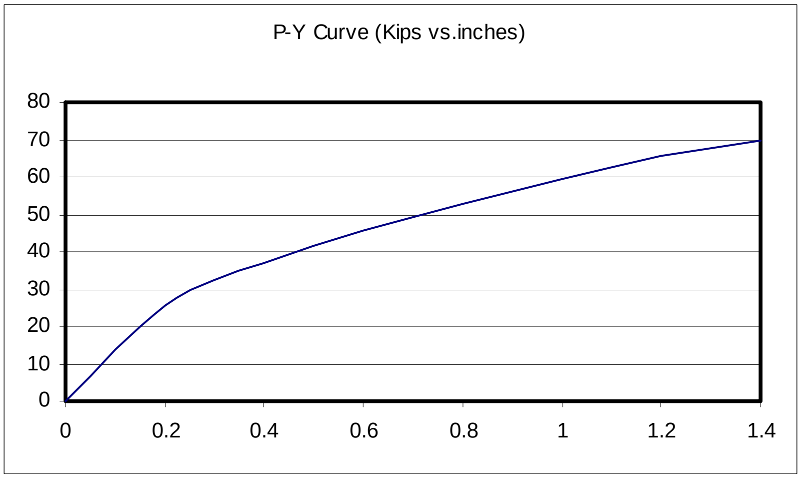
$K_x := K_{long\_single}\cdot N$

$K_x = 9.45\times 10^4 \quad \frac{k}{ft}$

$L := 80.417$ ft

$K_{tran\_single} := 51\cdot\frac{12}{1.4}$

$K_{tran\_single} = 437.143$



$K_y := K_{tran\_single} \cdot N$

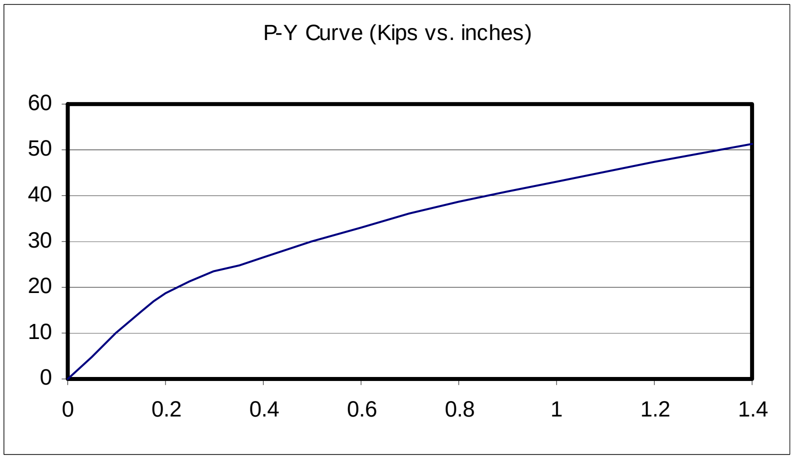
$K_y = 4.59 \times 10^4 \quad \frac{k}{ft}$

$A := \frac{34.4}{144}$

$A = 0.239 \quad ft^2$

$K_{axial} := A \cdot \frac{E}{L}$

$K_{axial} = 1.241 \times 10^4 \quad \frac{k}{ft}$



$K_z := K_{axial} \cdot N$

$K_z = 1.303 \times 10^6 \quad \frac{k}{ft}$

$$S_1 := \frac{10}{144} \cdot \left[(1 \cdot 75)^2 + (2 \cdot 75)^2 + (3 \cdot 75)^2 + (4 \cdot 75)^2 + (5 \cdot 75)^2 + (6 \cdot 75)^2 + (7 \cdot 75)^2 + (8 \cdot 75)^2 + (9 \cdot 75)^2 + (10 \cdot 75)^2\right]$$

$K_{rx} := K_{axial} \cdot S_1 \qquad S_2 := \frac{2}{144} \cdot \frac{N}{5} \cdot \left(1 \cdot 75^2 + 2 \cdot 75^2\right)$

$K_{ry} := K_{axial} \cdot S_2$

$$K_{rz} := \left[\left(\frac{K_{rx}}{K_{axial}}\right) + \left(\frac{K_{ry}}{K_{axial}}\right)\right] \cdot \frac{\left(K_{long\_single} + K_{tran\_single}\right)}{2}$$

$K_x = 9.45 \times 10^4$

$K_y = 4.59 \times 10^4$

$K_z = 1.303 \times 10^6$

$K_{rx} = 1.866 \times 10^{9}$

$K_{ry} = 6.106 \times 10^{7}$

$K_{rz} = 1.038 \times 10^{8}$

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

$$P_1 := \frac{V}{N} + M_{long} \cdot \frac{2 \cdot 75}{12 \cdot S_2} + M_{tran} \cdot \frac{10 \cdot 75}{12 \cdot S_1}$$

$P_1 = 211.246$ kips

$P_{al} := A \cdot 9 \cdot 144$

$P_{al} = 309.6$ kips Almost equal to P1 say OK

$Dis_L := .5$ in $\qquad M_1 := 2830$ k − in

$Dis_T := 1.18$ in $\qquad M_2 := 2950$ k − in

EQ in transverse direction

$M_{long} := 43800$ k − ft

$M_{tran} := 107000$ k − ft

$$P_2 := \frac{V}{N} + M_{long} \cdot \frac{2 \cdot 75}{12 S_2} + M_{tran} \cdot \frac{10 \cdot 75}{12 S_1}$$

$P_2 = 187.087$ kips

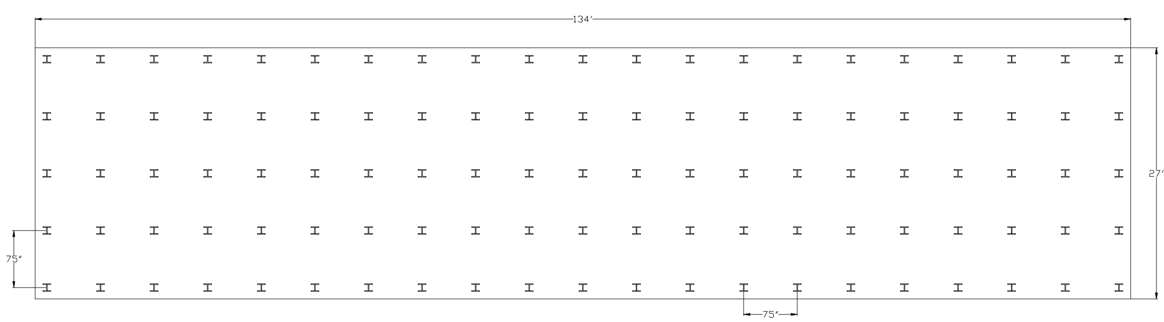


Fig 5. Piles Arrangement

$Dis_L := .39$ in

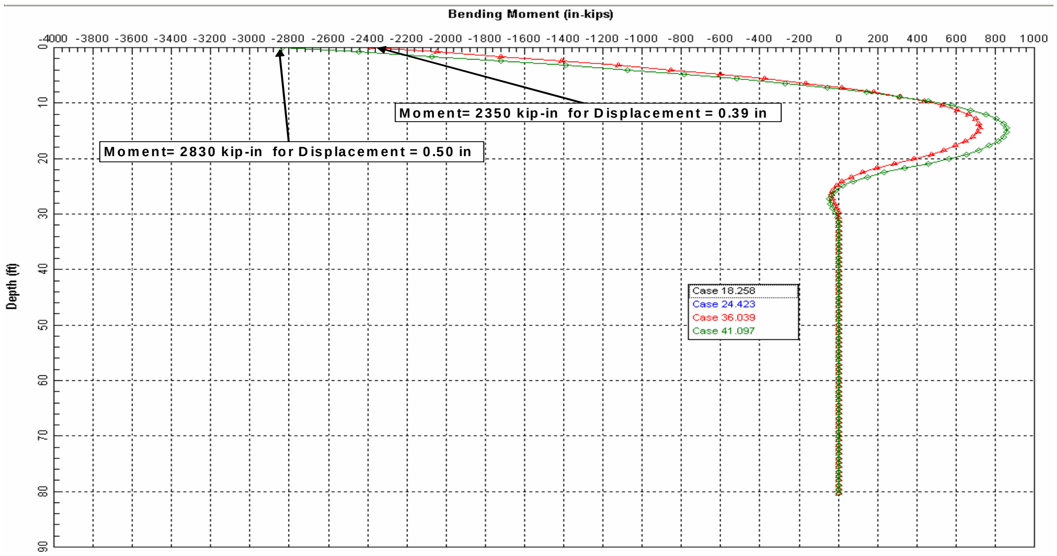
$M_3 := 2350$ k − in

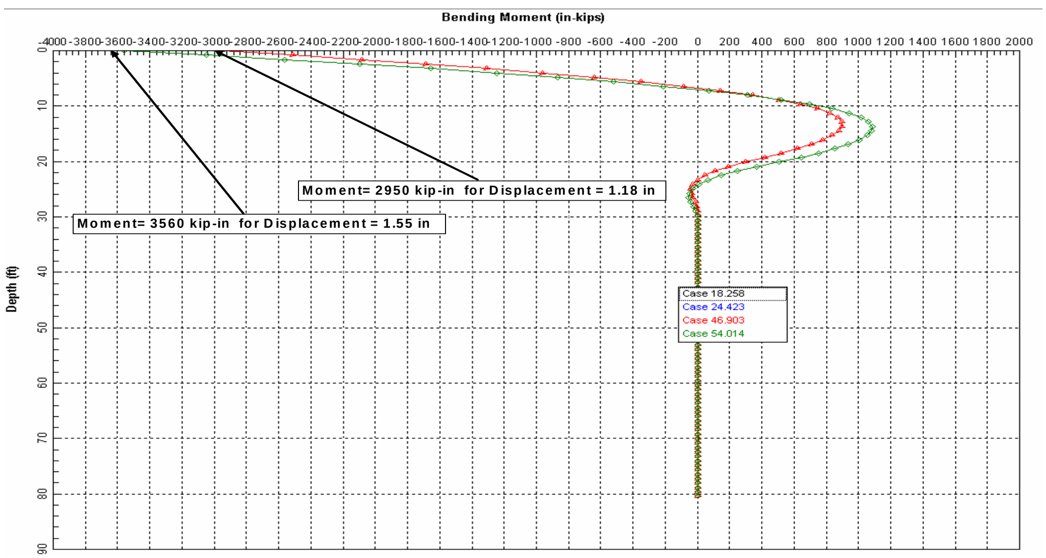
$Dis_T := 1.55$ in

$M_4 := 3560$ k − in

$$\frac{P_1}{0.85 \cdot 50 \cdot 34.4} + \frac{8}{9} \cdot \left(\frac{M_1}{50 \cdot 194} + \frac{M_2}{50 \cdot 91.4} \right) = 0.978$$

$$\frac{P_2}{0.85 \cdot 50 \cdot 34.4} + \frac{8}{9} \cdot \left(\frac{M_3}{50 \cdot 194} + \frac{M_4}{50 \cdot 91.4} \right) = 1.036$$





Forces and Moments from SAP2000 (MCE):

NCHRP		Forces & Moments-Trans(Y) EQ 100%Trans+40%Long				
Location		Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Exterior Column	top	347	609	5053	1494	833
	bottom	350	628	2572	5788	833
Middle Column	top	347	790	7339	1496	416
	bottom	350	810	2521	5790	416
Central Column	top	347	811	7605	1496	0
	bottom	350	830	2002	5574	0

NCHRP		Forces & Moments-Long(X) EQ 100%Long+40%Trans				
Location		Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Exterior Column	top	456	461	3831	1967	628
	bottom	460	476	1949	7624	628
Middle Column	top	456	599	5564	1971	314
	bottom	460	614	1911	7626	314
Central Column	top	456	615	5766	1971	0
	bottom	460	630	1517	7342	0

Final Forces and Moments due to MCE and DL by appliying Reduction Factor of 1.5 for designing columns:

NCHRP - MCE		Final Design Forces and Moments (EQ+DL) By R=1.5									
		Transvers					Longitudinal				
Location		Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips	Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Exterior Column	top	231	406	3369	996	1072	304	307	2554	1311	936
	bottom	233	419	1715	3859	1100	307	317	1299	5083	964
Middle Column	top	231	527	4893	997	794	304	399	3709	1314	726
	bottom	233	540	1681	3860	822	307	409	1274	5084	754
Central Column	top	231	541	5070	997	517	304	410	3844	1314	517
	bottom	233	553	1335	3716	545	307	420	1011	4895	545

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SUBJECT FILE:

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

BY: NA

CHECKED: AAA

DATE: 6/22/2004

TITLE:
Saint Clair County Bridge

NCHRP Guidelines

Sheet No: **20** of __

Forces and Moments from SAP2000 (FE):

NCHRP - FE		Forces & Moments-Trans(Y) EQ 100%Trans+40%Long				
Location		Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Exterior Column	top	16	32	273	65	43
	bottom	16	34	137	143	43
Middle Column	top	16	41	395	65	22
	bottom	16	43	135	263	22
Central Column	top	16	43	409	65	0
	bottom	16	45	134	263	0

NCHRP - FE		Forces & Moments-Long(X) EQ 100%Long+40%Trans				
Location		Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Exterior Column	top	31	37	312	131	48
	bottom	31	38	159	216	48
Middle Column	top	31	48	455	131	25
	bottom	31	49	156	516	25
Central Column	top	31	50	470	131	0
	bottom	31	51	155	516	0

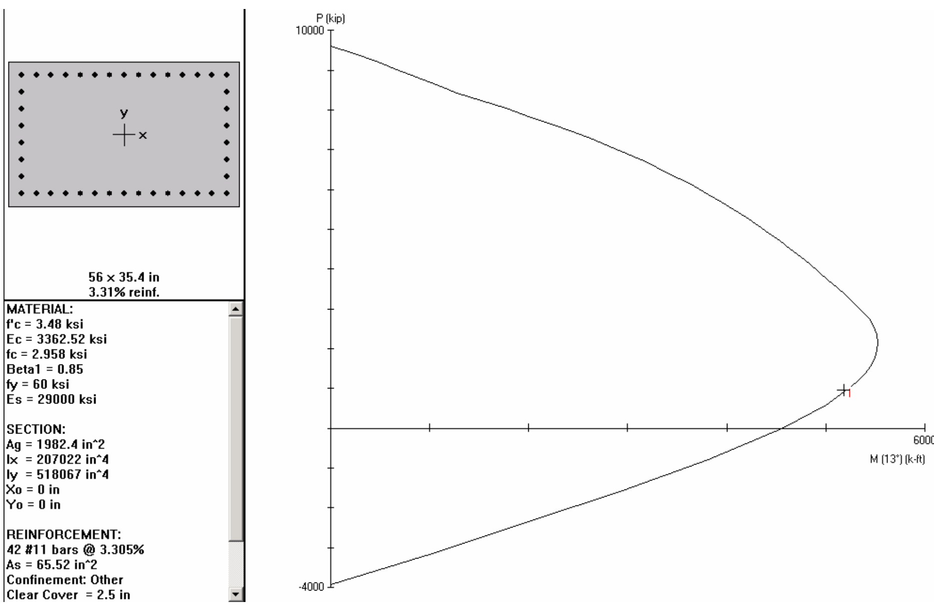
Final Forces and Moments due to FE and DL by appliying Reduction Factor of 0.849 for designing columns:

NCHRP - FE		Final Design Forces and Moments (EQ+DL) By R=0.849									
		Transverse					Longitudinal				
Location		Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips	Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Exterior Column	top	18	37	321	77	567	36	44	368	154	574
	bottom	18	40	161	168	595	36	45	187	254	602
Middle Column	top	18	49	465	77	542	36	57	535	154	546
	bottom	18	51	159	309	570	36	58	184	608	574
Central Column	top	18	50	482	77	517	36	59	554	154	517
	bottom	18	53	158	309	545	36	60	183	608	545

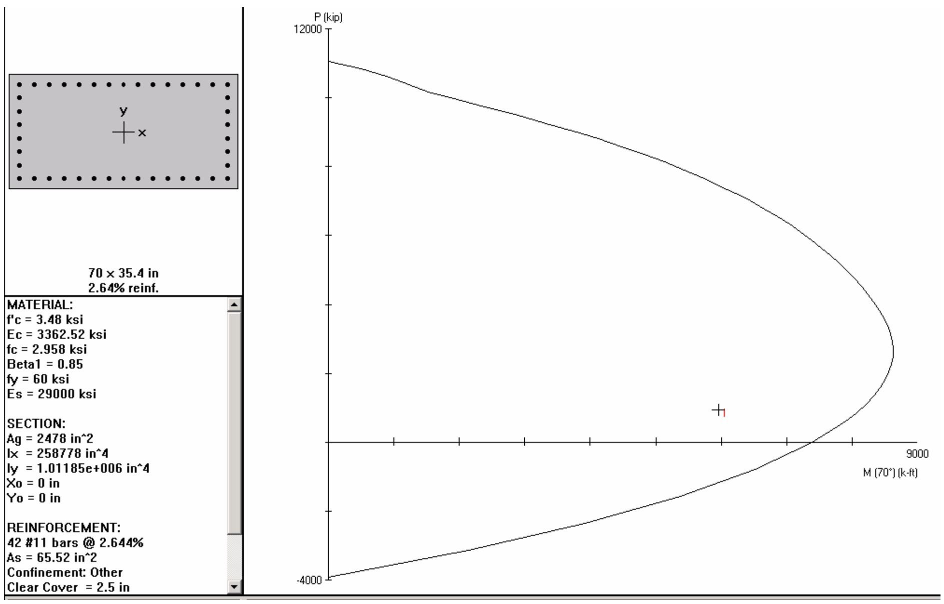
Design of Columns using PCACOL:

- 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
- ϕ 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:



Top Section:



Transverse Reinforcement In coloumns:

Method 1: Implicit Shear Detailing Approach [Guide Spec, Article 8.8.2.3]

(56 X 35.4 in ratio=3.31%)

$K_{shape} := 0.375$ Rectangular Section

$\Lambda := 1$ Fixity Factor

$\rho_t := 0.0331$ Longitudinal Steel Content

$\phi := 0.9$ Strength Reduction

$f_{yh} := 60000$ psi Yeild Strength of Ties

$f_{su} := 1.5 \cdot f_{yh}$ $\quad f_{su} = 9 \times 10^{4}$ psi

$L := 147.6$ in $\quad$ Column Height

$D_p := 35.4 - 2 \cdot \left(2 + \frac{1.41}{2}\right)$ in $\quad$ The distance b/w the outer layers of the longitudinal steel.

$d := 56 - 2$

$\alpha := \frac{D_p}{L}$ $\quad \alpha = 0.203$ $\quad$ Geometric aspect ratio

$\tan(\alpha \cdot deg) = 3.546 \times 10^{-3}$

$A_g := 35.4 \cdot 56$ $\quad$ Gross Section Area

$A_g = 1.982 \times 10^{3}$ in^2

$b_w := 35.4$

$A_v := b_w \cdot d$ $\quad$ Shear Area of Concrete

$A_v = 1.912 \times 10^{3}$ in^2

$b := 35.4$ in $\quad$ Width of column section

$h := 56$ in $\quad$ Hight of column section

Try $\quad s := \min\left(10, \frac{\min(b,h)}{2}\right)$ $\quad s = 10$ in

$A_{sh} := 2 \cdot 0.31$ in^2 $\quad$ No 5

$\rho_V := \dfrac{A_{sh}}{b_w \cdot s}$ Ratio of Transvers Reinforcement (8.8.2.3-2)

$\rho_V = 1.751 \times 10^{-3}$

$\tan\theta := \left(\dfrac{1.6 \cdot \rho_V \cdot A_v}{\Lambda \cdot \rho_t \cdot A_g}\right)^{0.25}$ $\tan\theta = 0.535$

$\rho_v := K_{shape} \cdot \Lambda \cdot \dfrac{\rho_t}{\phi} \cdot \dfrac{f_{su}}{f_{yh}} \cdot \dfrac{A_g}{A_v} \cdot \tan(\alpha \cdot deg) \cdot \tan\theta$ Ratio of Transvers Reinforcement (8.8.2.3-1)

$A_{sh} := b_w \cdot s \cdot \rho_v$ $\rho_v = 4.067 \times 10^{-5}$

$A_{sh} = 0.014$

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.

$f_c := 3.48 \cdot 10^3$ psi Compressive Strength of Concrete

$V_c := 2 \cdot \sqrt{f_c}$ Shear Strength of Concrete

$V_c = 117.983$ ksi

$\rho := \rho_v - 0.17 \cdot \dfrac{\sqrt{f_c}}{f_{yh}}$ Ratio of tensverse 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 zoneonly. 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]

$L := 147.6$ in — Column Height

$A_b := 1.56$ in^2 — Area of rebars for Long reinforcement

Aproximate the overstrength effects simply as 1.5 times the forces from the verification.

$P_d := 538$ kips — Axial load

$OS := 1.5$ — Overstrength Factor

$P_e := 750$ kips — Axial load due to EQ

$P_e := P_d + OS \cdot (P_e - P_d)$ — Axial load due to EQ

$P_e = 856$ kips

$M_{p\_top} := 5070$ kip − ft — Moment at the bottom of column

$M_{p\_bot} := 5083$ kip − ft — Moment at the top of column

$M_{p\_top} := OS \cdot M_{p\_top}$ — $M_{p\_top} = 7.605 \times 10^3$ kip − ft

$M_{p\_bot} := OS \cdot M_{p\_bot}$ — $M_{p\_bot} = 7.625 \times 10^3$ kip − ft

$L := \frac{L}{12}$ $L = 12.3$ ft Hieght of column

$V_u := \frac{\left(M_{p\_bot} + M_{p\_top}\right)}{L}$ Ultimate shear strength

$V_u = 1.238 \times 10^3$ kip

$V_c := 0.6 \cdot \sqrt{f_c} \cdot \frac{A_v}{1000}$ Shear strength of Concrete

$V_c = 67.661$ kip

$V_p := \frac{\Lambda \cdot P_e \cdot \tan(\alpha \cdot deg)}{2}$ The contribution due to arch action (8.8.2.3-7)

$V_p = 1.518$ kip

$\phi := 0.9$

$V_s := \frac{V_u}{\phi} - V_c - V_p$ Shear resistance provided by transvers reinfocement

$V_s = 1.307 \times 10^3$ kip

Guess $s := 10$ in $A_{sh} := 0.31 \cdot 2$ 2 legs No. 5

$A_{sh} = 0.62$ in^2

$\rho_v := \frac{A_{sh}}{s \cdot b_w}$ Ratio of Transvers Reinforcement (8.8.2.3-2)

$\rho_v = 1.751 \times 10^{-3}$

$\tan\theta := \left(\frac{1.6 \cdot \rho_v \cdot A_v}{\Lambda \cdot \rho_t \cdot A_g}\right)^{0.25}$ (8.8.2.3-4)

$\tan\theta = 0.535$

Because $\tan\theta$ is greater than $\tan\alpha$, use $\tan\theta$ to find Ash:

$$A_{vs} := s \cdot \frac{V_s}{f_{yh} \cdot b_w} \cdot \tan\theta \qquad (8.8.2.3\text{-}17)$$

$$A_{vs} = 3.288 \times 10^{-3}$$

Use #5 @ 10 in.

Transvers Reinforcement for confinement at plastic hinge Pmax column [Guide Spec, Article 8.8.2.4]

$f_y := 60$ ksi — yeild strength of reinforcing bars

$U_{sf} := 15.95$ ksi — Strain energy capacity (module of toughness) of the transverese reinforcement

$A_{sh} := .31 \cdot 2$ — 2 legs No . 5

$h := 54$ in — Height of section

$A_c := (b - 5) \cdot (h - 5)$ — Area of column core concrete

$A_c = 1.49 \times 10^3$ in^2

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

$\rho_s = 0.021$ — The volumetric ratio

$legs := 4$

$s := \dfrac{legs \cdot A_{sh}}{\rho_s \cdot b}$ (8.8.2.3-2)

$s = 3.395$ s max=3 in OK

No.5 @3 in for confinment in the end region of the Pmax column.

Method 2: Explicit Shear detailing approach- Pmin Column [Guide Spec, Article 8.8.2.3]

$L := 147.6$ Column Hieght

$A_b := 1.56$ in^2 Area of rebars for Long reinforcement

$P_d := 517$ kips Axial load

$P_e := 314$ kips Axial load due to EQ (Lower Compression)

$P_e := P_d + OS \cdot (P_e - P_d)$ Axial load due to EQ

$P_e = 212.5$ kips

$M_{p\_top} := 341$ kip − ft Moment at the bottom of column

$M_{p\_bot} := 853$ kip − ft Moment at the top of column

$OS := 1.5$

$M_{p\_top} := OS \cdot M_{p\_top}$ $M_{p\_top} = 511.5$ kip − ft

$M_{p\_bot} := OS \cdot M_{p\_bot}$ $M_{p\_bot} = 1.28 \times 10^3$ kip − ft

$L := \dfrac{L}{12}$ $L = 12.3$ ft

$$V_u := \frac{(M_{p\_bot} + M_{p\_top})}{L}$$ Ultimate shear strength

$V_u = 145.61$ kip

$$V_c := 0.6 \cdot \sqrt{f_c} \cdot \frac{A_v}{1000}$$ Shear strength of Concrete

$V_c = 67.661$ kip

$$V_p := \frac{\Lambda \cdot P_e \cdot \tan(\alpha \cdot deg)}{2}$$ The contribution due to arch action (8.8.2.3-7)

$V_p = 0.377$ kip

$\phi := 1.0$

$$V_s := \frac{V_u}{\phi} - V_c - V_p$$ Shear resistance provided by transvers reinfocement

$V_s = 77.572$ kip

Guess $s := 10$ in $A_{sh} := 0.11 \cdot 2$ No 3

$$\rho_v := \frac{2 \cdot A_{sh}}{s \cdot b_w}$$

$\rho_v = 1.243 \times 10^{-3}$

$$\tan\theta := \left(\frac{1.6 \cdot \rho_v \cdot A_v}{\Lambda \cdot \rho_t \cdot A_g}\right)^{0.25}$$ (8.8.2.3-4)

$\tan\theta = 0.491$

Because $\tan\theta$ is greater than $\tan\alpha$, use $\tan\theta$ to find Ash:

$$A_{vs} := s \cdot \frac{V_s}{f_{yh} \cdot b_w} \cdot \tan\theta \qquad (8.8.2.3\text{-}17)$$

$$A_{vs} = 1.792 \times 10^{-4}$$

Use 2#3 @ 10 in. it is more than required

Transvers reinforcement for confinment at plastic hinges Pmin Column [Guide Spec, Article 8.8.2.4]:

$f_y := 60$ ksi — yeild strength of reinforcing bars

$U_{sf} := 15.95$ ksi — Strain energy capacity (module of toughness) of the transverese reinforcement

$A_{sh} := 0.31 \cdot 2$ No . 5

$h := 56$ in — Height of section

$A_c := (b - 5) \cdot (h - 5)$ — Area of column core concrete

$A_c = 1.55 \times 10^3$ in^2

$\rho_t := 0.0331$ — The volumetric ratio

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

$$\rho_s = 0.014$$

legs := 3

$$s := \frac{legs \cdot A_{sh}}{\rho_s \cdot b_w} \qquad (8.8.2.3\text{-}2)$$

$s = 3.824$ s max=4 in

3 No.5 @4 in for confinment in the end region of the Pmin column.

Anti-Buckling steel [Guide Spec, Article 8.8.2.5]:

$d_b := 1.41$ in bar diameter for No. 11

$s_{new} := 6 \cdot d_b$

$s_{new} = 8.46$ in Required spacing of anti-buckling

$s := 8.46$ in

$A_b := 1.56 \cdot 42$

$$A_{bh} := 0.09 \cdot A_b \cdot \frac{f_y}{\frac{f_{yh}}{1000}} \qquad (8.8.2.5\text{-}3)$$

$A_{bh} = 5.897$

$legs\_antibuckling := 8$ Legs required for anti buckling

$$\frac{A_{bh}}{legs\_antibuckling \cdot 2} = 0.369$$

$S := 4$ in

Use No.6 @ 4 in for anti buckling

Extend of Shear Steel, Confinement and Anti-buckling Steel [Guide Spec, Article 8.8.2.6 and 4.9]:

$D := \frac{h-4}{12}$ $\quad D = 4.333$ ft — Maximum cross-sectional dimantion of column

$M_u := 5058$ kip − ft — Maximum Column Moment

$M_{p\_bot} := 8418$ kip − ft — Column plastic overstrength moment

$V_u = 145.61$ kip — Maximum column shear

$M_y := \frac{0.85 \cdot M_{p\_bot}}{1.5}$ kip − ft — Column yield moment

$d_b := 1.41$ in — Longitudinal bar diameter

$\varepsilon_y := 0.00207$ in — Yield strain of the longitudinal Reinforcement

$$L1 := D \cdot \left(\frac{1}{\tan\theta} + 0.5 \cdot \tan\theta \right) \qquad L1 = 9.896 \text{ ft}$$

$$L2 := 1.5 \cdot \left(0.08 \cdot \frac{M_u}{V_u} + 4400 \cdot \varepsilon_y \cdot \frac{d_b}{12} \right) \qquad L2 = 5.774 \text{ ft}$$

$$L3 := \frac{M_u}{V_u} \cdot \left(1 - \frac{M_y}{M_{p\_bot}} \right) \qquad L3 = 15.053 \text{ ft}$$

$$L := \max(L1, L2, L3)$$

$L = 15.053$ ft — Controls

Plastic zone length= 15 ft

Connections:

a) Connection at the top of columns to Cap beam:

(70 X 35.4 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 Reinforement per Guide Spec 8.8.2.3.

$B1 := 35.4 - 5 - 0.625 \quad B1 = 29.775$ in

$D1 := 70 - 5 - 0.625 \quad D1 = 64.375$ in

$B2 := 35.4 - 5 - 2\cdot 0.625 \quad B2 = 29.15$ in

$D2 := 70 - 5 - 2\cdot 0.625 \quad D2 = 63.75$ in

$\phi := 0.90$ — Strength Reduction Factor for shear

$f_{yh} := 60000$ psi — Yeild Strength of Ties

$f_{su} := 1.5\cdot f_{yh} \quad f_{su} = 9\times 10^{4}$ psi

$\rho_t := 0.0264$ — Longitudinal Steel Content

$A_g := 70\cdot 35.4 \quad A_g = 2.478\times 10^{3}$ in^2 — Cross-Sectional Area of Columns

$A_c := B1\cdot D1 \quad A_c = 1.917\times 10^{3}$ in^2 — Cross-Sectional Area of Column core

$D := 35.4$ in — Width of the column framing into the joint

$H_c := 43.3$ in — Height of the cap beam

$\tan\alpha := \frac{D}{H_c}$

$$\rho := 1.2 \cdot \frac{\frac{B1}{D1} + 0.5 \cdot \rho_t \cdot f_{su} \cdot A_g}{2 \cdot \frac{B1}{D1} + 2 \cdot \phi \cdot f_{yh} \cdot A_c} \cdot \left(\frac{D}{H_c}\right)^2$$

For a rectangular column, minimum ratio

$\rho = 0.011$

$A_{sh} := 2 \cdot .31$ 6 legs

$s := \frac{A_{sh}}{\rho \cdot B2}$ $s = 1.865$ maximum

No.5 @ 2 in

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:

$f_h := 0$ ksi Average axial stress in the horizontal direction

$P_{max} := 943$ kip At mid-depth of joint

$M_p := 5330$ kip − ft At mid-depth of joint

$b_b := 39.37$ in Width of the Cap Beam

$H_c := 43.3$ in Hight of the joint

$L_{mid\_depth\_jt} := D + H_c$

$A_{mid\_depth\_jt} := b_b \cdot L_{mid\_depth\_jt}$

$L_{mid\_depth\_jt} = 78.7$

$A_{mid\_depth\_jt} = 3.098 \times 10^3$ in^2

$$f_v := \frac{P_{max}}{A_{mid\_depth\_jt}}$$ Average axial stress in the horizontal direction

$f_v = 0.304$ ksi

$h_b := 43.3$ in — Cap beam Depth

$b_{je} := b_b$ — The effective joint width

$h_c := 70$ — The column lateral dimention

$$v_{hv} := \frac{M_p \cdot 12}{h_b \cdot h_c \cdot b_{je}}$$ The average shear stress within the plane of the connection.

$v_{hv} = 0.536$ ksi

$f_c := 3.48$ 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 tention stress (8.8.4.2-1).

$p_t = -0.405$ ksi

$$P_{t\_max} := 3.5 \cdot \sqrt{\frac{f_c}{1000}}$$ Maximum tention stress

$P_{t\_max} = 0.206$ ksi $<$ p_t

Therfore 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).

$p_c = 0.709$ ksi

$p_{c\_max} := 0.25 \cdot f_c$ — Maximum Compression stress [Guide spec, Article 8.8.4.2.3]

$p_{c\_max} = 0.87$ ksi > p_c OK

Case 2:

$P_{min} := 341$ kip

$M_p := 3216$ kip − ft

Obviously it will produce amaller principle srtesses than Case1.

Case 3:

$P_{DL} := 570$ kip — Axial Load due to Dead Load

$M_p := 4258$ kip − 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 $N := 42$

$A_{bar} := 1.56$ in^2 — Area of one longitudinal reinforcement

$A_{st} := N \cdot A_{bar}$ — Total Area pf Longitudinal steel

$A_{st} = 65.52$ in^2

$A_{jv} := 0.16 \cdot A_{st}$ — Area of the steel located within the distance 0.5D or 0.5h from the column or pier wall face.

$A_{jv} = 10.483$ in^2

$A_{stirrup\_leg} := 2 \cdot 0.31$ in^2 — Using 2 legs No. 5 Stirrups

$$\frac{A_{jv}}{A_{stirrup\_leg}} = 16.908$$

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)

$$A_h := 0.08 \cdot A_{st}$$

$f_y := 60$ ksi

$A_{bar} := 1.0$ in^2

Using No.9 bars,

$f_c := 3.48$ ksi

$$\frac{A_h}{A_{bar}} = 5.242$$

$\boxed{\text{6 \# 9 in bottom cap beam}}$

$d_b := 1.128$ in for No. 9

$$L_d := 1.25 \cdot A_{bar} \cdot \frac{f_y}{\sqrt{f_c}}$$

No more than

$$Ld := 0.4 \cdot f_y \cdot d_b$$

$$Ld = 27.072$$

$$L_d = 40.204$$

Hoop Reinforcement Guide Spec (8.8.4.3.4)

$f_y := 60$ ksi

$f_c := 3.48$ ksi

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

Cross-Section Area of bar #11

$$l_{ac} := 1.25 \cdot A_{bar} \cdot \frac{f_y}{\sqrt{f_c}}$$

Development length of # 11 per AASHTO LRFD 5.11.2.1

$l_{ac} = 62.719$ in

$\rho_s := 0.4 \cdot \frac{A_{st}}{l_{ac}^{2}}$ Guide Spec (8.8.4.3-1)

$\rho_s = 6.663 \times 10^{-3}$

$A_{hoop} := 0.31 \cdot 2 \quad in^2$ 2 legs No.5 for hoops

$s := 4 \cdot \frac{A_{hoop}}{\rho_s \cdot D2}$

$s = 5.839$

2 No. 5 @ 5 3/4 in pitch required in the joint

b) Connection at the bottom of columns to Footing:

(56 X 35.4 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 Reinforement per Guide Spec 8.8.2.3.

$B1 := 35.4 - 5 - 0.625 \quad B1 = 29.775 \quad in$

$D1 := 56 - 5 - 0.625 \quad D1 = 50.375 \quad in$

$B2 := 35.4 - 5 - 2 \cdot 0.625 \quad B2 = 29.15 \quad in$

$D2 := 56 - 5 - 2 \cdot 0.625 \qquad D2 = 49.75$ in

$\phi := 0.90$ — Strength Reduction Factor for shear

$f_{yh} := 60000$ psi — Yeild Strength of Ties

$f_{su} := 1.5 \cdot f_{yh} \qquad f_{su} = 9 \times 10^4$ psi

$\rho_t := 0.0331$ — Longitudinal Steel Content

$A_g := 56 \cdot 35.4 \qquad A_g = 1.982 \times 10^3$ in^2 — Cross-Sectional Area of Columns

$A_c := B2 \cdot D2 \qquad A_c = 1.45 \times 10^3$ in^2 — Cross-Sectional Area of Column core

$D := 35.4$ in — Width of the column framing into the joint

$H_c := 66.93$ in — Height of the Pile cap

$$\tan\alpha := \frac{D}{H_c}$$

$$\rho := 1.2 \cdot \frac{\frac{B1}{D1} + 0.5 \cdot \rho_t \cdot f_{su} \cdot A_g}{2 \cdot \frac{B1}{D1} + 2 \cdot \phi \cdot f_{yh} \cdot A_c} \cdot \left(\frac{D}{H_c}\right)^2$$

For a rectangular column, minimum ratio

$\rho = 6.329 \times 10^{-3}$

$A_{sh} := 2 \cdot .31$

$s := \frac{A_{sh}}{\rho \cdot B2} \qquad s = 3.361$ maximum

2 No.5 @ 3 1/4 in

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 overstrenght:

$f_h := 0 \quad ksi$ — Average axial stress in the horizontal direction

$P_{max} := 962 \quad kip$

At mid-depth of joint

$M_p := 5058 \quad kip - ft$

$b_b := 39.37 \quad in$ — Width of the pile Cap

$H_c := 66.93 \quad in$ — Hight of the joint

$L_{mid\_depth\_jt} := D + H_c$

$L_{mid\_depth\_jt} = 102.33 \quad in$

$A_{mid\_depth\_jt} := b_b \cdot L_{mid\_depth\_jt}$

$A_{mid\_depth\_jt} = 4.029 \times 10^3 \quad in^2$

$$f_v := \frac{P_{max}}{A_{mid\_depth\_jt}}$$

Average axial stress in the horizontal direction

$f_v = 0.239 \quad ksi$

$h_b := 43.3 \quad in$ — Pile Cap Depth

$b_{je} := b_b$ — The effective joint width

The column lateral dimention

$h_c := 56$

$$v_{hv} := \frac{M_p \cdot 12}{h_b \cdot h_c \cdot b_{je}}$$

The average shear stress within the plane of the connection.

$v_{hv} = 0.636$ ksi

$f_c := 3.48$ 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 tention stress (8.8.4.2-1).

$p_t = -0.528$ ksi

$$P_{t\_max} := 3.5 \cdot \sqrt{\frac{f_c}{1000}}$$

Maximum tention stress

$P_{t\_max} = 0.206$ 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).

$p_c = 0.766$ ksi

$p_{c\_max} := 0.25 \cdot f_c$ — Maximum Compression stress [Guide spec, Article 8.8.4.2.3]

$p_{c\_max} = 0.87$ ksi $>$ p_c OK

Case 2:

$P_{min} := 359$ kip

$M_p := 3778$ kip − ft

Obviously it will produce amaller principle srtesses than Case1.

Case 3:

$P_{DL} := 570$ kip — Axial Load due to Dead Load

$M_p := 853$ kip − 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 $\quad N := 42$

$A_{bar} := 1.56$ in^2 — Area of one longitudinal reinforcement

$A_{st} := N \cdot A_{bar}$ — Total Area pf Longitudinal steel

$A_{st} = 65.52$ in^2

$A_{jv} := 0.16 \cdot A_{st}$ — Area of the steel located within the distance 0.5D or 0.5h from the column or pier wall face.

$A_{jv} = 10.483$ in^2

$A_{stirrup\_leg} := 0.31 \cdot 2$ in^2 — Using No. 5 Stirrups

$\frac{A_{jv}}{A_{stirrup\_leg}} = 16.908$ — 9 # 5 legs requied

$A_{clamp} := 0.08 \cdot A_{st}$ — 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)

$A_h := 0.08 \cdot A_{st}$

$f_y := 60$ ksi

$A_{bar} := 1.0$ in^2

Using No.9 bars,

$f_c := 3.48$ ksi

$\frac{A_h}{A_{bar}} = 5.242$

6 # 9 in footing

$d_b := 1.128$ in for No. 9

$L_d := 1.25 \cdot A_{bar} \cdot \frac{f_y}{\sqrt{f_c}}$

No more than

$Ld := 0.4 \cdot f_y \cdot d_b$

$Ld = 27.072$

$L_d = 40.204$

Hoop Reinforcement Guide Spec (8.8.4.3.4)

$f_y := 60$ ksi

$f_c := 3.48$ ksi

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

Cross-Section Area of bar #11

$l_{ac} := 1.25 \cdot A_{bar} \cdot \frac{f_y}{\sqrt{f_c}}$

Development length of # 11 per AASHTO LRFD 5.11.2.1

$l_{ac} = 62.719$ in

$\rho_s := 0.4 \cdot \frac{A_{st}}{l_{ac}^2}$

Guide Spec (8.8.4.3-1)

$\rho_s = 6.663 \times 10^{-3}$

$A_{hoop} := 0.31 \cdot 2$ in^2

2 legs No.5 for hoops

$s := 4 \cdot \frac{A_{hoop}}{\rho_s \cdot D2}$

$s = 7.482$

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 Monent in Transvers direction

M2 := 29260 kip − ft Monent in Longitudinal direction

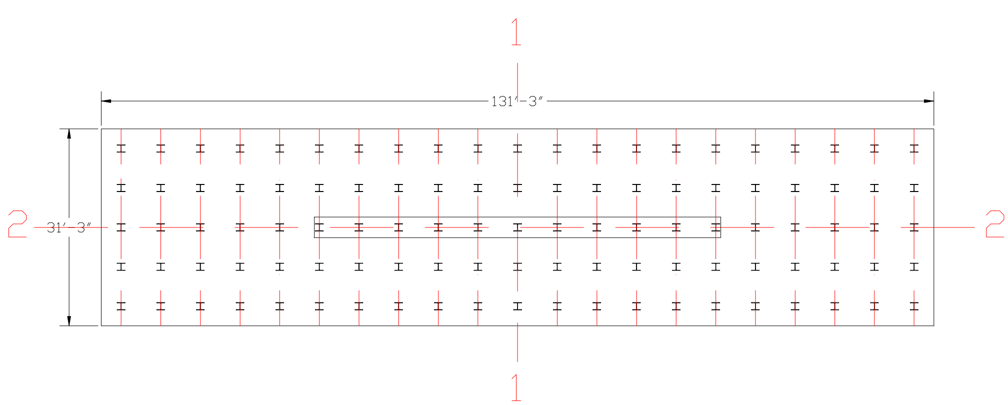
V := 2987 kip Monent in Transvers direction

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 — Distance between piles

$m1 := 5$ — Number of rows

$n1 := 21$ — Number of piles in the rows

$\Sigma d1 := \frac{s^2}{12} \cdot n1 \cdot \left(n1^2 - 1\right) \cdot m1$ — Sum of the squares of the distances to each pile from the center or gravity of piles

$\Sigma d1 = 1.504 \times 10^5$ pile ft^2

$m2 := 21$ — Number of rows

$n2 := 5$ — Number of piles in the rows

$\Sigma d2 := \frac{s^2}{12} \cdot n2 \cdot \left(n2^2 - 1\right) \cdot m2$

$\Sigma d2 = 8.203 \times 10^3$ pileft^2

$M1 := 25116$ kip − ft

$M2 := 29260$ kip − ft

$V := 2987$ kip

Piles Coordinates:

1st row

$x1 := 62.5$	$x4 := 43.75$	$x7 := 25$	$x10 := 6.25$	$x13 := -12.5$	$x16 := -31.25$	$x19 := -50$
$y1 := 12.5$	$y4 := 12.5$	$y7 := 12.5$	$y10 := 12.5$	$y13 := 12.5$	$y16 := 12.5$	$y19 := 12.5$
$x2 := 56.25$	$x5 := 37.5$	$x8 := 18.75$	$x11 := 0$	$x14 := -18.75$	$x17 := -37.5$	$x20 := -56.25$
$y2 := 12.5$	$y5 := 12.5$	$y8 := 12.5$	$y11 := 12.5$	$y14 := 12.5$	$y17 := 12.5$	$y20 := 12.5$

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SUBJECT FILE:

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

BY: **NA**

CHECKED: **AAA**

DATE: **6/22/2004**

TITLE:
Saint Clair County Bridge

NCHRP Guidelines

$x3 := 50$	$x6 := 31.25$	$x9 := 12.5$	$x12 := -6.25$	$x15 := -25$	$x18 := -43.75$	$x21 := -62.5$
$y3 := 12.5$	$y6 := 12.5$	$y9 := 12.5$	$y12 := 12.5$	$y15 := 12.5$	$y18 := 12.5$	$y21 := 12.5$

2nd row

$x22 := 62.5$	$x25 := 43.75$	$x28 := 25$	$x31 := 6.25$	$x34 := -12.5$	$x37 := -31.25$	$x40 := -50$
$y22 := 6.25$	$y25 := 6.25$	$y28 := 6.25$	$y31 := 6.25$	$y34 := 6.25$	$y37 := 6.25$	$y40 := 6.25$
$x23 := 56.25$	$x26 := 37.5$	$x29 := 18.75$	$x32 := 0$	$x35 := -18.75$	$x38 := -37.5$	$x41 := -56.25$
$y23 := 6.25$	$y26 := 6.25$	$y29 := 6.25$	$y32 := 6.25$	$y35 := 6.25$	$y38 := 6.25$	$y41 := 6.25$
$x24 := 50$	$x27 := 31.25$	$x30 := 12.5$	$x33 := -6.25$	$x36 := -25$	$x39 := -43.75$	$x42 := -62.5$
$y24 := 6.25$	$y27 := 6.25$	$y30 := 6.25$	$y33 := 6.25$	$y36 := 6.25$	$y39 := 6.25$	$y42 := 6.25$

3rd row

$x43 := 62.5$	$x46 := 43.75$	$x49 := 25$	$x52 := 6.25$	$x55 := -12.5$	$x58 := -31.25$	$x61 := -50$
$y43 := 0$	$y46 := 0$	$y49 := 0$	$y52 := 0$	$y55 := 0$	$y58 := 0$	$y61 := 0$
$x44 := 56.25$	$x47 := 37.5$	$x50 := 18.75$	$x53 := 0$	$x56 := -18.75$	$x59 := -37.5$	$x62 := -56.25$
$y44 := 0$	$y47 := 0$	$y50 := 0$	$y53 := 0$	$y56 := 0$	$y59 := 0$	$y62 := 0$
$x45 := 50$	$x48 := 31.25$	$x51 := 12.5$	$x54 := -6.25$	$x57 := -25$	$x60 := -43.75$	$x63 := -62.5$
$y45 := 0$	$y48 := 0$	$y51 := 0$	$y54 := 0$	$y57 := 0$	$y60 := 0$	$y63 := 0$

4th row

$x64 := 62.5$	$x67 := 43.75$	$x70 := 25$	$x73 := 6.25$	$x76 := -12.5$	$x79 := -31.25$	$x82 := -50$
$y64 := -6.25$	$y67 := -6.25$	$y70 := -6.25$	$y73 := -6.25$	$y76 := -6.25$	$y79 := -6.25$	$y82 := -6.25$
$x65 := 56.25$	$x68 := 37.5$	$x71 := 18.75$	$x74 := 0$	$x77 := -18.75$	$x80 := -37.5$	$x83 := -56.25$
$y65 := -6.25$	$y68 := -6.25$	$y71 := -6.25$	$y74 := -6.25$	$y77 := -6.25$	$y80 := -6.25$	$y83 := -6.25$

$x66 := 50$	$x69 := 31.25$	$x72 := 12.5$	$x75 := -6.25$	$x78 := -25$	$x81 := -43.75$	$x84 := -62.5$
$y66 := -6.25$	$y69 := -6.25$	$y72 := -6.25$	$y75 := -6.25$	$y78 := -6.25$	$y81 := -6.25$	$y84 := -6.25$

5th row

$x85 := 62.5$	$x88 := 43.75$	$x91 := 25$	$x94 := 6.25$	$x97 := -12.5$	$x100 := -31.25$	$x103 := -50$
$y85 := -12.5$	$y88 := -12.5$	$y91 := -12.5$	$y94 := -12.5$	$y97 := -12.5$	$y100 := -12.5$	$y103 := -12.5$
$x86 := 56.25$	$x89 := 37.5$	$x92 := 18.75$	$x95 := 0$	$x98 := -18.75$	$x101 := -37.5$	$x104 := -56.2$
$y86 := -12.5$	$y89 := -12.5$	$y92 := -12.5$	$y95 := -12.5$	$y98 := -12.5$	$y101 := -12.5$	$y104 := -12.5$
$x87 := 50$	$x90 := 31.25$	$x93 := 12.5$	$x96 := -6.25$	$x99 := -25$	$x102 := -43.75$	$x105 := -62.5$
$y87 := -12.5$	$y90 := -12.5$	$y93 := -12.5$	$y96 := -12.5$	$y99 := -12.5$	$y102 := -12.5$	$y105 := -12.5$

$A := 6.25 \cdot 6.25$ — Tributary Area for each pile

$A = 39.063$ ft^2

$\delta := 0.12$ $\frac{kip}{ft^3}$ — Soil unit weight

$h := 2$ ft — soil hight

$p := A \cdot \delta \cdot h$

$p = 9.375$ kips — Soil pressure for each pile

$n := 105$ — Number of piles in the group

Axial forces in each pile

$$P1 := \frac{V}{n} + M1 \cdot \frac{x1}{\Sigma d1} + M2 \cdot \frac{y1}{\Sigma d2} + p$$

$$P2 := \frac{V}{n} + M1 \cdot \frac{x2}{\Sigma d1} + M2 \cdot \frac{y2}{\Sigma d2} + p$$

$P1 = 92.847$ kips

$P2 = 91.803$ kips

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$$P3 := \frac{V}{n} + M1\cdot\frac{x3}{\Sigma d1} + M2\cdot\frac{y3}{\Sigma d2} + p$$

$P3 = 90.76$ 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

$$P12 := \frac{V}{n} + M1\cdot\frac{x12}{\Sigma d1} + M2\cdot\frac{y12}{\Sigma d2} + p$$

$P12 = 81.366$

$$P13 := \frac{V}{n} + M1\cdot\frac{x13}{\Sigma d1} + M2\cdot\frac{y13}{\Sigma d2} + p$$

kips

$P13 = 80.322$

$$P14 := \frac{V}{n} + M1\cdot\frac{x14}{\Sigma d1} + M2\cdot\frac{y14}{\Sigma d2} + p$$

$P14 = 79.278$

$$P15 := \frac{V}{n} + M1\cdot\frac{x15}{\Sigma d1} + M2\cdot\frac{y15}{\Sigma d2} + p$$

kips

$P15 = 78.234$

$$P16 := \frac{V}{n} + M1\cdot\frac{x16}{\Sigma d1} + M2\cdot\frac{y16}{\Sigma d2} + p$$

$P16 = 77.19$

$$P17 := \frac{V}{n} + M1 \cdot \frac{x17}{\Sigma d1} + M2 \cdot \frac{y17}{\Sigma d2} + p$$

kips

$P17 = 76.147$

$$P18 := \frac{V}{n} + M1 \cdot \frac{x18}{\Sigma d1} + M2 \cdot \frac{y18}{\Sigma d2} + p$$

$P18 = 75.103$

$$P19 := \frac{V}{n} + M1 \cdot \frac{x19}{\Sigma d1} + M2 \cdot \frac{y19}{\Sigma d2} + p$$

kips

$P19 = 74.059$

$$P20 := \frac{V}{n} + M1 \cdot \frac{x20}{\Sigma d1} + M2 \cdot \frac{y20}{\Sigma d2} + p$$

$P20 = 73.015$

$$P21 := \frac{V}{n} + M1 \cdot \frac{x21}{\Sigma d1} + M2 \cdot \frac{y21}{\Sigma d2} + p$$

kips

$P21 = 71.971$

$$P22 := \frac{V}{n} + M1 \cdot \frac{x22}{\Sigma d1} + M2 \cdot \frac{y22}{\Sigma d2} + p$$

$P22 = 70.554$

$$P23 := \frac{V}{n} + M1 \cdot \frac{x23}{\Sigma d1} + M2 \cdot \frac{y23}{\Sigma d2} + p$$

kips

$P23 = 69.51$

$$P24 := \frac{V}{n} + M1 \cdot \frac{x24}{\Sigma d1} + M2 \cdot \frac{y24}{\Sigma d2} + p$$

$P24 = 68.466$

$$P25 := \frac{V}{n} + M1 \cdot \frac{x25}{\Sigma d1} + M2 \cdot \frac{y25}{\Sigma d2} + p$$

$P25 = 67.422$ kips

$$P26 := \frac{V}{n} + M1 \cdot \frac{x26}{\Sigma d1} + M2 \cdot \frac{y26}{\Sigma d2} + p$$

$P26 = 66.379$

$$P27 := \frac{V}{n} + M1 \cdot \frac{x27}{\Sigma d1} + M2 \cdot \frac{y27}{\Sigma d2} + p$$

kips

$P27 = 65.335$

$$P28 := \frac{V}{n} + M1 \cdot \frac{x28}{\Sigma d1} + M2 \cdot \frac{y28}{\Sigma d2} + p$$

$P28 = 64.291$

$$P29 := \frac{V}{n} + M1\cdot\frac{x29}{\Sigma d1} + M2\cdot\frac{y29}{\Sigma d2} + p$$

$P29 = 63.247$ kips

$$P30 := \frac{V}{n} + M1\cdot\frac{x30}{\Sigma d1} + M2\cdot\frac{y30}{\Sigma d2} + p$$

$P30 = 62.204$

$$P31 := \frac{V}{n} + M1\cdot\frac{x31}{\Sigma d1} + M2\cdot\frac{y31}{\Sigma d2} + p$$

$P31 = 61.16$ kips

$$P32 := \frac{V}{n} + M1\cdot\frac{x32}{\Sigma d1} + M2\cdot\frac{y32}{\Sigma d2} + p$$

$P32 = 60.116$

$$P33 := \frac{V}{n} + M1\cdot\frac{x33}{\Sigma d1} + M2\cdot\frac{y33}{\Sigma d2} + p$$

$P33 = 59.072$ kips

$$P34 := \frac{V}{n} + M1\cdot\frac{x34}{\Sigma d1} + M2\cdot\frac{y34}{\Sigma d2} + p$$

$P34 = 58.028$

$$P35 := \frac{V}{n} + M1\cdot\frac{x35}{\Sigma d1} + M2\cdot\frac{y35}{\Sigma d2} + p$$

$P35 = 56.985$ kips

$$P36 := \frac{V}{n} + M1\cdot\frac{x36}{\Sigma d1} + M2\cdot\frac{y36}{\Sigma d2} + p$$

$P36 = 55.941$

$$P37 := \frac{V}{n} + M1\cdot\frac{x37}{\Sigma d1} + M2\cdot\frac{y37}{\Sigma d2} + p$$

$P37 = 54.897$ kips

$$P38 := \frac{V}{n} + M1\cdot\frac{x38}{\Sigma d1} + M2\cdot\frac{y38}{\Sigma d2} + p$$

$P38 = 53.853$

$$P39 := \frac{V}{n} + M1\cdot\frac{x39}{\Sigma d1} + M2\cdot\frac{y39}{\Sigma d2} + p$$

$P39 = 52.809$ kips

$$P40 := \frac{V}{n} + M1\cdot\frac{x40}{\Sigma d1} + M2\cdot\frac{y40}{\Sigma d2} + p$$

$P40 = 51.766$

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SUBJECT FILE:

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

BY: **NA**

CHECKED: **AAA**

DATE: **6/22/2004**

TITLE:
Saint Clair County Bridge

NCHRP Guidelines

$$P41 := \frac{V}{n} + M1\cdot\frac{x41}{\Sigma d1} + M2\cdot\frac{y41}{\Sigma d2} + p$$

$P41 = 50.722$ kips

$$P42 := \frac{V}{n} + M1\cdot\frac{x42}{\Sigma d1} + M2\cdot\frac{y42}{\Sigma d2} + p$$

$P42 = 49.678$ kips

$$P43 := \frac{V}{n} + M1\cdot\frac{x43}{\Sigma d1} + M2\cdot\frac{y43}{\Sigma d2} + p$$

$P43 = 48.26$ kips

$$P44 := \frac{V}{n} + M1\cdot\frac{x44}{\Sigma d1} + M2\cdot\frac{y44}{\Sigma d2} + p$$

$P44 = 47.217$ kips

$$P45 := \frac{V}{n} + M1\cdot\frac{x45}{\Sigma d1} + M2\cdot\frac{y45}{\Sigma d2} + p$$

$P45 = 46.173$ kips

$$P46 := \frac{V}{n} + M1\cdot\frac{x46}{\Sigma d1} + M2\cdot\frac{y46}{\Sigma d2} + p$$

$P46 = 45.129$ kips

$$P47 := \frac{V}{n} + M1\cdot\frac{x47}{\Sigma d1} + M2\cdot\frac{y47}{\Sigma d2} + p$$

$P47 = 44.085$ kips

$$P48 := \frac{V}{n} + M1\cdot\frac{x48}{\Sigma d1} + M2\cdot\frac{y48}{\Sigma d2} + p$$

$P48 = 43.042$ kips

$$P49 := \frac{V}{n} + M1\cdot\frac{x49}{\Sigma d1} + M2\cdot\frac{y49}{\Sigma d2} + p$$

$P49 = 41.998$ kips

$$P50 := \frac{V}{n} + M1\cdot\frac{x50}{\Sigma d1} + M2\cdot\frac{y50}{\Sigma d2} + p$$

$P50 = 40.954$ kips

$$P51 := \frac{V}{n} + M1\cdot\frac{x51}{\Sigma d1} + M2\cdot\frac{y51}{\Sigma d2} + p$$

$P51 = 39.91$ kips

$$P52 := \frac{V}{n} + M1\cdot\frac{x52}{\Sigma d1} + M2\cdot\frac{y52}{\Sigma d2} + p$$

$P52 = 38.866$ kips

$$P53 := \frac{V}{n} + M1\cdot\frac{x53}{\Sigma d1} + M2\cdot\frac{y53}{\Sigma d2} + p$$

$$P54 := \frac{V}{n} + M1\cdot\frac{x54}{\Sigma d1} + M2\cdot\frac{y54}{\Sigma d2} + p$$

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$P53 = 37.823$ kips

$P54 = 36.779$ kips

$$P55 := \frac{V}{n} + M1 \cdot \frac{x55}{\Sigma d1} + M2 \cdot \frac{y55}{\Sigma d2} + p$$

$P55 = 35.735$ kips

$$P56 := \frac{V}{n} + M1 \cdot \frac{x56}{\Sigma d1} + M2 \cdot \frac{y56}{\Sigma d2} + p$$

$P56 = 34.691$ kips

$$P57 := \frac{V}{n} + M1 \cdot \frac{x57}{\Sigma d1} + M2 \cdot \frac{y57}{\Sigma d2} + p$$

$P57 = 33.647$ kips

$$P58 := \frac{V}{n} + M1 \cdot \frac{x58}{\Sigma d1} + M2 \cdot \frac{y58}{\Sigma d2} + p$$

$P58 = 32.604$ kips

$$P59 := \frac{V}{n} + M1 \cdot \frac{x59}{\Sigma d1} + M2 \cdot \frac{y59}{\Sigma d2} + p$$

$P59 = 31.56$ kips

$$P60 := \frac{V}{n} + M1 \cdot \frac{x60}{\Sigma d1} + M2 \cdot \frac{y60}{\Sigma d2} + p$$

$P60 = 30.516$ kips

$$P61 := \frac{V}{n} + M1 \cdot \frac{x61}{\Sigma d1} + M2 \cdot \frac{y61}{\Sigma d2} + p$$

$P61 = 29.472$ kips

$$P62 := \frac{V}{n} + M1 \cdot \frac{x62}{\Sigma d1} + M2 \cdot \frac{y62}{\Sigma d2} + p$$

$P62 = 28.429$ kips

$$P63 := \frac{V}{n} + M1 \cdot \frac{x63}{\Sigma d1} + M2 \cdot \frac{y63}{\Sigma d2} + p$$

$P63 = 27.385$ kips

$$P64 := \frac{V}{n} + M1 \cdot \frac{x64}{\Sigma d1} + M2 \cdot \frac{y64}{\Sigma d2} + p$$

$P64 = 25.967$ kips

$$P65 := \frac{V}{n} + M1 \cdot \frac{x65}{\Sigma d1} + M2 \cdot \frac{y65}{\Sigma d2} + p$$

$P65 = 24.923$ kips

$$P66 := \frac{V}{n} + M1 \cdot \frac{x66}{\Sigma d1} + M2 \cdot \frac{y66}{\Sigma d2} + p$$

$P66 = 23.88$ kips

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$$P67 := \frac{V}{n} + M1 \cdot \frac{x67}{\Sigma d1} + M2 \cdot \frac{y67}{\Sigma d2} + p$$

$P67 = 22.836$ kips

$$P68 := \frac{V}{n} + M1 \cdot \frac{x68}{\Sigma d1} + M2 \cdot \frac{y68}{\Sigma d2} + p$$

$P68 = 21.792$ kips

$$P69 := \frac{V}{n} + M1 \cdot \frac{x69}{\Sigma d1} + M2 \cdot \frac{y69}{\Sigma d2} + p$$

$P69 = 20.748$ kips

$$P70 := \frac{V}{n} + M1 \cdot \frac{x70}{\Sigma d1} + M2 \cdot \frac{y70}{\Sigma d2} + p$$

$P70 = 19.704$ kips

$$P71 := \frac{V}{n} + M1 \cdot \frac{x71}{\Sigma d1} + M2 \cdot \frac{y71}{\Sigma d2} + p$$

$P71 = 18.661$ kips

$$P72 := \frac{V}{n} + M1 \cdot \frac{x72}{\Sigma d1} + M2 \cdot \frac{y72}{\Sigma d2} + p$$

$P72 = 17.617$ kips

$$P73 := \frac{V}{n} + M1 \cdot \frac{x73}{\Sigma d1} + M2 \cdot \frac{y73}{\Sigma d2} + p$$

$P73 = 16.573$ kips

$$P74 := \frac{V}{n} + M1 \cdot \frac{x74}{\Sigma d1} + M2 \cdot \frac{y74}{\Sigma d2} + p$$

$P74 = 15.529$ kips

$$P75 := \frac{V}{n} + M1 \cdot \frac{x75}{\Sigma d1} + M2 \cdot \frac{y75}{\Sigma d2} + p$$

$P75 = 14.486$ kips

$$P76 := \frac{V}{n} + M1 \cdot \frac{x76}{\Sigma d1} + M2 \cdot \frac{y76}{\Sigma d2} + p$$

$P76 = 13.442$ kips

$$P77 := \frac{V}{n} + M1 \cdot \frac{x77}{\Sigma d1} + M2 \cdot \frac{y77}{\Sigma d2} + p$$

$P77 = 12.398$ kips

$$P78 := \frac{V}{n} + M1 \cdot \frac{x78}{\Sigma d1} + M2 \cdot \frac{y78}{\Sigma d2} + p$$

$P78 = 11.354$ kips

$$P79 := \frac{V}{n} + M1 \cdot \frac{x79}{\Sigma d1} + M2 \cdot \frac{y79}{\Sigma d2} + p$$

$P79 = 10.31$ kips

$$P80 := \frac{V}{n} + M1 \cdot \frac{x80}{\Sigma d1} + M2 \cdot \frac{y80}{\Sigma d2} + p$$

$P80 = 9.267$ kips

$$P81 := \frac{V}{n} + M1 \cdot \frac{x81}{\Sigma d1} + M2 \cdot \frac{y81}{\Sigma d2} + p$$

$P81 = 8.223$ kips

$$P82 := \frac{V}{n} + M1 \cdot \frac{x82}{\Sigma d1} + M2 \cdot \frac{y82}{\Sigma d2} + p$$

$P82 = 7.179$ kips

$$P83 := \frac{V}{n} + M1 \cdot \frac{x83}{\Sigma d1} + M2 \cdot \frac{y83}{\Sigma d2} + p$$

$P83 = 6.135$ kips

$$P84 := \frac{V}{n} + M1 \cdot \frac{x84}{\Sigma d1} + M2 \cdot \frac{y84}{\Sigma d2} + p$$

$P84 = 5.091$ kips

$$P85 := \frac{V}{n} + M1 \cdot \frac{x85}{\Sigma d1} + M2 \cdot \frac{y85}{\Sigma d2} + p$$

$P85 = 3.674$ kips

$$P86 := \frac{V}{n} + M1 \cdot \frac{x86}{\Sigma d1} + M2 \cdot \frac{y86}{\Sigma d2} + p$$

$P86 = 2.63$ kips

$$P87 := \frac{V}{n} + M1 \cdot \frac{x87}{\Sigma d1} + M2 \cdot \frac{y87}{\Sigma d2} + p$$

$P87 = 1.586$ kips

$$P88 := \frac{V}{n} + M1 \cdot \frac{x88}{\Sigma d1} + M2 \cdot \frac{y88}{\Sigma d2} + p$$

$P88 = 0.542$ kips

$$P89 := \frac{V}{n} + M1 \cdot \frac{x89}{\Sigma d1} + M2 \cdot \frac{y89}{\Sigma d2} + p$$

$P89 = -0.501$ kips

$$P90 := \frac{V}{n} + M1 \cdot \frac{x90}{\Sigma d1} + M2 \cdot \frac{y90}{\Sigma d2} + p$$

$P90 = -1.545$ kips

$P91 := \frac{V}{n} + M1 \cdot \frac{x91}{\Sigma d1} + M2 \cdot \frac{y91}{\Sigma d2} + p$

$P91 = -2.589$ kips

$P92 := \frac{V}{n} + M1 \cdot \frac{x92}{\Sigma d1} + M2 \cdot \frac{y92}{\Sigma d2} + p$

$P92 = -3.633$ kips

$P93 := \frac{V}{n} + M1 \cdot \frac{x93}{\Sigma d1} + M2 \cdot \frac{y93}{\Sigma d2} + p$

$P93 = -4.676$ kips

$P94 := \frac{V}{n} + M1 \cdot \frac{x94}{\Sigma d1} + M2 \cdot \frac{y94}{\Sigma d2} + p$

$P94 = -5.72$ kips

$P95 := \frac{V}{n} + M1 \cdot \frac{x95}{\Sigma d1} + M2 \cdot \frac{y95}{\Sigma d2} + p$

$P95 = -6.764$ kips

$P96 := \frac{V}{n} + M1 \cdot \frac{x96}{\Sigma d1} + M2 \cdot \frac{y96}{\Sigma d2} + p$

$P96 = -7.808$ kips

$P97 := \frac{V}{n} + M1 \cdot \frac{x97}{\Sigma d1} + M2 \cdot \frac{y97}{\Sigma d2} + p$

$P97 = -8.852$ kips

$P98 := \frac{V}{n} + M1 \cdot \frac{x98}{\Sigma d1} + M2 \cdot \frac{y98}{\Sigma d2} + p$

$P98 = -9.895$ kips

$P99 := \frac{V}{n} + M1 \cdot \frac{x99}{\Sigma d1} + M2 \cdot \frac{y99}{\Sigma d2} + p$

$P99 = -10.939$ kips

$P100 := \frac{V}{n} + M1 \cdot \frac{x100}{\Sigma d1} + M2 \cdot \frac{y100}{\Sigma d2} + p$

$P100 = -11.983$ kips

$P101 := \frac{V}{n} + M1 \cdot \frac{x101}{\Sigma d1} + M2 \cdot \frac{y101}{\Sigma d2} + p$

$P101 = -13.027$ kips

$P102 := \frac{V}{n} + M1 \cdot \frac{x102}{\Sigma d1} + M2 \cdot \frac{y102}{\Sigma d2} + p$

$P102 = -14.071$ kips

$$P103 := \frac{V}{n} + M1 \cdot \frac{x103}{\Sigma d1} + M2 \cdot \frac{y103}{\Sigma d2} + p$$

$P103 = -15.114$ kips

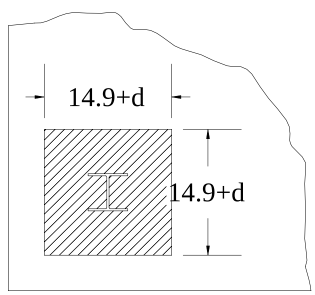
$$P104 := \frac{V}{n} + M1 \cdot \frac{x104}{\Sigma d1} + M2 \cdot \frac{y104}{\Sigma d2} + p$$

$P104 = -16.158$ kips

$$P105 := \frac{V}{n} + M1 \cdot \frac{x105}{\Sigma d1} + M2 \cdot \frac{y105}{\Sigma d2} + p$$

$P105 = -17.202$ kips

Pile punching shear check:



$Vu := P1$

$Vu = 92.847$ Kips — Maximum axial load in piles

$d := 45$ in — Concrete slab thickness for 3 ft footing

$b := 191.6$ in — Perimeter acting in punching shear

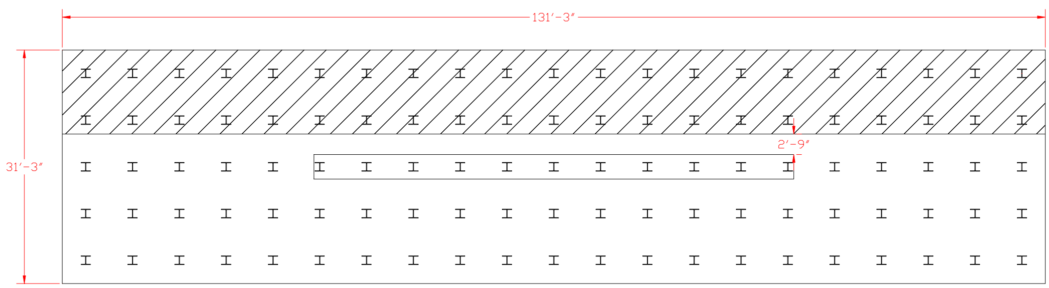
$$Vc := 4 \cdot \sqrt{Fc} \cdot b \cdot \frac{d}{1000}$$ Shear strength

$Vc = 2.034 \times 10^3$ kips

$\phi := 0.85$ Strength reduction factor

$\frac{Vu}{\phi} = 109.232$ $\frac{Vu}{\phi} < Vc$ OK

One way shear Check:



$V1 := 2.933 \cdot 10^3$ Kips V1 is equale to sum of P1 to P42

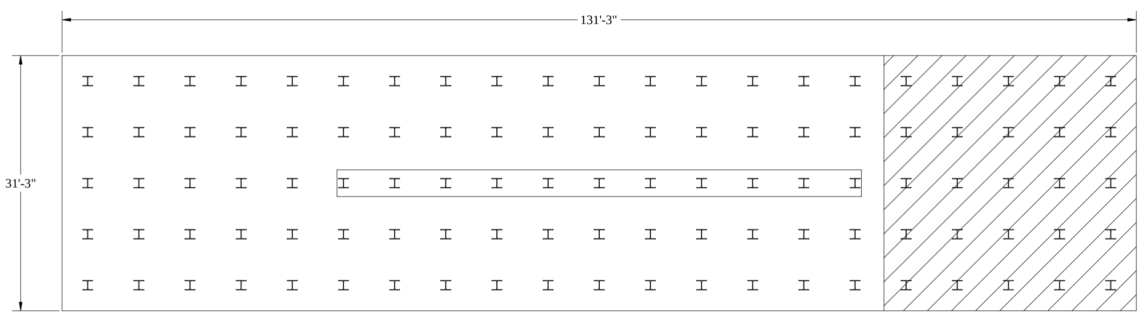
$V2 := 549.898$ Kips V2 is equale to sum of P43 to P105

$b2 := 1575$ in Length of pilecap

$Vc2 := 4 \cdot \sqrt{Fc} \cdot b2 \cdot \frac{d}{1000}$

$\mathrm{Vc2} = 1.672 \times 10^{4}$

$\frac{\mathrm{V1}}{\phi} = 3.451 \times 10^{3} \quad \frac{\mathrm{V1}}{\phi} < \mathrm{Vc2}$ OK



$\mathrm{v1} := (\mathrm{P1} + \mathrm{P2} + \mathrm{P3} + \mathrm{P4} + \mathrm{P5})$

$\mathrm{v2} := (\mathrm{P22} + \mathrm{P23} + \mathrm{P24} + \mathrm{P25} + \mathrm{P26})$

$\mathrm{v3} := (\mathrm{P43} + \mathrm{P44} + \mathrm{P45} + \mathrm{P46} + \mathrm{P47})$

$\mathrm{v4} := (\mathrm{P64} + \mathrm{P65} + \mathrm{P66} + \mathrm{P67} + \mathrm{P68})$

$\mathrm{v5} := (\mathrm{P85} + \mathrm{P86} + \mathrm{P87} + \mathrm{P88} + \mathrm{P89})$

$\mathrm{v} := \mathrm{v1} + \mathrm{v2} + \mathrm{v3} + \mathrm{v4} + \mathrm{v5}$

$\mathrm{v} = 1.154 \times 10^{3}$

$$\frac{v}{0.85} = 1.358 \times 10^3$$

$b3 := 375$

$Fc := 3480$

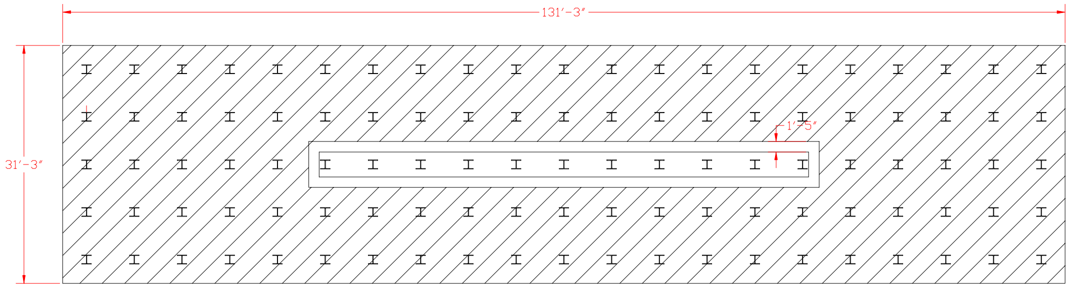
$d := 36$

$$Vc3 := 4 \cdot \sqrt{Fc} \cdot b3 \cdot \frac{d}{1000}$$

$Vc3 = 3.186 \times 10^3$

$\frac{v}{0.85} < Vc3$ OK

Two-way shear check:



$V_u := V1 + V2$

$V_u = 3.483 \times 10^3$

$b_0 := 801.91 \cdot 72.37$ Perimter of inside rectangular

$b_0 = 5.803 \times 10^4$ in

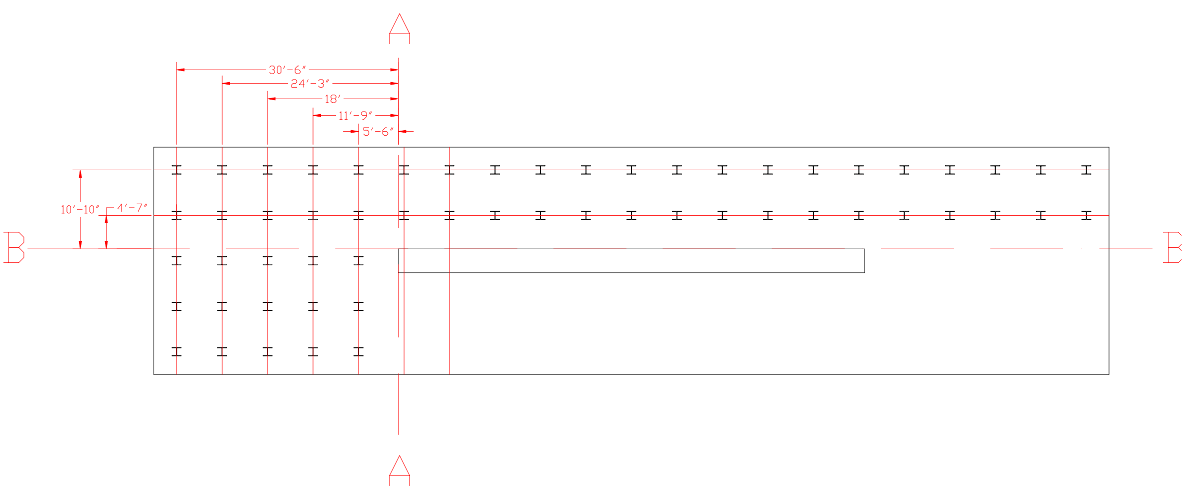
$V_c := 4 \cdot \sqrt{Fc} \cdot b_0 \cdot d$

$V_c = 4.93 \times 10^8$

$\phi := 0.85$ Strength reduction factor

$\frac{V_u}{\phi} = 4.098 \times 10^3$ < V_c ok

Flexure bending design:



SECTION A-A

$M1 := (P17 + P38 + P59 + P80 + P101) \cdot 66 \cdot 1000$

$M2 := (P18 + P39 + P60 + P81 + P102) \cdot 141 \cdot 1000$

$M3 := (P19 + P40 + P61 + P82 + P103) \cdot 216 \cdot 1000$

$M4 := (P20 + P41 + P62 + P83 + P104) \cdot 291 \cdot 1000$

$M5 := (P21 + P42 + P63 + P84 + P105) \cdot 366 \cdot 1000$

$Mu := M1 + M2 + M3 + M4 + M5$

$Mu = 1.552 \times 10^{8}$ $lb - in$

$\Phi := 0.9$ Strength reduction factor

$As := \dfrac{Mu}{\Phi \cdot Fy \cdot 0.9 \cdot d}$ Area of steel

$As = 88.727$ in^{2}

$Asmin := 0.0018 \cdot 45 \cdot 375$ Minimum reinfocement

$Asmin = 30.375$ in^{2} less than As OK

In long Direction : 90#9

Use 31 # 9 in long direction at the top of footing

SECTION B-B

$V_1 := 1.731 \cdot 10^3$

$V_2 := 1.262 \cdot 10^3$

$M1 := (V_1) \cdot 55 \cdot 1000$ V1 is equal to sumation of P1 to P21

$M2 := (V_2) \cdot 130 \cdot 1000$ V2 is equal to sumation of P22 to P42

$Mu := M1 + M2$

$Mu = 2.593 \times 10^8$ $lb - in$

$\Phi := 0.9$ Strength reduction factor

$As := \dfrac{Mu}{\Phi \cdot Fy \cdot 0.9 \cdot d}$ Area of steel

$As = 148.185$ in^2

$Asmin := 0.0018 \cdot 45 \cdot 1575$

$Asmin = 127.575$ Minimum reinfocement

$Asmin = 127.575$ in^2 less than As OK

$\boxed{\text{In short direction: 149 \# 9}}$

$\boxed{\text{Use 103 \# 9 in short direction at the top of footing}}$

CAP Beam Design:

For Cap-beam design, a simple 2D model under developed. (11 concentrated loads applying on the beam as shown below).

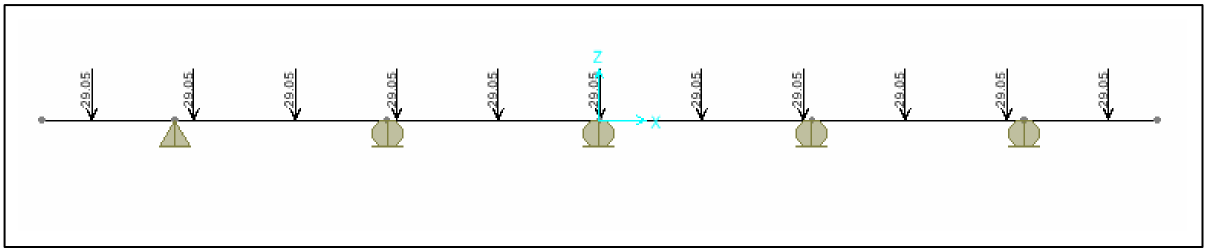


Fig.4. 2D model of Cap-beam

NCHRP	Forces & Moments-dead load			
Location	Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft
Beam	49	0	0	208

NCHRP	Forces & Moments-Trans(Y) EQ 100%Trans+40%Long			
Location	Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft
Beam	593	5	8	5522

NCHRP	Forces & Moments-Long(X) EQ 100%Long+40%Trans			
Location	Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft
Beam	783	6	11	4186

Final Design Forces and Moments:

NCHRP	Final design Forces and Moments (EQ+DL)							
	Transvers				Longitudinal			
Location	Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft
Beam	642	5	8	5730	832	6	11	4394

Table 1

Reinforcement Design for continuous deep beam:

Deep beam design (Source: Reinforcement concrete, Nawy, 3rd edition, section 6.9)

Shear reinforcement:

$f_c := 3480$ psi — Compressive Strength of Concrete

$f_y := 60000$ psi — Yield strength of reinforcing bars

$l := 175.19$ in — span

$l_n := 105.2$ in — clear span

$h := 56.5$ in — Hight of the beam

$b_w := 39.37$ in — Width of beam

$d := 0.9 \cdot h$

$d = 50.85$ in

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$\frac{l_n}{d} = 2.069$ < 5 , hence treat as a deep beam

$0.15 \cdot l_n = 15.78$ in Distance of the critical section (6.17a)

$V := 642$ kip Design shear force from table 1

$$V_u := \frac{V \cdot l_n}{2} - V \cdot \frac{0.15 \cdot l_n}{12}$$

$V_u = 3.292 \times 10^4$ kip

$\phi := 0.85$ Strength reduction factor

$\phi V_n := \phi \cdot 8 \cdot \sqrt{f_c} \cdot b_w \cdot d$ factored shear force (6-18a)

$\phi V_n = 8.031 \times 10^5$ kip > Vu OK

$$M_u := \frac{V \cdot \left(\frac{l_n}{12}\right)}{2} \cdot \left(\frac{0.15 \cdot l_n}{12}\right) - \frac{V \cdot \left(\frac{0.15 l_n}{12}\right)^2}{2}$$

$M_u = 3.145 \times 10^3$

$3.5 - 2.5 \cdot \frac{M_u}{V_u \cdot d} = 3.495$ > 2.5 Use 2.5

$\rho_w := \frac{9 \cdot 0.79}{b_w \cdot d}$ Steel Ratio in tension (9 No.8)

$\rho_w = 3.552 \times 10^{-3}$

$$V_c := 2.5\left(1.9\cdot\sqrt{f_c} + 2500\cdot\rho_w\cdot\frac{V_u\cdot d}{M_u}\right)\cdot b_w\cdot d$$

$V_c = 2.421 \times 10^7$ lb

$V_c := 6\cdot\sqrt{f_c}\cdot b_w\cdot d = \left(7.086 \times 10^5\right)$ Controls

Assume No.5 bars placed both horizontally and vertically on both faces

$$V_s := \frac{V_u}{\phi} - \frac{V_c}{1000}$$

$V_s = 3.803 \times 10^4$

$A_v := .31\cdot 4$ 4 No. 5

$A_{vh} := 6\cdot 0.44$ 6 No.6 in horizontal position

$A_{vh} = 2.64$

Try $S_1 := 18$

$$V_s := \left(\frac{A_v}{S_1}\cdot\frac{1 + \frac{l_n}{d}}{12} + \frac{A_{vh}}{S_1}\cdot\frac{11 - \frac{l_n}{d}}{12}\right)\cdot f_y\cdot d$$

$V_s = 3.868 \times 10^5$

$\frac{d}{5} = 10.17$ or 18 in Maximum Spacing

$\frac{d}{3} = 16.95$ or 18 in

$S := 10$ Controls

$A_v := 0.0015 \cdot b_w \cdot S$ Minimum reinforcement

$A_v = 0.591$ less than Av=0.62 ok

$A_{vh} := 0.0025 \cdot b_w \cdot S$ Minimum reinforcement

$A_{vh} = 0.984$ less than Avh=3.52. ok

Flexural Steel:

$M := 5370$ kip − ft

$\phi := 0.9$ Strength reduction factor

$M_n := \frac{M_u}{\phi}$

$jd := 0.2(l + 1.5 \cdot h)$ Lever arm

$jd = 51.988$ in

$A_s := \frac{M_n \cdot 12000}{f_y \cdot jd}$ Area of tension steel

$A_s = 13.445$ in^2

$A_{s1} := \frac{200 \cdot b_w \cdot d}{f_y}$ $\qquad$ $A_{s2} := \frac{3 \cdot \sqrt{f_c}}{f_y} \cdot b_w \cdot d$

$A_{s1} = 6.673$ $\qquad$ $A_{s2} = 5.905$ Controls

$A_{s\_min} := \max\left(A_{s1}, A_{s2}\right)$

$A_{s\_min} = 6.673$ in^2 less than As OK

$A_s = 13.445$ in^2

Use 14 # 9 as flexural reinforcement

Checks:

AASHTO LRFD SEC.5.7.3.3.2:

1)

$\rho_{min} := 0.03 \cdot \frac{f_c}{f_y}$

$\rho_{min} = 1.74 \times 10^{-3}$

$\rho min := \frac{A_s}{b_w \cdot d}$

$\rho min = 6.716 \times 10^{-3}$ $\qquad$ $\rho min > \rho_{min}$ OK

2)

$$F_r := 7.5 \cdot \sqrt{f_c}$$

$$I_g := \frac{b_w \cdot h^3}{12}$$ Moment of Inertia

$$y_t := \frac{h}{2}$$

$$M_{cr} := \frac{F_r \cdot \frac{I_g}{y_t}}{12000}$$ Cracking Moment

$$M_{cr} = 772.291$$

$$a := A_s \cdot \frac{f_y}{0.85 \cdot f_c \cdot b_w}$$

$$a = 6.927$$

$$M_u := \phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) \cdot \frac{1}{12000}$$

$$M_u = 2.867 \times 10^3$$

$$M_u \ge 1.2 \cdot M_{cr}$$ OK

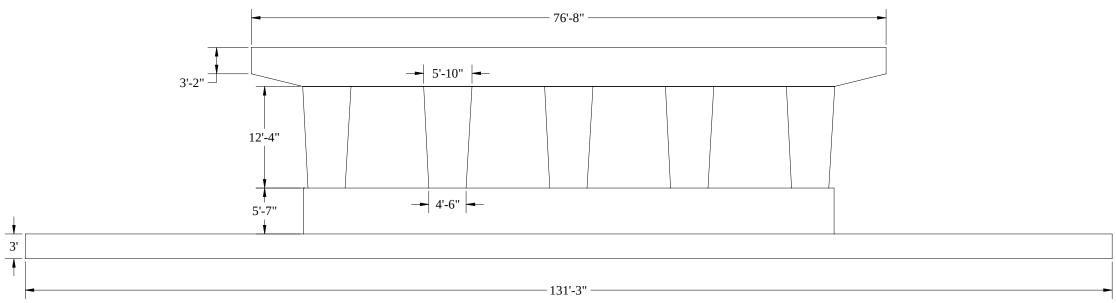


Fig.5. bent front view with dimensions

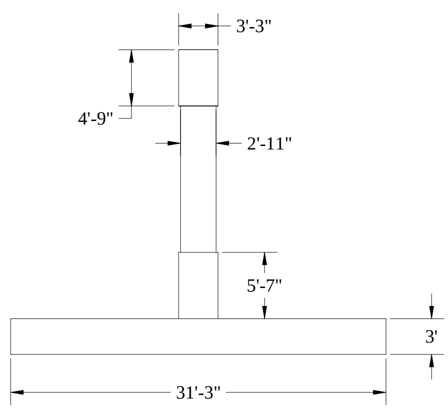


Fig.6. bent side view with dimensions

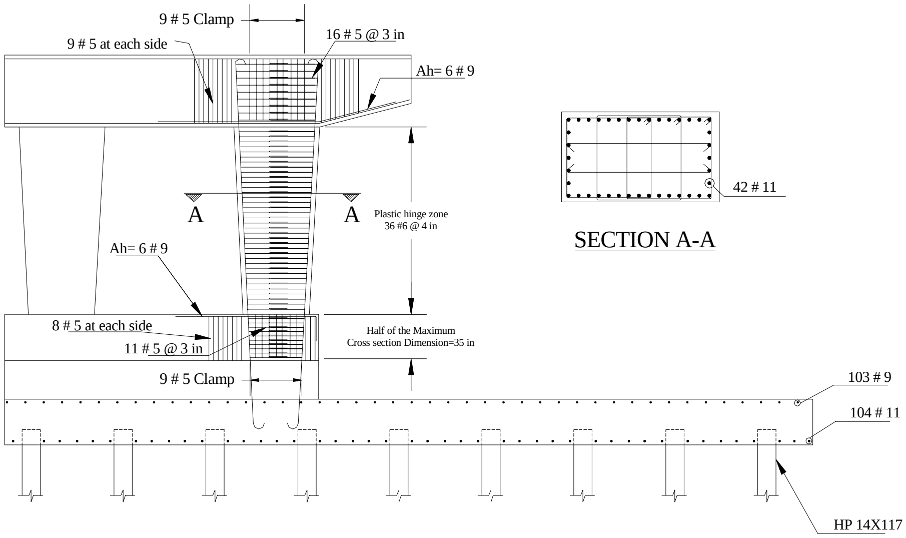


Fig.7. Reinforcement details for Columns, Footing and Cap beam (front view)

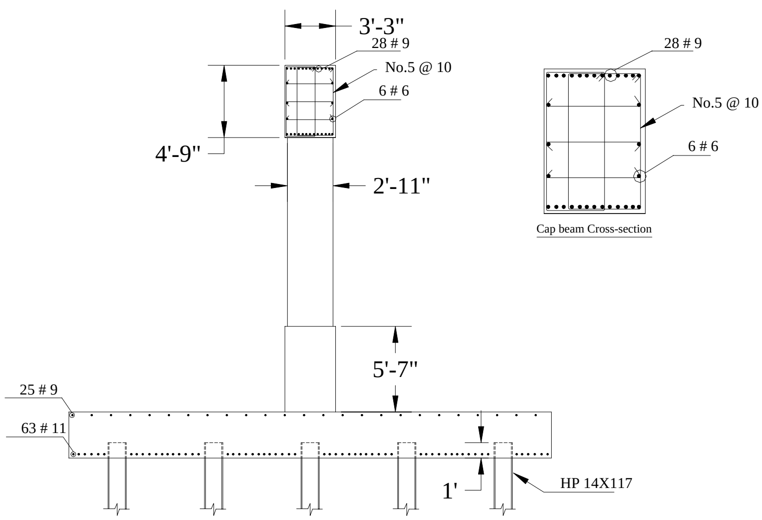


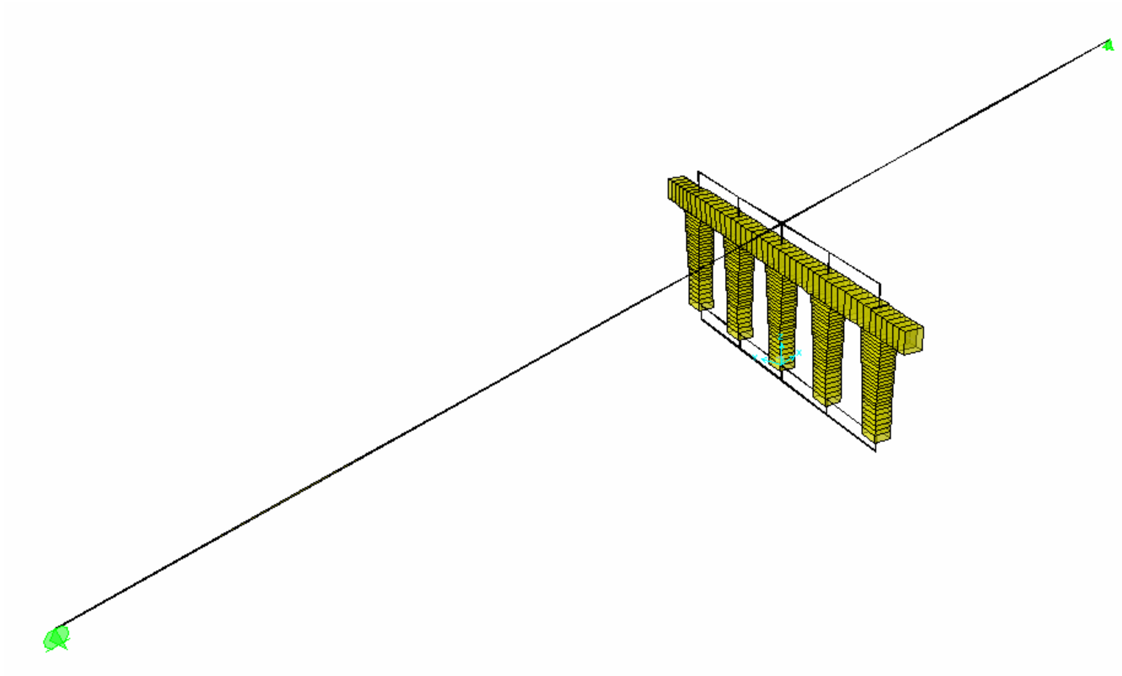
Fig.8. Reinforcement details for Footing and Cap beam (Side view)

Appendix D -- Detailed Computations for Seismic Analysis and Design Check of the St. Clair County Bridge Using AASHTO Specifications

Table of Contents of Appendix- D Page

SAP 2000 Model

View of SAP 2000 Model (with concrete extrusion shown):



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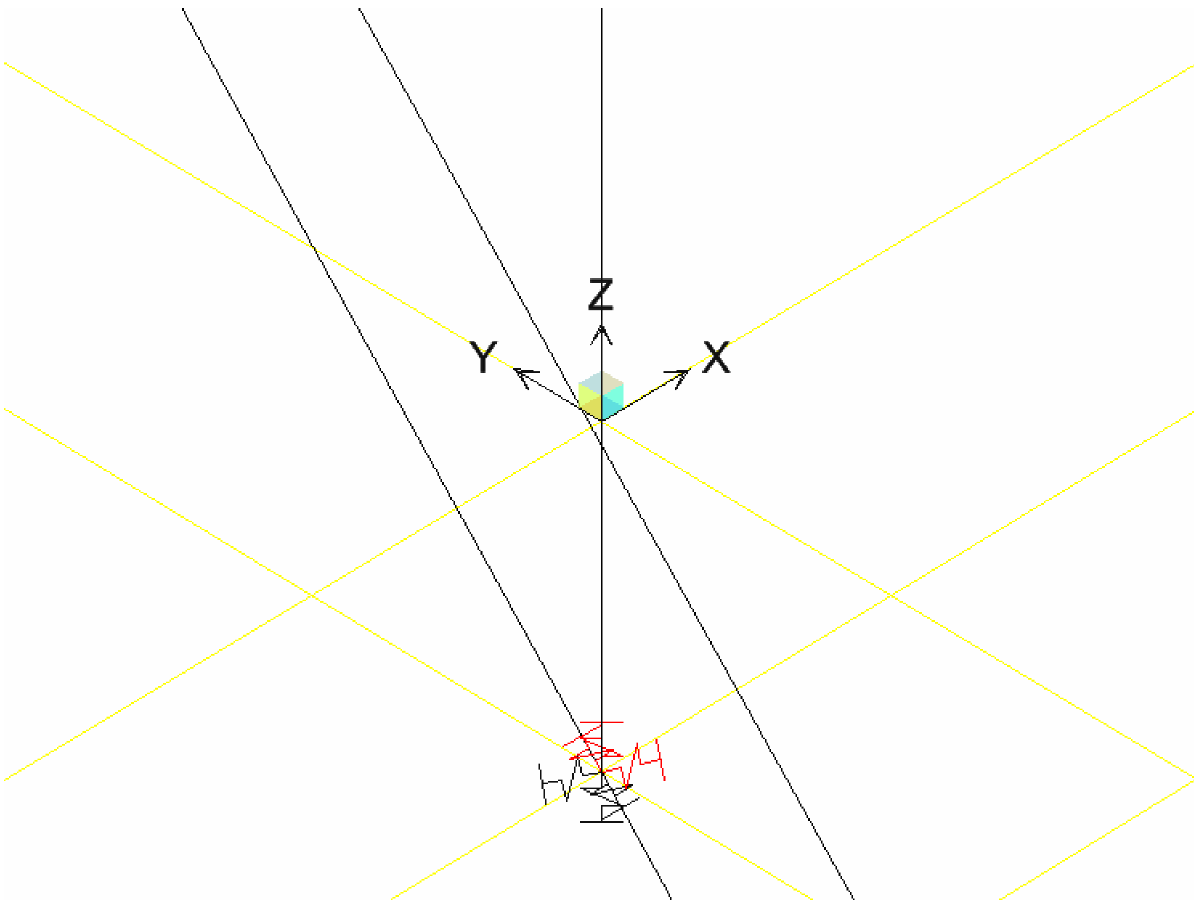
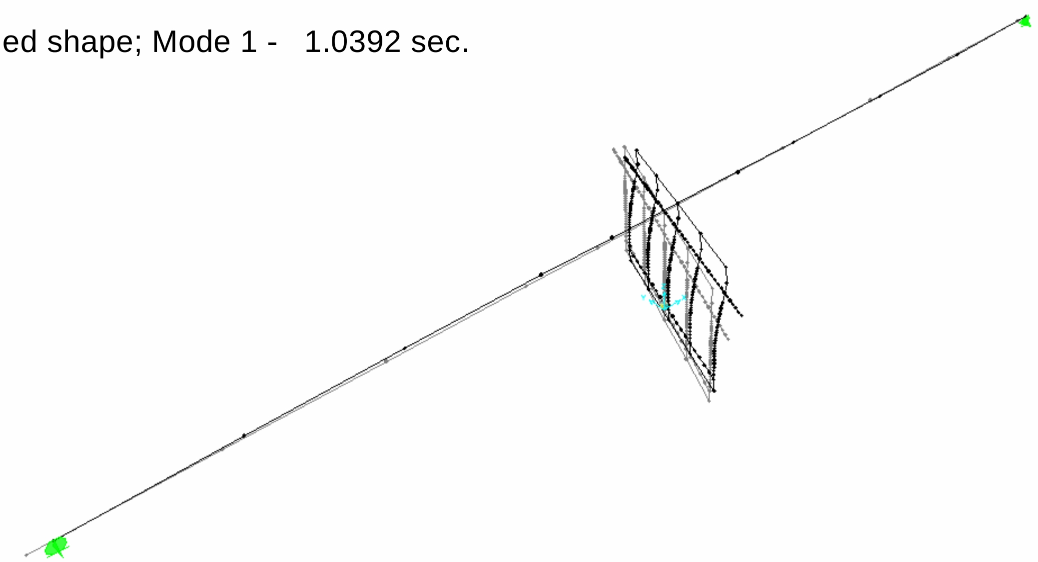
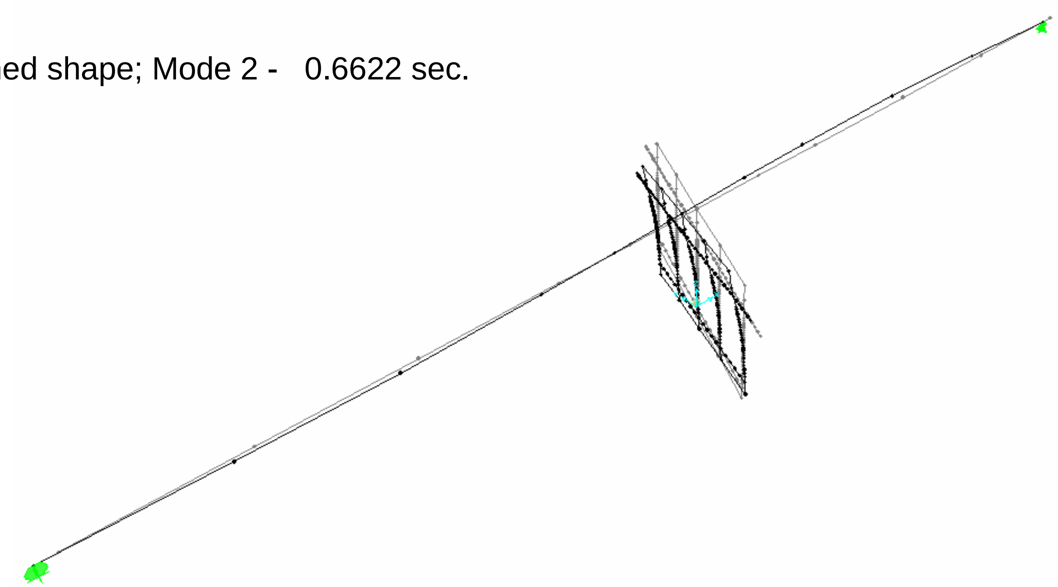
Enlarged View of Translational and Rotational Springs:



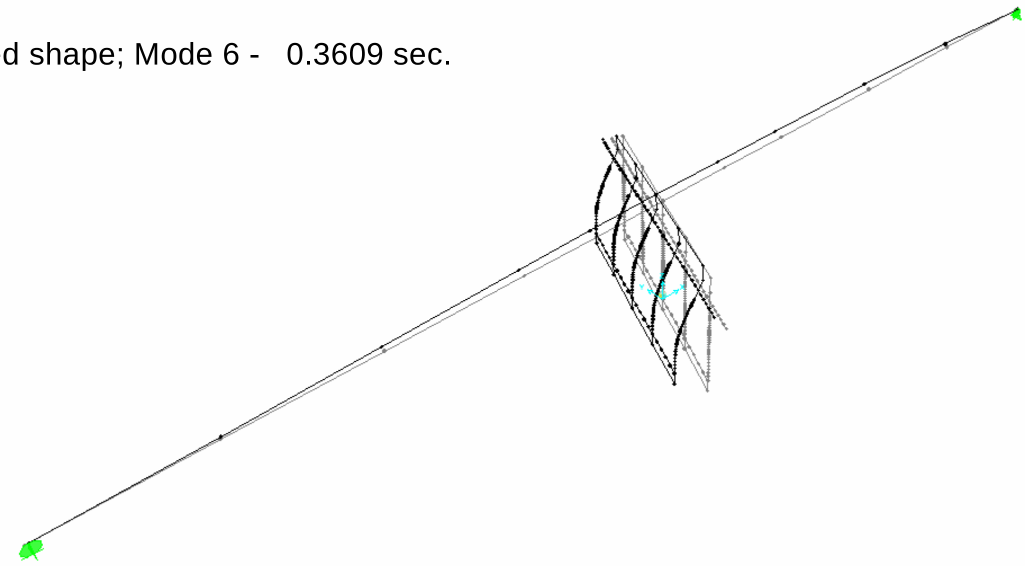
Deformed shape; Mode 1 - 1.0392 sec.



Deformed shape; Mode 2 - 0.6622 sec.



Deformed shape; Mode 6 - 0.3609 sec.



Deformed shape; Mode 8 - 0.3168 sec.

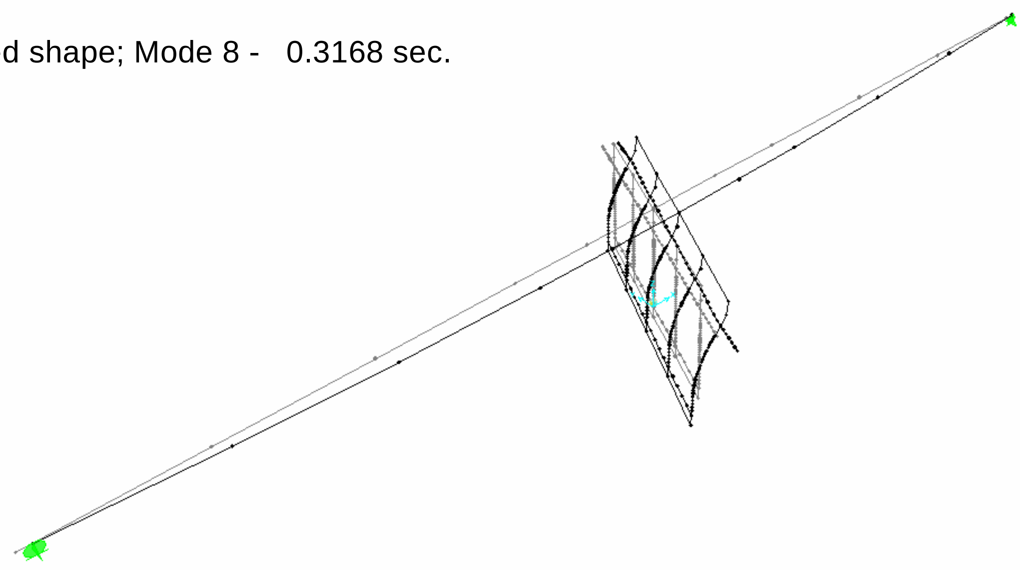
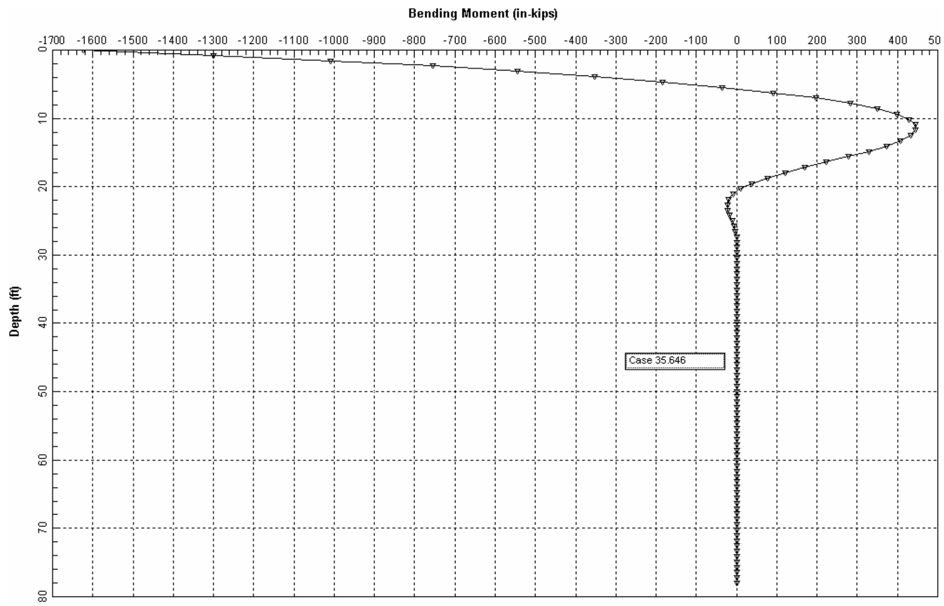


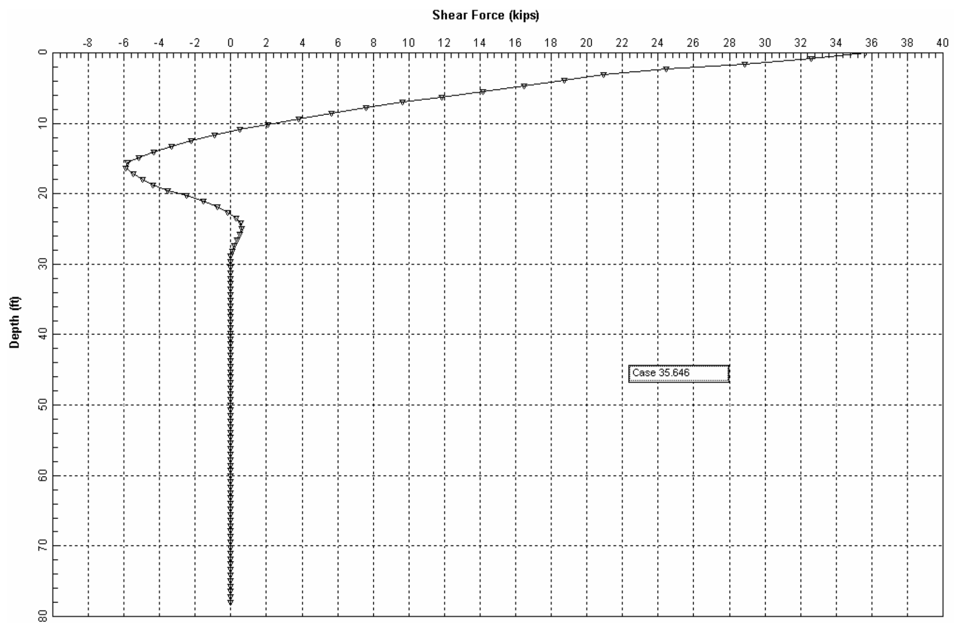
TABLE: Modal Participating Mass Ratios				
Mode Number	Period Sec	UX %	UY %	UZ %
1	1.0392	40.16	0.45	0
2	0.6622	34.43	8.99	0
3	0.6434	0	0	98.66
4	0.6002	2.08	0.07	0
5	0.4894	0	0	0
6	0.3609	16.02	49.67	0
7	0.3214	0	0	0.24
8	0.3168	7.30	30.02	0
9	0.1644	0.01	7.00	0
10	0.1482	0	0	0
11	0.1115	0	0	1.00
12	0.0707	0	0	0
13	0.0613	0	0	0.10
14	0.0506	0	0	0
15	0.0359	0	0	0
16	0.0305	0	0.03	0
17	0.0289	0	0.05	0
18	0.0271	0	2.69	0
19	0.0259	0	0	0
20	0.0246	0	0	0
21	0.0243	0	0.17	0
22	0.0237	0	0.06	0
23	0.0230	0	0	0
24	0.0225	0	0.03	0
25	0.0205	0	0	0
		100.00	99.23	100.00

SOUTHERN ILLINOIS UNIVERSITY EDWARDSVILLE **Civil Engineering** www.siue.edu/civil SIUE, Edwardsville, IL 62025-1800 (618)650-2533 Fax: (618)650-2555	SUBJECT FILE: **Evaluation of Comprehensive Seismic Design of Bridges (LRFD) in Illinois**	BY: **G.V.I.**
		CHECKED: **W.N.M.**
		DATE: **6/24/2004**
TITLE: **St. Clair County Bridge**	**AASHTO Standard Specifications**	Sheet No: **8** of **33**

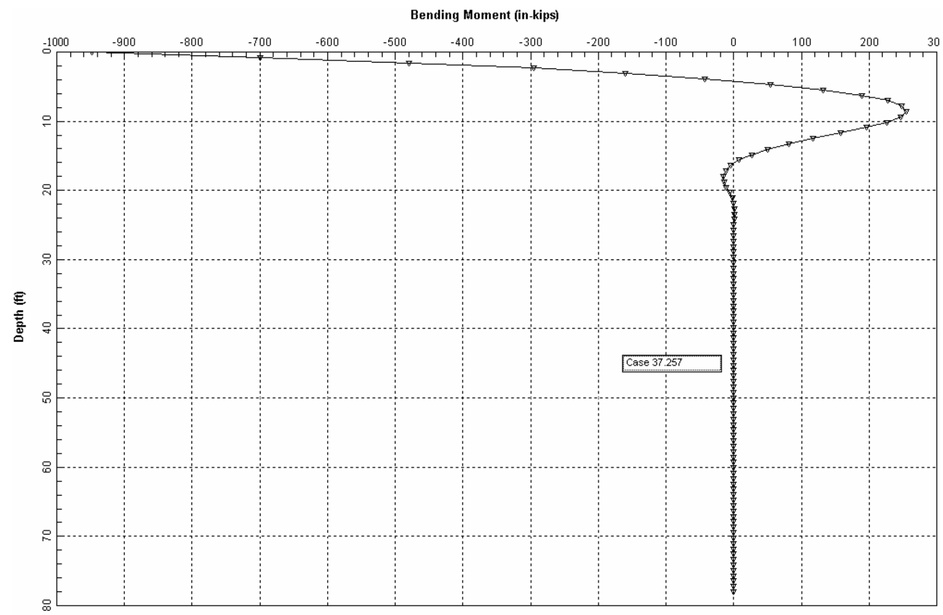
Piles (L-pile results)

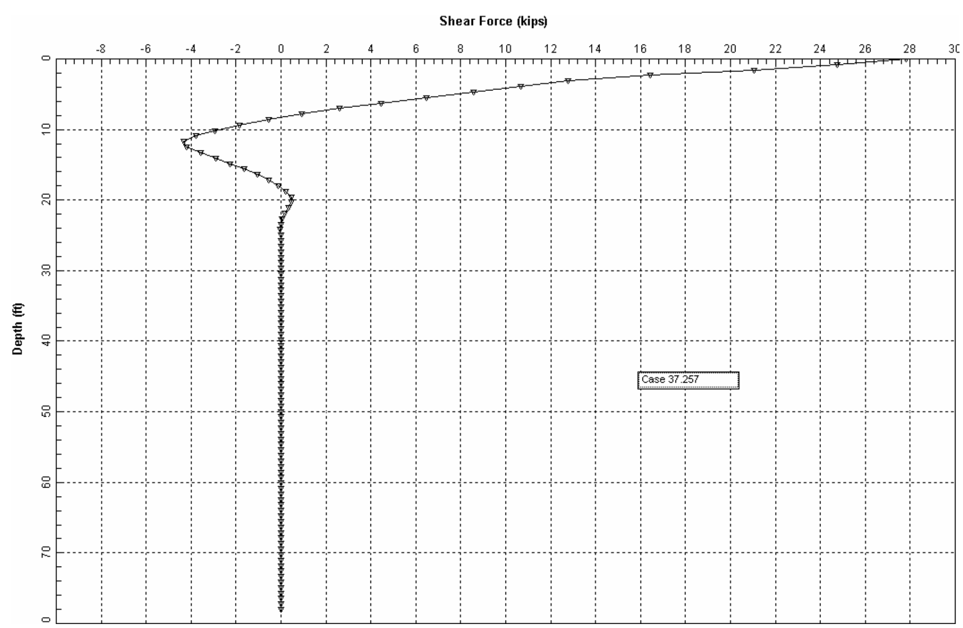
Longitudinal direction of the bridge:





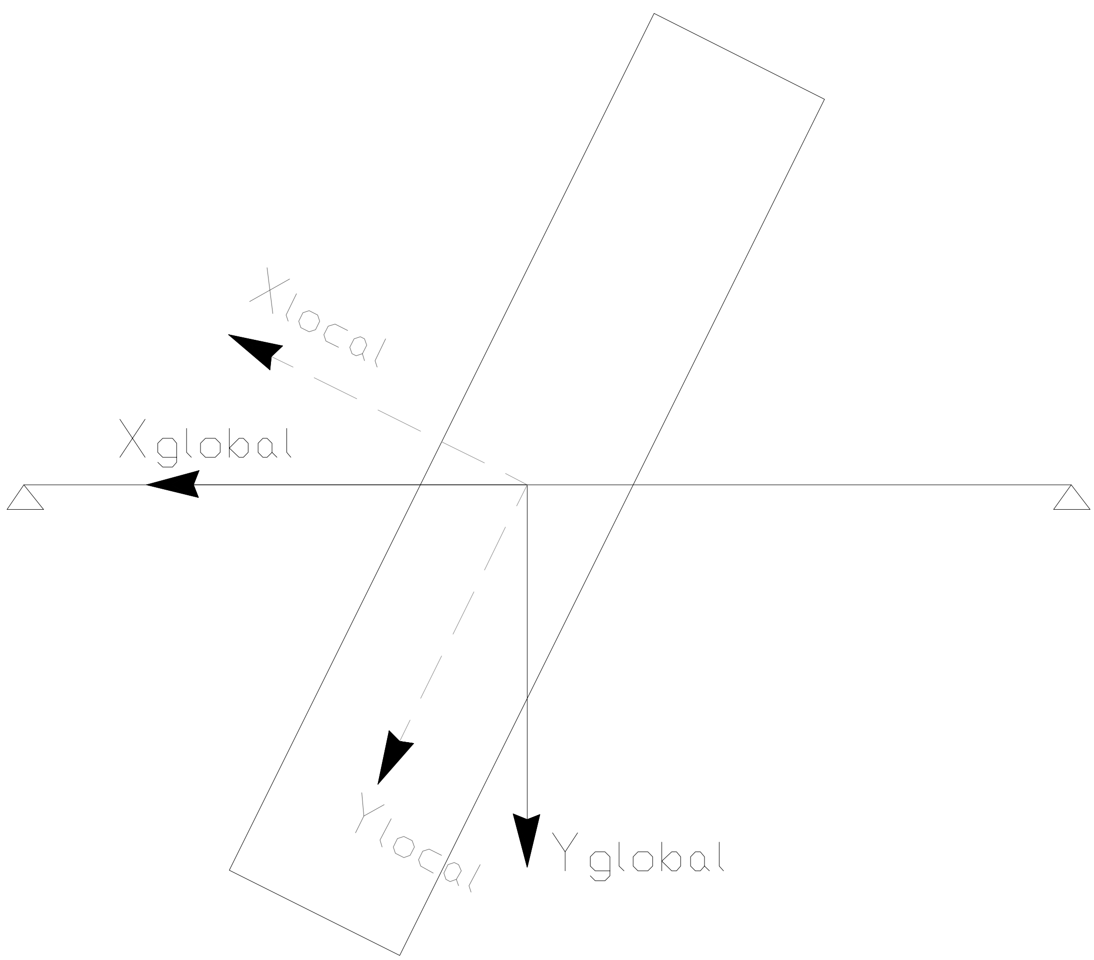
Transverse direction of the bridge:





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 :

$\phi := 26.528$ degrees — Skew of the bridge

$R_{sc} := 5$ — Response Modification Factor for a pile bent:

$R_r := R_{sc} \cdot 0.5$ — Reduced Response Modification Factor (AASHTO article 6.6.2)

$M_{Yglob} := \dfrac{21913}{R_r}$ $\quad M_{Yglob} = 8.765 \times 10^3$ k – ft $\quad$ Longitudinal

$M_{YlocalY} := M_{Yglob} \cdot \sin(\phi)$ $\quad M_{YlocalY} = 8.631 \times 10^3$ k – ft

$M_{Xglob} := \dfrac{5531}{R_r}$ $\quad M_{Xglob} = 2.212 \times 10^3$ k – ft $\quad$ Transverse

$M_{XlocaY} := M_{Yglob} \cdot \cos(\phi)$ $\quad M_{XlocaY} = 1.531 \times 10^3$ k – ft

$M_{XlocalX} := M_{Xglob} \cdot \sin(\phi)$

$M_{YlocaX} := M_{Xglob} \cdot \cos(\phi)$

$M_{eql} := M_{YlocalY} + M_{YlocaX}$ $\quad M_{eql} = 9.017 \times 10^3$ k – ft $\quad$ longitudinal

$M_{eqt} := M_{XlocaY} + M_{XlocalX}$ $\quad M_{eqt} = 3.709 \times 10^3$ k – ft $\quad$ transverse

$P_W := 3291$ kips

The distance from center line of foundation to center of pile (ft):

$d1 := 3.281$ $\quad d2 := 3.002$ $\quad d3 := 9.006$ $\quad d4 := 14.747$ $\quad d5 := 20.488$ $\quad d6 := 26.229$ $\quad d7 := 31.97$

Sum of squares of the distances to each pile from the center line of the foundation:

$\Sigma d1 := 24 \cdot \left(d1^2\right)$ $\qquad \Sigma d1 = 258.359 \ ft^2$ (longitudinal)

$\Sigma d2 := 6 \cdot \left(d2^2\right) + 6 \cdot \left(d3^2\right) + 6 \cdot \left(d4^2\right) + 6 \cdot \left(d5^2\right) + 6 \cdot \left(d6^2\right) + 6 \cdot \left(d7^2\right)$

$\Sigma d2 = 1.462 \times 10^4 \ ft^2$ (transverse)

Piles Loads:

$F_{pile1} := \left(\frac{P_w}{36}\right) + \left(M_{eql} \cdot \frac{d1}{\Sigma d1}\right) + \left(M_{eqt} \cdot \frac{d7}{\Sigma d2}\right)$ $\qquad F_{pile1} = 214$ kips

$F_{pile12} := \left(\frac{P_w}{36}\right) + \left(M_{eql} \cdot \frac{d1}{\Sigma d1}\right) - \left(M_{eqt} \cdot \frac{d7}{\Sigma d2}\right)$ $\qquad F_{pile12} = 197.82$ kips

$F_{pile25} := \left(\frac{P_w}{36}\right) - \left(M_{eql} \cdot \frac{d1}{\Sigma d1}\right) + \left(M_{eqt} \cdot \frac{d7}{\Sigma d2}\right)$ $\qquad F_{pile25} = -14.98$ kips

$F_{pile36} := \left(\frac{P_w}{36}\right) - \left(M_{eql} \cdot \frac{d1}{\Sigma d1}\right) - \left(M_{eqt} \cdot \frac{d7}{\Sigma d2}\right)$ $\qquad F_{pile36} = -31.2$ kips

*Note: Q.ult = Q.s + Q.t - W.self (Compression) and .7*Q.s + W (uplift);*
Q.allow = Q.ult / FS

Forces from SAP2000 model: COMB 1: F(longitud.) = 1367.04 kips, F(transverse) = 1327.71 kips,
M(longitud.) = 5530.71 k-ft, M(transverse) = 21912.78 k-ft

COMB 2: F(longitud.) = 1148.89 kips, F(transverse) = 1021.76 kips,
M(longitud.) = 2988.43 k-ft, M(transverse) = 11482.57 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

$M_{px} := F_y \cdot Z_x$

$M_{px} = 2.664 \times 10^3$ $k \cdot in$ full plastic moment about X-axis

$M_{py} := F_y \cdot Z_y$

$M_{py} = 1.159 \times 10^3$ $k \cdot in$ full plastic moment about Y-axis

$P_{pile} := 214$ k axial load at top of pile

LPile output (local coordinates):

$M_{xlpile} := 1623$ $k \cdot in$ moment about X-axis(single pile)

$M_{ylpile} := 948.2$ $k \cdot in$ moment about Y-axis (single pile)

$V_{xlpile} := 35.7$ k longitudinal lateral load (single pile)

$V_{ylpile} := 27.8$ k transverse lateral load (single pile)

Capacity of the piles (deflection limited to 0.5in):

$n_p := 36$ number of piles

$V_{Xfull} := V_{xlpile} \cdot n_p$

$V_{Xfull} = 1.285 \times 10^3$ k longitudinal direction

$V_{Yfull} := V_{ylpile} \cdot n_p$

$V_{Yfull} = 1.001 \times 10^3$ k transverse direction

Maximum Loads (SAP 2000 Model output):

$\phi := 26.528$ degrees skew of the bridge

$V_{Xglob} := 1367$ k

$V_{Yglob} := 1327$ k

Converting loads from Global to Local coordinate system:

$V_{XlocalY} := V_{Yglob} \cdot \cos(\phi)$ $V_{XlocalY} = 231.744$ k X component (local coord. syst.) of transverse load in global coord. syst.

$V_{YlocalY} := V_{Yglob} \cdot \sin(\phi)$ $V_{YlocalY} = 1.307 \times 10^3$ k Y component (local coord. syst.) of transverse load in global coord. syst.

$V_{XlocalX} := V_{Xglob} \cdot \sin(\phi)$ $V_{XlocalX} = 1.346 \times 10^3$ k X component (local coord. syst.) of longitudinal load in global coord. syst.

$V_{YlocalX} := V_{Xglob} \cdot \cos(\phi)$ $V_{YlocalX} = 238.729$ k Y component (local coord. syst.) of longitudinal load in global coord. syst.

$V_{Xmax} := V_{XlocalY} + V_{XlocalX}$ $V_{Xmax} = 1.578 \times 10^3$ k transverse load (local coordinates)

$V_{Ymax} := V_{YlocalY} + V_{YlocalX}$ $V_{Ymax} = 1.545 \times 10^3$ k longitudinal load (local coordinates)

$R_p := 5$ * 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:

$$M_x := \frac{M_{xlpile} \cdot \left(\frac{V_{Xmax}}{V_{Xfull}} \right)}{R_{pm}} \qquad M_x = 796.971 \quad k \cdot in$$

$$M_y := \frac{M_{ylpile} \cdot \left(\frac{V_{Ymax}}{V_{Yfull}} \right)}{R_{pm}} \qquad M_y = 585.647 \quad k \cdot in$$

$$\left(\frac{P_{pile}}{0.85 \cdot A_{pile} \cdot F_y} \right) + \frac{8}{9} \cdot \left[\left(\frac{M_x}{M_{px}} \right) + \left(\frac{M_y}{M_{py}} \right) \right] = 1.166$$

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.

Pile Information for L-Pile input:

$ft := 12in$

$k := 1000lb$

$m = 3.281\,length$

HP 12x53 - piles

$L_{pile} := 936 \cdot in$ $\qquad L_{pile} = 78\,ft$

$A_{pile} := 15.5 \cdot in^2$ $\qquad A_{pile} = 0.108\,ft^2$

$E_S := 29000000 \cdot \frac{lb}{in^2}$ $\qquad E_S = 4176000000\,\frac{lb}{ft^2}$

$\Delta := 0.5 \cdot in$ top of pile displacement limit.

Results from L-Pile (displacement is limited to 0.5 in):

$M_{utrans} := 9.482 \cdot 10^5 lb \cdot in$ $\qquad M_{utrans} = 7.902 \times 10^4\,lb \cdot ft$ Transverse

$V_{utrans} := 2.7803 \cdot 10^4 lb$ $\qquad V_{utrans} = 2.78 \times 10^4\,lb$ Transverse

$M_{ulong} := 1.623 \cdot 10^6 lb \cdot in$ $\qquad M_{ulong} = 1.353 \times 10^5\,lb \cdot ft$ Longitudinal

$V_{ulong} := 3.5646 \cdot 10^4 lb$ $\qquad V_{ulong} = 3.565 \times 10^4\,lb$ Longitudinal

Spring Stiffness:

$k_{axial} := A_{pile} \cdot \frac{E_S}{L_{pile}}$ $\qquad k_{axial} = 5.763 \times 10^6\,\frac{lb}{ft}$ Axial

$k_{long} := \frac{V_{ulong}}{\Delta}$ $\qquad k_{long} = 8.555 \times 10^5\,\frac{lb}{ft}$ Longitudinal

$k_{trans} := \frac{V_{utrans}}{\Delta}$ $\qquad k_{trans} = 6.673 \times 10^5\,\frac{lb}{ft}$ Transverse

Lateral Springs:

$k_{longB} := 173.8 \cdot 10^3 \cdot \frac{lb}{ft}$ for battered pile

$K_{long} := 24 \cdot (k_{longB} + k_{long}) + 12 k_{long}$ $K_1 := 3.497 \times 10^4 \cdot \frac{k}{ft}$

$K_{trans} := 36 \cdot k_{trans}$ $K_2 := 2.402 \times 10^4 \cdot \frac{k}{ft}$

Axial Springs:

$K_{axial} := 36 \cdot k_{axial}$ $K_3 := 2.075 \times 10^5 \cdot \frac{k}{ft}$

Rotational Springs:

Distance between piles (transverse)	Distance between footing C.L. and piles C.L.	Distance between piles (longitudinal)		Distance from center of footing to center of piles
$l_1 := 3.002 \cdot ft$	$d_1 := 3.002 \cdot ft$	$w := 3.28 \cdot ft$	$n_1 := \sqrt{d_1^2 + w^2}$	$n_1 = 4.446\,ft$
$l_2 := 6.004 \cdot ft$	$d_2 := 9.006 \cdot ft$		$n_2 := \sqrt{d_2^2 + w^2}$	$n_2 = 9.585\,ft$
$l_3 := 5.741 \cdot ft$	$d_3 := 14.747 \cdot ft$		$n_3 := \sqrt{d_3^2 + w^2}$	$n_3 = 15.107\,ft$
$l_4 := 5.741 \cdot ft$	$d_4 := 20.488 \cdot ft$		$n_4 := \sqrt{d_4^2 + w^2}$	$n_4 = 20.749\,ft$
$l_5 := 5.741 \cdot ft$	$d_5 := 26.229 \cdot ft$		$n_5 := \sqrt{d_5^2 + w^2}$	$n_5 = 26.433\,ft$
$l_6 := 5.741 \cdot ft$	$d_6 := 31.97 \cdot ft$		$n_6 := \sqrt{d_6^2 + w^2}$	$n_6 = 32.138\,ft$

$$K_{rx} := k_{axial} \cdot 6 \cdot \left[(d_1)^2 + (d_2)^2 + (d_3)^2 + (d_4)^2 + (d_5)^2 + (d_6)^2 \right]$$

$$K_{r1} := 8.428 \times 10^7 \cdot \frac{k}{ft}$$ about X axis

$K_{ry} := k_{axial} \cdot 33 \cdot 3.28^2$ $\qquad K_{ry} = 2.046 \times 10^9\, mass\, length^{-1}$

$$K_{r2} := 24.55 \times 10^6 \cdot \frac{k}{ft}$$ about Y axis

$$K_{rz} := k_{long} \cdot \left[4 \cdot \left[(n_1)^2 + (n_2)^2 + (n_3)^2 + (n_4)^2 + (n_5)^2 + (n_6)^2 \right] \ldots \\ + 2 \cdot \left[(d_1)^2 + (d_2)^2 + (d_3)^2 + (d_4)^2 + (d_5)^2 + (d_6)^2 \right] \right]$$

$$K_{r3} := 1.273 \times 10^7 \cdot \frac{k}{ft}$$ about Z axis

Substructure

Supports (both abutments): Longitudinal - Roller ; Transverse - Pin

	Forces and Moments - Load Combination 1 - (1.0*EQL + 0.3*EQT)								
	Longitudinal (Weak Direction of Pier)				Transverse (Strong Direction of Pier)				Axial
Support/Location	Shear X (kips) V3		Moment Y (kip-ft) M2		Shear Y (kips) V2		Moment X (kip-ft) M3		(kips)
Top of Cap Beam									
1(external)	80.00	26.67	285.00	95.00	150.00	30.00	534.00	106.80	23.00
3(middle)	199.00	66.33	708.00	236.00	172.00	34.40	613.00	122.60	0.00
Top of column									
1(external)	127.00	42.33	1041.00	347.00	174.00	34.80	754.00	150.80	168.00
3(middle)	166.00	55.33	1521.00	507.00	174.00	34.80	754.00	150.80	0.00
Bottom of column									
1(external)	127.00	42.33	515.00	171.67	175.00	35.00	2903.00	580.60	163.00
3(middle)	166.00	55.33	518.00	172.67	175.00	35.00	2903.00	580.60	0.00
Bottom of wall									
1(external)	127.00	42.33	1244.00	414.67	174.00	34.80	3913.00	782.60	161.00
3(middle)	167.00	55.67	1478.00	492.67	175.00	35.00	3912.00	782.40	0.00

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)

	Forces and Moments - Load Combination 2 - (0.3*EQL + 1.0*EQT)								
	Longitudinal (Weak Direction of Pier)				Transverse (Strong Direction of Pier)				Axial
Support/Location	Shear X (kips) V3		Moment Y (kip-ft) M2		Shear Y (kips) V2		Moment X (kip-ft) M3		(kips)
Top of Cap Beam									
1(external)	46.00	15.33	162.00	54.00	82.00	16.40	292.00	58.40	14.00
3(middle)	175.00	58.33	620.00	206.67	123.00	24.60	437.00	87.40	0.00
Top of column									
1(external)	120.00	40.00	970.00	323.33	95.00	19.00	410.00	82.00	157.00
3(middle)	157.00	52.33	1422.00	474.00	95.00	19.00	410.00	82.00	0.00
Bottom of column									
1(external)	177.00	59.00	483.00	161.00	93.00	18.60	1560.00	312.00	153.00
3(middle)	155.00	51.67	489.00	163.00	93.00	18.60	1560.00	312.00	0.00
Bottom of wall									
1(external)	116.00	38.67	1155.00	385.00	92.00	18.40	2090.00	418.00	151.00
3(middle)	154.00	51.33	1376.00	458.67	92.00	18.40	2089.00	417.80	0.00

	Forces and Moments - Dead Loads				
	Longitudinal (Weak Direction of Pier)		Transverse (Strong Direction of Pier)		Axial
Support/Location	Shear X (kips)	Moment Y (kip-ft)	Shear Y (kips)	Moment X (kip-ft)	(kips)
Top of column					
1	0.17	3.23	0	0	516
3	0.17	3.23	0	0	516
Bottom of column					
1	0.9	4.5	0	0	542
3	0	0	0	0	542
Bottom of wall	0.9	0.7	0	0	551

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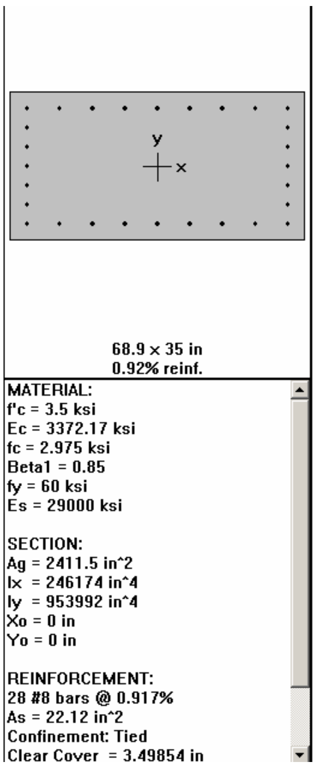
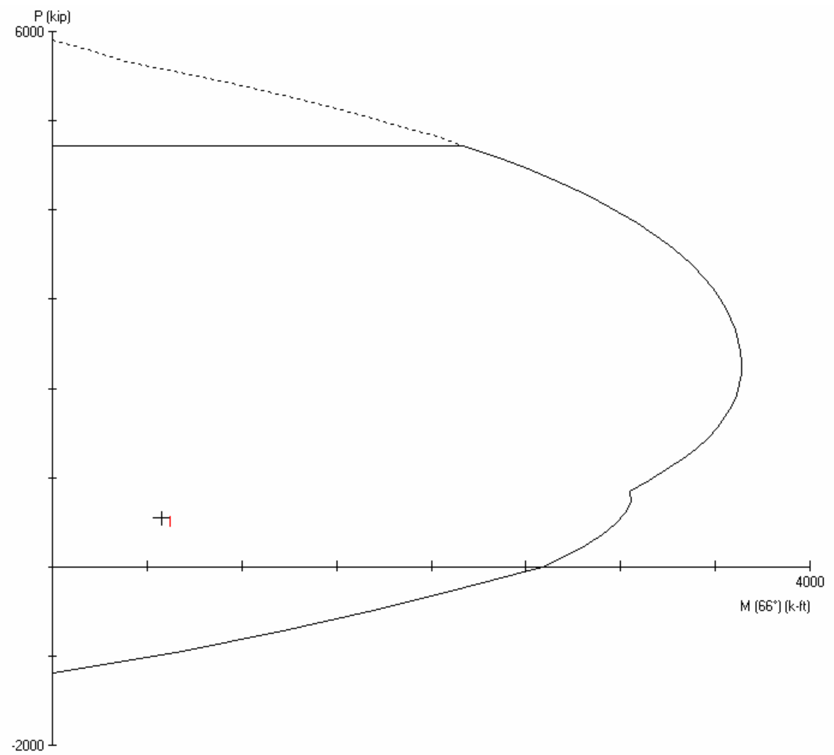
TABLE: Joint Displacements									
Joint	**OutputCase**	**CaseType**	**StepType**	**U1**	**U2**	**U3**	**R1**	**R2**	**R3**
Text	Text	Text	Text	in	in	in	Radians	Radians	Radians
1	COMB1	Combination	Max	0.4749	0.5925	-0.6920	0.0037	0.0003	0.0000
1	COMB2	Combination	Max	0.3950	0.4745	-1.2310	0.0020	0.0001	0.0000
2	COMB1	Combination	Max	0.4749	0.5923	-0.8226	0.0037	0.0003	0.0000
2	COMB2	Combination	Max	0.3950	0.4744	-1.3017	0.0020	0.0001	0.0000
3	COMB1	Combination	Max	0.4749	0.5922	-0.9536	0.0037	0.0003	0.0000
3	COMB2	Combination	Max	0.3950	0.4744	-1.3727	0.0020	0.0001	0.0000
4	COMB1	Combination	Max	0.4749	0.5921	-1.0841	0.0037	0.0003	0.0000
4	COMB2	Combination	Max	0.3950	0.4743	-1.4433	0.0020	0.0001	0.0000
5	COMB1	Combination	Max	0.4749	0.5920	-1.3435	0.0037	0.0003	0.0000
5	COMB2	Combination	Max	0.3950	0.4743	-1.5830	0.0020	0.0001	0.0000

TABLE: Base Reactions									
Number	**OutputCase**	**CaseType**	**StepType**	**GlobalFX**	**GlobalFY**	**GlobalFZ**	**GlobalMX**	**GlobalMY**	**GlobalMZ**
	Text	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
	DEAD	LinStatic		0	0	4545	0	0	0
	COMB1	Combination	Max	1393	1183	4545	30459	27144	0
	COMB2	Combination	Max	1076	1750	4545	31323	13564	0

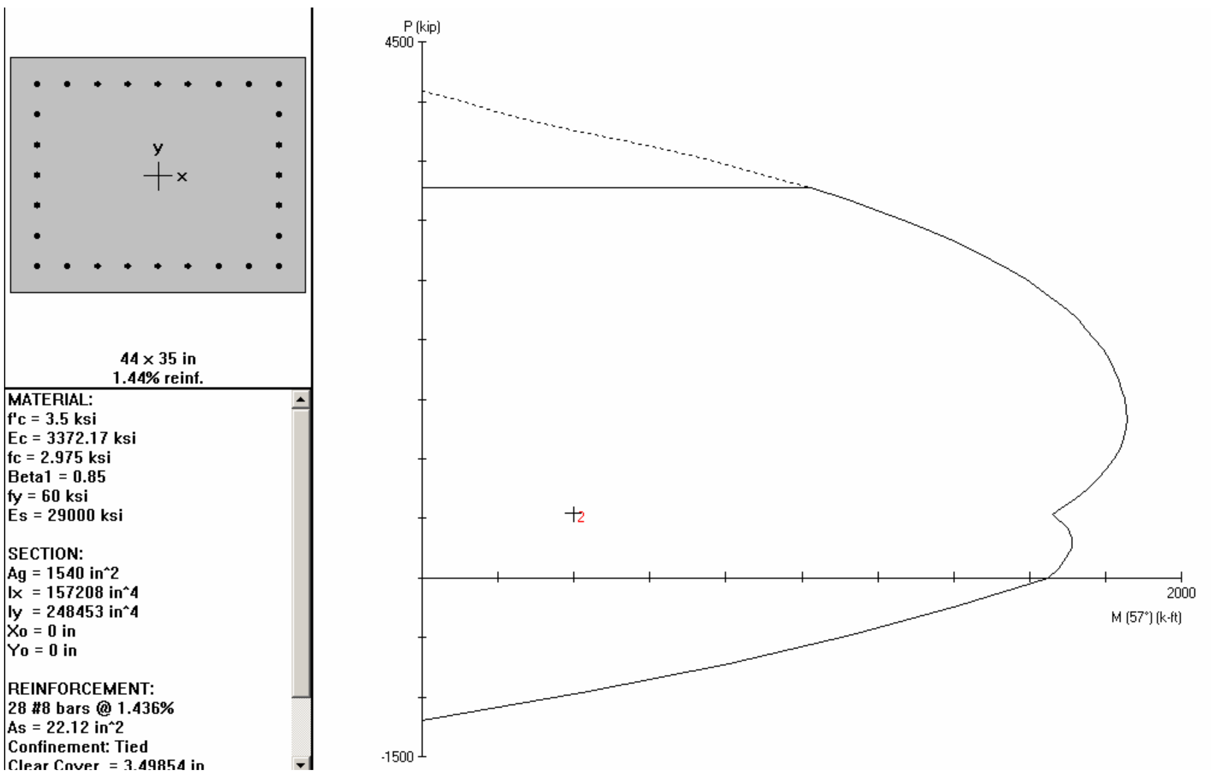
TABLE: Joint Reactions - Spring Forces									
Joint	**OutputCase**	**CaseType**	**StepType**	**U1**	**U2**	**U3**	**R1**	**R2**	**R3**
Text	Text	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
J3	DEAD	LinStatic		0	0	3225	0	0	0
J3	COMB1	Combination	Max	1367	1328	3225	5531	21913	0
J3	COMB2	Combination	Max	1149	1022	3225	2988	11483	0

Top of a typical column

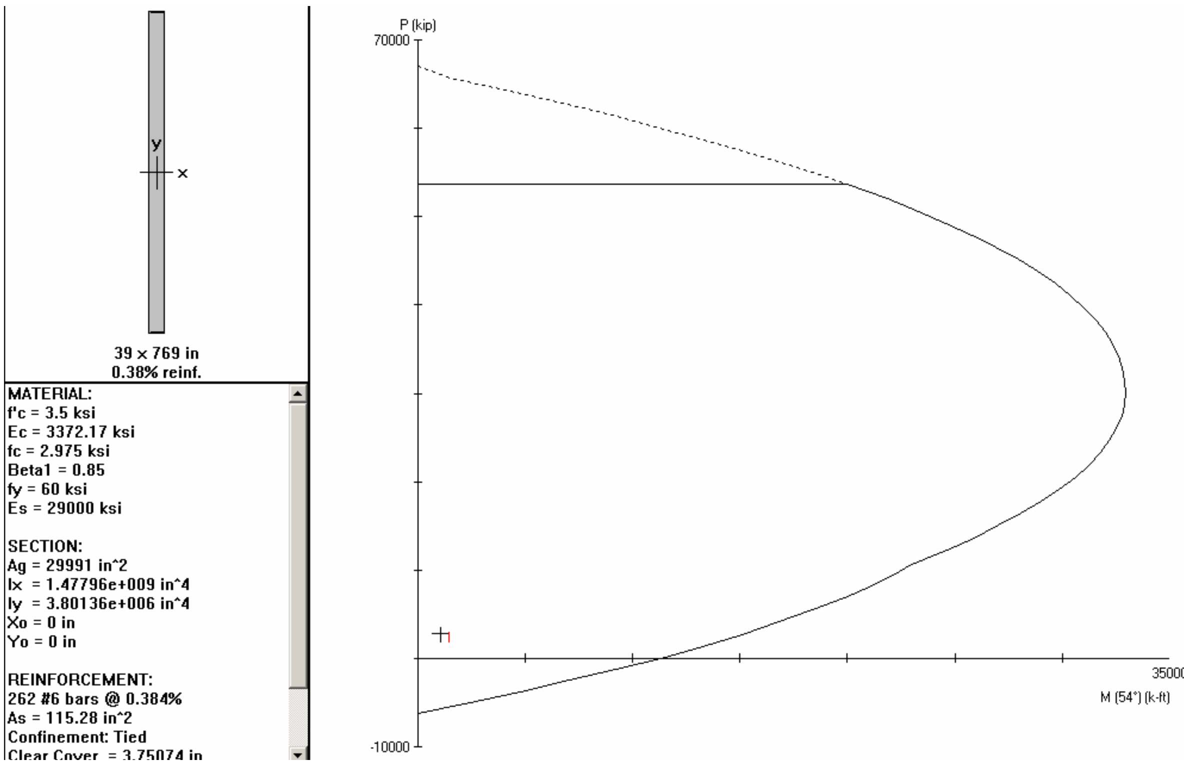




Bottom of a typical column



Bottom of Crash Wall



Section Properties of Superstructure

Pertinent Information about Materials used, etc. :

$pconv := \left(1.45 \times 10^{-4}\right)$ **N/m^2 to psi conversion factor.**

$feet := \frac{1}{.3048}$ $feet = 3.281$ **meter to feet conversion.**

$inch := feet \cdot 12$ $inch = 39.37$ **meter to inches conversion.**

$f_c := 24 \cdot \left(10^6\right) \cdot pconv$ $f_c = 3480.000$

$f_y := 400 \cdot \left(10^6\right) \cdot pconv$ $f_y = 5.8 \times 10^4$ (Reinforcement)(psi)

$F_{y1} := 345\left(10^6\right) \cdot pconv$ $F_{y1} = 5.003 \times 10^4$ (Structural)

$F_{y2} := 250 \cdot \left(10^6\right) \cdot pconv$ $F_{y2} = 3.625 \times 10^4$ (Structural)

$E_c := 57000 \cdot \left(\sqrt{f_c}\right)$ $E_c = 3.363 \times 10^6$ **Modulus of Elasticity of concrete.**

$E_s := 29000000$ $E_s = 2.9 \times 10^7$ **Modulus of Elasticity of steel**.

$n := \frac{E_s}{E_c}$ $n = 8.624$ **Modular Ratio**

$boltD := 24 \cdot \left(10^{-3}\right) \cdot inch$ $boltD = 0.945$ **Bolt Diameter (in.)**

$Skew := 26 + \left(\frac{31}{60}\right) + \left(\frac{40}{3600}\right)$ $Skew = 26.528$ **Skew Angle (degrees)**

Calculations for Moment of Inertia for Superstructure

$L_{eff} := 41.50 \cdot inch$	$L_{eff} = 1.634 \times 10^3$	Effective length of a span (in)
$t_{avg} := .195 \cdot inch$	$t_{avg} = 7.677$	Slab thickness (in)
$t_w := .014 \cdot inch$	$t_w = 0.551$	Web thickness (in)
$b_{f1} := .300 \cdot inch$	$b_{f1} = 11.811$	Top flange width outerline (in)
$b_{f2} := .600 \cdot inch$	$b_{f2} = 23.622$	Top flange width innerline (in)
$gspace := 1.905 \cdot inch$	$gspace = 75$	Girderline spacing (in)

Effective Flange Width of interior maybe be taken as least of (in inches):

$efw_1 := .25 \cdot L_{eff}$	$efw_1 = 408.465$	
$efw_2 := (12 \cdot t_{avg}) + t_w$	$efw_2 = 92.677$	
$efw_3 := (12 t_{avg}) + (.5 \cdot b_{f1})$	$efw_3 = 98.031$	(outer)
$efw_{3b} := (12 \cdot t_{avg}) + (.5 \cdot b_{f2})$	$efw_{3b} = 103.937$	(inner)
$efw_4 := gspace$	$efw_4 = 75$	**This controls**

$EFW_{inside} := gspace$ $\quad EFW_{inside} = 75$

Effective Flange Width of exterior maybe be taken as .5 EFW plus least of (in inches):

$efw_5 := \frac{1}{8} \cdot L_{eff}$	$efw_5 = 204.232$	
$efw_6 := (6 \cdot t_{avg}) + (.5 \cdot t_w)$	$efw_6 = 46.339$	
$efw_7 := (6 \cdot t_{avg}) + (.25 \cdot b_{f1})$	$efw_7 = 49.016$	
$efw_8 := (6 \cdot t_{avg}) + (.25 \cdot b_{f2})$	$efw_8 = 51.969$	
$ohw := .925 \cdot inch$	$ohw = 36.417$	**This controls**

$EFW_{outside} := (.5 \cdot EFW_{inside}) + ohw$ $\quad EFW_{outside} = 73.917$

We can use all of the slab for moment of inertia.

$I_{yy1} := 36.2084$ $\qquad I_{zz1} := 3676.5792$

$I_{yy2} := 61.4485$ $\qquad I_{zz2} := 5343.1700$

Torsional Properties:

We assume only the deck will resist torsion.

$b_{deck} := 20.8 \cdot feet$ $\qquad b_{deck} = 68.241$ ft

$h_{deck} := .195 \cdot feet$ $\qquad h_{deck} = 0.64$ ft

$J := (b_{deck}) \cdot \frac{(h_{deck})^3}{3}$ $\qquad J = 5.956$ ft^4

Cross-Sectional Area of Superstructure:

$b_{slab} := 20.8 \cdot \frac{feet}{n}$

$b_{slab} = 7.913$ ft

$h_{slab} := .195 \cdot feet$

$h_{slab} = 0.64$ ft $\qquad area_{slab} := h_{slab} \cdot b_{slab}$ $\qquad area_{slab} = 5.062$ ft^2

$area_{girder1} := 11 \cdot [(.02 \cdot .3) + (.02 \cdot .6) + (1.372 \cdot .014)] \cdot feet \cdot feet$ $\qquad area_{girder1} = 4.406$ ft^2

$area_{girder2} := 11 \cdot [(.045 \cdot .6) + (.045 \cdot .6) + (1.372 \cdot .014)] \cdot feet \cdot feet$ $\qquad area_{girder2} = 8.668$ ft^2

$area_{super1} := (area_{slab} + area_{girder1})$ $\qquad area_{super1} = 9.468$ ft^2

$area_{super2} := (area_{slab} + area_{girder2})$ $\qquad area_{super2} = 13.73$ ft^2

$area_{barrier} := [(.2 \cdot .535) + (.065 \cdot .535) + (.320 \cdot .330) + (.5 \cdot .180 \cdot .330)] \cdot feet \cdot feet$

$area_{median} := \left[.150 \cdot \left[\frac{(1.3 + 1.236)}{2} \right] \right] \cdot feet \cdot feet$

$area_{barrier} = 2.982$ ft^2

$area_{median} = 2.047$ ft^2

Loads :

Dead Loads of the superstructure

$w_{barrier} := 2 \cdot area_{barrier} \cdot 150$ — $w_{barrier} = 895$ plf

$w_{slab} := (19.05 \cdot feet) \cdot (.195 \cdot feet) \cdot 150$ — $w_{slab} = 5998$ plf

$w_{girder1} := 11[(1.372 \cdot feet) \cdot (.014 \cdot feet) + (.020 \cdot feet) \cdot (.900 \cdot feet)] \cdot 490$ — $w_{girder1} = 2159$ plf

$w_{girder2} := 11[(1.372 \cdot feet)(.014 \cdot feet) + (.045 \cdot feet) \cdot (1.200 \cdot feet)] \cdot 490$ — $w_{girder2} = 4247$ plf

$w_{median} := area_{median} \cdot .190$ — $w_{median} = 0.389$ 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 13 cross frames per length total number = 130

$w_{cframe} := 13 \cdot 10 \cdot [(2.546 + 2.546 + 1.904) \cdot feet] \cdot \frac{8.16}{272}$ $w_{cframe} = 89.516$ plf

Future Wearing Surface was 2.4 kN/m^2 or about 50 psf.

$w_{bridge} := (3 + 4.2 + .3 + 3.9 + 4.2 + 3) \cdot feet$ — $w_{bridge} = 61.024$ ft

$w_{fws} := 2.4 \cdot 1000 \cdot pconv \cdot 144 \cdot w_{bridge}$ — $w_{fws} = 3.058 \times 10^3$ plf

Adding the individual pieces together we get:

$w_{outer} := w_{barrier} + w_{median} + w_{cframe} + w_{slab} + w_{girder1} + w_{fws}$ $\quad w_{outer} = 12199$ plf

$w_{inner} := w_{barrier} + w_{median} + w_{cframe} + w_{slab} + w_{girder2} + w_{fws}$ $\quad w_{inner} = 14288$ 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 girder line which are the continuous part over the central pier.

Pier Column Dimensions and properties:

$b_{coltop} := 1.75 \cdot feet$ $\quad b_{coltop} = 5.741$ ft

$b_{colbot} := 1.125 \cdot feet$ $\quad b_{colbot} = 3.691$ ft

$l_{col} := 3.75 \cdot feet$ $\quad l_{col} = 12.303$ ft

$w_{col} := .90 \cdot feet$ $\quad w_{col} = 2.953$ ft

number of column increments used: $incr_{col} := 5$

length of column increments: $l_{incr} := \dfrac{l_{col}}{incr_{col}}$ $\quad l_{incr} = 2.461$ ft

Dimensions used to model:

column1 .. ∎ $\quad b_{col1} := 1.75 \cdot feet$ $\quad b_{col1} = 5.741$ ft

column2 .. ∎ $\quad b_{col2} := 1.625 \cdot feet$ $\quad b_{col2} = 5.331$ ft

column3 .. ∎ $\quad b_{col3} := 1.5 \cdot feet$ $\quad b_{col3} = 4.921$ ft

column4 .. ∎ $\quad b_{col4} := 1.375 \cdot feet$ $\quad b_{col4} = 4.511$ ft

column5 .. ∎ $\quad b_{col5} := 1.25 \cdot feet$ $\quad b_{col5} = 4.101$ ft

column6 .. ∎ $\quad b_{col6} := 1.125 \cdot feet$ $\quad b_{col6} = 3.691$ ft

All columns are a width of 2.953 ft over their entire length.

Columns designed as non-prismatic sections in SAP-2000.

Pier Cap Dimensions and properties:

$$y_{bar} := \frac{\left[\left(23.36\cdot.984\cdot\frac{.984}{2}\right) + 2\cdot(.5\cdot1.905\cdot.3)\cdot(.984 + .1) + (19.55\cdot.3)\cdot\left(.984 + \frac{.3}{2}\right)\right]\cdot\text{feet}}{[(23.36\cdot.984) + 2\cdot(.5\cdot1.905\cdot.3) + (19.55\cdot.3)]}$$

$y_{bar} = 2.072$ ft Measured from the top of the cap.

Section taken in middle of cap:

$b_{cap} := 1\cdot\text{feet}$ $b_{cap} = 3.281$ ft

$h_{cap} := \text{feet}(.984 + .300)$ $h_{cap} = 4.213$ ft

Section taken at end of cap:

$h_{capend} := .984\cdot\text{feet}$ $h_{capend} = 3.228$ ft

Width is constant across the cap.

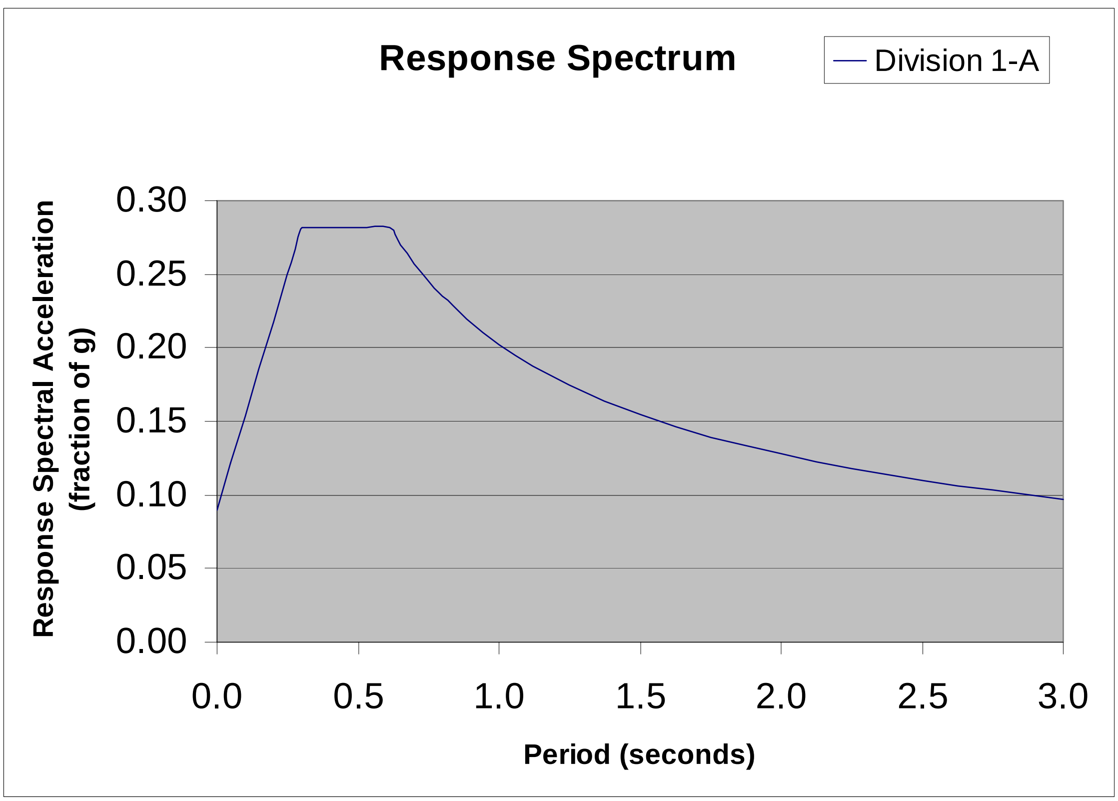
Saint Clair County Bridge - AASHTO Division 1-A - Design Checks

Earthquake Data:

Seismic Performance Category (SPC) is B
Bedrock Acceleration Coefficient (A) is .1125g $\quad a := .1125$
Site Coefficient is 1.5 $\quad sc := 1.5$

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)

$V_c := 2 \cdot \left(1 + \frac{593000}{2000 \cdot 44 \cdot 35}\right) \cdot \sqrt{3500} \cdot 44 \cdot 32$ $\quad V_c = 198672$ lbs $\quad .85 \cdot V_c = 1.689 \times 10^5$

$V_{c2} := 2 \cdot \sqrt{3500} \cdot 44 \cdot 32$ $\quad V_{c2} = 1.666 \times 10^5$ lbs

Use value of 168 kips since it takes into account the axial load that is present.

$v_{col1} := \sqrt{53^2 + 29^2}$ $\quad v_{col1} = 60.415$ kips

$v_{col2} := \sqrt{26^2 + 24.5^2}$ $\quad v_{col2} = 35.725$ kips

Applied shear is less than 2x V.c, thus column satisfies requirements for shear with only minimum reinforcement

Shear - Crashwall

$V_{c3} := 2\cdot\sqrt{3500}\cdot 769\cdot 36.4$ $\quad V_{c3} = 3.312\times 10^{6}$ lbs

$.85\cdot V_{c3} = 2.815\times 10^{6}$ lbs

Use value of 2815 kips

$v_{wall1} := \sqrt{79.5^2 + 72.6^2}$ $\quad v_{wall1} = 107.66$ kips

$v_{wall2} := \sqrt{39^2 + 61^2}$ $\quad v_{wall2} = 72.402$ kips

Applied shear is less than 2x V.c, thus wall satisfies requirements for shear with only minimum reinforcement

Slenderness Effects (8.16.5)

$r := .3\cdot 35.4$ $\quad r = 10.62$ in $\quad k := 2$

$l_u := 147.6$ in $\quad \text{slenderness} := k\cdot\frac{l_u}{r}$ $\quad \text{slenderness} = 27.8$

$M_{1b} := -236$ k – ft

$M_{2b} := 471$ k – ft

$\beta_d := 1$ $\quad E_c := 1820\cdot\sqrt{3500}$ $\quad E_s := 29000$ $\quad C_m := .6 + .4\cdot\left(\frac{M_{1b}}{M_{2b}}\right)$

$I_g := \frac{1}{12}\cdot 44.3\cdot 35.4^3$ $\quad I_g = 1.638\times 10^{5}$ in^4 $\quad C_m = 0.4$

$$EI := \frac{\left(E_c\cdot\frac{I_g}{2.5}\right)}{(1+\beta_d)} \qquad EI = 3.527\times 10^{9}$$

$P_c := \frac{(\pi^2 \cdot EI)}{(2 \cdot l_u)^2}$ $\quad\quad$ $P_c = 3.994 \times 10^5$ k

$\phi := .9 - 2 \cdot \frac{567}{3.5 \cdot 44.3 \cdot 35.4}$ $\quad\quad$ $\phi = 0.693$ $\quad\quad$ *Use ϕ equals .70 (pure compression)*

$\phi \cdot P_c = 2.77 \times 10^5$ k

$\delta_b := \frac{.4}{1 - \frac{567}{276960}}$ $\quad\quad$ $\delta_b = 0.4$ $\quad\quad$ *Use δb equals 1.0*

$\delta_s := 1.0$ $\quad\quad$ *Use values obtain from SAP2000 with their respective R factors as the design moments*

AASHTO Division 1-A; Section 6; SPC (B); Checklist

6.3.1 $\quad$ $N := (203 + 1.67 \cdot 83 + 6.66 \cdot 6.72) \cdot (1 + .000125 \cdot 26.528 \cdot 26.528)$

$N = 420.353$ mm $\quad\quad$ $\frac{N}{25.4} = 16.549$ in

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 Computations for Seismic Analysis and Design of the Pulaski County Bridge using Proposed NCHRP Specification

Table of Contents of Appendix- E

Site Class = Medium Stiff Clay

Site Class D MCEER Guide Spec. 3.4.2.1

Type III per AASHTO Spec.

$s := 1.5$ AASHTO Table 3.5.1

$A := 0.22\ g$

- Design Response Spectrum Development - MCE

$S_s := 3.16$ 0.2-second period spectral acceleration

$S_1 := 0.919$ 1-second period spectral acceleration

$F_a := 1$ Site coefficient for short period (MCEER Table 3.4.2.3-1)

$F_v := 1.5$ Site coefficient for long period (MCEER Table 3.4.2.3-2)

$S_{DS} := F_a \cdot S_s$

$S_{DS} = 3.16$ Design earthquake response spectral acceleration at short period

$S_{D1} := F_v \cdot S_1$

$S_{D1} = 1.379$ Design earthquake response spectral acceleration at long period

$0.4 \cdot S_{DS} = 1.264$

$T_S := \dfrac{S_{D1}}{S_{DS}}$ Period at the end of construction design spectral acceleration plateau

$T_S = 0.436$ Sec

$T_0 := 0.2 \cdot T_S$ Period at the beginning of construction design spectral acceleration plateau

$T_0 = 0.087$ 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)

$S_{DS} := F_a \cdot S_s$

$S_{DS} = 0.164$ Design earthquake response spectral acceleration at short period

$S_{D1} := F_v \cdot S_1$

$S_{D1} = 0.048$ Design earthquake response spectral acceleration at long period

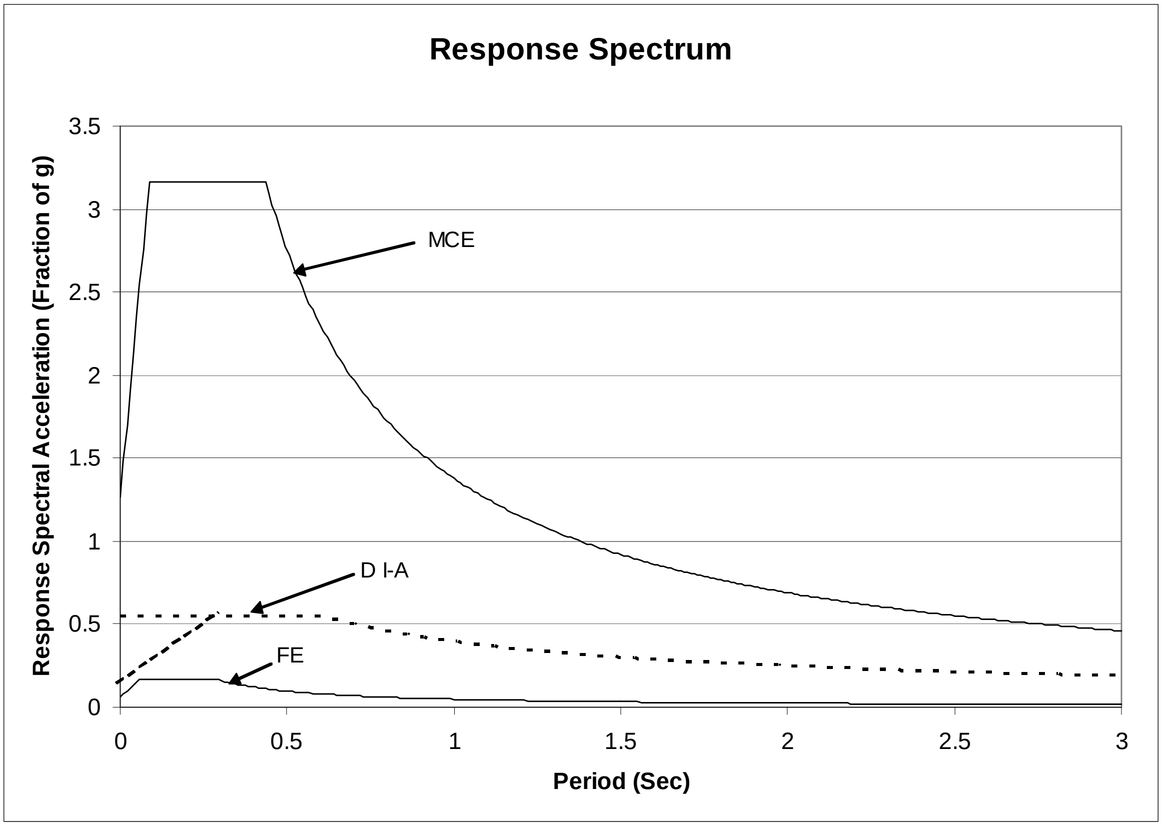
$0.4 \cdot S_{DS} = 0.066$

$T_S := \frac{S_{D1}}{S_{DS}}$ Period at the end of construction design spectral acceleration plateau

$T_S = 0.292$ Sec

$T_0 := 0.2 \cdot T_S$ Period at the beginning of construction design spectral acceleration plateau

$T_0 = 0.058$ Sec



Superstructure Section Properties

33W130

$L_{bridge} := 232$

$A_{girder} := 38.3$ in^2

$d_{girder} := 33.1$ in

$I_x := 6710$ in^4

$S_x := 406$ in^3

$I_y := 218$ in^4

$S_y := 37.9$ in^3

5 girder at 6' 6"
Slab thickness = 6.5'
Hung = 1"

$N := 8$

$hung := 1$ in

$b_{slab} := 30 \cdot 12$

$b_{slab} = 360$ in

Effective width of the composite section (AASHTO LRFD 4.6.2.6.2)

$L_1 := 40.66 \cdot \frac{12}{4}$ 1/4 of the span length

$L_1 = 121.98$ in

$L_2 := 6.5 \cdot 12$ Girder spacing

$L_2 = 78$ in

$L_3 := 6.5 \cdot 12 + \max(.58, .5 \cdot .855)$

$L_3 = 78.58$ in

$b_{effective} := \min(L_1, L_2, L_3)$

$b_{effective} = 78$ in

$h_{slab} := 6.5$ in

$A_{slab} := b_{slab} \cdot \frac{h_{slab}}{N}$

$A_{slab} = 292.5$ in^2

$$I_{x\_slab} := b_{slab} \cdot \frac{h_{slab}^{3}}{12 \cdot N}$$

$I_{x\_slab} = 1.03 \times 10^{3}$ in^{4}

$$Y_{slab} := \frac{h_{slab}}{2}$$

$Y_{slab} = 3.25$ in

$$Y_{x\_comp} := \frac{\left[A_{slab} \cdot Y_{slab} + 5A_{girder} \cdot \left(\frac{d_{girder}}{2} + 2 \cdot Y_{slab} + hung\right)\right]}{\left(5A_{girder} + A_{slab}\right)}$$

$Y_{x\_comp} = 11.48$ in From top

$$I_{x\_comp} := 5 \cdot I_x + I_{x\_slab} + 5 \cdot A_{girder} \cdot \left(\frac{d_{girder}}{2} + 2 \cdot Y_{slab} + hung - Y_{x\_comp}\right)^2 + A_{slab} \cdot \left(Y_{slab} - Y_{x\_comp}\right)^2$$

$I_{x\_comp} = 8.465 \times 10^{4}$ in^{4}

$$S_{x\_comp} := \frac{I_{x\_comp}}{Y_{x\_comp}}$$

$S_{x\_comp} = 7.374 \times 10^{3}$ in^{4}

$$A_{comp} := A_{girder} \cdot 5 + A_{slab}$$

$A_{comp} = 484$ in^{2}

$$r_x := \sqrt{\frac{I_{x\_comp}}{A_{comp}}}$$

$r_x = 13.225$ in

$$I_{y\_slab} := h_{slab} \cdot \frac{b_{slab}^{3}}{12 \cdot N}$$

$I_{y\_slab} = 3.159 \times 10^{6}$ in^4

$I_{y\_comp} := 5 \cdot I_y + 2 \cdot A_{girder} \cdot \left(6.5^2 + 13^2\right) + I_{y\_slab}$

$I_{y\_comp} = 3.176 \times 10^{6}$ in^4

$S_{y\_comp} := \frac{I_{y\_comp}}{42 \cdot .5 \cdot 12}$

$S_{y\_comp} = 1.26 \times 10^{4}$ in^3

$r_y := \sqrt{\frac{I_{y\_comp}}{A_{comp}}}$

$r_y = 81.01$ in

$A_{shear} := 0.8 \cdot A_{comp}$

$A_{shear} = 387.2$ in^2

$J := b_{slab} \cdot \frac{h_{slab}^{\ 3}}{12 \cdot N} + h_{slab} \cdot \frac{b_{slab}^{\ 3}}{12 \cdot N}$

$J = 3.16 \times 10^{6}$

$S_{x\_plastic} := S_{x\_comp} \cdot 1.5$

$S_{x\_plastic} = 1.106 \times 10^{4}$ in^3

$S_{y\_plastic} := S_{y\_comp} \cdot 1.5$

$S_{y\_plastic} = 1.891 \times 10^{4}$ in^3

Mass Calculation

$W_{slab} := 30 \cdot \frac{6.5 \cdot L_{bridge}}{12} \cdot .15$

$W_{slab} = 565.5$ kips

$W_{girders} := \frac{130}{1000} \cdot 5 \cdot L_{bridge} \cdot 1.15$

$W_{girders} = 173.42$ kips

$Bar := 5.2 \cdot L_{bridge} \cdot .15$

$Bar = 180.96$ kips

$FWS := \frac{35}{1000} \cdot 24 \cdot L_{bridge}$

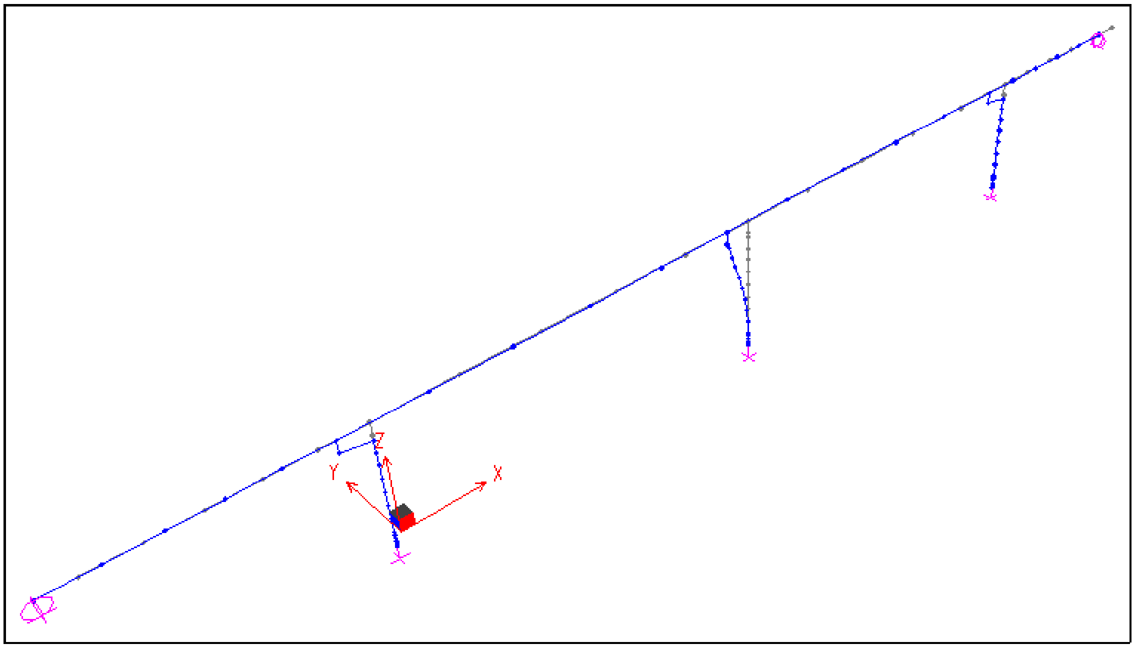
$FWS = 194.88$ kips

$W_{total} := W_{slab} + W_{girders} + Bar + FWS$

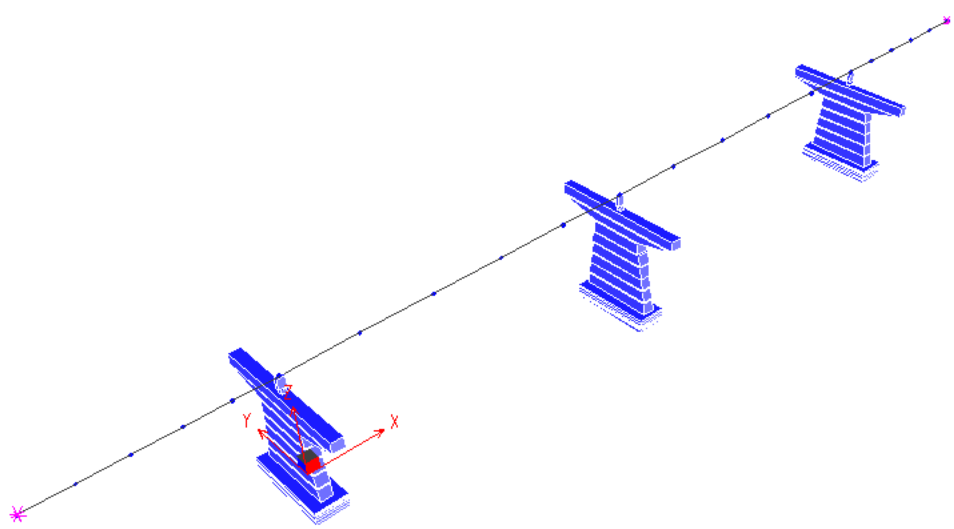
$W_{total} = 1.115 \times 10^{3}$ kips

$w_{total} := \frac{W_{total}}{L_{bridge}}$

$w_{total} = 4.805$ klf



Finite Element Model with fix base
Period = 0.874 Sec

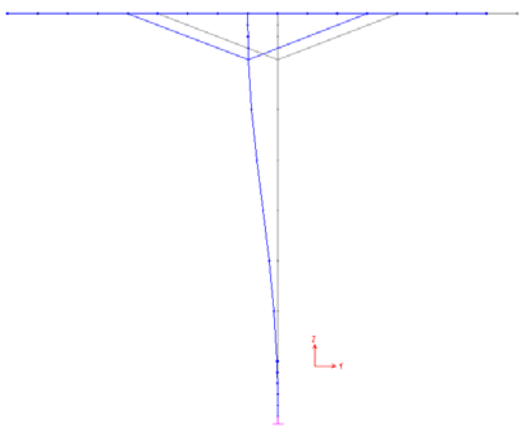


MODAL PARTICIPATING MASS RATIOS				
With fixed base				
MODE	PERIOD	INDIVIDUAL MODE (%)		
		UX	UY	UZ
1	0.8738	65.68	0.00	0.00
2	0.2681	0.03	0.00	0.00
3	0.2116	0.00	14.74	0.00
4	0.1859	0.00	0.00	19.70
5	0.1594	0.00	0.00	0.00
6	0.1396	0.00	4.52	0.00
7	0.1219	6.32	0.00	0.00
8	0.1219	1.89	0.00	0.00
9	0.1025	0.00	0.00	0.00
10	0.0915	0.00	0.00	0.00
11	0.0857	0.00	4.45	0.00
12	0.0856	0.00	0.00	0.00
13	0.0840	0.00	0.00	22.44
14	0.0764	0.00	35.69	0.00
15	0.0683	0.00	0.00	0.00
16	0.0672	0.00	0.00	0.00
17	0.0612	0.00	3.64	0.00
18	0.0602	0.00	0.00	2.67
19	0.0527	0.00	0.00	0.00
50	0.0133	0.00	0.00	8.64
		78.09	76.59	61.99

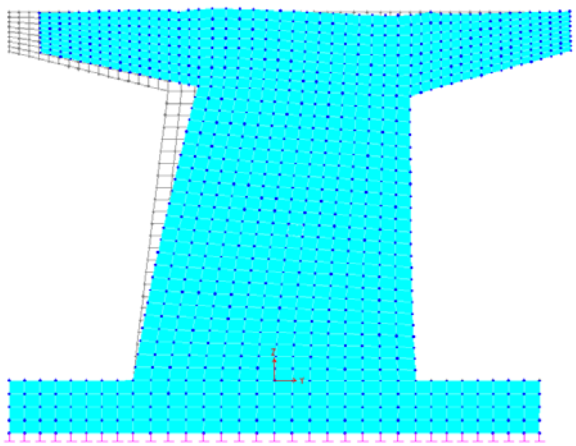
Center of gravity on top of the hammer head:

$$Y := \frac{[(2.25\cdot .5 + 2.25)\cdot(30\cdot 2.25) + (2.25\cdot .5)\cdot(12\cdot 2.25) + 2\cdot(2.25\cdot .66)\cdot(.5\cdot 9\cdot 2.25)]}{(30\cdot 2.25 + 9\cdot 2.25 + 12\cdot 2.25)}$$

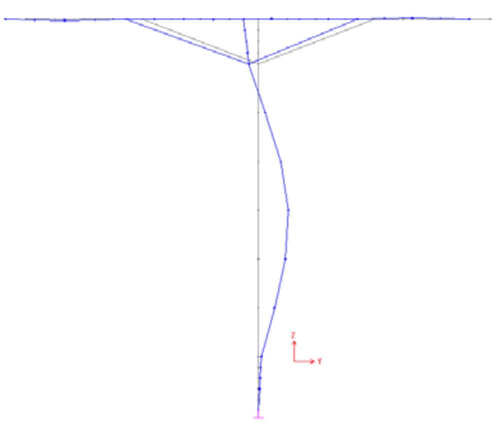
$Y = 2.512$ ft from the bottom



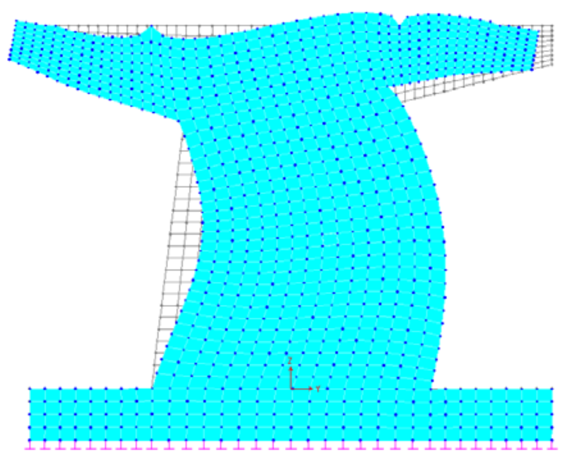
First main mode shape
T= 0.0226 Sec



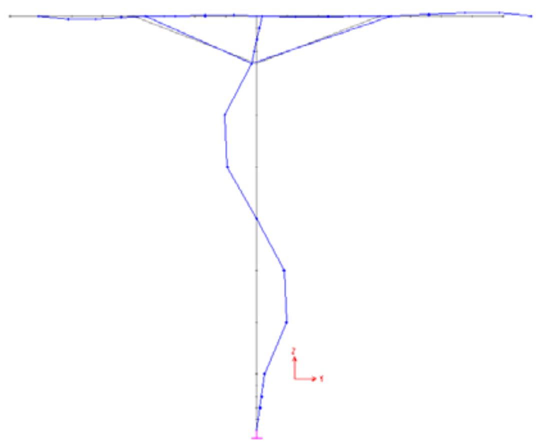
First main mode shape
T= 0.0209 Sec



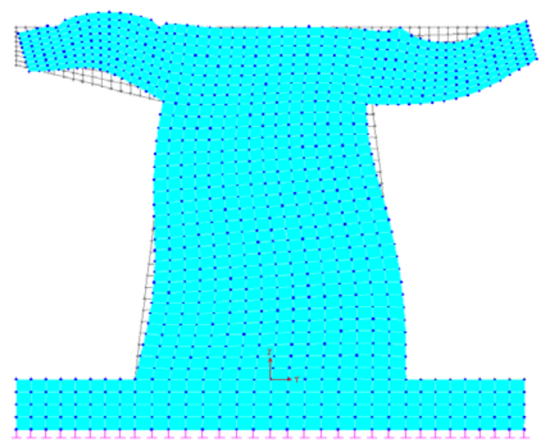
Second main mode shape
T= 0.0060 Sec



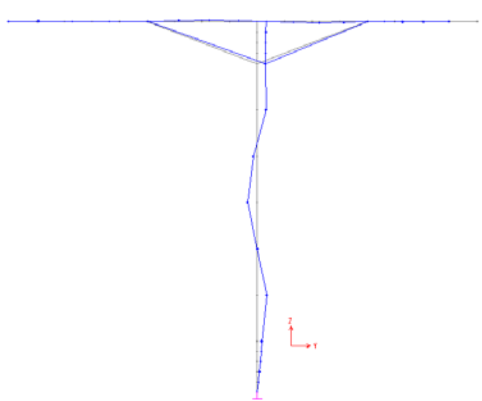
Second main mode shape
T= 0.0059 Sec



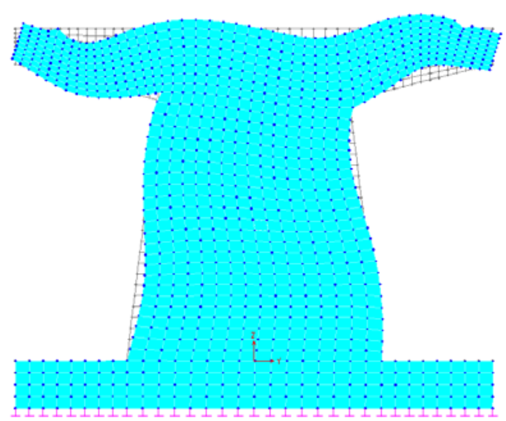
Third main mode shape
T=0.0034 Sec



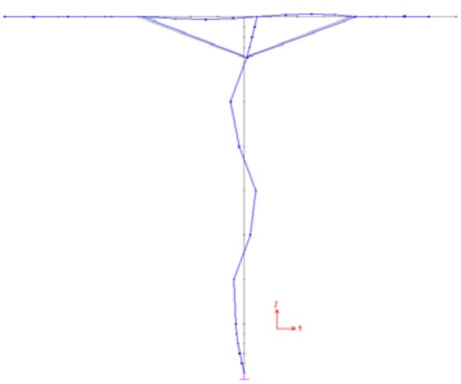
Third main mode shape
T= 0.0035 Sec



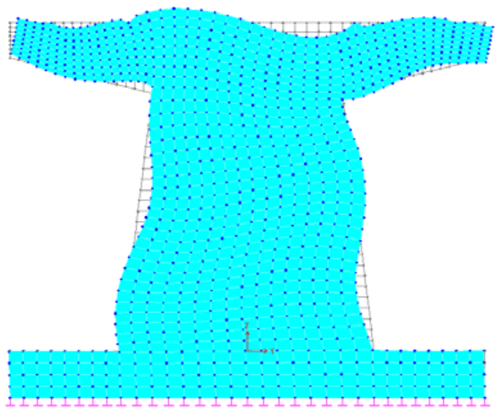
Fourth main mode shape
T=0.0024 Sec



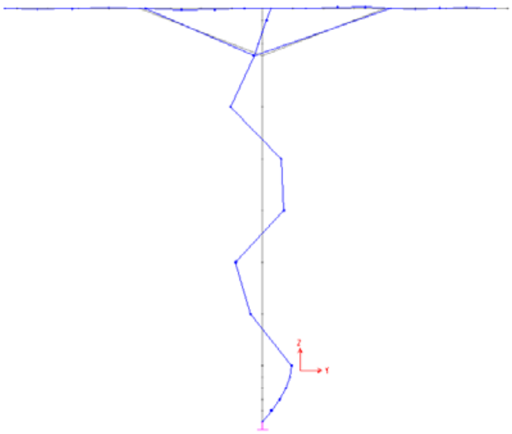
Fourth main mode shape
T=0.0029 Sec



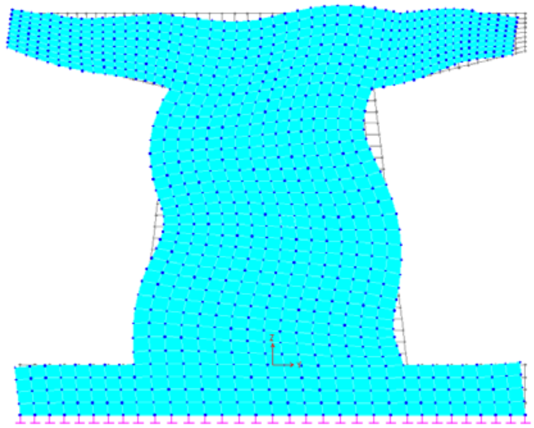
Fifth main mode shape
T=0.0022 Sec



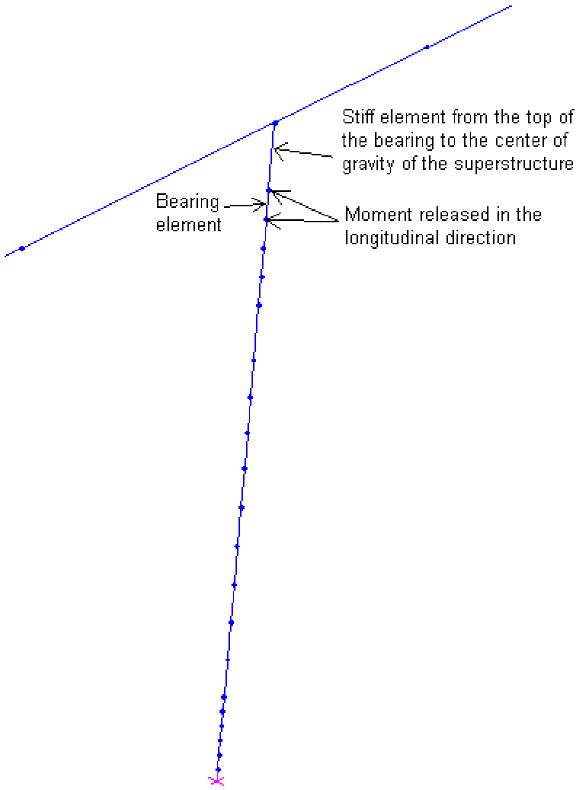
Fifth main mode shape
T=0.0022 Sec

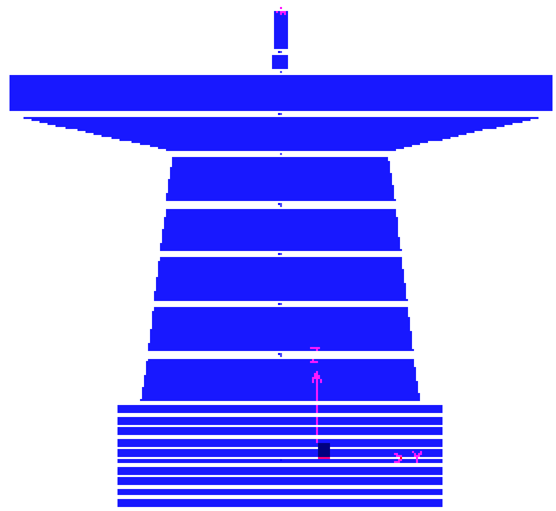


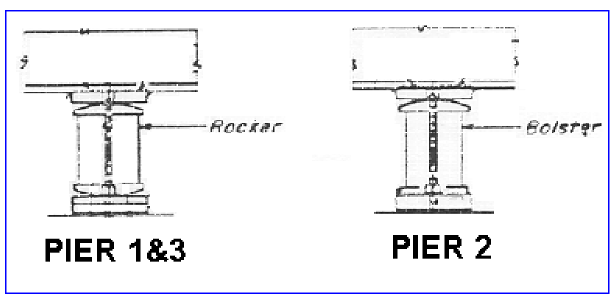
Sixth main mode shape
T=0.0019 Sec



Sixth main mode shape
T=0.0020 Sec


Stiff element from the top of the bearing to the center of gravity of the superstructure
Bearing element
Moment released in the longitudinal direction




Rocker
Bolster
PIER 1&3
PIER 2

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SUBJECT FILE:

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

NCHRP Guidelines

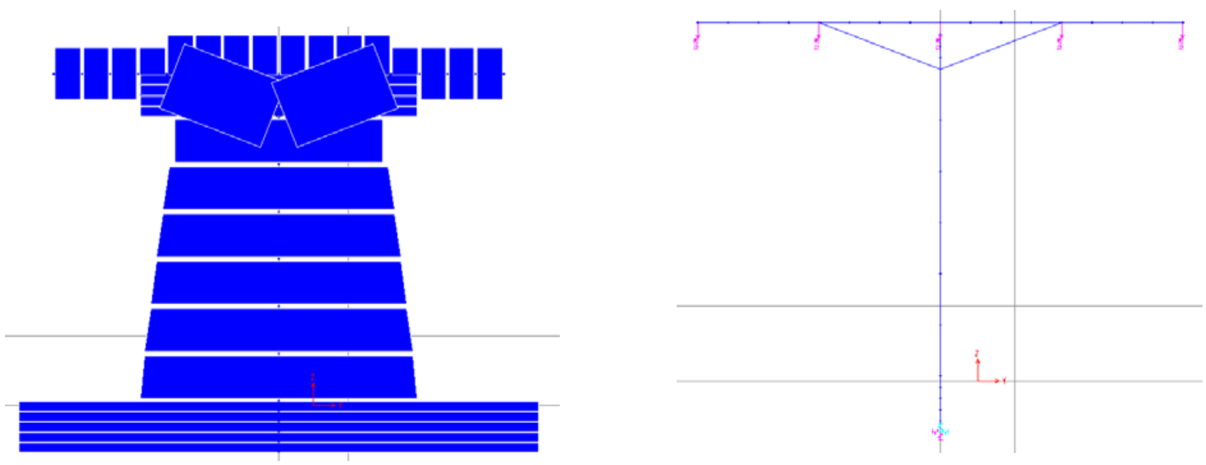
BY: **NA**

CHECKED: **AAA**

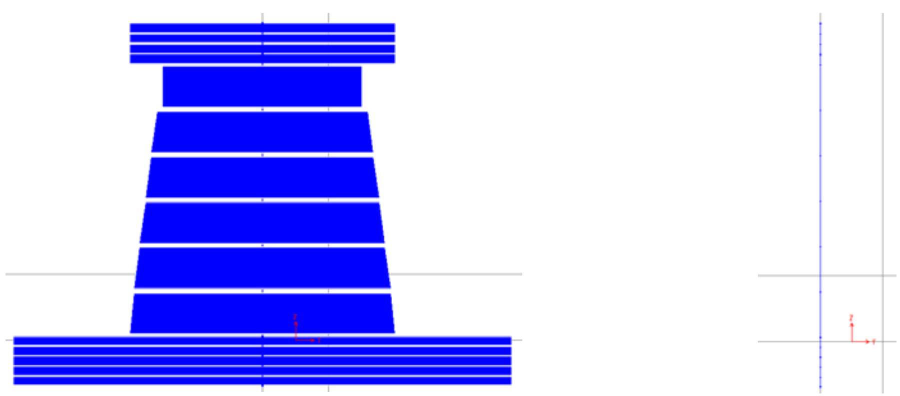
DATE: **6/22/2004**

Title: Pulaski County Bridge

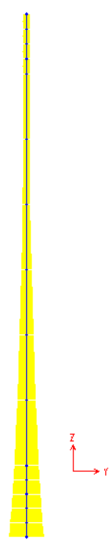
Sheet No: **16**



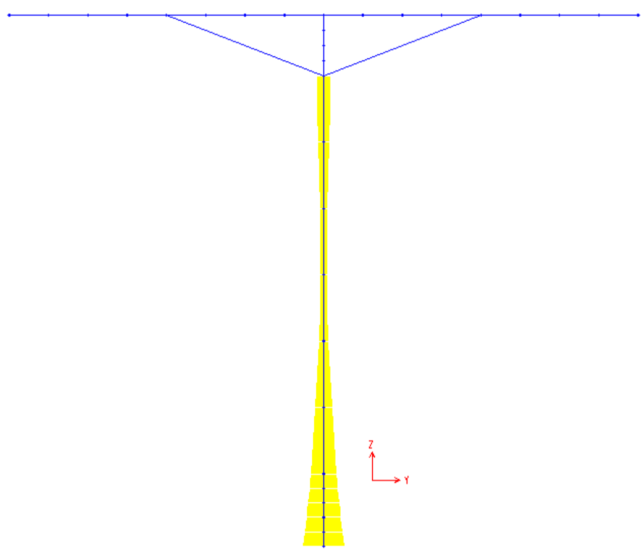
2D Model of Intermediate Bent With Masses at Real Location of Girders



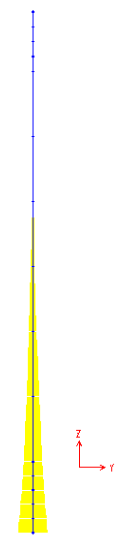
2D Model of Intermediate Bent With Single Vertical Column



With Rotational Mass
M = 2780 k-ft



With Masses at Location of Girders
M = 2804 k-ft



Without Rotational Mass
M = 2120 k-ft

Group Pile Stiffness At Fixed Bent:

Use HP 14X117 for the NCHRP design

A=34.4 in^2
Ixx=1220 in^4 Zxx=194 in^3
Iyy=443 in^4 Zyy=91.4 in^3
Depth=14.2 in
Width=14.9 in

$$E := \frac{29000000 \cdot 144}{1000} \quad \text{ksf}$$

$N := 40$ 4X10 (70" in long and 70" spacing in transverse direction)

Results from L-pile:

$\Delta_{long} = 0.56\,\text{in}$

$\Delta_{trans} = 0.71\text{in}$

Results from SAP m

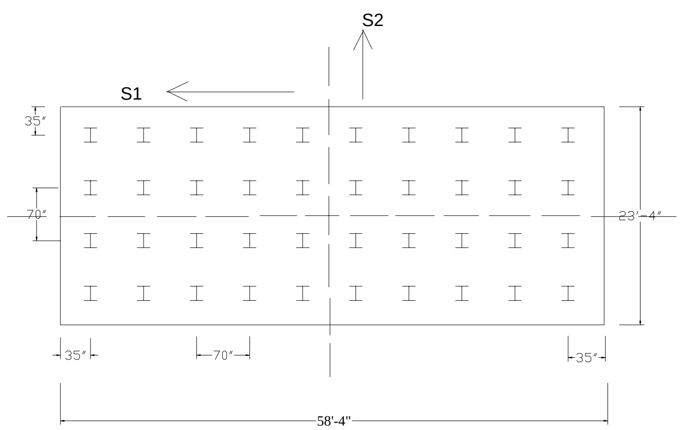
$V_{long\_single} = 96\,\text{Kips}$

$V_{trans\_single} = 74\ \ \text{Kip}$

$$K_{long\_single} = \frac{V_{long\_s}}{\Delta_{lor}}$$

$$K_{long\_single} := 96 \cdot \frac{12}{.56}$$

$$K_{long\_single} = 2.057 \times 10^{3} \ \frac{k}{ft}$$



$K_x := K_{long\_single} \cdot N$

$K_x = 8.229 \times 10^4 \quad \frac{k}{ft}$

$L := 60 \ ft$

$K_{tran\_single} = \frac{V_{trans\_single}}{\Delta_{trans}}$

$K_{tran\_single} := 74 \cdot \frac{12}{.71}$

$K_{tran\_single} = 1.251 \times 10^3$

$K_y := K_{tran\_single} \cdot N$

$K_y = 5.003 \times 10^4 \quad \frac{k}{ft}$

$A := \frac{34.4}{144}$

$A = 0.239 \quad ft^2$

$K_{axial} := A \cdot \frac{E}{L}$

$K_{axial} = 1.663 \times 10^4 \quad \frac{k}{ft}$

$K_z := K_{axial} \cdot N$

$K_z = 6.651 \times 10^5 \quad \frac{k}{ft}$

$S_1 := \frac{8}{144} \cdot \left[(.5 \cdot 70)^2 + (1.5 \cdot 70)^2 + (2.5 \cdot 70)^2 + (3.5 \cdot 70)^2 + (4.5 \cdot 70)^2 \right]$

$S_2 := \frac{2}{144} \cdot \frac{N}{4} \cdot \left[(.5 \cdot 70)^2 + (1.5 \cdot 70)^2 \right]$

$K_{rx} := K_{axial} \cdot S_1$

$K_{ry} := K_{axial} \cdot S_2$

$K_{rz} := \left(K_{long\_single} \cdot S_1 + K_{tran\_single} \cdot S_2\right)$

$K_x = 8.229 \times 10^4 \ \frac{k}{ft}$

$K_y = 5.003 \times 10^4 \ \frac{k}{ft}$

$K_z = 6.651 \times 10^5 \ \frac{k}{ft}$

$K_{rx} = 1.867 \times 10^8 \ \frac{ft - Kips}{rad}$

$K_{ry} = 2.829 \times 10^7 \ \frac{ft - Kips}{rad}$

$K_{rz} = 2.523 \times 10^7 \ \frac{ft - Kips}{rad}$

Total DL = 1109 kips

Axial capacity (IDOT Bridge Manual 3.8.1) = 9000 psi

$P_{cap} := A \cdot 1.5 \cdot 9 \cdot 144$ A is area of cross-section of single pile

$P_{cap} = 464.4$ kips Ultimate axial capacity of pile

$N = 40$ No of piles used

$V := 1109$ kips

Results from SAP :

$M_{long} := 46161$ k − ft $M_{tran} := 15526$ k − ft

$$P_1 := \frac{V}{N} + M_{long} \cdot \frac{1.5 \cdot 75}{12 \cdot S_2} + M_{tran} \cdot \frac{4.5 \cdot 75}{12 \cdot S_1}$$

$P_1 = 320.969$ kips

$P_1 < P_{cap}$ Ok

Results from L-pile:

$Dis_L := .39$ in $M_1 := 4080$ k − in

$Dis_T := .23$ in $M_2 := 1800$ k − in

Compressive Resistance (Combined axial compression and flexure)

$$\frac{P_1}{0.85 \cdot 50 \cdot 34.4} + \frac{8}{9} \cdot \left(\frac{M_1}{50 \cdot 194} + \frac{M_2}{50 \cdot 91.4} \right) = 0.944 < 1 \text{ OK}$$

$f_y = 50 \cdot ksi$ for steel pile

EQ in transverse direction

Results from SAP :

$M_{long} := 18646$ k − ft $M_{tran} := 38811$ k − ft

$$P_2 := \frac{V}{N} + M_{long} \cdot \frac{1.5 \cdot 75}{12S_2} + M_{tran} \cdot \frac{4.5 \cdot 75}{12S_1}$$

$P_2 = 227.676$ kips $< P_{cap}$ OK

Results from L-pile:

$Dis_L := .15$ in $M_3 := 2230$ k − in

$Dis_T := .57$ in $M_4 := 3055$ k − in

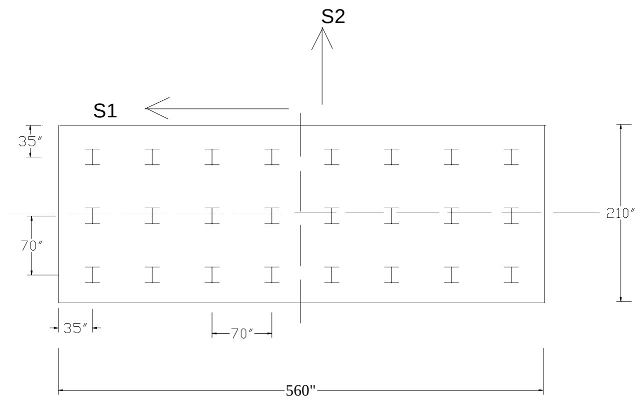
$$\frac{P_2}{0.85 \cdot 50 \cdot 34.4} + \frac{8}{9} \cdot \left(\frac{M_3}{50 \cdot 194} + \frac{M_4}{50 \cdot 91.4}\right) = 0.954$$ <1 OK $\left(\text{AASHTO LRFD}\,6.9.2.2 - 2\right)$

Spring Stiffness at Expansion Bent

$N := 24$

3X8 (70" spacing in both direction)

$K_{long\_single} = 2.057 \times 10^3 \quad \frac{k}{ft}$



$K_x := K_{long\_single} \cdot N$

$K_x = 4.937 \times 10^4 \quad \frac{k}{ft}$

$L := 60 \quad ft$

$K_{tran\_single} = 1.251 \times 10^3$

$K_y := K_{tran\_single} \cdot N$

$K_y = 3.002 \times 10^4 \quad \frac{k}{ft}$

$A := \frac{34.4}{144}$

$A = 0.239 \quad ft^2$

$K_{axial} := A \cdot \frac{E}{L}$

$K_{axial} = 1.663 \times 10^4 \quad \frac{k}{ft}$

$K_z := K_{axial} \cdot N$

$K_z = 3.99 \times 10^5 \quad \frac{k}{ft}$

$S_1 := \frac{6}{144} \cdot \left[(.5 \cdot 70)^2 + (1.5 \cdot 70)^2 + (2.5 \cdot 70)^2 + (3.5 \cdot 70)^2 \right] \qquad S_1 = 4.287 \times 10^3 \quad ft^2$

$S_2 := \frac{2}{144} \cdot \frac{N}{3} \cdot \left(70^2\right) \qquad S_2 = 544.444 \quad ft^2$

$K_{rx} := K_{axial} \cdot S_1$

$K_{ry} := K_{axial} \cdot S_2$

$K_{rz} := S_1 \cdot K_{long\_single} + S_2 \cdot K_{tran\_single}$

$K_{rz} = 9.501 \times 10^6 \quad \frac{ft - Kip}{rad}$

$K_x = 4.937 \times 10^4 \quad \frac{K}{ft}$

$K_y = 3.002 \times 10^4 \quad \frac{K}{ft}$

$K_z = 3.99 \times 10^5 \quad \frac{K}{ft}$

$K_{rx} = 7.129 \times 10^7 \quad \frac{ft - Kip}{rad}$

$K_{ry} = 9.052 \times 10^6 \quad \frac{ft - Kip}{rad}$

$K_{rz} = 9.501 \times 10^6 \quad \frac{ft - Kip}{rad}$

Total DL = 820 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

$P_{cap} = 464.4$ kips Ultimate axial capacity of pile

$N = 24$ No of piles used

$V := 820$ kips Axial load

Results from SAP:

$M_{long} := 8243$ k − ft

$M_{tran} := 15508$ k − ft

$$P_1 := \frac{V}{N} + M_{long} \cdot \frac{1 \cdot 75}{12 \cdot S_2} + M_{tran} \cdot \frac{3.5 \cdot 75}{12 \cdot S_1}$$

$P_1 = 207.915$ kips

$P_1 < P_{cap}$ Ok

Results from L-pile:

$Dis_L := .28$ in $M_1 := 3350$ k − in

$Dis_T := .39$ in $M_2 := 2500$ k − in

Compressive Resistance (Combined axial compression and flexure)

$f_y = 50$ ksi for steel pile

$$\frac{P_1}{0.85 \cdot 50 \cdot 34.4} + \frac{8}{9} \cdot \left(\frac{M_1}{50 \cdot 194} + \frac{M_2}{50 \cdot 91.4} \right) = 0.935 < 1$$ Ok

(AASHTO LRFD 6.9.2.2)

Earthquake in transverse direction:

Results from SAP Model:

$M_{long} := 3298$ k − ft

$M_{tran} := 38766$ k − ft

$$P_2 := \frac{V}{N} + M_{long} \cdot \frac{1 \cdot 75}{12 S_2} + M_{tran} \cdot \frac{3.5 \cdot 75}{12 S_1}$$

$P_2 = 269.812$ kips $< P_{cap}$ OK

Results from L-pile:

$Dis_L := .11$ in $\quad M_3 := 1700$ k − in

$Dis_T := .75$ in $\quad M_4 := 3380$ k − in

Compressive Resistance (Combined axial compression and flexure)

$$\frac{P_2}{0.85 \cdot 50 \cdot 34.4} + \frac{8}{9} \cdot \left(\frac{M_3}{50 \cdot 194} + \frac{M_4}{50 \cdot 91.4} \right) = 0.998 \quad :1 \quad \text{Ok}$$

MCEER Guide Spec. Table 4.7-1

Base Response Modification Factor (R_B) SDAP D Operational Performance	
Single column	1.5
Wall pier	1.0
Vertical pile	1
All elements for FE	0.9
Superstructure to abutment	0.8
Column and pile to cap beam	0.8

$T_s := .436$ Sec

$T := .874$ Sec

$R_{col} := 1 + (1.5 - 1)\cdot\left(\frac{T}{1.25\cdot T_s}\right)$ MCEER Guide Spec. 4.7

$R_{col} = 1.802$ < 1.5

$R_{wall} := 1 + (1 - 1)\left(\frac{T}{1.25\cdot T_s}\right)$

$R_{wall} = 1$

$T_s := .292$ Sec

$T := .874$ Sec

$R_{FE} := 1 + (0.9 - 1)\left(\frac{T}{1.25\cdot T_s}\right)$

$R_{FE} = 0.761$ > 0.9

$R_{FE} := 0.9$

Response Modification Factor (R) SDAP D Operational Performance	
Single column	1.50
Wall pier	1.0
Vertical pile	1
All elements for FE	0.76
Superstructure to abutment	0.8
Column and pile to cap beam	0.8

Dead Load	
Location	Axial kips
Fix Pier	515
Exp Pier	424

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		DATE: 6/22/2004
Title: Pulaski County Bridge	**NCHRP Guidelines**	Sheet No: **27**

MCE		**Forces & Moments - EQ_{trans}**			
		Transverse			
Support / Location		Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	1139	24709	0	0
	Bottom		35753	0	
Exp Pier	Top	1510	11777	0	0
	Bottom		33204	0	

MCE		**Forces & Moments - EQ_{long}**			
		Longitudinal			
Support / Location		Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	1854	0	0	0
	Bottom		0	40481	
Exp Pier	Top	462	0	0	68
	Bottom		0	6494	

MCE		**100% + 40% Forces & Moments - EQ**									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	742	1139	0	24709	0	456	1854	9884	0	0
	Bottom			16192	35753				14301	40481	
Exp Pier	Top	185	1510	0	11777	27	604	462	4711	0	68
	Bottom			2598	33204				13282	6494	

MCE		**Factored Forces & Moments - EQ by R**									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top	494	1139	0	24709	0	456	1236	9884	0	0
	Bottom			10795	35753				14301	26987	
Exp Pier	Top	123	1510	0	11777	18	604	308	4711	0	45
	Bottom			1732	33204				13282	4329	

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NCHRP Guidelines

BY: **NA**

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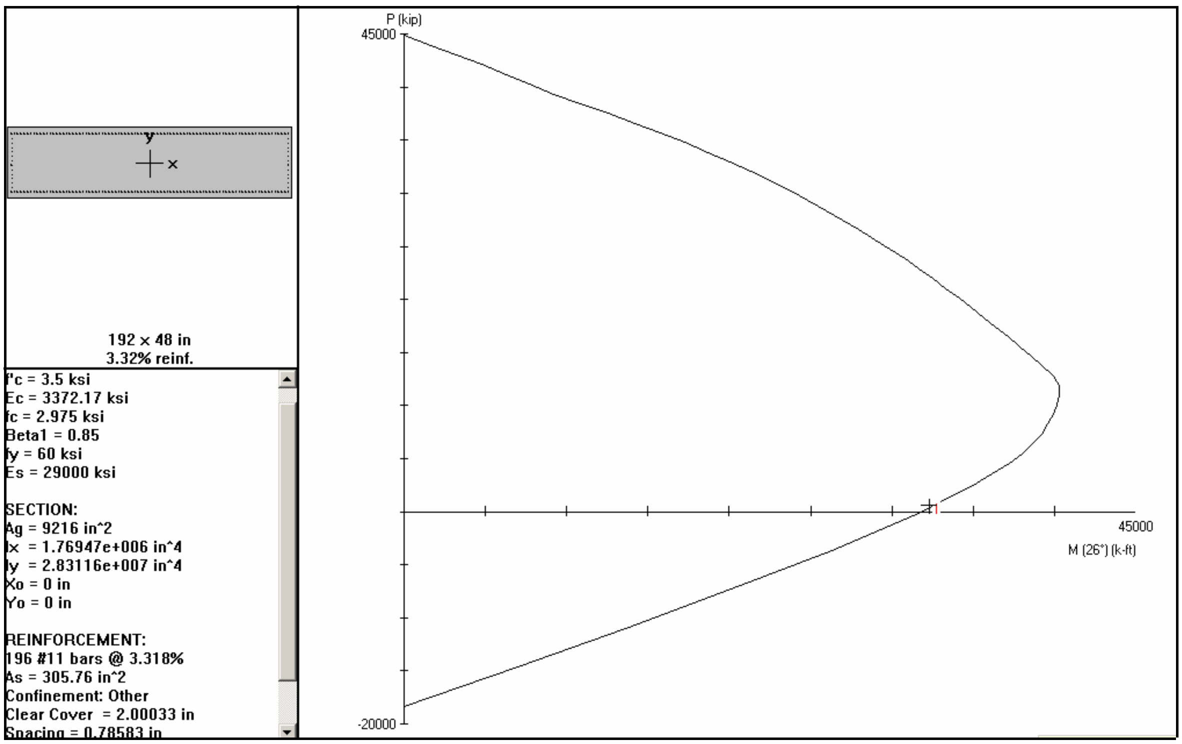
DATE: **6/22/2004**

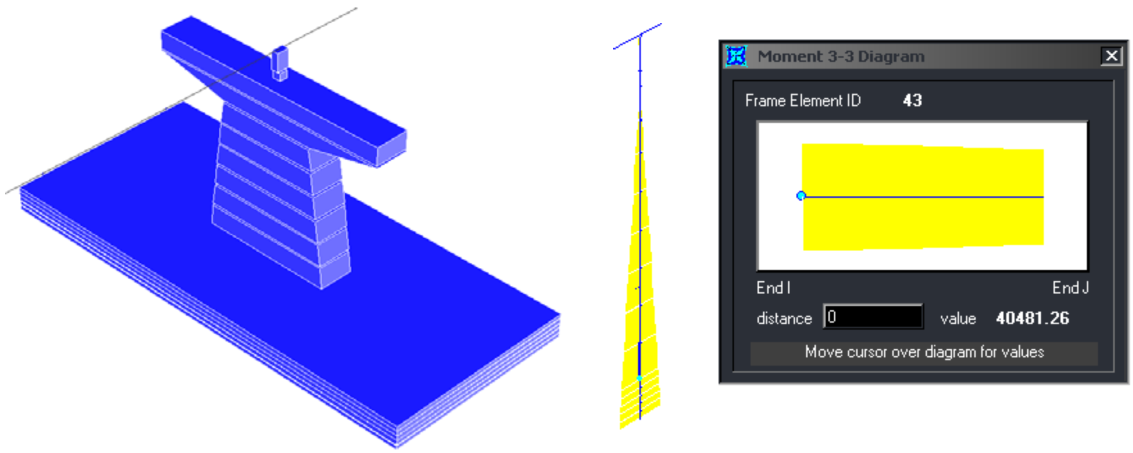
Title: Pulaski County Bridge

Sheet No: **28**

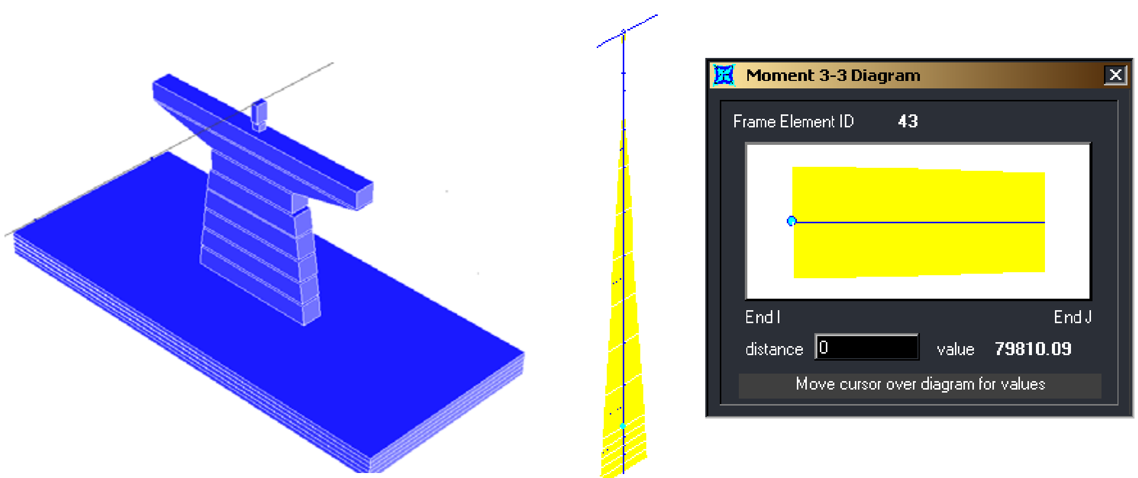
MCE		**Final Design Forces & Moments (EQ + DL)**									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Fix Pier	Top Bottom	494	1139	0 10795	24709 35753	570	456	1236	9884 14301	0 26987	570
Exp Pier	Top Bottom	123	1510	0 1732	11777 33204	488	604	308	4711 13282	0 4329	515



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)

$E := 29000000 \cdot \frac{144}{1000}$

$E = 4.176 \times 10^6$ ksf

$N := 40$ 4X10 (70" in long and 70" spacing in transverse direction)

Results from SAP model:

$V_{long\_single} = 74$ Kips

$V_{trans\_single} = 75$ Kips

Results from L-pile:

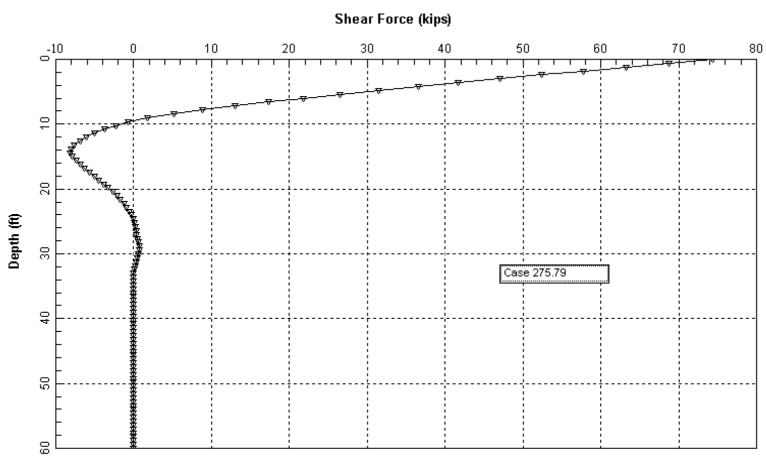
$\Delta_{long} = 0.24$ in

$\Delta_{trans} = 0.66$ in

$K_{long\_single} = \frac{V_{long\_single}}{\Delta_{long}}$

$K_{long\_single} := 74 \cdot \frac{12}{.24}$

$K_{long\_single} = 3.7 \times 10^3 \ \frac{k}{ft}$



$K_x := K_{long\_single} \cdot N$

$K_x = 1.48 \times 10^5 \ \frac{k}{ft}$

$L := 60$ ft

$K_{tran\_single} = \frac{V_{trans\_single}}{\Delta_{trans}}$

$K_{tran\_single} := 75 \cdot \frac{12}{.66}$

$K_{tran\_single} = 1.364 \times 10^3$

$K_y := K_{tran\_single} \cdot N$

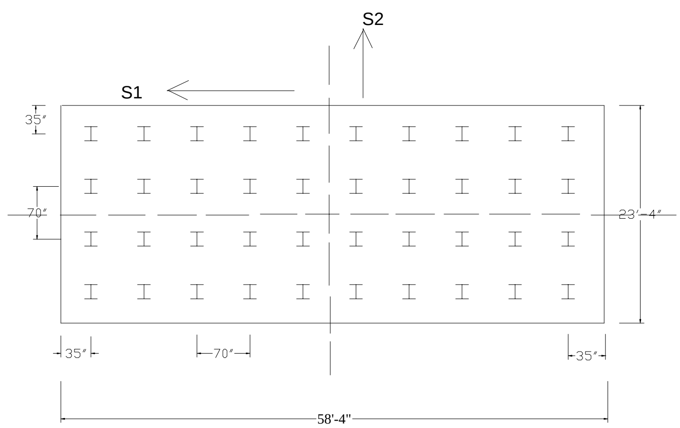
$K_y = 5.455 \times 10^4$

$A := \frac{34.4}{144}$

$A = 0.239 \quad ft^2$

$K_{axial} := A \cdot \frac{E}{L}$

$K_{axial} = 1.663 \times 10^4$



$K_z := K_{axial} \cdot N$

$K_z = 6.651 \times 10^5 \quad \frac{k}{ft}$

$S_1 := \frac{6}{144} \cdot \left[(.5 \cdot 70)^2 + (1.5 \cdot 70)^2 + (2.5 \cdot 70)^2 + (3.5 \cdot 70)^2 + (4.5 \cdot 70)^2 \right]$

$S_2 := \frac{2}{144} \cdot \frac{N}{4} \cdot \left[(.5 \cdot 70)^2 + (1.5 \cdot 70)^2 \right]$

$K_{rx} := K_{axial} \cdot S_1$

$K_{ry} := K_{axial} \cdot S_2$

$K_{rz} := \left(K_{long\_single} \cdot S_1 + K_{tran\_single} \cdot S_2 \right)$

$K_x = 1.48 \times 10^5 \quad \frac{k}{ft}$

$K_y = 5.455 \times 10^4 \quad \frac{k}{ft}$

$K_z = 6.651 \times 10^5 \quad \frac{k}{ft}$

$K_{rx} = 1.4 \times 10^8 \quad \frac{k - ft}{rad}$

$K_{ry} = 2.829 \times 10^7 \quad \frac{k - ft}{rad}$

$K_{rz} = 3.348 \times 10^7 \quad \frac{k - ft}{rad}$

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

$P_{cap} = 464.4$ kips Ultimate axial capacity of pile

$N = 40$ No of piles used

$V := 1065$ kips

Results from SAP:

$M_{long} := 31836 \quad k - ft$

$M_{tran} := 17848 \quad k - ft$

$$P_1 := \frac{V}{N} + M_{long} \cdot \frac{1.5 \cdot 70}{12 \cdot S_2} + M_{tran} \cdot \frac{4.5 \cdot 70}{12 \cdot S_1}$$

$P_1 = 245.983$ kips

$P_1 < P_{cap}$ Ok

Results from L-pile:

$Dis_L := .24$ in $\qquad M_1 := 3055$ k − in

$Dis_T := .20$ in $\qquad M_2 := 1660$ k − in

Compressive Resistance (Combined axial compression and flexure)

$$\frac{P_1}{0.85 \cdot 50 \cdot 34.4} + \frac{8}{9} \cdot \left(\frac{M_1}{50 \cdot 194} + \frac{M_2}{50 \cdot 91.4} \right) = 0.771 \,.\, < 1 \text{ OK} \qquad (\text{AASHTO LRFD6.9.2.2} - 2)$$

EQ in transverse direction

Results from SAP:

$M_{long} := 12746$ k − ft

$M_{tran} := 44615$ k − ft

$$P_2 := \frac{V}{N} + M_{long} \cdot \frac{1.5 \cdot 75}{12S_2} + M_{tran} \cdot \frac{4.5 \cdot 75}{12S_1}$$

$P_2 = 245.851$ kips $< P_{cap}$ OK

Results from L-pile:

$Dis_L := .097$ in $\qquad M_3 := 1640$ k − in

$Dis_T := .51$ in $\qquad M_4 := 2890$ k − in

Compressive Resistance (Combined axial compression and flexure)

$$\frac{P_2}{0.85 \cdot 50 \cdot 34.4} + \frac{8}{9} \cdot \left(\frac{M_3}{50 \cdot 194} + \frac{M_4}{50 \cdot 91.4} \right) = 0.88 \,.\, < 1 \text{ OK}$$

MCEER Guide Spec. Table 4.7-1

Base Response Modification Factor (R_B) SDAP D Operational Performance	
Single column	1.5
Wall pier	1.0
Vertical pile	1
All elements for FE	0.9
Superstructure to abutment	0.8
Column and pile to cap beam	0.8

$T_s := 0.436$ Sec

$T := .8738$ Sec

$R_{col} := 1 + (1.5 - 1)\cdot\left(\frac{T}{1.25\cdot T_s}\right)$ 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	
Single column	1.50
Wall pier	1.0
Vertical pile	1
All elements for FE	0.9
Superstructure to abutment	0.8
Column and pile to cap beam	0.8

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BY: **NA**

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DATE: **6/22/2004**

NCHRP Guidelines

Title: Pulaski County Bridge

MCE		Forces & Moments - EQ_{trans}			
		Transverse			
Support / Location		Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Pier 2	Top	1199	28687	0	0
	Bottom		41430	0	
Pier 1&3	Top	1651	16116	0	0
	Bottom		39190	0	

Dead Load	
Location	Axial kips
Pier 2	570
Pier 1&3	470

MCE		Forces & Moments - EQ_{long}			
		Longitudinal			
Support / Location		Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Pier 2	Top	1342	0	0	0
	Bottom		0	27654	
Pier 1&3	Top	1758	0	0	184
	Bottom		0	32521	

MCE		100% + 40% Forces & Moments - EQ									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Pier 2	Top	537	1199	0	28687	0	480	1342	11475	0	0
	Bottom			11062	41430				16572	27654	
Pier 1&3	Top	703	1651	0	16116	74	660	1758	6446	0	184
	Bottom			13008	39190				15676	32521	

MCE		Factored Forces & Moments - EQ by R									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Pier 2	Top	358	1199	0	28687	0	480	895	11475	0	0
	Bottom			7374	41430				16572	18436	
Pier 1&3	Top	469	1651	0	16116	49	660	1172	6446	0	123
	Bottom			8672	39190				15676	21681	

MCE		Final Design Forces & Moments (EQ + DL)									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Pier 2	Top	358	1199	0	28687	570	480	895	11475	0	570
	Bottom			7374	41430				16572	18436	
Pier 1&3	Top	469	1651	0	16116	519	660	1172	6446	0	593
	Bottom			8672	39190				15676	21681	

FE		Forces & Moments- EQtrans			
		Transverse			
Support/Location		Shear 3 Kips	Moment 2 Kip-ft	Moment 3 Kip-ft	Axial Kips
Pier 2	Top	30	393	0	0
	Bottom		703	0	
Pier 1&3	Top	17	245	0	0
	Bottom		359	0	

FE		Forces & Moments- EQlong			
		Longitudinal			
Support/Location		Shear 2 Kips	Moment 2 Kip-ft	Moment 3 Kip-ft	Axial Kips
Pier 2	Top	36	0	0	0
	Bottom		0	753	
Pier 1&3	Top	51	0	0	4
	Bottom		0	942	

FE		100% + 40% Forces & Moments - EQ									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips	Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Pier 2	Top	14	30	393	0	0	36	12	157	0	0
	Bottom			703	301				281	753	
Pier 1&3	Top	21	17	245	0	2	51	7	98	0	4
	Bottom			359	377				144	942	

FE		Factored Forces & Moments - EQ/R									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips	Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Pier 2	Top	16	34	437	0	0	40	14	175	0	0
	Bottom			781	335				312	837	
Pier 1&3	Top	24	19	272	0	2	57	8	109	0	5
	Bottom			399	419				160	1047	

FE		Final Design Forces & Moments - (EQ/R+DL)									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips	Shear 2 kips	Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Pier 2	Top	16	34	437	0	570	40	14	175	0	570
	Bottom			781	335				312	837	
Pier 1&3	Top	24	19	272	0	472	57	8	109	0	475
	Bottom			399	419				160	1047	

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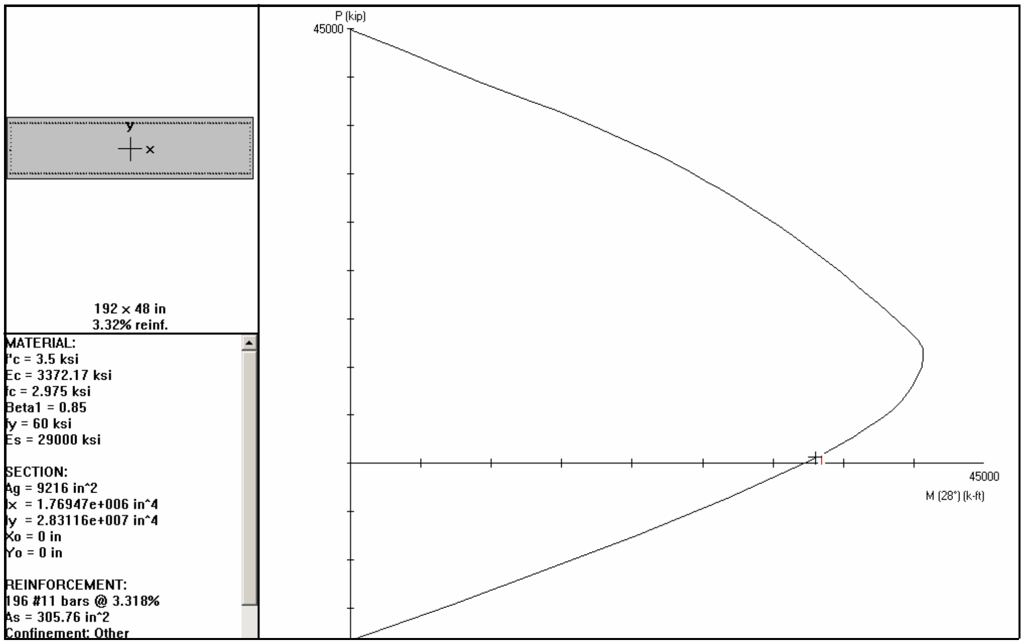
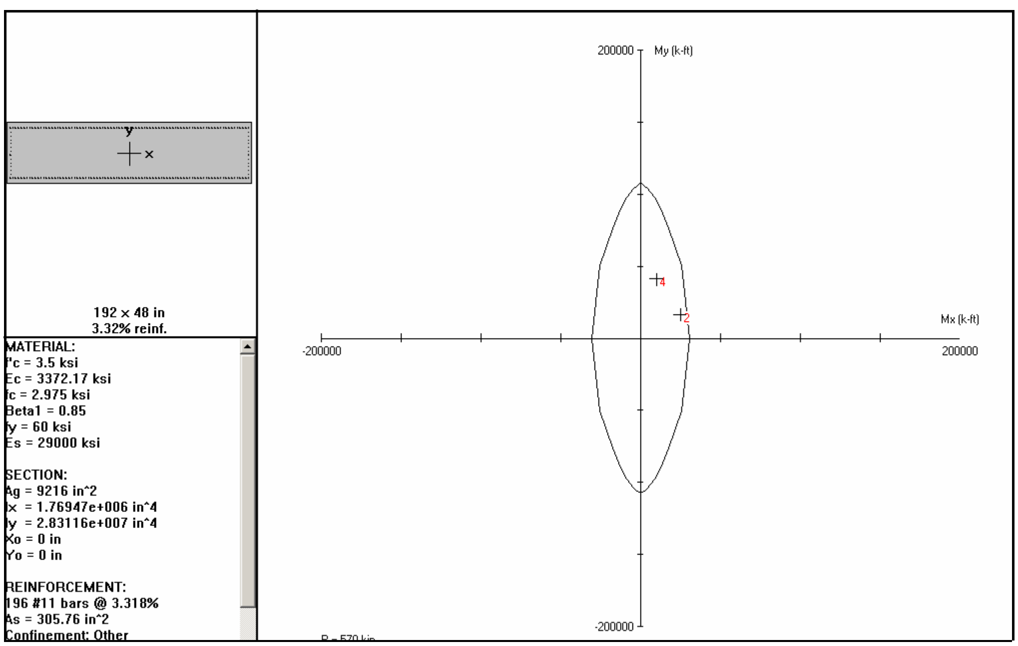
Therefore, MCE design forces govern.





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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

MCE		Forces & Moments - EQ_{trans}			
		Transverse			
Support / Location		Shear 3 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Pier 2	Top	1146	28687	0	0
	Bottom		37284	0	
Pier 1&3	Top	1601	16116	0	0
	Bottom		31133	0	

Dead Load	
Location	Axial kips
Pier 2	519
Pier 1&3	421

MCE		Forces & Moments - EQ_{long}			
		Longitudinal			
Support / Location		Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Pier 2	Top	1312	0	0	0
	Bottom		0	20367	
Pier 1&3	Top	1745	0	0	184
	Bottom		0	22915	

MCE		100% + 40% Forces & Moments - EQ									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Pier 2	Top	525	1146	0	28687	0	458	1312	11475	0	0
	Bottom			8147	37284				14914	20367	
Pier 1&3	Top	698	1601	0	16116	74	640	1745	6446	0	184
	Bottom			9166	31133				12453	22915	

MCE		Factored Forces & Moments - EQ by R									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Pier 2	Top	350	1146	0	28687	0	458	875	11475	0	0
	Bottom			5431	37284				14914	13578	
Pier 1&3	Top	465	1601	0	16116	49	640	1163	6446	0	123
	Bottom			6111	31133				12453	15277	

MCE		Final Design Forces & Moments (EQ + DL)									
		Transverse					Longitudinal				
Support / Location		Shear 2 kips	Shear 3 kips	Moment 3 kip-ft	Moment 2 kip-ft	Axial kips	Shear 3 kips	Shear 2 kips	Moment 2 kip-ft	Moment 3 kip-ft	Axial kips
Pier 2	Top	350	1146	0	28687	519	458	875	11475	0	519
	Bottom			5431	37284				14914	13578	
Pier 1&3	Top	465	1601	0	16116	470	640	1163	6446	0	544
	Bottom			6111	31133				12453	15277	

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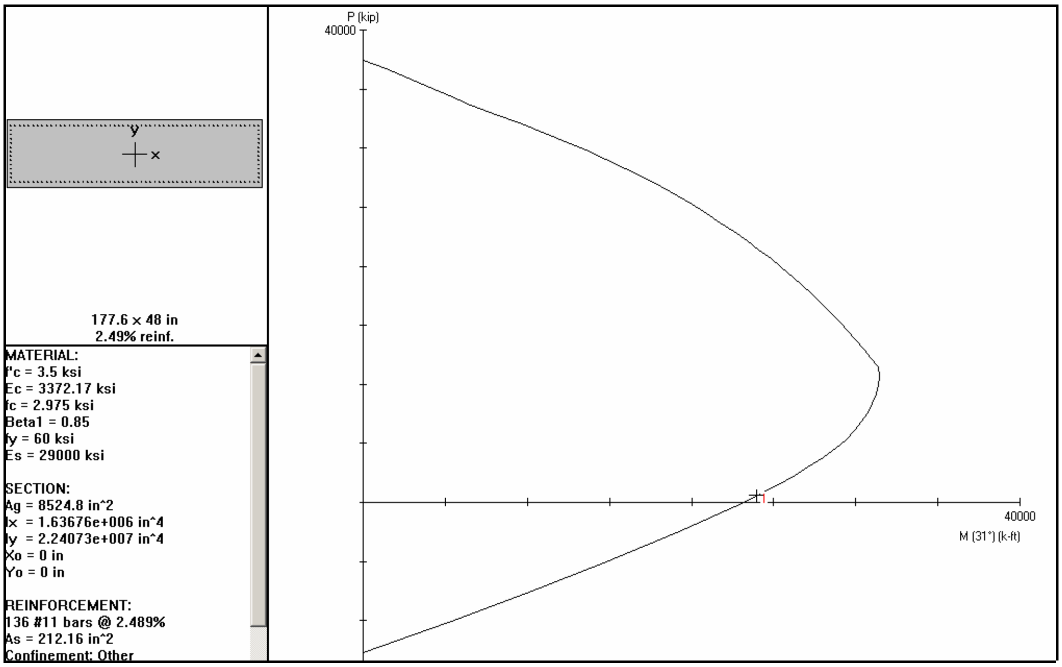
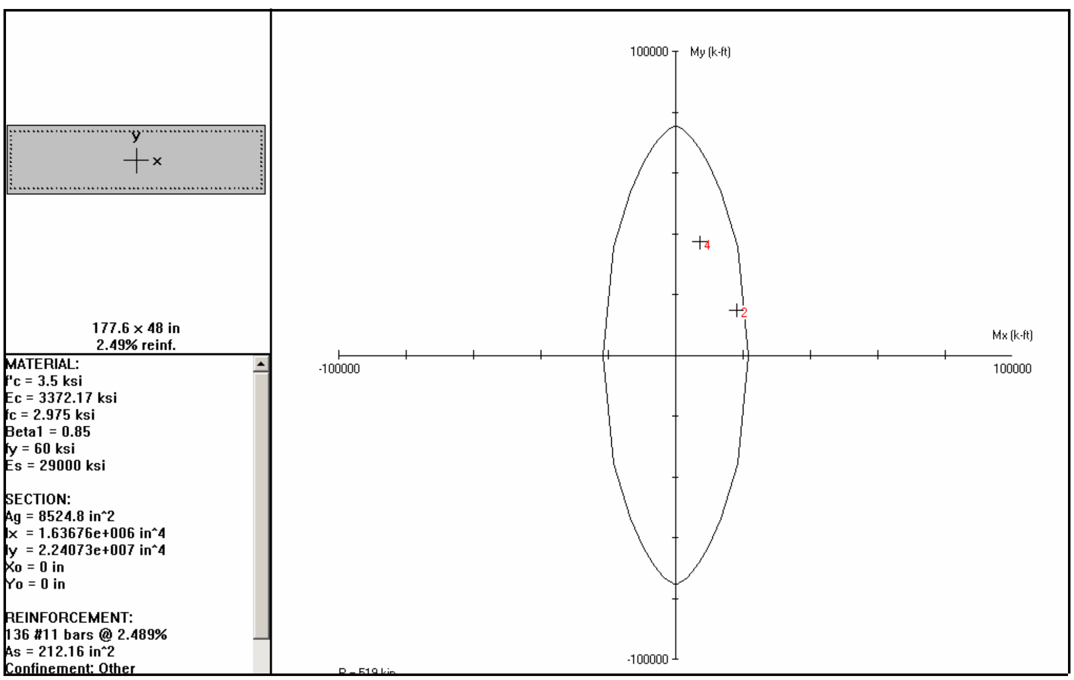
Sheet No: **40**





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

$h := 48$ in   thickness of the wall

1-

$D := h - 3$ in

$D := 48$ in

2-

$H := 256.25$ in

$\frac{1}{6} \cdot H = 42.708$ in

3-

18 in

4-

$\varepsilon_y := 0.00207$

$d_b := 1.41$ in   for #11

$M := 24711$ $k - ft$

$V := 1199$ k

$1.5\left(0.08 \cdot M \cdot \frac{12}{V} + 4400 \cdot \varepsilon_y \cdot d_b\right) = 48.941$ in   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)

$N := 196$ number of longitudinal reinforcement

$R := 0.8$ R-factor for joint (table 3.1-2)

$L := H$ in Height of the pier

$A_b := 1.56$ 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.

$P_d := 570$ kips Axial dead load in the pier

$P_e := 0$ kips Axial earthquake force

$M_{topL} := 0$ k – ft

$M_{botL} := 24711$ k – ft

$OS := 1.5$ Over strength factor

$M_{p\_topL} := OS \cdot M_{topL}$

$M_{p\_topL} = 0$ k – ft

$M_{p\_botL} := OS \cdot M_{botL}$

$M_{p\_botL} = 3.707 \times 10^4$ k – ft

$$V_{uL} := \frac{\left(M_{p\_topL} + M_{p\_botL}\right) \cdot 12}{L \cdot R}$$

$V_{uL} = 2.17 \times 10^3$ kips

$V_{u\_analysis} := \frac{V}{R}$ kips

$V_u := \max(V_{uL}, V_{u\_analysis})$

$V_u = 2.17 \times 10^3$ kips

$f_c := 3500$ psi

$b_w := 192$ in

$d := D$ in

$h = 48$ in

$V_c := .6\left[\sqrt{(f_c)}\right] \cdot b_w \cdot \frac{d}{1000}$ 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

$\frac{D_p}{L} = 0.168$

$V_p := \frac{\Lambda}{2} \cdot P_e \cdot \frac{D_p}{L}$

$V_p = 0$ kips Contribution due to arch action

$\phi := 0.90$ 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

$K_{shape} := .5$ For wall in weak axis (MCEER 8.8.2.3)

$f_{yh} := 60$ ksi — For the transverse and longitudinal steel

$f_{su} := 1.5 \cdot f_{yh}$ — Ultimate tensile stress of the longitudinal reinforcement

$f_{su} = 90$ ksi

$A_v := b_w \cdot d$ — Shear area

$A_v = 9.216 \times 10^3$ in^2

$\rho_t := .0332$ — Longitudinal reinforcement ratio

$s := 4$ in — < 4" OK (MCEER 8.8.2.4) for plastic hinge zone

$A_{sh} := .2$ #4 — Area of transverse reinforcement (ties)

$legs := 10$ — Number of legs

$$\rho_{v\_pr} := legs \cdot \frac{A_{sh}}{b_w \cdot s}$$

$$\rho_{v\_pr} = 2.604 \times 10^{-3}$$

$A_g := b_w \cdot h$

$A_g = 9.216 \times 10^3$ in^2

$A_c := (b_w - 6) \cdot (h - 6)$ — Area of column core concrete

$A_c = 7.812 \times 10^3$ in^2

$$\tan\theta := \left(1.6 \cdot \rho_{v\_pr} \cdot \frac{A_v}{\Lambda \cdot \rho_t \cdot A_g}\right)^{.25}$$

$\tan\theta = 0.595$

$D_{pp} := h - 4$ — Width of the perimeter transverse direction

$$A_{vs} := s \cdot \frac{V_s}{f_{yh} \cdot D_{pp}} \cdot \tan\theta$$

$$\frac{A_{vs}}{legs} = 0.188 \quad in^2$$ < Ash OK

Method 1: Implicit Shear Detailing Approach
(Guide Spec. 8.8.2.3)

$\rho_v := K_{shape} \cdot 2\Lambda \cdot \frac{\rho_t}{\phi} \cdot \frac{f_{su}}{f_{yh}} \cdot \frac{A_g}{A_c} \cdot \frac{D_p}{L} \cdot \tan\theta$ For fixed - fixed case of transverse direction

$\rho_v = 6.52 \times 10^{-3}$

$A_{vs} := \rho_v \cdot b_w \cdot s$

legs_new := 26

$\frac{A_{vs}}{legs\_new} = 0.193$ in^2 < Ash provided OK

Outside the plastic hinge zone
Method 2: Explicit Approach
8.8.2.3

legs_outplastic := 9

$V_c := 2 \cdot \sqrt{f_c} \cdot b_w \cdot \frac{d}{1000}$ Shear resistance of concrete out side of the plastic hinge zone

$V_c = 1.09 \times 10^3$ kips

$V_p = 0$

$V_s := \frac{V_u}{\phi} - (V_p + V_c)$

$V_s = 1.32 \times 10^3$ kips

$s_m := 6$ in Maximum spacing (MCEER 8.8.2.6)

$A_{shm} := .2$ #4

$\rho_{v\_pr} := legs\_outplastic \cdot \frac{A_{shm}}{b_w \cdot s_m}$

$\rho_{v\_pr} = 1.563 \times 10^{-3}$

$A_g = 9.216 \times 10^3$

$\tan\theta := \left(1.6 \cdot \rho_{v\_pr} \cdot \frac{A_v}{\Lambda \cdot \rho_t \cdot A_g}\right)^{.25}$

$\tan\theta = 0.524$

$D_{pp} = 44$ in

$A_{vs} := s_m \cdot \frac{V_s}{f_{yh} \cdot D_{pp}} \cdot \tan\theta$

$\frac{A_{vs}}{legs\_outplastic} = 0.175$ in^2 < Ash OK

Method 1: Implicit Shear Detailing Approach

$legs\_new := 25$

$f_{yh} = 60$

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

$\rho_{vstar} = 4.548 \times 10^{-3}$

$\rho_{pr} := legs\_new \cdot \frac{A_{shm}}{b_w \cdot s_m}$

$\rho_{pr} = 4.34 \times 10^{-3}$ Use method 2

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

$legs = 10$

$f_{yh} = 60$ ksi

$U_{sf} := 15.95$ ksi Strain energy capacity

$A_{sh} = 0.2$ in^2

$A_c := (b_w - 6)\cdot(h - 6)$

$A_c = 7.812 \times 10^3$ in^2

$s := 4$ in < 4" OK (MCEER 8.8.2.4)

$$\rho_{s1} := legs\cdot\frac{A_{sh}}{D_{pp}\cdot s}$$

$\rho_{s1} = 0.011$

$\rho_t = 0.033$

$A_g = 9.216 \times 10^3$

$f_c = 3.5 \times 10^3$

$P_e := P_d + P_e$

$P_e = 570$ kips Factored axial load include seismic effect

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

$\rho_{s2} = 0.011$ OK

$D_p = 43$

$legs = 10$

$legs\_pmax := 10$

$$\frac{legs\_pmax\cdot A_{sh}}{s\cdot D_p} = 0.012$$

Include the area of total legs in both direction of cross section (Guide Spec. 8.8.2.4)

$$s_{max} := legs\_pmax\cdot\frac{A_{sh}}{D_{pp}\cdot\rho_{s2}}$$

$s_{max} = 4.184$ in OK

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

$\rho_s = 0.011$

$s = 4$ in

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

$P_d = 570$ kips

$P_{eminL} := P_d \cdot .8$ kips For the uplift effect

$P_{eminL} = 456$ kips

$M_{p\_top} := 0$ $k - ft$

$M_{botL} = 2.471 \times 10^4$ $k - ft$

$OS = 1.5$

$P_{eL} := P_d + OS \cdot (P_{eminL} - P_d)$

$P_{eL} = 399$ kips

$M_{p\_bot} := OS \cdot M_{botL}$

$$V_u := \frac{M_{p\_bot} \cdot 12}{L \cdot R}$$

$V_u = 2.17 \times 10^3$ kips

$A_v = 9.216 \times 10^3$

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SUBJECT FILE:

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

NCHRP Guidelines

BY: **NA**

CHECKED: **AAA**

DATE: **6/22/2004**

Title: Pulaski County Bridge

Sheet No: **49**

$$V_c := 0.6 \cdot \sqrt{f_c} \cdot \frac{A_v}{1000}$$

$V_c = 327.136$ kips

$$V_p := \Lambda \cdot P_{eL} \cdot \frac{D_p}{L \cdot 2}$$

$V_p = 33.477$ kips

$$V_s := \frac{V_u}{\phi} - V_c - V_p$$

$V_s = 2.05 \times 10^3$

$s := 4$ in

$A_{sh} = 0.2$ in^2

$$\rho_v := legs \cdot \frac{A_{sh}}{b_w \cdot s}$$

$\rho_v = 2.604 \times 10^{-3}$

$$\tan\theta := \left[1.6 \cdot \rho_v \cdot \frac{A_v}{(\Lambda \cdot \rho_t \cdot A_g)} \right]^{.25}$$

$\tan\theta = 0.595$

$$\tan\alpha := \frac{D_p}{L}$$

$\tan\alpha = 0.168$ OK

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

$\frac{A_s}{legs} = 0.185$ OK < Ash OK

Anti buckling steel
(Guide Spec. 8.8.2.5)

$legs\_antibuckling := 70$

$d_b = 1.41$ in Diameter of longitudinal rebars

$s_{new} := 6 \cdot d_b$ Maximum spacing of ties

$s_{new} = 8.46$ in Required spacing of anti-buckling

$s = 4$ in

$A_{sh} := 0.44$ #6

$A_b := 1.56 \cdot N$

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

$$\frac{A_{bh}}{legs\_antibuckling} = 0.393 \quad in^2 < A_{sh} \quad \text{OK}$$

$s_{max} := .25 \cdot 60$

$s_{max} = 15$ OK

Final check for the plastic hinge zone
(Guide Spec. 4.9.1)

$\tan\theta = 0.595$

$$L_p := \frac{(h-4)}{12} \cdot \left(\frac{1}{\tan\theta} + .5 \cdot \tan\theta \right)$$

$L_p = 7.252$ ft

$V_u = 2.17 \times 10^3$

$M_{p\_botL} = 3.707 \times 10^4$ k − ft

$V_{uL} = 2.17 \times 10^3$ kips

$$\frac{M_{p\_botL}}{V_u}\cdot\left[1-.85\cdot\frac{\left(\frac{M_{p\_botL}}{1.5}\right)}{M_{p\_botL}}\right]=7.403 \quad \text{ft} \qquad \text{controls}$$

$$\frac{1.5}{12}\cdot\left(4400\cdot\varepsilon_y\cdot d_b+.08\cdot\frac{12M_{p\_botL}}{V_u}\right)=3.655$$

Summary of transverse reinforcement pier 2
#6@8" with 70 legs in 7.5' of bottom
#4@8" with 23 legs in 7.5' 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 2)

$f_c := 3500$ psi

$f_y := 60000$ psi

$h := 16$ ft

$b := 4$ ft

$d := .9\cdot h$

$d = 14.4$ ft

$l := \frac{H}{12}$ ft

$l = 21.354$ ft

$\frac{l}{d} = 1.483$ <5 so design anthology applies

$\Phi := 0.90$ for shear

$V = 1.199 \times 10^3$ kips

$M := 41430$ k − ft

$\Phi \cdot \left(8 \cdot \sqrt{f_c} \cdot b \cdot d \cdot \frac{144}{1000}\right) = 3.533 \times 10^3$ kips ΦVn

$A_1 := \frac{M}{V \cdot d}$

$A_1 = 2.4$

$A_2 := 3.5 - 2.5 \cdot \frac{M}{V \cdot d}$

$A_2 = -2.499$ A2 < 3.5 OK

But A2 <1 Hence use $A_2 := 1$

$\rho_t = 0.033$

$V_c := A_2 \cdot \left(1.9 \cdot \sqrt{f_c} + 2500 \cdot \rho_t \cdot \frac{V \cdot d}{M}\right) \cdot b \cdot d \cdot \frac{144}{1000}$

$V_c = 1.219 \times 10^3$ kips

$V_{cmax} := 6 \cdot \sqrt{f_c} \cdot b \cdot d \cdot \frac{144}{1000}$

$V_{cmax} = 2.944 \times 10^3$

$V_c := \min\left(V_c, V_{cmax}\right)$

$V_c = 1.219 \times 10^3$ kips

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

$V_s = 234.91$ kips Shear reinforcement is not required

$$K_1 := \frac{305.75}{4.5} \cdot \frac{\left(11 - \frac{h}{d}\right)}{12}$$

$$K_2 := \frac{V_s \cdot 1000}{f_y \cdot d \cdot 12}$$

$$S_v := \frac{\left[2 \cdot .79 \cdot \left(1 + \frac{h}{d}\right)\right]}{\left[12 \cdot \left(K_1 - K_2\right)\right]}$$

$$S_v := \left(\frac{12}{2 \cdot .79 \cdot 1 + \frac{h}{d}}\right) \cdot \left(K_1 - K_2\right)$$

$S_v = 249.571$ in

$$S_{vmax} := \min\left(18, \frac{b \cdot 12}{3}\right)$$

$S_{vmax} = 16$ in

Flexure design for deep beam

$\Phi := 1$

$M = 4.143 \times 10^4$ $k - ft$

$\frac{l}{h} = 1.335$ > 1 so jd=0.6l

$$j := .2 \frac{(l + 2 \cdot h)}{d}$$

$j = 0.741$

$j \cdot d = 10.671$ ft

$$A_s := \frac{M \cdot 12000}{\Phi \cdot f_y \cdot j \cdot d \cdot 12}$$

$A_s = 64.709$ in^2 < 307.75 in2 OK

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

$A_{smin} = 27.648$ in^2 < 305.75 in^2 OK $b := 2.5 \cdot 12$

Shear design in transverse direction

MCEER Guide Spec. 8.8.3

$h := 48$

$\rho_{hmin} := .0025$

$A_{sh\_rq} := \rho_{hmin} \cdot (b_w - 4) \cdot h$

$A_{sh\_rq} = 22.56$ in^2

$A_{sh} = 0.44$ 2 #6

$$N\_h := \frac{A_{sh\_rq}}{A_{sh}}$$

$N\_h = 51.273$ Number of horizontal reinforcement

$s_h := 4$ in

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

$V_r = 1.61 \times 10^3$ kips

$V_n := \phi\cdot\left(.756\cdot\sqrt{f_c} + \rho_{hmin}\cdot f_{yh}\cdot 1000\right)\cdot\left(b_w - 3\right)\cdot\frac{h}{1000}$

$V_n = 1.59\times 10^3$ kips

$V := \min\left(V_n, V_r\right)$

$V = 1.59\times 10^3$ kips

$V_{rq} := 1651$ kips (So minimum horizontal reinforcement is not adequate for this pier)

$\rho_h := .0029$

$A_{sh\_rq} := \rho_h\cdot\left(b_w - 4\right)\cdot h$

$A_{sh\_rq} = 26.17$ in^2

$A_{sh} := 0.88$ 2 #6

$N\_h := \frac{A_{sh\_rq}}{A_{sh}}$

$N\_h = 29.738$ Number of horizontal reinforcement

$s_h := 4$ in

$V_n := \phi\cdot\left(.756\cdot\sqrt{f_c} + \rho_h\cdot f_{yh}\cdot 1000\right)\cdot\left(b_w - 3\right)\cdot\frac{h}{1000}$

$V_n = 1.786\times 10^3$ kips

$V_{rq} = 1.651\times 10^3$ 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

$h := 48$ in thickness of the wall

1-

$D := h - 3$ in

$D := 48$ in

2-

$H := 256.25$ in

$\frac{1}{6} \cdot H = 42.708$ n

3-

18 in

4-

$\varepsilon_y := 0.00207$

$d_b := 1.41$ in for #11

$M := 29060$ k − ft

$V := 1651$ k

$1.5\left(0.08 \cdot M \cdot \frac{12}{V} + 4400 \cdot \varepsilon_y \cdot d_b\right) = 44.61$ in 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)

$L := H$ in Height of the pier

$A_b := 1.56$ 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.

$P_d := 470$ kips Axial dead load in the pier

$P_e := 164$ kips Axial earthquake force

$M_{topL} := 0$ k − ft

$M_{botL} := M$

$OS := 1.5$ Over strength factor

$M_{p\_topL} := OS \cdot M_{topL}$

$M_{p\_topL} = 0$ k − ft

$M_{p\_botL} := OS \cdot M_{botL}$

$M_{p\_botL} = 4.359 \times 10^4$ k − ft

$$V_{uL} := \frac{\left(M_{p\_topL} + M_{p\_botL}\right) \cdot 12}{L \cdot R}$$

$V_{uL} = 2.552 \times 10^3$ kips

$V_{u\_analysis} := \frac{V}{R}$ kips

$V_u := \max(V_{uL}, V_{u\_analysis})$

$V_u = 2.552 \times 10^3$ kips

$f_c := 3500$ psi

$b_w := 192$ in

$d := D$ in

$h = 48$ in

$V_c := .6\left[\sqrt{(f_c)}\right] \cdot b_w \cdot \frac{d}{1000}$ Shear resistance in the end regions (plastic hinge zone)

$V_c = 327.136$ kips

$L := 253.5$ in

$D_p := h - 5$ Width of the column core

$D_p = 43$ in

$\frac{D_p}{L} = 0.17$

$V_p := \frac{\Lambda}{2} \cdot P_e \cdot \frac{D_p}{L}$

$V_p = 13.909$ kips Contribution due to arch action

$\phi := 0.90$ For shear

$V_u = 2.552 \times 10^3$ kips

$V_s := \frac{V_u}{\phi} - (V_p + V_c)$

$V_s = 2.494 \times 10^3$ kips

$K_{shape} := .5$ For wall in weak axis (MCEER 8.8.2.3)

$f_{yh} := 60$ ksi — For the transverse and longitudinal steel

$f_{su} := 1.5 \cdot f_{yh}$ — Ultimate tensile stress of the longitudinal reinforcement

$f_{su} = 90$ ksi

$A_v := b_w \cdot d$ — Shear area

$A_v = 9.216 \times 10^3$ in^2

$\rho_t := .0332$ — Longitudinal reinforcement ratio

$s := 4$ in — < 4" OK (MCEER 8.8.2.4) for plastic hinge zone

$A_{sh} := .2$ #4 — Area of transverse reinforcement (ties)

$legs := 13$ — Number of legs

$$\rho_{v\_pr} := legs \cdot \frac{A_{sh}}{b_w \cdot s}$$

$$\rho_{v\_pr} = 3.385 \times 10^{-3}$$

$$A_g := b_w \cdot h$$

$A_g = 9.216 \times 10^3$ in^2

$$\tan\theta := \left(1.6 \cdot \rho_{v\_pr} \cdot \frac{A_v}{\Lambda \cdot \rho_t \cdot A_g}\right)^{.25}$$

$$\tan\theta = 0.636$$

$D_{pp} := h - 4$ — Width of the perimeter transverse direction

$$A_{vs} := s \cdot \frac{V_s}{f_{yh} \cdot D_{pp}} \cdot \tan\theta$$

$\frac{A_{vs}}{legs} = 0.185$ in^2 < Ash OK

Method 1: Implicit Shear Detailing Approach
(Guide Spec. 8.8.2.3)

$\rho_v := K_{shape} \cdot 2\Lambda \cdot \frac{\rho_t}{\phi} \cdot \frac{f_{su}}{f_{yh}} \cdot \frac{A_g}{A_v} \cdot \frac{D_p}{L} \cdot \tan\theta$ For fixed - fixed case of transverse direction

$\rho_v = 5.965 \times 10^{-3}$

$A_{vs} := \rho_v \cdot b_w \cdot s$

legs_new := 26

$\frac{A_{vs}}{legs\_new} = 0.176$ in^2 < Ash provided OK

Outside the plastic hinge zone
Method 2: Explicit Approach
8.8.2.3

legs_outplastic := 13

$V_c := 2 \cdot \sqrt{f_c} \cdot b_w \cdot \frac{d}{1000}$ Shear resistance of concrete out side of the plastic hinge zone

$V_c = 1.09 \times 10^3$ kips

$V_p = 13.909$

$V_s := \frac{V_u}{\phi} - (V_p + V_c)$

$V_s = 1.731 \times 10^3$ kips

$s_m := 6$ in   Maximum spacing (MCEER 8.8.2.6)

$A_{shm} := .2$ #4

$\rho_{v\_pr} := legs\_outplastic \cdot \frac{A_{shm}}{b_w \cdot s_m}$

$\rho_{v\_pr} = 2.257 \times 10^{-3}$

$A_g = 9.216 \times 10^3$

$\tan\theta := \left(1.6 \cdot \rho_{v\_pr} \cdot \frac{A_v}{\Lambda \cdot \rho_t \cdot A_g}\right)^{.25}$

$\tan\theta = 0.574$

$D_{pp} = 44$ in

$A_{vs} := s_m \cdot \frac{V_s}{f_{yh} \cdot D_{pp}} \cdot \tan\theta$

$\frac{A_{vs}}{legs\_outplastic} = 0.174$ in^2 < Ash OK

Method 1: Implicit Shear Detailing Approach

$legs\_new := 27$

$f_{yh} = 60$

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

$\rho_{vstar} = 3.993 \times 10^{-3}$

$\rho_{pr} := legs\_new \cdot \frac{A_{shm}}{b_w \cdot s_m}$

$\rho_{pr} = 4.688 \times 10^{-3}$ Use method 2

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

$legs = 13$

$f_{yh} = 60$ ksi

$U_{sf} := 15.95$ ksi Strain energy capacity

$A_{sh} = 0.2$ in^2

$A_c := (b_w - 6)\cdot(h - 6)$

$A_c = 7.812 \times 10^3$ in^2

$s := 4$ in < 4" OK (MCEER 8.8.2.4)

$$\rho_{s1} := legs \cdot \frac{A_{sh}}{D_{pp} \cdot s}$$

$\rho_{s1} = 0.015$

$\rho_t = 0.033$

$A_g = 9.216 \times 10^3$

$f_c = 3.5 \times 10^3$

$P_e := P_d + P_e$

$P_e = 634$ kips Factored axial load include seismic effect

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

$\rho_{s2} = 0.011$ OK

$D_p = 43$

$legs = 13$

$legs\_pmax := 10$

$$\frac{legs\_pmax \cdot A_{sh}}{s \cdot D_p} = 0.012$$

Include the area of total legs in both direction of cross section (Guide Spec. 8.8.2.4)

$$s_{max} := legs\_pmax \cdot \frac{A_{sh}}{D_{pp} \cdot \rho_{s2}}$$

$s_{max} = 4.151$ in OK

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

$\rho_s = 0.015$

$s = 4$ in

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

$P_d = 470$ kips

$P_{eminL} := P_d \cdot .8$ kips For the uplift effect

$P_{eminL} = 376$ kips

$M_{p\_top} := 0$ k − ft

$M_{botL} = 2.906 \times 10^4$ k − ft

$OS = 1.5$

$P_{eL} := P_d + OS \cdot (P_{eminL} - P_d)$

$P_{eL} = 329$ kips

$M_{p\_bot} := OS \cdot M_{botL}$

$V_u := \dfrac{M_{p\_bot} \cdot 12}{L \cdot R}$

$V_u = 2.579 \times 10^3$ ips

$A_v = 9.216 \times 10^3$

$V_c := 0.6 \cdot \sqrt{f_c} \cdot \dfrac{A_v}{1000}$

$V_c = 327.136$ kips

$$V_p := \Lambda \cdot P_{eL} \cdot \frac{D_p}{L \cdot 2}$$

$V_p = 27.903$ kips

$$V_s := \frac{V_u}{\phi} - V_c - V_p$$

$V_s = 2.511 \times 10^3$

$s := 4$ in

$A_{sh} = 0.2$ in^2

$$\rho_v := legs \cdot \frac{A_{sh}}{b_w \cdot s}$$

$\rho_v = 3.385 \times 10^{-3}$

$$\tan\theta := \left[1.6 \cdot \rho_v \cdot \frac{A_v}{(\Lambda \cdot \rho_t \cdot A_g)}\right]^{.25}$$

$\tan\theta = 0.636$

$$\tan\alpha := \frac{D_p}{L}$$

$\tan\alpha = 0.17$ OK

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

$\frac{A_s}{legs} = 0.186$ OK < Ash OK

Anti buckling steel
(Guide Spec. 8.8.2.5)

legs_antibuckling := 70

$d_b = 1.41$ in Diameter of longitudinal rebars

$s_{new} := 6 \cdot d_b$ Maximum spacing of ties

$s_{new} = 8.46$ in Required spacing of anti-buckling

$s = 4$ in

$A_{sh} := .44$ #6

$A_b := 1.56 \cdot N$

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

$\frac{A_{bh}}{legs\_antibuckling} = 0.393$ < Ash OK

$s_{max} := .25 \cdot 60$

$s_{max} = 15$ OK

Final check for the plastic hinge zone (Guide Spec. 4.9.1)

$\tan\theta = 0.636$

$L_p := \frac{(h-4)}{12} \cdot \left(\frac{1}{\tan\theta} + .5 \cdot \tan\theta \right)$

$L_p = 6.934$ ft

$V_u = 2.579 \times 10^3$

$M_{p\_botL} = 4.359 \times 10^4$ k − ft

$V_{uL} = 2.552 \times 10^3$ kips

$\frac{M_{p\_botL}}{V_u} \cdot \left[1 - .85 \cdot \frac{\left(\frac{M_{p\_botL}}{1.5} \right)}{M_{p\_botL}} \right] = 7.323$ ft controls

$$\frac{1.5}{12}\cdot\left(4400\cdot\varepsilon_y\cdot d_b + .08\cdot\frac{12M_{p\_botL}}{V_u}\right) = 3.633$$

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)

$f_c := 3500$ psi

$f_y := 60000$ psi

$h := 16$ ft

$b := 4$ ft

$d := .9\cdot h$

$d = 14.4$ ft

$l := \frac{L}{12}$ ft

$l = 21.125$ ft

$\frac{l}{d} = 1.467$ <5 so design anthology applies

$\Phi := 0.90$ for shear

$V = 1.651 \times 10^3$ kips

$M := 39190$ k − ft

$\Phi \cdot \left(8 \cdot \sqrt{f_c} \cdot b \cdot d \cdot \frac{144}{1000}\right) = 3.533 \times 10^3$ kips ΦVn

$A_1 := \frac{M}{V \cdot d}$

$A_1 = 1.648$

$A_2 := 3.5 - 2.5 \cdot \frac{M}{V \cdot d}$

$A_2 = -0.621$ A2 < 3.5 OK

$\rho_t = 0.033$

$V_c := A_2 \cdot \left(1.9 \cdot \sqrt{f_c} + 2500 \cdot \rho_t \cdot \frac{V \cdot d}{M}\right) \cdot b \cdot d \cdot \frac{144}{1000}$

$V_c = -838.372$ kips use 0

$V_{cmax} := 6 \cdot \sqrt{f_c} \cdot b \cdot d \cdot \frac{144}{1000}$

$V_{cmax} = 2.944 \times 10^3$

$V_c := \min(V_c, V_{cmax})$

$V_c := 0$ kips

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

$V_s = 1.834 \times 10^3$ kips Shear reinforcement is not required

$K_1 := \frac{305.75}{4.5} \cdot \frac{\left(11 - \frac{h}{d}\right)}{12}$

$K_2 := \frac{V_s \cdot 1000}{f_y \cdot d \cdot 12}$

$$S_v := \frac{\left[2\cdot .79\cdot\left(1+\frac{h}{d}\right)\right]}{\left[12\cdot\left(K_1 - K_2\right)\right]}$$

$$S_v := \left(\frac{12}{2\cdot .79\cdot 1 + \frac{h}{d}}\right)\cdot\left(K_1 - K_2\right)$$

$S_v = 248.883$ in

$S_{vmax} := \min\left(18, \frac{b\cdot 12}{3}\right)$

$S_{vmax} = 16$ in

Flexure design for deep beam

$\Phi := 1$

$M = 3.919\times 10^4$ k − ft

$\frac{l}{h} = 1.32$ > 1 so jd=0.6l

$j := .2\,\frac{(l + 2\cdot h)}{d}$

$j = 0.738$

$j\cdot d = 10.625$ ft

$$A_s := \frac{M\cdot 12000}{\Phi\cdot f_y\cdot j\cdot d\cdot 12}$$

$A_s = 61.475$ in^2 < 307.75 in2 OK

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

$A_{smin} = 27.648$ in^2 < 305.75 in^2 OK

Shear design in transverse direction

MCEER Guide Spec. 8.8.3

$h := 48$

$\rho_{hmin} := .0025$

$A_{sh\_rq} := \rho_{hmin} \cdot (b_w - 4) \cdot h$

$A_{sh\_rq} = 22.56$ in^2

$A_{sh} = 0.44$ 2 #6

$N\_h := \frac{A_{sh\_rq}}{A_{sh}}$

$N\_h = 51.273$ Number of horizontal reinforcement

$s_h := 4$ in

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

$V_r = 1.61 \times 10^3$ kips

$V_n := \phi \cdot \left(.756 \cdot \sqrt{f_c} + \rho_{hmin} \cdot f_{yh} \cdot 1000\right) \cdot (b_w - 3) \cdot \frac{h}{1000}$

$V_n = 1.59 \times 10^3$ kips

$V := \min(V_n, V_r)$

$V = 1.59 \times 10^3$ kips

$V_{rq} := 1651$ kips (So minimum horizontal reinforcement is not adequate for this pier)

$\rho_h := .0029$

$A_{sh\_rq} := \rho_h \cdot (b_w - 4) \cdot h$

$A_{sh\_rq} = 26.17$ in^2

$A_{sh} := 0.88$ 2 #6

$N\_h := \dfrac{A_{sh\_rq}}{A_{sh}}$

$N\_h = 29.738$ Number of horizontal reinforcement

$s_h := 4$ in

$V_n := \phi \cdot \left(.756 \cdot \sqrt{f_c} + \rho_h \cdot f_{yh} \cdot 1000\right) \cdot (b_w - 3) \cdot \dfrac{h}{1000}$

$V_n = 1.786 \times 10^3$ kips

$V_{rq} = 1.651 \times 10^3$ kips (So 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)

$N := 196$ number of longitudinal reinforcement

$b_w := 48$ in Height of pile cap

$H_c := 48$ in Height of the joint

$h := 48$ in Dimension of the column

$D := h - 3$ in

$D = 45$ in

$R := 0.8$ R-factor for joint (table 3.1-2)

$\varepsilon_y := .00207$ For the longitudinal steel

$L := 256.25$ in Height of the pier

$A_b := 1.56$ in^2 Area of rebars for longitudinal rebar

$P_d := 570$ kips Axial dead load in the pier

$P_e := 0$ kips Axial earthquake force

$M_{botL} := 24711$ $k - ft$

$OS := 1.5$ Over strength factor

$M_{p\_botL} := OS \cdot M_{botL}$

$M_{p\_botL} = 3.707 \times 10^4$ $k - ft$

$f_c := 3500$ psi

$b_w := 192$ in

$d := D$ in

$L = 256.25$ in

$D_p := h - 4$ Width of the column core

$D_p = 44$ in

$\phi := 0.90$ For shear

$f_{yh} := 60$ ksi For the transverse and longitudinal steel

$f_{su} := 1.5 \cdot f_{yh}$ Ultimate tensile stress of the longitudinal reinforcement

$f_{su} = 90$ ksi

$A_v := b_w \cdot d$ Shear area

$A_v = 8.64 \times 10^3$ in^2

$\rho_t := .0332$ Longitudinal reinforcement ratio

$s := 4$ in < 4" OK (MCEER 8.8.2.4) for plastic hinge zone

$A_{sh} := .2$ #4 Area of transverse reinforcement (ties)

$legs := 16$ Number of legs

$$\rho_{v\_pr} := legs \cdot \frac{A_{sh}}{b_w \cdot s}$$

$\rho_{v\_pr} = 4.167 \times 10^{-3}$

$A_g := b_w \cdot h$

$A_g = 9.216 \times 10^3$ in^2

$L = 256.25$ in Length of the column

$A_b := 1.56$ in^2 For longitudinal rebar

$D_p = 44$ in Width of the column core

$\phi := 0.90$ For shear

$A_v := D_p^{\,2}$ Shear area Av=bw*d

$A_g := D^2$

$f_{yh} := 60$ ksi

$f_{su} := 1.5 \cdot f_{yh}$

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

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

$A_{sc} = 305.76$ 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

$P := P_d + P_e$

$P = 570$ kips Maximum axial force include earthquake effect

$M_{p\_botL} = 3.707 \times 10^4$ $k - ft$ From push over analysis

$b_b := 20 \cdot 12$ in Width of pile cap

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

$L_{mid\_dept\_jt} = 93$ in

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

$A_{mid} = 2.232 \times 10^4$ in^2

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

$f_v = 0$ ksi

$h_b := H_c$ Joint depth

$h_b = 48$ in

$h_c := D$ Dimension of column

$h_c = 45$ in

$b_{je} := 2h_c$ effective joint width for shear stress calculations

$b_{je} = 90$ in

$v_{hv} := \frac{M_{p\_botL} \cdot 12}{h_b \cdot h_c \cdot b_{je}}$ Joint shear stress

$v_{hv} = 2.288$ ksi

$p_t := \frac{\left(f_h + f_v\right)}{2} - \sqrt{\left[\frac{\left(f_h - f_v\right)}{2}\right]^2 + v_{hv}^2}$ Principal tension stress

$p_t = -2.288$

$p_{t\_max} := \frac{3.5}{1000} \cdot \sqrt{f_c}$ 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

$p_c = 2.288$

$$p_{c\_max} := \frac{.25}{1000} \cdot f_c$$ 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)

$A_{jv} = 48.922$ in^2

Use #7

$\frac{A_{jv}}{.6} = 81.536$ $\boxed{\text{72\#7 vertical legs in (development length) each side}}$

$A_{clamp} := .08 \cdot A_{sc}$ Vertical reinforcement (Clamping reinforcement) (Guide Spec. 8.8.4.3.2)

$A_{clamp} = 24.461$

$\frac{A_{clamp}}{.6} = 40.768$ $\boxed{\text{31\#7 in 36" of the joint}}$

$$l_{d9} := .63 \cdot 1.128 \cdot \frac{f_{yh}}{\sqrt{\frac{f_c}{1000}}}$$ Development length for #9 (AASHTO LRFD 5.11.2.2.1-1)

$0.3 \cdot 1.128 \cdot f_{yh} = 20.304$ in

$l_{d9} = 22.791$ in

$A_h := 0.08 \cdot A_{sc}$ Horizontal reinforcement (Guide Spec. 8.8.4.3.3)

$A_h = 24.461$ $\boxed{\text{25\#9 in 40" length each side}}$

$l_{d11} := 1.25 \cdot 1.56 \cdot \dfrac{f_{yh}}{\sqrt{\dfrac{f_c}{1000}}}$

$l_{d11} = 62.539$

$.4 \cdot 1.41 \cdot f_{yh} = 33.84$

$\rho_s := .4 \dfrac{A_{sc}}{l_{d11}^{\,2}}$ Hoops (Guide Spec. 8.8.4.3.4)

$\rho_s = 0.031$

$\rho_{s\_min} := \dfrac{3.5}{1000} \cdot \dfrac{\sqrt{f_c}}{f_{yh}}$ Minimum required horizontal reinforcement (Guide Spec. 8.8.4.2.2)

$\rho_{s\_min} = 3.451 \times 10^{-3}$ OK

$A_{sh} := .2$ in^2

$s = 4$ $in \le 4$ in OK (MCEER8.8.2.4)

$legs := 23$

$s_{max} := legs \cdot \dfrac{A_{sh}}{D_{pp} \cdot \rho_s}$

$s_{max} = 4.745$ 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

$b_w := 48$ in Height of pile cap

$H_c = 48$ in Height of the joint

$h = 48$ in Dimension of the column

$D := h - 3$ in

$D = 45$ in

$R := 0.8$ R-factor for joint (table 3.1-2)

$\varepsilon_y := .00207$ For the longitudinal steel

$L := 253.5$ in Height of the pier

$A_b := 1$ 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.

$P_d := 470$ kips Axial dead load in the pier

$P_e := 164$ kips Axial earthquake force

$M_{botL} := 29060$ k – ft

$OS := 1.5$ Over strength factor

$M_{p\_botL} := OS \cdot M_{botL}$

$M_{p\_botL} = 4.359 \times 10^4$ k – ft

$f_c := 3500$ psi

$b_w := 196$ in

$d := D$ in

$L = 253.5$ in

$D_p := h - 4$ Width of the column core

$D_p = 44$ in

$\phi := 0.85$ For shear

$f_{yh} := 60$ ksi For the transverse and longitudinal steel

$f_{su} := 1.5 \cdot f_{yh}$ Ultimate tensile stress of the longitudinal reinforcement

$f_{su} = 90$ ksi

$A_v := b_w \cdot d$ Shear area

$A_v = 8.82 \times 10^3$ in^2

$\rho_t = 0.033$ Longitudinal reinforcement ratio

$s := 4$ in < 4" OK (MCEER 8.8.2.4) for plastic hinge zone

$A_{sh} := .2$ #4 Area of transverse reinforcement (ties)

$legs := 16$ Number of legs

$$\rho_{v\_pr} := legs \cdot \frac{A_{sh}}{b_w \cdot s}$$

$$\rho_{v\_pr} = 4.082 \times 10^{-3}$$

$A_g := b_w \cdot h$

$A_g = 9.408 \times 10^3$ in^2

$L = 253.5$ in Length of the column

$A_b = 1$ in^2 For longitudinal rebar

$D_p = 44$ in Width of the column core

$\phi := 0.85$ For shear

$A_v := D_p^{\,2}$ Shear area Av=bw*d

$A_g := D^2$

$f_{yh} := 60$ 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

$A_{sc} = 196$ 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

$P := P_d + P_e$

$P = 634$ kips — Maximum axial force include earthquake effect

$M_{p\_botL} = 4.359 \times 10^4$ $k - ft$ — From push over analysis

$b_b := 25 \cdot 12$ in — Width of pile cap

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

$L_{mid\_dept\_jt} = 93$ in

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

$A_{mid} = 2.79 \times 10^4$ in^2

$f_v := \dfrac{P_e}{A_{mid}}$ — Average axial stress in the vertical direction

$f_v = 5.878 \times 10^{-3}$ ksi

$h_b := H_c$ — Joint depth

$h_b = 48$ in

$h_c := D$ — Dimension of column

$h_c = 45$ in

$b_{je} := 2h_c$ — effective joint width for shear stress calculations

$b_{je} = 90$ in

$v_{hv} := \frac{M_{p\_botL} \cdot 12}{h_b \cdot h_c \cdot b_{je}}$ Joint shear stress

$v_{hv} = 2.691$ 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

$p_t = -2.688$

$p_{t\_max} := \frac{3.5}{1000} \cdot \sqrt{f_c}$ 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

$p_c = 2.694$

$p_{c\_max} := \frac{.25}{1000} \cdot f_c$ Maximum compression stress

$p_{c\_max} = 0.875$ Not OK, we should provide some horizontal reinforcement

$A_{jv} := .16 A_{sc}$ Vertical reinforcement (Stirrups) (Guide Spec. 8.8.4.3.2)

$A_{jv} = 31.36$ in^2

Use #7

$\frac{A_{jv}}{.6} = 52.267$ 53#7 vertical legs in (development length) each side

$A_{clamp} := .08 \cdot A_{sc}$ Vertical reinforcement (Clamping reinforcement) (Guide Spec. 8.8.4.3.2)

$A_{clamp} = 15.68$

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$\frac{A_{clamp}}{.6} = 26.133$ $\boxed{\text{27\#7 in 36" of the joint}}$

$l_{d9} := .63 \cdot 1.128 \cdot \frac{f_{yh}}{\sqrt{\frac{f_c}{1000}}}$ Development length for #9 (AASHTO LRFD 5.11.2.2.1-1)

$0.3 \cdot 1.128 \cdot f_{yh} = 20.304$ in

$l_{d9} = 22.791$ in

$A_h := 0.08 \cdot A_{sc}$ Horizontal reinforcement (Guide Spec. 8.8.4.3.3)

$A_h = 15.68$ $\boxed{\text{16\#9 in 40" length each side}}$

$l_{d11} := 1.25 \cdot 1.56 \cdot \frac{f_{yh}}{\sqrt{\frac{f_c}{1000}}}$

$l_{d11} = 62.539$

$.4 \cdot 1.41 \cdot f_{yh} = 33.84$

$\rho_s := .4 \frac{A_{sc}}{l_{d11}^{\,2}}$ Hoops (Guide Spec. 8.8.4.3.4)

$\rho_s = 0.02$

$\rho_{s\_min} := \frac{3.5}{1000} \cdot \frac{\sqrt{f_c}}{f_{yh}}$ Minimum required horizontal reinforcement (Guide Spec. 8.8.4.2.2)

$\rho_{s\_min} = 3.451 \times 10^{-3}$ OK

$A_{sh} := .2$ in^2

$s := 4$ in

$legs := 23$

$s_{max} := legs \cdot \frac{A_{sh}}{D_{pp} \cdot \rho_s}$ $s_{max} = 5.337$ in $\boxed{\text{Use \#4@4" with 23 legs}}$

P-Δ Requirements

(Guide Spec. 8.3.4)

Pier 2

$W := 525 + 2 \cdot 308$ kips — Average axial force at wall (Axial load taken from mid hight of Pier 2=525 kips +Axial load at top of Pier 1&3 =308 kip)

$V := 3636$ kips — from SAP model

$$H := \frac{256.25}{12}$$

$H = 21.354$ ft

$$C := \frac{V}{W}$$

$C = 3.187$

$\Delta_{limit} := .25 \cdot C \cdot H$

$\Delta_{limit} = 17.012$ ft

$\Delta := .52$ ft < Δlimit OK — Displacement of 0.52 ft is taken from SAP model

Pier 1&3

$W := 1197$ kips — Average axial force at column (Axial load taken from mid hight of Pier 1,2&3= 1197 KIPS)

$V := 2448$ kips

$$H := \frac{L}{12}$$

$H = 21.125$ ft

$$C := \frac{V}{W}$$

$C = 2.045$

$\Delta_{limit} := .25 \cdot C \cdot H$

$\Delta_{limit} = 10.801$ ft

$\Delta := 0.122$ ft < Δlimit OK Displacement of 0.122 ft is taken from SAP model

Minimum seat requirement

$L := 232 \cdot .3048$

$L = 70.714$ m Length of bridge

$H := \frac{253.5}{12} \cdot .3048$

$H = 6.439$ m Height of the tallest pier

$B := 30 \cdot .3048$

$B = 9.144$ m Width of superstructure

$F_v := 1.5$

$S_1 := .919$

$\alpha := 0$

$$N := \left[.1 + .0017 \cdot L + .007 \cdot H + .05 \cdot \sqrt{H} \cdot \sqrt{1 + \left(2 \cdot \frac{B}{L}\right)^2} \right] \cdot \left(1 + 1.25 \cdot F_v \cdot S_1\right)$$

$N = 1.079$ m

$N := \frac{N}{.3048}$

$N = 3.541$ ft 43" Minimum seat width

Moment check at the cantilever part of the wall pier

$M_{eq} := 99$ k − ft

$M_{DL} := 495$ k − ft

$R := 1.0$

$M := \frac{M_{eq}}{R} + M_{DL}$

$M = 594$ k − ft

$b := 4 \cdot 12$ in

$h := 4.5 \cdot 12$

$d := h - 4$

$A := h \cdot b$ Section area at hammer part of the wall pier

$A = 2.592 \times 10^3$ in^2

$\rho := \left(\frac{4}{2.5}\right) \cdot (10) \cdot \frac{1}{A}$ The existing ratio of steel has increased by the ratio of new wall (4' thick) to the existing wall thickness (2.5')

$\rho = 6.173 \times 10^{-3}$

$R := \frac{M \cdot 12000}{b \cdot d^2}$

$R = 59.4$

$\rho_{rq} := .0025$ $< \rho$ OK

Pile cap design procedure for Pulasky County bridge

$f_c := 3500$ psi

$Fy := 60000$ psi

Pile punching shear check:

$P_{axial} := 245$ kips (from sheet 30)

$Vu := P_{axial}$

$Vu = 245$ Kips

$d := 45$ in

$b := (14.2 + d)\cdot 4$ in

$Vc := 4\cdot\sqrt{f_c}\cdot\frac{b\cdot d}{1000}$

$Vc = 2.522 \times 10^3$ kips

$\phi := 0.9$

$\frac{Vu}{\phi} = 272.222$ $\frac{Vu}{\phi} < Vc$ OK

One way shear Check:

$V1 := 10P_{axial}$

$V1 = 2.45 \times 10^3$ Kips

$b2 := 55\cdot 12$ in

$b2 = 660$ in

$Vc2 := 4\cdot\sqrt{f_c}\cdot b2\cdot\frac{d}{1000}$

$Vc2 = 7.028 \times 10^3$

$\frac{V1}{\phi} = 2.722 \times 10^3$ $\frac{V1}{\phi} < Vc2$ OK

Flexure bending design: Only in one direction

$Mu := 10 \cdot P_{axial} \cdot 1000 \cdot (1.5 \cdot 70)$

$Mu = 2.572 \times 10^8$ lb − in $\quad \Phi := 0.9 \quad y_t := \dfrac{d+3}{2} \quad y_t = 24$ in

$f_r := 7.5 \cdot \sqrt{f_c} \quad f_r = 443.706$

$I_g := \dfrac{b2 \cdot (d+3)^3}{12} \quad I_g = 6.083 \times 10^6$ in^4

$M_{cr} := \dfrac{f_r \cdot I_g}{y_t} \quad M_{cr} = 1.125 \times 10^8$ lb − in

$Mu > 1.2 M_{cr}$ OK

$As := \dfrac{Mu}{\Phi \cdot Fy \cdot 0.9 \cdot d}$

$As = 117.627$ in^2

$Asmin := 0.0018 \cdot (d+3) \cdot b2$

$Asmin = 57.024$ in^2

$N_L := \dfrac{As}{.79}$

$N_L = 148.895$

In short Direction : 149 # 8

$Asmin := 0.0018 \cdot (d+3) \cdot 280$

$Asmin = 24.192$ in^2

$N_T := \dfrac{Asmin}{.44}$

$N_T = 54.982$

In long direction: 55 # 6

Axial Capacity for uplift:

$S_1 := 11230$

$S_2 := 1361$

$M_{long} := 31836 \quad k - ft$

$M_{tran} := 17848 \quad k - ft$

$N := 40$

$V := 1065 \quad kips$

$$P_{uplift} := \frac{V}{N} - M_{long} \cdot 1.5 \cdot \frac{70}{12 \cdot S_2} - M_{tran} \cdot 4.5 \cdot \frac{70}{12 \cdot S_1}$$

$P_{uplift} = -219.771 \quad kips$

$$L_{pile} := \frac{(14.2 \cdot 2 + 14.9 \cdot 2)}{12} ft$$

$L_{pile} = 4.85$ Area of the pile

$L1 := 4 \quad ft$

$L2 := 41 \quad ft$

$L := 45 \quad ft$ length of the pile

$$qu1 := \frac{38 \cdot 2000}{144} \quad psi$$

$qu1 = 527.778$

$$qu2 := \frac{50 \cdot 2000}{144} \quad psi$$

$qu2 = 694.444$

$f1 := 2.5\cdot\sqrt{qu1}$ $f1 = 57.434$ psi Less than $.15\cdot qu1 = 79.167$ psi

$f2 := 2.5\cdot\sqrt{qu2}$ $f2 = 65.881$ psi less than $.158qu2 = 109.722$ psi

$Qu1 := \dfrac{L_{pile}\cdot L1\cdot f1\cdot 144}{1000}$ $Qu1 = 160.446$ kips

$Qu2 := \dfrac{L_{pile}\cdot L2\cdot f2\cdot 144}{1000}$ $Qu2 = 1.886\times 10^{3}$ kips

$Wpile := .490\cdot\left(\dfrac{34.4}{144}\right)\cdot L$ $Wpile = 5.268$ kips

$$Qallow := 1.33\cdot\frac{(Qu1 + Qu2 - Wpile)}{2}$$

$Qallow = 1.358\times 10^{3}$ kips

$$Qallowuplift := 1.33\cdot\frac{(0.7Qu1 + 0.7Qu2 + Wpile)}{2}$$

$Qallowuplift = 956.336$ kips

Flexural Design for uplift:

$Mu := 10\cdot P_{uplift}\cdot 1000\cdot(1.5\cdot 70)$

$Mu := 2.308\cdot 10^{8}$ lb − in $\Phi := 0.9$ $y_t := \dfrac{d+3}{2}$ $y_t = 24$ in

$f_r := 7.5\cdot\sqrt{f_c}$ $f_r = 443.706$

$I_g := \dfrac{b2\cdot(d+3)^3}{12}$ $I_g = 6.083\times 10^{6}$ in^4

$M_{cr} := \frac{f_r \cdot I_g}{y_t}$ $\quad M_{cr} = 1.125 \times 10^8$ lb − in

$Mu > 1.2M_{cr}$ OK

$As := \frac{Mu}{\Phi \cdot Fy \cdot 0.9 \cdot d}$

$As = 105.533$ in^2

$Asmin := 0.0018 \cdot (d + 3) \cdot b2$

$Asmin = 57.024$ in^2

$N_L := \frac{As}{.79}$

$N_L = 133.586$

In short Direction : 134 # 8

$Asmin := 0.0018 \cdot (d + 3) \cdot 280$

$Asmin = 24.192$ in^2

$N_T := \frac{Asmin}{.44}$

$N_T = 54.982$

In long direction: 55 # 6

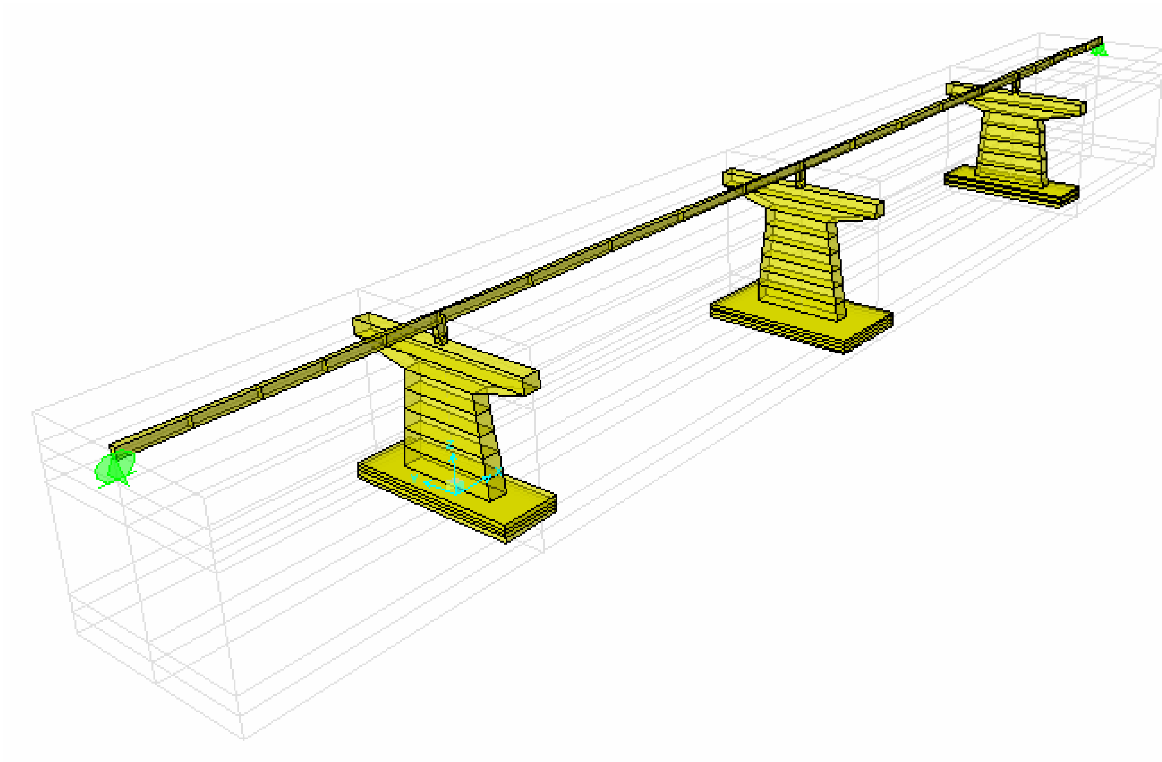
Appendix F -- Detailed Computations for Seismic Analysis and Design of the Pulaski County Bridge Using AASHTO Specifications

Table of contents of Appendix- F

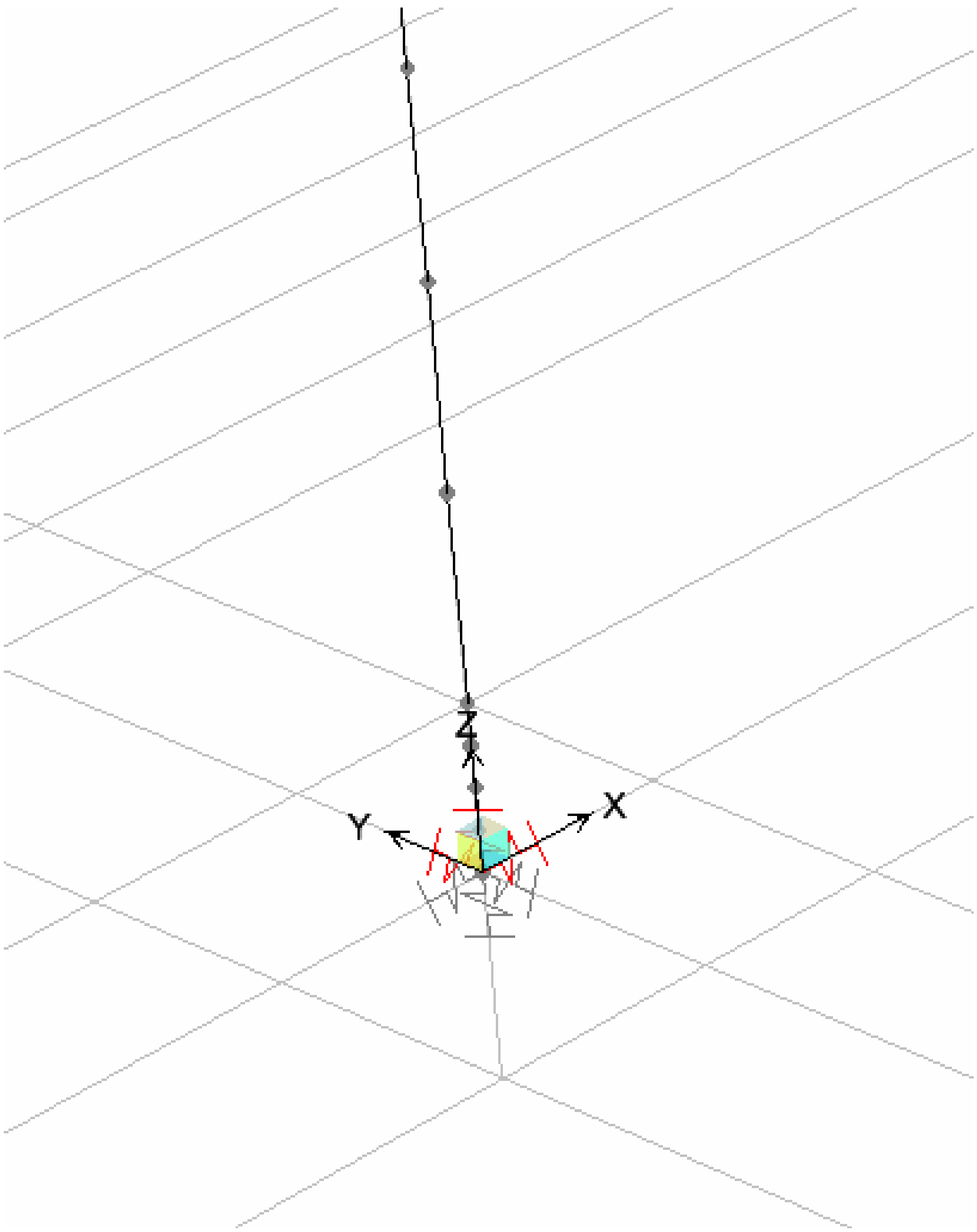
Page

SAP 2000 Model

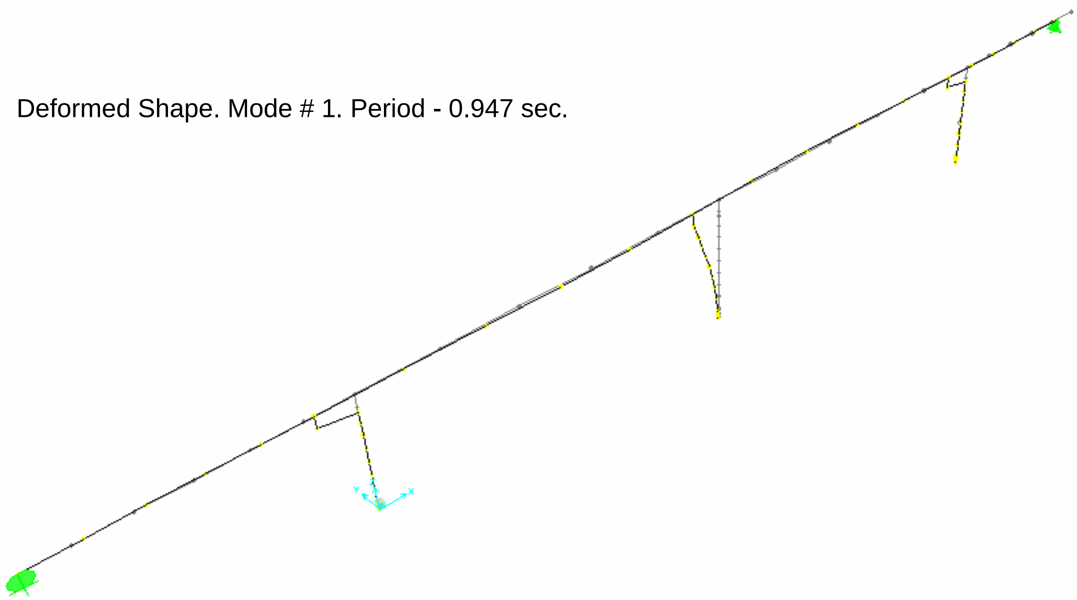
View of SAP 2000 Model (with concrete extrusion shown):



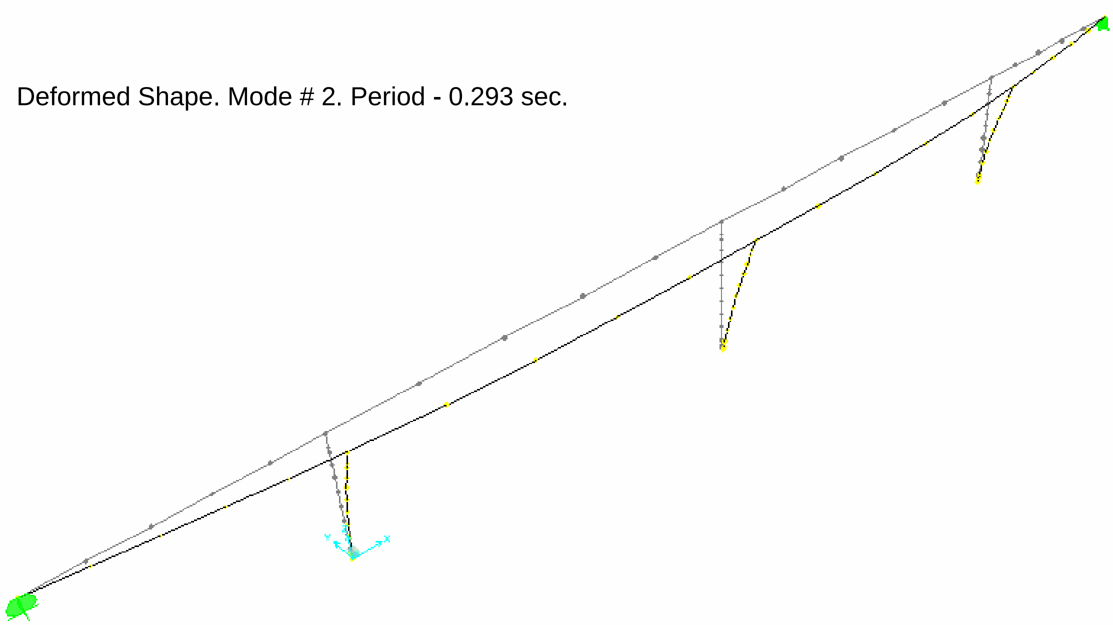
Enlarged View of Translational and Rotational Springs:



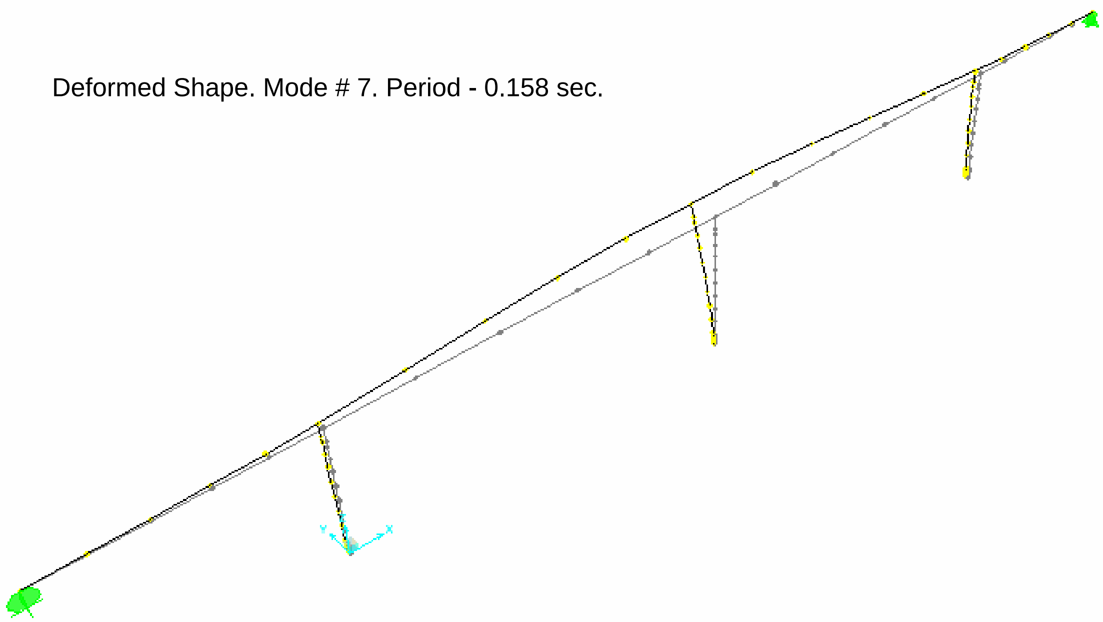
Deformed Shape. Mode # 1. Period - 0.947 sec.



Deformed Shape. Mode # 2. Period - 0.293 sec.



Deformed Shape. Mode # 7. Period - 0.158 sec.



Deformed Shape. Mode # 9. Period - 0.102 sec.

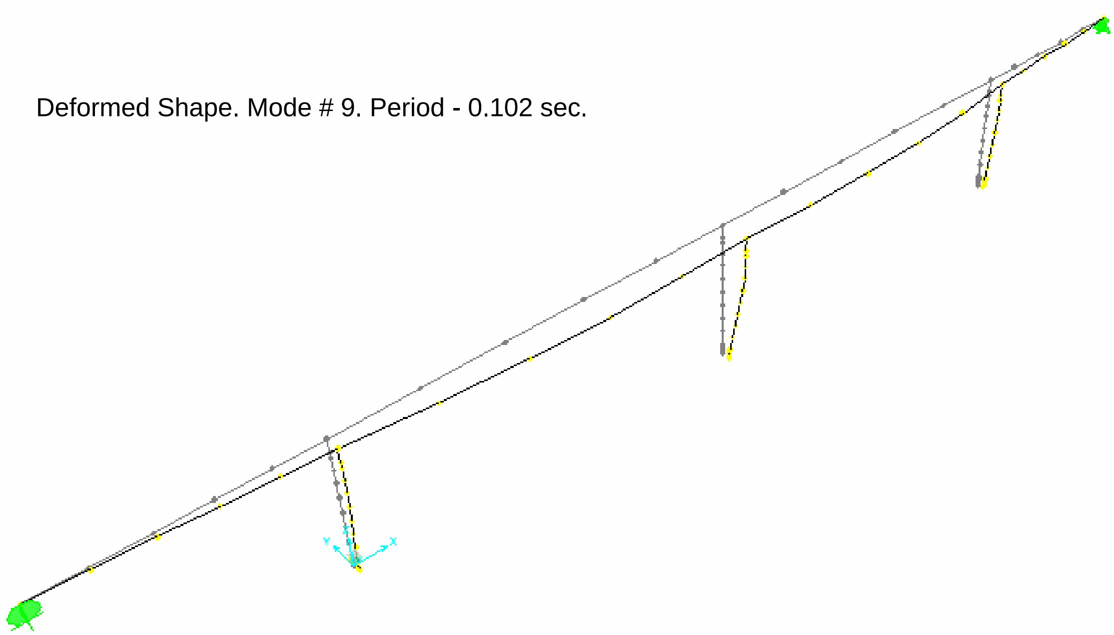


TABLE: Modal Participating Mass Ratios				
StepNum	Period	UX	UY	UZ
Unitless	Sec	%	%	%
1	0.947	68.06	0.00	0.00
2	0.293	0.00	17.44	0.00
3	0.270	0.03	0.00	0.00
4	0.226	0.00	0.00	0.00
5	0.189	0.00	0.00	23.80
6	0.179	5.44	0.00	0.00
7	0.158	0.00	12.86	0.00
8	0.114	0.00	0.00	0.00
9	0.102	0.00	41.93	0.00
10	0.093	0.00	0.00	0.00
11	0.088	0.00	0.00	34.78
12	0.076	0.00	2.15	0.00
13	0.075	0.01	0.00	0.00
14	0.068	0.00	0.00	2.41
15	0.058	0.00	6.24	0.00
16	0.057	0.00	3.74	0.00
17	0.048	0.00	0.00	0.00
18	0.046	0.00	0.00	34.45
19	0.045	4.26	0.00	0.00
20	0.044	2.85	0.00	0.00
21	0.044	0.00	4.99	0.00
22	0.039	0.00	0.00	0.00
23	0.037	0.00	0.15	0.00
24	0.036	0.08	0.00	0.00
25	0.035	0.00	4.15	0.00
26	0.035	9.00	0.00	0.00
27	0.031	0.00	2.68	0.00
28	0.031	0.00	0.00	1.87
29	0.028	0.00	0.55	0.00
	Total	89.74	96.87	97.31

Pulaski County Bridge - AASHTO Division 1-A

Earthquake Data:

Seismic Performance Category (SPC) - C

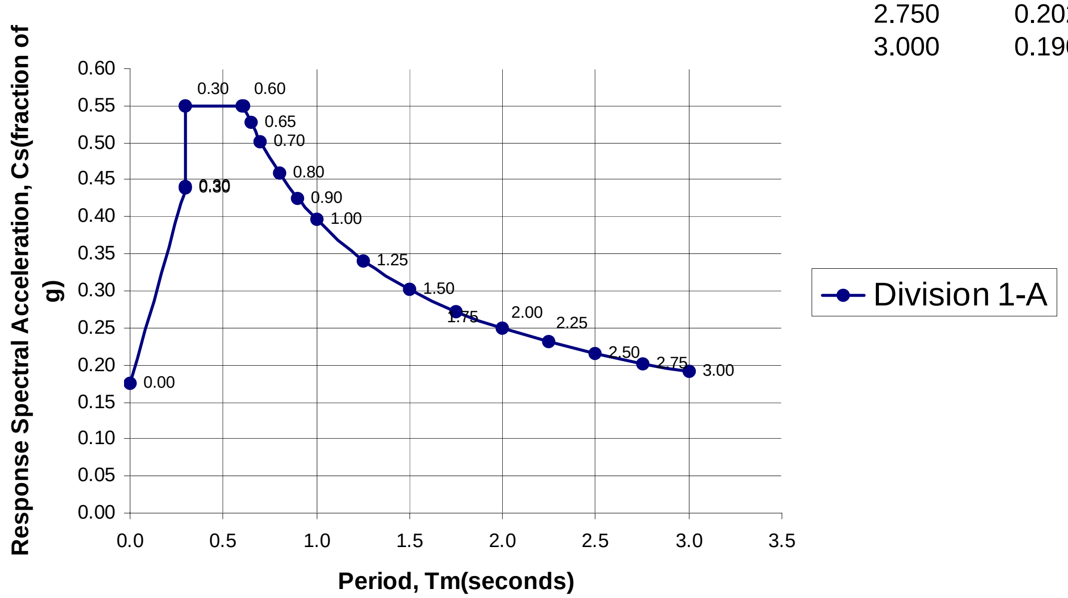
Bedrock Acceleration Coefficient (A) - 0.22*g $\quad A := 0.22$

Soil Profile Type - III

Site Coefficient (S) - 1.5 $\quad Sc := 1.5$

Tm	Cs
0.000	0.176
0.300	0.440
0.300	0.440
0.300	0.550
0.300	0.550
0.611	0.550
0.600	0.550
0.650	0.528
0.700	0.502
0.800	0.460
0.900	0.425
1.000	0.396
1.250	0.341
1.500	0.302
1.750	0.273
2.000	0.249
2.250	0.231
2.500	0.215
2.750	0.202
3.000	0.190

Response Spectrum



Load Combinations

COMB1 - 100% load in X (longitudinal) direction + 30% load in Y (transverse) direction

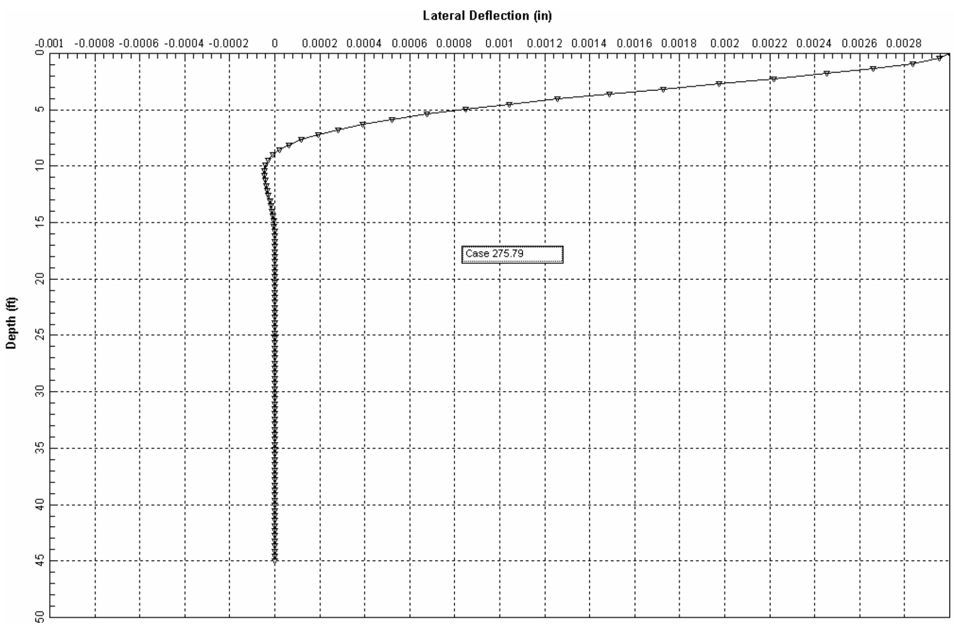
COMB2 - 100% load in Y (transverse) direction + 30% load in X (longitudinal) direction

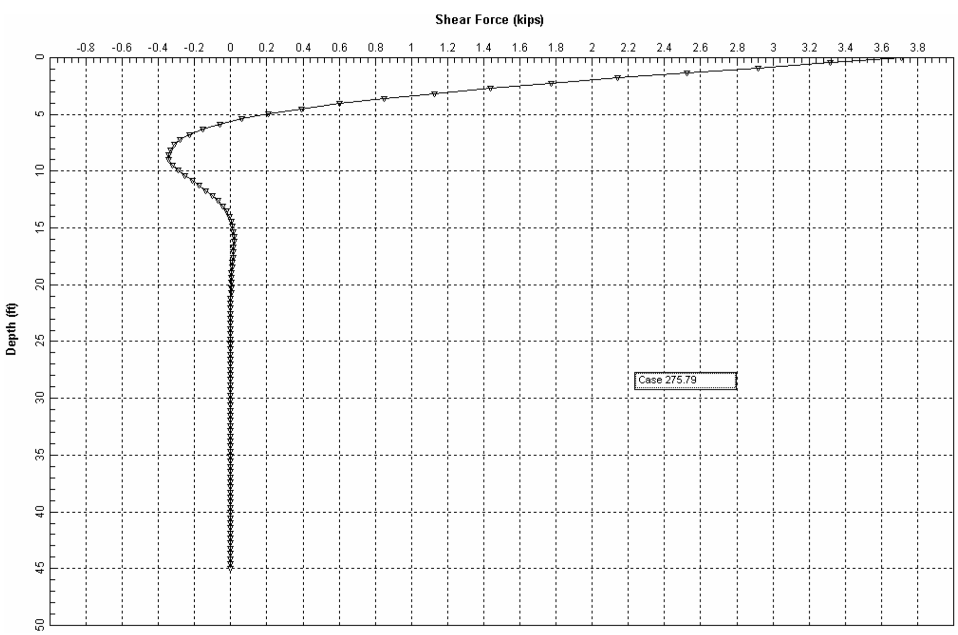
COMD1 - COMB1 + Dead Load

COMD2 - COMB2 + Dead Load

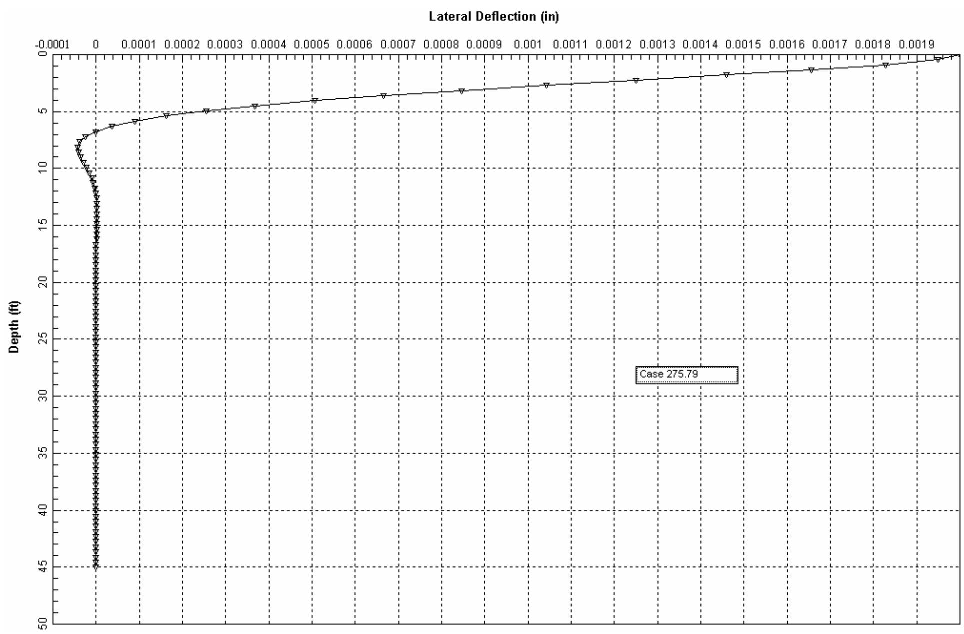
Piles (L-pile results)

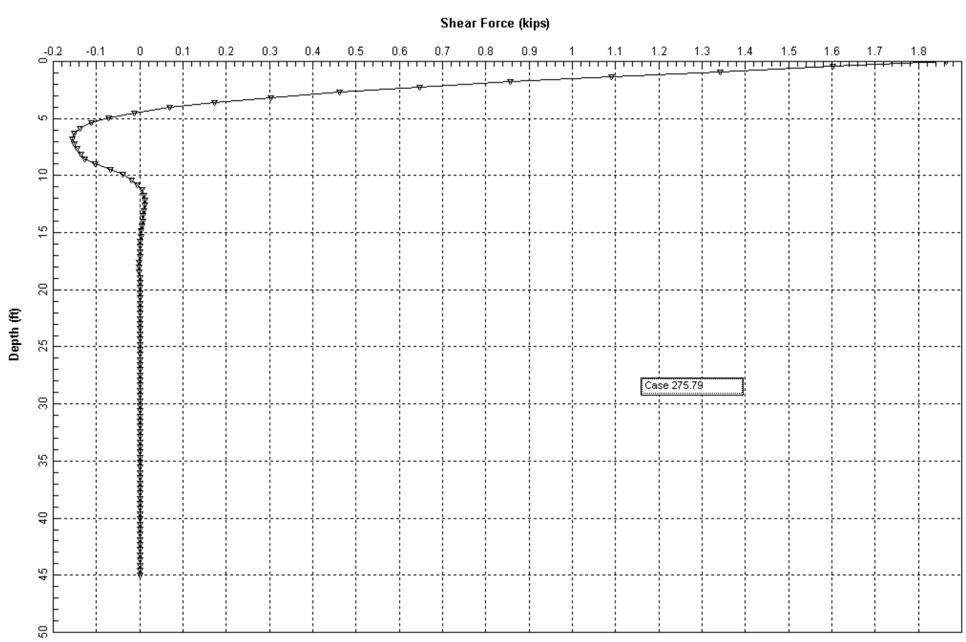
Longitudinal direction of the bridge: (expansion pier)



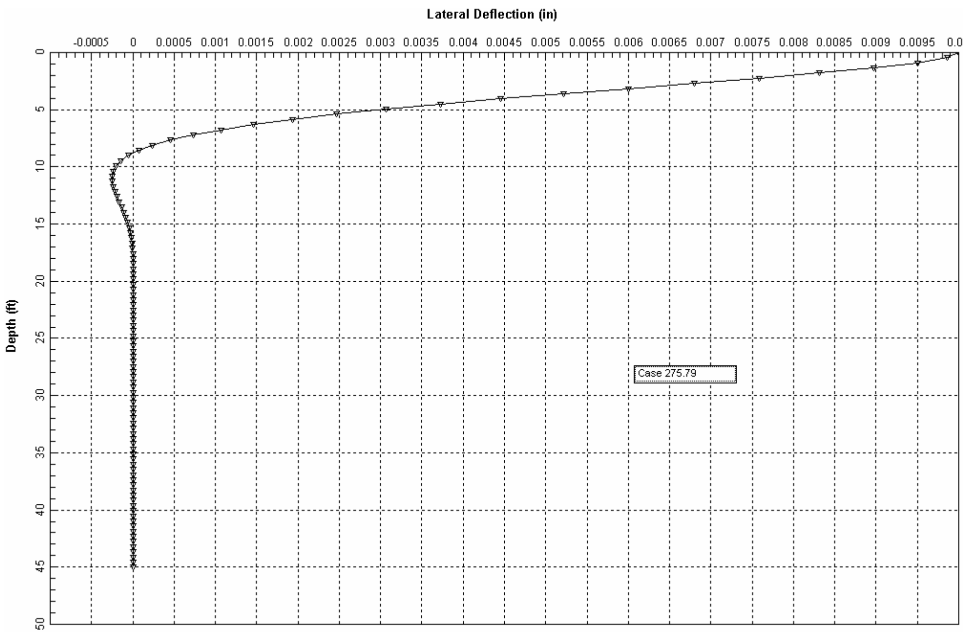


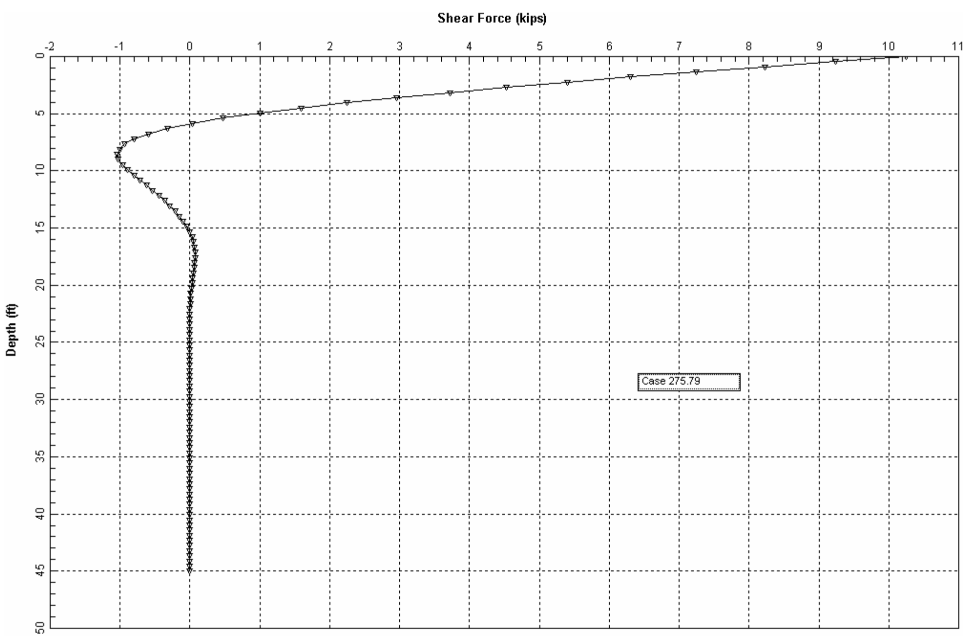
Transverse direction of the bridge: (expansion pier)





Longitudinal direction of the bridge: (fixed pier)





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SUBJECT FILE:

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

BY: **G.V.I.**

CHECKED: **W.N.M.**

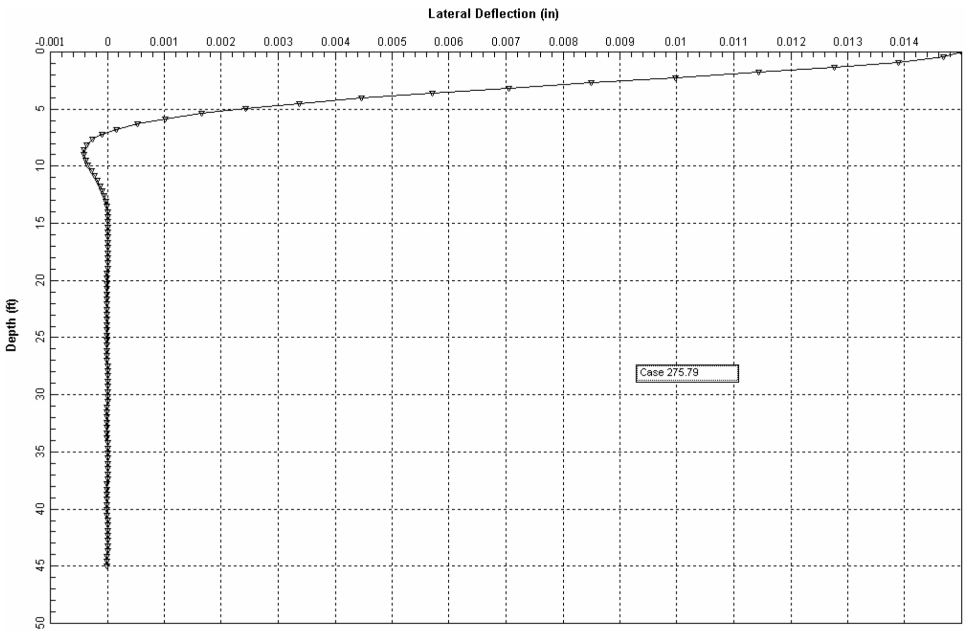
DATE: **6/24/2004**

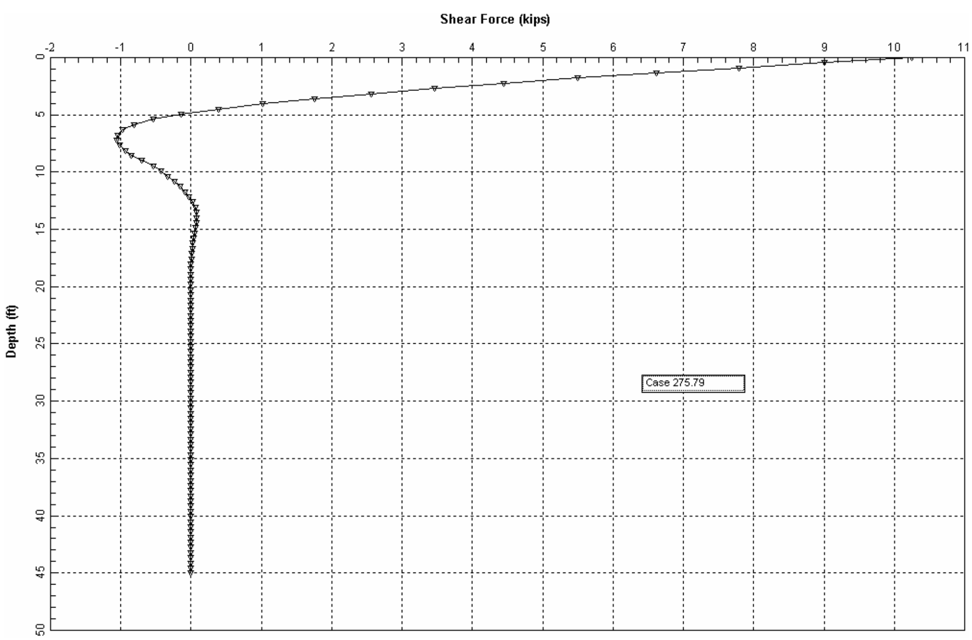
TITLE:

Pulaski County Bridge

AASHTO
Standard Specifications

Transverse direction of the bridge: (fixed pier)





Pile Capacity:

$A_{pile} := 26.1$ in^2 Cross sectional area of **HP14x89 pile**

$Q_{pile} := 9 \cdot A_{pile} \cdot 1.5$ $Q_{pile} = 352.35$ kip Pile Capacity

Group effect

$d_{pile} := 14$ 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:

$D_{b.pile} := 5 \cdot d_{pile}$

$D_{b.pile} = 70$ in

-no larger than: 8 ft.

Set $D_{b.pile}$ equal to 6 ft - Distance between piles (on center)

Axial force on a pile.

Pier #1 & #3 (expansion)

$\phi := 0$ degrees Skew of the bridge

$R := 1$ Response Modification Factor (applied in both directions)

Load Combination **COMD1**

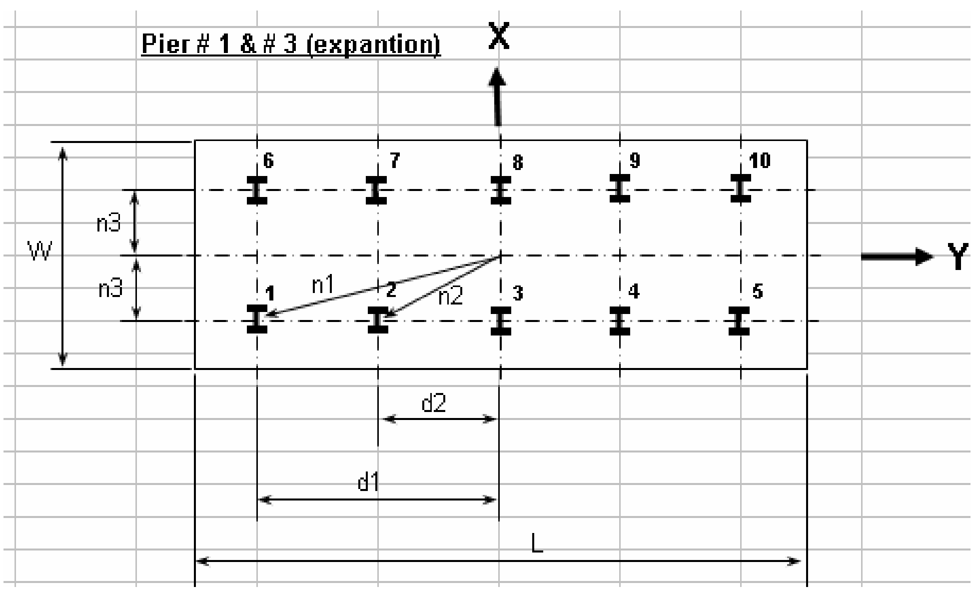
$M_{xunf.} := 646.0$ k – ft Longitudinal (unfactored)

$M_{yunf.} := 480.1$ k – ft Transverse (unfactored)

$P := 516.1$ kips Axial load

$M_y := \frac{M_{yunf.}}{R}$ $M_y = 480.1$ k – ft Longitudinal (factored)

$M_x := \frac{M_{xunf.}}{R}$ $M_x = 646$ k – ft Transverse (factored)



$L := 26.5$ ft

$W := 8.5$ ft

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:

$\Sigma d_1 := 10 \cdot \left(n_3{}^2\right)$ $\qquad \Sigma d_1 = 90$ $\quad ft^2$ (longitudinal)

$\Sigma d_2 := 4 \cdot \left(d_2{}^2\right) + 4 \cdot \left(d_1{}^2\right)$ $\qquad \Sigma d_2 = 720$ $\quad ft^2$ (transverse)

Piles Loads:

$$F_{pile1} := \frac{P}{10} + M_y \cdot \frac{n_3}{\Sigma d_1} + M_x \cdot \frac{d_1}{\Sigma d_2} \qquad F_{pile1} = 78.4 \text{ kips}$$

$$F_{pile5} := \frac{P}{10} + M_y \cdot \frac{n_3}{\Sigma d_1} - M_x \cdot \frac{d_1}{\Sigma d_2} \qquad F_{pile5} = 56.85 \text{ kips}$$

$$F_{pile6} := \frac{P}{10} - M_y \cdot \frac{n_3}{\Sigma d_1} + M_x \cdot \frac{d_1}{\Sigma d_2} \qquad F_{pile6} = 46.37 \text{ kips}$$

$$F_{pile10} := \frac{P}{10} - M_y \cdot \frac{n_3}{\Sigma d_1} - M_x \cdot \frac{d_1}{\Sigma d_2} \qquad F_{pile10} = 24.84 \text{ kips}$$

Pile Capacity is 352.35 kips

Load Combination **COMD2**

$M_{xunf.} := 2153.4$ k – ft Longitudinal (unfactored)

$M_{yunf.} := 144.0$ k – ft Transverse (unfactored)

$P := 504.2$ kips Axial load

$M_y := \frac{M_{yunf.}}{R}$ $M_y = 144$ k – ft Longitudinal (factored)

$M_x := \frac{M_{xunf.}}{R}$ $M_x = 2.153 \times 10^3$ 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

$d_1 := 12$ $d_2 := 6$ $d_3 := 0$ $n_3 := 3$

Sum of squares of the distances to each pile from the center line of the foundation:

$\Sigma d_1 := 10 \cdot \left(n_3^{\,2}\right)$ $\Sigma d_1 = 90$ ft^2 (longitudinal)

$\Sigma d_2 := 4 \cdot \left(d_2^{\,2}\right) + 4 \cdot \left(d_1^{\,2}\right)$ $\Sigma d_2 = 720$ ft^2 (transverse)

Piles Loads:

$F_{pile1} := \frac{P}{10} + M_y \cdot \frac{n_3}{\Sigma d_1} + M_x \cdot \frac{d_1}{\Sigma d_2}$ $F_{pile1} = 91.1$ kips

$F_{pile5} := \frac{P}{10} + M_y \cdot \frac{n_3}{\Sigma d_1} - M_x \cdot \frac{d_1}{\Sigma d_2}$ $F_{pile5} = 19.33$ kips

$F_{pile6} := \frac{P}{10} - M_y \cdot \frac{n_3}{\Sigma d_1} + M_x \cdot \frac{d_1}{\Sigma d_2}$ $F_{pile6} = 81.51$ kips

$F_{pile10} := \frac{P}{10} - M_y \cdot \frac{n_3}{\Sigma d_1} - M_x \cdot \frac{d_1}{\Sigma d_2}$ $F_{pile10} = 9.73$ kips

Capacity is 352.35 kips

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Axial Force on a Pile.

Pier #2 (fixed)

$\phi := 0$ degrees Skew of the bridge

$R := 1$ Response Modification Factor (applied in both directions)

Load Combination **COMD1**
(from SAP 2000; file: (FINAL) springs4

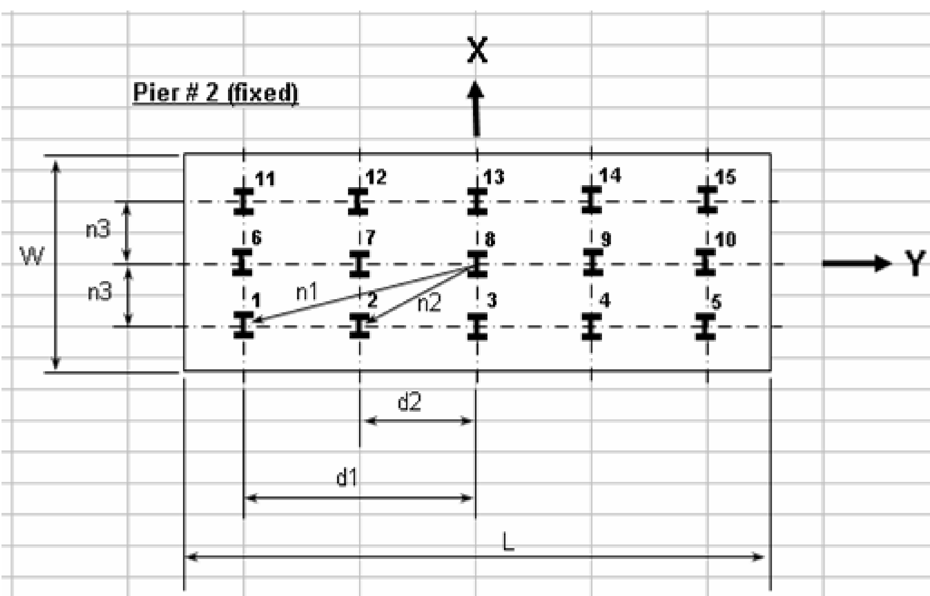
$M_{xunf.} := 1108.4$ k – ft Longitudinal (unfactored)

$M_{yunf.} := 11784.1$ k – ft Transverse (unfactored)

$P := 646.9$ kips Axial load

$M_y := \dfrac{M_{yunf.}}{R}$ $M_y = 1.178 \times 10^4$ k – ft Longitudinal (factored)

$M_x := \dfrac{M_{xunf.}}{R}$ $M_x = 1.108 \times 10^3$ k – ft Transverse (factored)



$L := 27$ ft

$W := 14.5$ ft

The distance from center line of foundation to center of pile (ft):

Distance between piles is set to avoid group effect

$d_1 := 12$ $d_2 := 6$ $d_3 := 0$ $n_3 := 6$

Sum of squares of the distances to each pile from the center line of the foundation:

$\Sigma d_1 := 10 \cdot \left(n_3^{\,2}\right)$ $\Sigma d_1 = 360$ ft^2 (longitudinal)

$\Sigma d_2 := 6 \cdot \left(d_2^{\,2}\right) + 6 \cdot \left(d_1^{\,2}\right)$ $\Sigma d_2 = 720$ ft^2 (transverse)

Piles Loads:

$F_{pile1} := \frac{P}{10} + M_y \cdot \frac{n_3}{\Sigma d_1} + M_x \cdot \frac{d_1}{\Sigma d_2}$ $F_{pile1} = 273.4$ kips

$F_{pile5} := \frac{P}{10} + M_y \cdot \frac{n_3}{\Sigma d_1} - M_x \cdot \frac{d_1}{\Sigma d_2}$ $F_{pile5} = 248.78$ kips

$F_{pile11} := \frac{P}{10} - M_y \cdot \frac{n_3}{\Sigma d_1} + M_x \cdot \frac{d_1}{\Sigma d_2}$ $F_{pile11} = -119.4$ kips

$F_{pile15} := \frac{P}{10} - M_y \cdot \frac{n_3}{\Sigma d_1} - M_x \cdot \frac{d_1}{\Sigma d_2}$ $F_{pile15} = -144.027$ kips

Pile Capacity is 352.35 kips

$F_{pile3} := \frac{\left(F_{pile1} + F_{pile5}\right)}{2}$ $F_{pile3} = 261.092$ kips

$F_{pile2} := \frac{\left(F_{pile3} + F_{pile1}\right)}{2}$ $F_{pile2} = 267.249$ kips

$F_{pile4} := \frac{\left(F_{pile3} + F_{pile5}\right)}{2}$ $F_{pile4} = 254.934$ kips

Load Combination **COMD2**
(from SAP 2000; file: (FINAL) springs4)

$M_{xunf.} := 3694.7$ k – ft Longitudinal (unfactored)

$M_{yunf.} := 3535.2$ k – ft Transverse (unfactored)

$P := 646.9$ kips Axial load

$M_y := \frac{M_{yunf.}}{R}$ $M_y = 3.535 \times 10^3$ k – ft Longitudinal (factored)

$M_x := \frac{M_{xunf.}}{R}$ $M_x = 3.695 \times 10^3$ 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

$d_1 := 12$ $d_2 := 6$ $d_3 := 0$ $n_3 := 6$

Sum of squares of the distances to each pile from the center line of the foundation:

$\Sigma d_1 := 10 \cdot \left(n_3^{\,2}\right)$ $\Sigma d_1 = 360$ ft^2 (longitudinal)

$\Sigma d_2 := 6 \cdot \left(d_2^{\,2}\right) + 6 \cdot \left(d_1^{\,2}\right)$ $\Sigma d_2 = 1.08 \times 1($ ft^2 (transverse)

Piles Loads:

$F_{pile1} := \frac{P}{10} + M_y \cdot \frac{n_3}{\Sigma d_1} + M_x \cdot \frac{d_1}{\Sigma d_2}$ $F_{pile1} = 164.7$ kips

$F_{pile5} := \frac{P}{10} + M_y \cdot \frac{n_3}{\Sigma d_1} - M_x \cdot \frac{d_1}{\Sigma d_2}$ $F_{pile5} = 82.56$ kips

$F_{pile11} := \frac{P}{10} - M_y \cdot \frac{n_3}{\Sigma d_1} + M_x \cdot \frac{d_1}{\Sigma d_2}$ $F_{pile11} = 46.82$ kips

$F_{pile15} := \frac{P}{10} - M_y \cdot \frac{n_3}{\Sigma d_1} - M_x \cdot \frac{d_1}{\Sigma d_2}$ $F_{pile15} = -35.282$ 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:

*All axes are **Global** to SAP 2000 model*
e.g.. designation "longitudinal" and "transverse" refer to location of Global coordinate system in SAP model.

$F_y := 36$ ksi

$A_{pile} := 26.1$ in^2

$Z_x := 67.7$ in^3

$Z_y := 146.0$ in^3

$M_{px} := F_y \cdot Z_x$

$M_{px} = 2.437 \times 10^3$ $k \cdot in$ full plastic moment about X-axis (transverse)

$M_{py} := F_y \cdot Z_y$

$M_{py} = 5.256 \times 10^3$ $k \cdot in$ full plastic moment about Y-axis (longitudinal)

Pier #1 & #3 (expansion)

Load combination: COMD 1

Maximum axial force on a pile:

(as determined in "Axial force on a pile")

$P := 78.4$ k — axial load (single pile)

Maximum Loads (from LPILE output):

$M_{xLpile} := 32.7$ $k \cdot in$ — moment about X-axis(single pile)

$M_{yLpile} := 84.0$ $k \cdot in$ — moment about Y-axis (single pile)

$R_p := 1$ Response Modification Factor for piles (SPC - **C**)

$$\left(\frac{P}{0.85 \cdot A_{pile} \cdot F_y}\right) + \left[\left(\frac{M_{xLpile}}{M_{px}}\right) + \left(\frac{M_{yLpile}}{M_{py}}\right)\right] = 0.128$$

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")

$P := 91.1$ k axial load (single pile)

Maximum Loads (from LPILE output):

$M_{xLpile} := 32.7$ k·in moment about X-axis(single pile)

$M_{yLpile} := 84.0$ k·in moment about Y-axis (single pile)

$R_p := 1$ Response Modification Factor for piles (SPC - **C**)

$$\left(\frac{P}{0.85 \cdot A_{pile} \cdot F_y}\right) + \left[\left(\frac{M_{xLpile}}{M_{px}}\right) + \left(\frac{M_{yLpile}}{M_{py}}\right)\right] = 0.143$$

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")

$P := 273.4$ k — axial load (single pile)

Maximum Loads (from LPILE output):

$M_{xLpile} := 205.4$ k·in — moment about X-axis(single pile)

$M_{yLpile} := 251.6$ k·in — moment about Y-axis (single pile)

$R_p := 1$ Response Modification Factor for piles (SPC - **C**)

$$\left(\frac{P}{0.85 \cdot A_{pile} \cdot F_y}\right) + \left[\left(\frac{M_{xLpile}}{M_{px}}\right) + \left(\frac{M_{yLpile}}{M_{py}}\right)\right] = 0.474$$

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")

$P := 164.7$ k axial load (single pile)

Maximum Loads (from LPILE output):

$M_{xLpile} := 205.4$ $k \cdot in$ moment about X-axis(single pile)

$M_{yLpile} := 251.6$ $k \cdot in$ moment about Y-axis (single pile)

$R_p := 1$ Response Modification Factor for piles (SPC - **C**)

$$\left(\frac{P}{0.85 \cdot A_{pile} \cdot F_y}\right) + \left[\left(\frac{M_{xLpile}}{M_{px}}\right) + \left(\frac{M_{yLpile}}{M_{py}}\right)\right] = 0.338$$

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

$L_{pile} := 540$ in

$A_{pile} := 26.1$ in^2

$E_s := 29000000$ $\frac{lb}{in^2}$

Results from L-Pile:

Piers #1 & #3 Long.HP 14x89_(iteration3).lpd
Piers #1 & #3 Trans.HP 14x89_(iteration3).lpd

$\Delta_t := 0.002$ in Displacement (transverse)

$\Delta_l := 0.003$ in Displacement (longitudinal)

$V_{utrans} := 1864.3$ lb Transverse

$V_{ulong} := 3712.2$ lb Longitudinal

Spring Stiffness (per one pile):

$k_{axial} := A_{pile} \cdot \frac{E_s}{L_{pile}}$ $k_{axial} = 1.402 \times 10^6$ $\frac{lb}{in}$ Axial

$k_{trans} := \frac{V_{utrans}}{\Delta_t}$ $k_{trans} = 9.322 \times 10^5$ $\frac{lb}{in}$ Transverse

$k_{long} := \frac{V_{ulong}}{\Delta_l}$ $k_{long} = 1.237 \times 10^6$ $\frac{lb}{in}$ Longitudinal

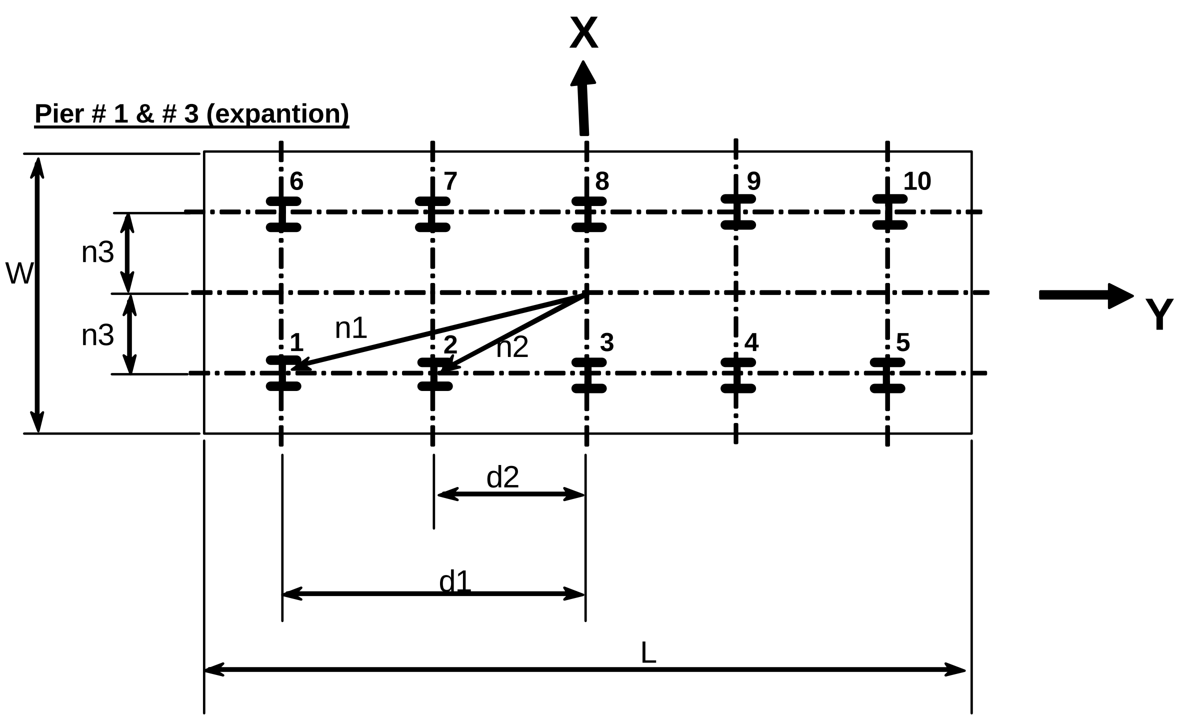
Lateral Springs:

$K_{long} := 10 \cdot k_{long}$ $\quad K_{long} = 1.237 \times 10^{7} \frac{lb}{in}$ $\quad$ Along X axis (K_{u1})

$K_{trans} := 10 \cdot k_{trans}$ $\quad K_{trans} = 9.322 \times 10^{6} \frac{lb}{in}$ $\quad$ Along Y axis (K_{u2})

Axial Springs:

$K_{axial} := 10 \cdot k_{axial}$ $\quad K_{axial} = 1.402 \times 10^{7} \frac{lb}{in}$ $\quad$ Along Z axis (K_{u3})



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
$l_{12} := 72$	$d_1 := 144$	$w := 36$	$n_1 := \sqrt{d_1^{\,2} + w^2}$
$l_{23} := 72$	$d_2 := 72$		$n_2 := \sqrt{d_2^{\,2} + w^2}$
	$d_3 := 0$		$n_3 := \sqrt{d_3^{\,2} + w^2}$

$K_{rx} := k_{axial} \cdot \left[4 \cdot (d_1)^2 + 4 (d_2)^2\right]$ $\qquad K_{rx} = 1.453 \times 10^{11} \ \frac{lb \cdot in}{rad}$ $\qquad$ about X axis $\qquad (K_{r1})$

$K_{ry} := k_{axial} \cdot 10 \cdot n_3^{\,2}$ $\qquad K_{ry} = 1.817 \times 10^{10} \ \frac{lb \cdot in}{rad}$ $\qquad$ about Y axis $\qquad (K_{r2})$

$$K_{rz} := k_{long} \cdot \left(4 \cdot d_1^{\,2} + 4 \cdot d_2^{\,2}\right) + k_{trans} \cdot (10 n_3)^2$$

$K_{rz} = 2.491 \times 10^{11} \ \frac{lb \cdot in}{rad}$ $\qquad$ about Z axis $\qquad (K_{r3})$

Pier #2 (fixed)

Pile Information for L-Pile input:

HP 14x89 - piles

$L_{pile} := 540$ in

$A_{pile} := 26.1$ in^2

$E_s := 29000000$ $\frac{lb}{in^2}$

Results from L-Pile:

Pier #2 Long.HP 14x89_(iteration4).lpd
Pier #2 Trans.HP 14x89_(iteration4).lpd

$\Delta_t := 0.015$ in Displacement (transverse)

$\Delta_l := 0.01$ in Displacement (longitudinal)

$V_{utrans} := 10250.2$ lb Transverse

$V_{ulong} := 10248.6$ lb Longitudinal

Spring Stiffness (per one pile):

$k_{axial} := A_{pile} \cdot \frac{E_s}{L_{pile}}$ $k_{axial} = 1.402 \times 10^6$ $\frac{lb}{in}$ Axial

$k_{trans} := \frac{V_{utrans}}{\Delta_t}$ $k_{trans} = 6.833 \times 10^5$ $\frac{lb}{in}$ Transverse

$k_{long} := \frac{V_{ulong}}{\Delta_l}$ $k_{long} = 1.025 \times 10^6$ $\frac{lb}{in}$ Longitudinal

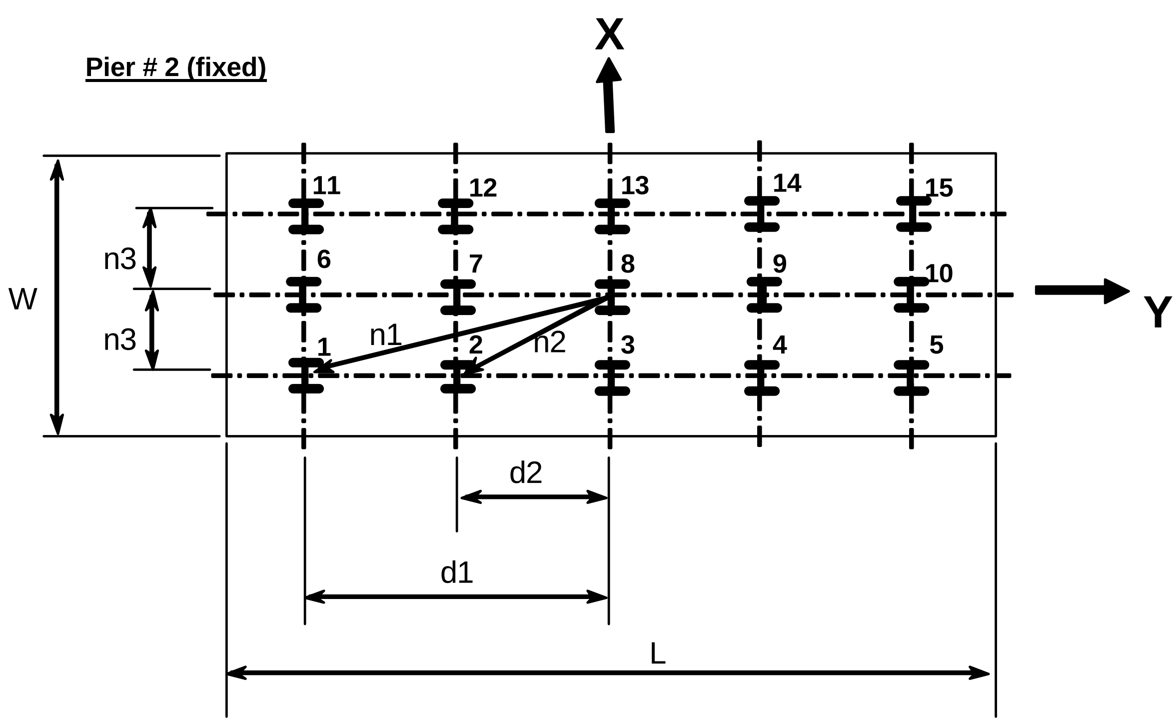
Lateral Springs:

$K_{long} := 15 \cdot k_{long}$ $\quad K_{long} = 1.537 \times 10^7 \ \frac{lb}{in}$ $\quad$ Along X axis $\quad (K_{u1})$

$K_{trans} := 15 \cdot k_{trans}$ $\quad K_{trans} = 1.025 \times 10^7 \ \frac{lb}{in}$ $\quad$ Along Y axis $\quad (K_{u2})$

Axial Springs:

$K_{axial} := 15 \cdot k_{axial}$ $\quad K_{axial} = 2.103 \times 10^7 \ \frac{lb}{in}$ $\quad$ Along Z axis $\quad (K_{u3})$



NOTE: Distance between piles is set to avoid group effect

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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	
$l_{12} := 72$	$d_1 := 144$	$w := 72$	$n_1 := \sqrt{d_1^2 + w^2}$	$n_1 = 160.997$
$l_{23} := 72$	$d_2 := 72$		$n_2 := \sqrt{d_2^2 + w^2}$	$n_2 = 101.823$
	$d_3 := 0$		$n_3 := \sqrt{d_3^2 + w^2}$	$n_3 = 72$

$K_{rx} := k_{axial} \cdot \left[6 \cdot (d_1)^2 + 6 (d_2)^2\right]$ $\quad K_{rx} = 2.18 \times 10^{11}$ $\frac{lb \cdot in}{rad}$ about X axis (K_{r1})

$K_{ry} := k_{axial} \cdot 10 \cdot n_3^2$ $\quad K_{ry} = 7.266 \times 10^{10}$ $\frac{lb \cdot in}{rad}$ about Y axis (K_{r2})

$K_{rz} := k_{long} \cdot \left(6 \cdot d_1^2 + 6 \cdot d_2^2\right) + k_{trans} \cdot (10 n_3)^2$

$K_{rz} = 5.136 \times 10^{11}$ $\frac{lb \cdot in}{rad}$ about Z axis (K_{r3})

Displacements at bottom of foundation

SAP 2000 File: (FINAL)springs5

TABLE: Joint Displacements							
Joint	OutputCase	U1	U2	U3	R1	R2	R3
Text	Text	in	in	in	Radians	Radians	Radians
66	DEAD	0.000	0.000	-0.036	0.000	0.000	0.000
	COMD1	0.003	0.002	-0.034	0.000	0.000	0.000
	COMD2	0.001	0.007	-0.035	0.000	0.000	0.000
70	DEAD	0.000	0.000	-0.036	0.000	0.000	0.000
	COMD1	0.003	0.002	-0.034	0.000	0.000	0.000
	COMD2	0.001	0.007	-0.035	0.000	0.000	0.000
74	DEAD	0.000	0.000	-0.031	0.000	0.000	0.000
	COMD1	0.032	0.004	-0.031	0.000	0.002	0.000
	COMD2	0.010	0.014	-0.031	0.000	0.001	0.000

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 LPILE:
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

LPile output See Appendices for actual LPile 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)

TABLE: Element Joint Forces - Frames				V2	V3	P	M2	M3	M1
Frame	**FrameElem**	**Joint**	**OutputCase**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
33	8	**4**	DEAD	0.0	0.0	-352.2	0.0	0.0	0.0
			COMD1	21.0	16.4	-335.6	756.5	51.2	6.4
			COMD2	6.3	54.8	-347.2	2521.5	15.3	21.3
42	25	**40**	DEAD	0.0	0.0	-352.2	0.0	0.0	0.0
			COMD1	21.0	16.4	-335.6	756.5	51.2	6.4
			COMD2	6.3	54.8	-347.2	2521.5	15.3	21.3
45	28	**5**	DEAD	0.0	0.0	-423.1	0.0	0.0	0.0
			COMD1	31.7	18.0	-406.4	655.1	409.1	6.4
			COMD2	9.5	60.1	-418.1	2183.6	122.7	21.3
70	32	**30**	DEAD	0.0	0.0	-423.1	0.0	0.0	0.0
			COMD1	31.7	18.0	-406.4	655.1	409.1	6.4
			COMD2	9.5	60.1	-418.1	2183.6	122.7	21.3
Pulaski County (Pier #2 at neck Joint # 23; at bottom Joint #14)									
20	11	**23**	DEAD	0.0	0.0	427.6	0.0	0.0	0.0
			COMD1	480.4	34.8	427.6	634.5	2613.0	0.0
			COMD2	144.1	116.1	427.6	2115.0	783.9	0.0
75	36	**14**	DEAD	0.0	0.0	-514.8	0.0	0.0	0.0
			COMD1	491.9	39.2	-514.8	1035.2	10676.9	0.0
			COMD2	147.6	130.7	-514.8	3450.6	3203.1	0.0

With Response Modification Factor

Pulaski County (Pier #1 & #3 at neck Joints #4 & #40; at bottom Joints #5 and #30)

TABLE: Element Joint Forces - Frames				V2	V3	P	M2	M3	M1
Frame	**FrameElem**	**Joint**	**OutputCase**	**F1**	**F2**	**F3**	**M1**	**M2**	**M3**
Text	Text	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
33	8	**4**	DEAD	0.0	0.0	-352.2	0.0	0.0	0.0
			COMD1	21.0	16.4	-335.6	378.2	25.6	6.4
			COMD2	6.3	54.8	-347.2	1260.8	7.7	21.3
							0.0	0.0	
42	25	**40**	DEAD	0.0	0.0	-352.2	0.0	0.0	0.0
			COMD1	21.0	16.4	-335.6	378.2	25.6	6.4
			COMD2	6.3	54.8	-347.2	1260.8	7.7	21.3
							0.0	0.0	
45	28	**5**	DEAD	0.0	0.0	-423.1	0.0	0.0	0.0
			COMD1	31.7	18.0	-406.4	327.5	204.5	6.4
			COMD2	9.5	60.1	-418.1	1091.8	61.4	21.3
							0	0	
70	32	**30**	DEAD	0.0	0.0	-423.1	0.0	0.0	0.0
			COMD1	31.7	18.0	-406.4	327.5	204.5	6.4
			COMD2	9.5	60.1	-418.1	1091.8	61.4	21.3

Pulaski County (Pier #2 at neck Joint # 23; at bottom Joint #14)

20	11	**23**	DEAD	0.0	0.0	427.6	0.0	0.0	0.0
			COMD1	480.4	34.8	427.6	317.2	1306.5	0.0
			COMD2	144.1	116.1	427.6	1057.5	392.0	0.0
							0.0	0.0	
75	36	**14**	DEAD	0.0	0.0	-514.8	0.0	0.0	0.0
			COMD1	491.9	39.2	-514.8	517.6	5338.5	0.0
			COMD2	147.6	130.7	-514.8	1725.3	1601.5	0.0

Response Modification Factors:
Piers:
R=1 Shear
R=2 Flexure
Piles:
R=1 longitudinal and transverse

Spring Forces

SAP 2000 File: (FINAL)springs4

TABLE: Joint Reactions - Spring Forces							
Joint	**OutputCase**	**U1**	**U2**	**U3**	**R1**	**R2**	**R3**
Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
66	DEAD	0.0	0.0	499.1	0.0	0.0	0.0
	COMD1	36.1	19.4	516.1	646.0	480.1	6.4
	COMD2	10.8	64.6	504.2	2153.4	144.0	21.3
70	DEAD	0.0	0.0	499.1	0.0	0.0	0.0
	COMD1	36.1	19.4	516.1	646.0	480.1	6.4
	COMD2	10.8	64.6	504.2	2153.4	144.0	21.3
74	DEAD	0.0	0.0	646.9	0.0	0.0	0.0
	COMD1	493.2	42.2	646.9	1108.4	11784.1	0.0
	COMD2	148.0	140.8	646.9	3694.7	3535.2	0.0

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 (expantion)

Load Combination COMD2

Deep beam design (Reinforcement concrete, Nawy, 4rd edition, section 6.9)

Shear design for pier #1& #3 (expansion)

$f_c := 3500$ psi

$f_y := 60000$ psi

$h := 15.5$ ft -Depth of beam

$b := 2.5$ ft -Width of beam

$d := 0.95 \cdot h$ (assumed)

$d = 14.725$ ft -Distance from reinforcement to extreme compression fiber

$l := 18.25$ ft -Hight of beam

$\Phi := 0.85$ (for shear)

$V := 60.1$ k

$M_{un} := 2183.6$ k – ft unfactored

$R := 2$

$M := \dfrac{M_{un}}{R}$ factored

$\frac{l}{d} = 1.239$ < 2.5 - criteria determining Deep Beam

For $\frac{l}{d} < 2.0$ - use following eq.

$$V_u := \Phi \cdot \left(8 \cdot \sqrt{f_c} \cdot b \cdot d\right) \cdot \frac{144}{1000}$$

$V_u = 2.133 \times 10^3$ k -Factored shear force

$V = 60.1$ k $<$ $V_u = 2.133 \times 10^3$ k OK

$$V_c := 2 \cdot \left(\sqrt{f_c}\right) \cdot b \cdot d \cdot \frac{144}{1000}$$

$V_c = 627.223$ k -Nominal shear resisting force (**controls**)

$$V_{cmax} := 6 \cdot \sqrt{f_c} \cdot b \cdot d \cdot \frac{144}{1000}$$

$V_{cmax} = 1.882 \times 10^3$ k -Maximum nominal shear resisting force

$\Phi \cdot V_c = 533.139$ k

$V_u = 2.133 \times 10^3$ must be less or equal than $\Phi \cdot \left(V_c + V_s\right)$

$\Phi \cdot V_c$ $<$ V_u -Shear reinforcement is required

$$\boxed{V_u - \Phi \cdot V_c \le \Phi \cdot V_s}$$

$$V_s := \frac{V_u}{\Phi} - V_c$$

$V_s = 1.882 \times 10^3$ k

$A_{s1} := 1.56$ in^2 Cross sectional area of # 11 rebar

$A_l := A_{s1} \cdot \frac{2}{144}$ $A_l = 0.022$ ft^2 Area of trans. rebars placed at distance s_{tr}

$A_{str} := 0.44$ in^2 Cross sectional area of # 6 rebar

$A_{tr} := A_{str} \cdot \frac{2}{144}$ $A_{tr} = 6.111 \times 10^{-3}$ ft^2 Area of long. rebars placed at distance s_l

$s_{trmax} := \frac{d}{5}$ $s_{trmax} = 2.945$ ft -Maximum distance between rebars in transverse direction (on center)

or 1.5 ft -use this value

$s_h := 1.5$ ft -Distance between horizontal (transverse) rebars (on center)

$t_r := 10.44$ in -Clear spacing between rebars;

$$\frac{(t_r + 1.56)}{12} = 1$$

$s_v := 1.0$ ft -Distance between vertical rebars (on center);PCACOL (file:#11BOT13)

$$\left[\left(\frac{A_{tr}}{s_h}\right) \cdot \frac{\left[1 + \left(\frac{h}{d}\right)\right]}{12}\right] + \left[\left(\frac{A_l}{s_v}\right) \cdot \frac{\left[11 - \left(\frac{h}{d}\right)\right]}{12}\right] \cdot f_y \cdot d = V_s$$

$K_1 := \left(\frac{A_{tr}}{s_h}\right) \cdot \frac{\left[1 + \left(\frac{h}{d}\right)\right]}{12}$ $K_1 = 6.969 \times 10^{-4}$

$K_2 := \left(\frac{A_l}{s_v}\right) \cdot \frac{\left[11 - \left(\frac{h}{d}\right)\right]}{12}$ $K_2 = 0.018$

$\left(K_1 + K_2\right) \cdot f_y \cdot d \cdot \frac{144}{1000} = 2.374 \times 10^3$ must be equal or greater than $V_s = 1.882 \times 10^3$ OK

Check for minimum steel: (pier # 1 & # 3)

Rebars # 6 placed in transverse direction on both faces of the pier

$A_{tr} = 6.111 \times 10^{-3}$ ft^2 - transverse

$A_{trmin} := 0.0015 \cdot b \cdot s_h$

$A_{trmin} = 5.625 \times 10^{-3}$ ft^2

$A_{tr} = 6.111 \times 10^{-3}$ 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

$A_l = 0.022$ ft^2 - longitudinal

$A_{lmin} := 0.0025 \cdot b \cdot s_h$

$A_{lmin} = 9.375 \times 10^{-3}$ ft^2

$A_l = 0.022$ 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)

$\Phi := 0.9$ - for flexure

$M = 1.092 \times 10^3$ k – ft

$\frac{l}{h} = 1.177$ > 1 so jd=0.2*(l+2h)

$j := .2\frac{(l + 2 \cdot h)}{d}$

$j = 0.669$

$j \cdot d = 9.85$ ft

Minimum reinforcement area

$A_s := \frac{M \cdot 144}{\Phi \cdot f_y \cdot j \cdot d \cdot \frac{144}{1000}}$ $A_s = 2.053$ in^2

$A_{smin1} := \max\left(3 \cdot \frac{\sqrt{f_c}}{f_y} \cdot b \cdot d \cdot 144\right)$ $A_{smin1} = 15.681$ in^2

$A_{smin2} := 200 \cdot b \cdot \frac{d \cdot 144}{f_y}$ $A_{smin2} = 17.67$ in^2 -controls

$y := 0.25 \cdot h - 0.05 \cdot l$

$y = 2.963$ ft -Depth over which flex. reinf. is to be distributed (in transverse direction)

$s_v = 1$ 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)

$A_{s1} := 1.56$ in^2 -cross section area of # 11 rebar

$A_s := A_{s1} \cdot n_{tr}$ $A_s = 12.363$ in^2 -Total cross section area of rebars available to resist flexure

$A_s = 12.363$ in^2 must be equal or greater than $A_{smin2} = 17.67$ in^2 not satisfactory

$A_{s1} := 1.56$ in^2 -cross section area of #11 rebar

$$N_{req} := \frac{A_{smin2}}{A_{s1}}$$

$N_{req} = 11.327$ -number of #11bars 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

$b_f := 14.695$ in -Cross sectional dimensions of HP 14x89

$b_{pc} := 26.5 \cdot 12$ 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

$d_{aver} := h - 3 - d_{bar8}$

$d_{aver} = 32$ in -Average depth of the footing slab (from top to bottom reinforcement)

$\phi := 0.85$ - for shear

$\Phi := 0.9$ - for flexure

Pile punching shear check:

$Vu := P_{axial}$

$Vu = 91.1$ k

$b_o := (d_p + d_{aver})\cdot 2 + (b_f + d_{aver})\cdot 2$

$b_o = 185.05$ in -Perimeter of critical cross section

$\beta_c := \dfrac{b_f}{d_p}$

$\beta_c = 1.063$

$V_{c1} := \left(2 + \dfrac{4}{\beta_c}\right)\cdot\sqrt{f_c}\cdot b_o \cdot d_{aver}$

$V_{c1} = 2.019 \times 10^6$ k -Nominal shear strength

$\alpha := 20$

$V_{c2} := \left(2 + \dfrac{\alpha \cdot d_{aver}}{b_o}\right)\cdot\sqrt{f_c}\cdot b_o \cdot d_{aver}$

$V_{c2} = 1.912 \times 10^6$ k -Nominal shear strength

$V_{c.max} := 4\cdot\sqrt{f_c}\cdot\dfrac{b_o \cdot d_{aver}}{1000}$

$V_{c.max} = 1.401 \times 10^3$ k -Maximum nominal shear strength (controles)

$\dfrac{Vu}{\phi} = 107.176$ k

$\dfrac{Vu}{\phi} < V_{c.max}$ OK

One way shear check:

$P_{axial} = 91.1$ k

$V_u := 5P_{axial}$

$V_u = 455.5$ k

$$V_{c1} := 2\cdot\sqrt{f_c}\cdot b_{pc}\cdot\frac{d_{aver}}{1000}$$

$\phi\cdot V_{c1} = 1.023\times10^{3}$ k

$\phi\cdot V_{c1} > V_u$ OK

Two way shear check:

$V_{u1} := 10P_{axial}$

$V_{u1} = 911$ k

$b_o := 2\cdot(15.5 + 2.5)\cdot12 + d_{aver}$ in - Perimeter of the failure plane

$b_o = 464$ in

$$Vc := 4\cdot\sqrt{f_c}\cdot b_o\cdot\frac{d_{aver}}{1000}$$

$Vc = 3.514\times10^{3}$

$\phi\cdot Vc = 2.987\times10^{3}$ k

$\phi\cdot Vc > V_{u1}$ OK

Flexure bending design: (longitudinal direction;short bars)

$M_u := 5 \cdot (P_{axial}) \cdot 18 \cdot 1000$

$M_u = 8.199 \times 10^6$ lb − in $\quad \Phi := 0.9$

$$A_{smin} := \frac{(200 \cdot b_{pc} \cdot d_{aver})}{f_y}$$

$A_{smin} = 33.92$ in^2

$a := 0.25 \cdot d_{aver}$ (assumed)

$$A_s := \frac{M_u}{\Phi \cdot f_y \cdot a}$$

$A_s = 18.979$ in^2

$$a := \frac{(A_{smin} \cdot f_y)}{0.85 \cdot f_c \cdot b_{pc}}$$

$a = 2.151$ in

$$M_n := A_{smin} \cdot f_y \cdot \left(d_{aver} - \frac{a}{2}\right)$$

$M_n = 6.294 \times 10^7$ lb − in

$\Phi \cdot M_n = 5.664 \times 10^7$ lb − in

$\Phi \cdot M_n > M_u$ OK

$y_t := \frac{d_{aver}}{2}$ $\quad y_t = 16$ in

$f_r := 7.5 \cdot \sqrt{f_c}$ $\quad f_r = 443.706$ psi

$I_g := \frac{b_{pc} \cdot d_{aver}^3}{12}$ $\quad I_g = 8.684 \times 10^5$ in^4

$$M_{cr} := \frac{f_r \cdot I_g}{y_t}$$

$M_{cr} = 2.408 \times 10^{7}$ lb − in

$\Phi \cdot M_n = 5.664 \times 10^{7}$ lb − in

$1.2M_{cr} = 2.89 \times 10^{7}$ lb − in

$\Phi \cdot M_n > 1.2M_{cr}$ OK

$$N_L := \frac{A_{smin}}{a_{bar8}}$$

$N_L = 42.937$ - Number of short bars

Short Bars use: 43 # 8

Flexure bending design: (transverse direction; long bars)

$M_u := 2 \cdot P_{axial} \cdot 4.25 \cdot 12 \cdot 1000$

$M_u = 9.292 \times 10^6$ lb – in $\Phi := 0.9$

$A_{smin} := \dfrac{(200 \cdot w_{pc} \cdot d_{aver})}{f_y}$

$A_{smin} = 10.88$ in^2

$a := 0.25 \cdot d_{aver}$ (assumed)

$A_s := \dfrac{M_u}{\Phi \cdot f_y \cdot a}$

$A_s = 21.51$ in^2 –governs

$a := \dfrac{(A_s \cdot f_y)}{0.85 \cdot f_c \cdot w_{pc}}$

$a = 4.253$ in

$M_n := 40 \cdot f_y \cdot \left(d_{aver} - \dfrac{a}{2}\right)$

$M_n = 7.17 \times 10^7$ lb – in

$\Phi \cdot M_n = 6.453 \times 10^7$ lb – in

$\Phi \cdot M_n > M_u$ OK

$y_t := \dfrac{d_{aver}}{2}$ $y_t = 16$ in

$f_r := 7.5 \cdot \sqrt{f_c}$ $f_r = 443.706$ psi

$I_g := \dfrac{b_{pc} \cdot d_{aver}^3}{12}$ $I_g = 8.684 \times 10^5$ in^4

$M_{cr} := \frac{f_r \cdot I_g}{y_t}$

$M_{cr} = 2.408 \times 10^7$ lb − in

$\Phi \cdot M_n = 6.453 \times 10^7$ lb − in

$1.2M_{cr} = 2.89 \times 10^7$ lb − in

$\Phi \cdot M_n > 1.2M_{cr}$ OK

$N_L := \frac{A_s}{a_{bar8}}$

$N_L = 27.227$ - Number of short bars Long Bars use: 28 # 8

Top reinforcement

There is no uplift force on piles at Pier #1 and #3 (expension), therefore shrinkage reinforcement is sufficient

$b_{pc} := 26.5 \cdot 12$ in - Length of pile cap $b_{pc} = 318$ in

$w_{pc} := 8.5 \cdot 12$ in - Width of pile cap $w_{pc} = 102$ in

$h := 36$ in -Thickness of the pile cap

$A_{g1} := b_{pc} \cdot h$ $A_{g1} = 1.145 \times 10^4$ in^2

$A_{smin1} := 0.0018 \cdot A_{g1}$ $A_{smin1} = 20.606$ in^2

$N_{short} := \frac{A_{smin1}}{a_{bar8}}$ $N_{short} = 26.084$

Provide 27 # 8 bars at 11.5 in (on center) along short direction.

$A_{g2} := w_{pc} \cdot h$ $\qquad A_{g2} = 3.672 \times 10^3$ in^2

$A_{smin2} := 0.0018 \cdot A_{g2}$

$A_{smin2} = 6.61$ in^2

$N_{long} := \dfrac{A_{smin2}}{a_{bar8}}$ $\qquad N_{long} = 8.367$

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

$f_c := 3500$ psi

$f_y := 60000$ psi

$h := 16.5$ ft -Depth of beam

$b := 2.5$ ft -Width of beam

$d := 0.95 \cdot h$ (assumed)

$d = 15.675$ ft -Distance from reinforcement to extreme compression fiber

$l := 21$ ft -Hight of beam

$\Phi := 0.85$ - for shear

$V := 130.7$ k

$M_{un} := 3450.6$ k – ft

$R := 2$

$M := \frac{M_{un}}{R}$ factored

$\frac{l}{d} = 1.34$ < 2.5 - criteria determining Deep Beam

For $\frac{l}{d} < 2.0$ - use following eq.

$$V_u := \Phi \cdot \left(8 \cdot \sqrt{f_c} \cdot b \cdot d\right) \cdot \frac{144}{1000}$$

$V_u = 2.27 \times 10^3$ k -Factored shear force

$V = 130.7\,k < V_u = 2.27 \times 10^3$ k OK

$$V_c := 2 \cdot \left(\sqrt{f_c}\right) \cdot b \cdot d \cdot \frac{144}{1000}$$

$V_c = 667.689$ k -Nominal shear resisting force (**controls**)

$$V_{cmax} := 6 \cdot \sqrt{f_c} \cdot b \cdot d \cdot \frac{144}{1000}$$

$V_{cmax} = 2.003 \times 10^3$ k -Maximum nominal shear resisting force

$\Phi \cdot V_c = 567.535$ k

$V_u = 2.27 \times 10^3$ must be less or equal than $\Phi \cdot \left(V_c + V_s\right)$

$\Phi \cdot V_c < V_u$ -Shear reinforcement is required

$$\boxed{V_u - \Phi \cdot V_c \le \Phi \cdot V_s}$$

$$V_s := \frac{V_u}{\Phi} - V_c$$

$V_s = 2.003 \times 10^3$ k

$A_{s1} := 1.56$ in^2 Cross sectional area of # 11 rebar

$A_l := A_{s1} \cdot \frac{2}{144}$ $A_l = 0.022$ ft^2 Area of trans. rebars placed at distance s_{tr}

$A_{str} := 0.44$ in^2 Cross sectional area of # 6 rebar

$A_{tr} := A_{str} \cdot \frac{2}{144}$ $A_{tr} = 6.111 \times 10^{-3}$ ft^2 Area of long. rebars placed at distance s_l

$s_{trmax} := \frac{d}{5}$ $s_{trmax} = 3.135$ ft -Maximum distance between rebars in transverse direction (on center)

or 1.5 ft -use this value

$s_h := 1.5$ ft -Distance between horizontal (transverse) rebars (on center)

$t_r := 10.44$ in -Clear spacing between rebars;

$\frac{(t_r + 1.56)}{12} = 1$ ft

$s_v := 1.0$ ft -Distance between vertical rebars (on center);PCACOL (file:#11BOT2)

$$\left[\left(\frac{A_{tr}}{s_h}\right) \cdot \frac{\left[1 + \left(\frac{h}{d}\right)\right]}{12}\right] + \left[\left(\frac{A_l}{s_v}\right) \cdot \frac{\left[11 - \left(\frac{h}{d}\right)\right]}{12}\right] \cdot f_y \cdot d = V_s$$

$$K_1 := \left(\frac{A_{tr}}{s_h}\right) \cdot \frac{\left[1 + \left(\frac{h}{d}\right)\right]}{12} \qquad K_1 = 6.969 \times 10^{-4}$$

$$K_2 := \left(\frac{A_l}{s_v}\right) \cdot \frac{\left[11 - \left(\frac{h}{d}\right)\right]}{12} \qquad K_2 = 0.018$$

$\left(K_1 + K_2\right) \cdot f_y \cdot d \cdot \frac{144}{1000} = 2.527 \times 10^3$ must be equal or greater than $V_s = 2.003 \times 10^3$ OK

Check for minimum steel: (pier # 2)

Rebars # 6 placed in transverse direction on both faces of the pier

$A_{tr} = 6.111 \times 10^{-3}$ ft^2 - transverse

$A_{trmin} := 0.0015 \cdot b \cdot s_h$

$A_{trmin} = 5.625 \times 10^{-3}$ ft^2

$A_{tr} = 6.111 \times 10^{-3}$ 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

$A_l = 0.022$ ft^2 - longitudinal

$A_{lmin} := 0.0025 \cdot b \cdot s_h$

$A_{lmin} = 9.375 \times 10^{-3}$ ft^2

$A_l = 0.022$ 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 # 2)

$\Phi := 0.9$

$M = 1.725 \times 10^{3}$ – ft

$\frac{l}{h} = 1.273$ > 1 so jd=0.2*(l+2h)

$j := .2\frac{(l + 2\cdot h)}{d}$

$j = 0.689$

$j\cdot d = 10.8$ ft

Minimum reinforcement area

$A_s := \frac{M\cdot 144}{\Phi\cdot f_y\cdot j\cdot d\cdot\frac{144}{1000}}$ $A_s = 2.958$ in^2

$A_{smin1} := \max\left(3\cdot\frac{\sqrt{f_c}}{f_y}\cdot b\cdot d\cdot 144\right)$ $A_{smin1} = 16.692$ in^2

$A_{smin2} := 200\cdot b\cdot\frac{d\cdot 144}{f_y}$ $A_{smin2} = 18.81$ in^2 -controls

$y := 0.25\cdot h - 0.05\cdot l$ -Depth over which flex. reinf. is to be distributed (in transverse direction)

$y = 3.075$ ft

$s_v = 1$ ft -Distance between vertical rebars (on center) as acquired from PCACOL design (file:#11BOT2)

$n_{tr} := \frac{y}{s_v}\cdot 2 + 2$

$n_{tr} = 8.15$ -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

$A_s = 12.714$ in^2 must be equal or greater than $A_{smin2} = 18.81$ in^2 not satisfactory

Number of rebars required for flexure:

$$N_{req} := \frac{A_{smin2}}{A_{s1}}$$

$N_{req} = 12.058$ -number of #11bars to be placed within distance $y = 3.075$ ft from extreme tension fiber

Pile cap design Pier # 2 (fixed)

$f_c := 3500$ psi

$f_y := 60000$ psi

$P_{axial} := 273.4$ k -Vertical force at top of most loaded pile

$d_{bar} := 1.0$ in -Diameter of # 8 bar

$a_{bar} := 0.79$ in^2 -Area of # 8 bar

$d_p := 13.83$ in
$b_f := 14.695$ in -Cross sectional dimensions of HP 14x89

$h := 46$ in -Thickness of the pile cap

$b_{pc} := 27.0 \cdot 12$ in - Length of pile cap

$w_{pc} := 14.5 \cdot 12$ in - Width of pile cap

$d_{emb} := 12$ in -Depth of pile embedment

$d_{aver} := h - 3 - d_{bar}$

$d_{aver} = 42$ in -Average depth of the footing slab (from top to bottom reinforcement)

$\phi := 0.85$ - for shear

$\Phi := 0.9$ - for flexure

Pile punching shear check:

$Vu := P_{axial}$

$Vu = 273.4$ k

$b_o := (d_p + d_{aver}) \cdot 2 + (b_f + d_{aver}) \cdot 2$

$b_o = 225.05$ in -Perimeter of critical cross section

$\beta_c := \dfrac{b_f}{d_p}$ $\beta_c = 1.063$

$V_{c1} := \left(2 + \dfrac{4}{\beta_c}\right) \cdot \sqrt{f_c} \cdot b_o \cdot d_{aver}$

$V_{c1} = 3.223 \times 10^6$ k -Nominal shear strength

$\alpha := 20$

$V_{c2} := \left(2 + \dfrac{\alpha \cdot d_{aver}}{b_o}\right) \cdot \sqrt{f_c} \cdot b_o \cdot d_{aver}$

$V_{c2} = 3.206 \times 10^6$ k -Nominal shear strength

$V_{c.max} := 4 \cdot \sqrt{f_c} \cdot \dfrac{b_o \cdot d_{aver}}{1000}$

$V_{c.max} = 2.237 \times 10^3$ k -Maximum nominal shear strength (**controles**)

$\dfrac{Vu}{\phi} = 321.647$ k $\dfrac{Vu}{\phi} < V_{c.max}$ **OK**

One way shear check:

$P_{axial} = 273.4$ k

$V_u := 5P_{axial}$

$V_u = 1.367 \times 10^3$ k

$V_{c1} := 2 \cdot \sqrt{f_c} \cdot b_{pc} \cdot \frac{d_{aver}}{1000}$ $\qquad d_{aver} = 42$ in

$\phi \cdot V_{c1} = 1.369 \times 10^3$ k

$\phi \cdot V_{c1} > V_u$ OK

Two way shear check:

$V_{u1} := 2 \cdot (6 \cdot P_{axial})$

$V_{u1} = 3.281 \times 10^3$ k

$b_o := 2 \cdot (16 + 2.5) \cdot 12 + d_{aver}$ in - Perimeter of the failure plane

$b_o = 486$ in

$Vc := 4 \cdot \sqrt{f_c} \cdot b_o \cdot \frac{d_{aver}}{1000}$

$Vc = 4.83 \times 10^3$

$\phi \cdot Vc = 4.106 \times 10^3$ k

$\phi \cdot Vc > V_{u1}$ OK

Flexure bending design: (longitudinal direction;short bars)

$M_u := (5P_{axial}) \cdot 1000 \cdot 57$

$M_u = 7.792 \times 10^7$ lb − in $\qquad \Phi := 0.9$

$A_{smin} := \frac{(200 \cdot b_{pc} \cdot d_{aver})}{f_y}$

$A_{smin} = 45.36$ in^2

$a := 0.25 \cdot d_{aver}$ (assumed)

$A_s := \frac{M_u}{\Phi \cdot f_y \cdot a}$

$A_s = 137.423$ in^2

$a := \frac{(A_s \cdot f_y)}{0.85 \cdot f_c \cdot b_{pc}}$

$a = 8.554$ in

$M_n := A_s \cdot f_y \cdot \left(d_{aver} - \frac{a}{2}\right)$

$M_n = 3.11 \times 10^8$ lb – in

$\Phi \cdot M_n = 2.799 \times 10^8$ lb – in

$\Phi \cdot M_n > M_u$ OK

$y_t := \frac{d_{aver}}{2}$ $y_t = 21$ in

$f_r := 7.5 \cdot \sqrt{f_c}$ $f_r = 443.706$ psi

$I_g := \frac{b_{pc} \cdot d_{aver}^3}{12}$ $I_g = 2 \times 10^6$ in^4

$M_{cr} := \frac{f_r \cdot I_g}{y_t}$

$M_{cr} = 4.227 \times 10^7$ lb – in

$\Phi \cdot M_n = 2.799 \times 10^8$ lb – in

$1.2M_{cr} = 5.072 \times 10^7$ lb – in

$\Phi \cdot M_n > 1.2M_{cr}$ OK

$N_L := \dfrac{A_{smin}}{a_{bar8}}$

$N_L = 57.418$ - Number of short bars

Short Bars use: 58 # 8

Flexure bending design: (transverse direction; long bars)

$M_u := 3 \cdot P_{axial} \cdot 1000 \cdot 4.25 \cdot 12$

$M_u = 4.183 \times 10^7$ lb – in $\Phi := 0.9$

$A_{smin} := \dfrac{(200 \cdot w_{pc} \cdot d_{aver})}{f_y}$

$A_{smin} = 24.36$ in^2

$a := 0.25 \cdot d_{aver}$ (assumed)

$A_s := \dfrac{M_u}{\Phi \cdot f_y \cdot a}$

$A_s = 73.775$ in^2 –governs

$a := \dfrac{(A_s \cdot f_y)}{0.85 \cdot f_c \cdot w_{pc}}$

$a = 8.551$ in

$M_n := 40 \cdot f_y \cdot \left(d_{aver} - \frac{a}{2}\right)$

$M_n = 9.054 \times 10^7$ lb − in

$\Phi \cdot M_n = 8.148 \times 10^7$ lb − in

$\Phi \cdot M_n > M_u$ OK

$y_t := \frac{d_{aver}}{2}$ $y_t = 21$ in

$f_r := 7.5 \cdot \sqrt{f_c}$ $f_r = 443.706$ psi

$I_g := \frac{b_{pc} \cdot d_{aver}^3}{12}$ $I_g = 2 \times 10^6$ in^4

$M_{cr} := \frac{f_r \cdot I_g}{y_t}$

$M_{cr} = 4.227 \times 10^7$ lb − in

$\Phi \cdot M_n = 8.148 \times 10^7$ lb − in

$1.2M_{cr} = 5.072 \times 10^7$ lb − in

$\Phi \cdot M_n > 1.2M_{cr}$ OK

$N_L := \frac{A_{smin}}{a_{bar8}}$

$N_L = 30.835$ - Number of short bars

Long Bars use: 30 # 6

Top reinforcement

$F_{uplift} := 144.0$ k - Uplift force on a single pile (Fixed pier ; load combination COMD1)

$n := 5$ - Number of piles

$Marm := 4.75$ ft - Moment arm

$f_c = 3.5 \times 10^3$ psi

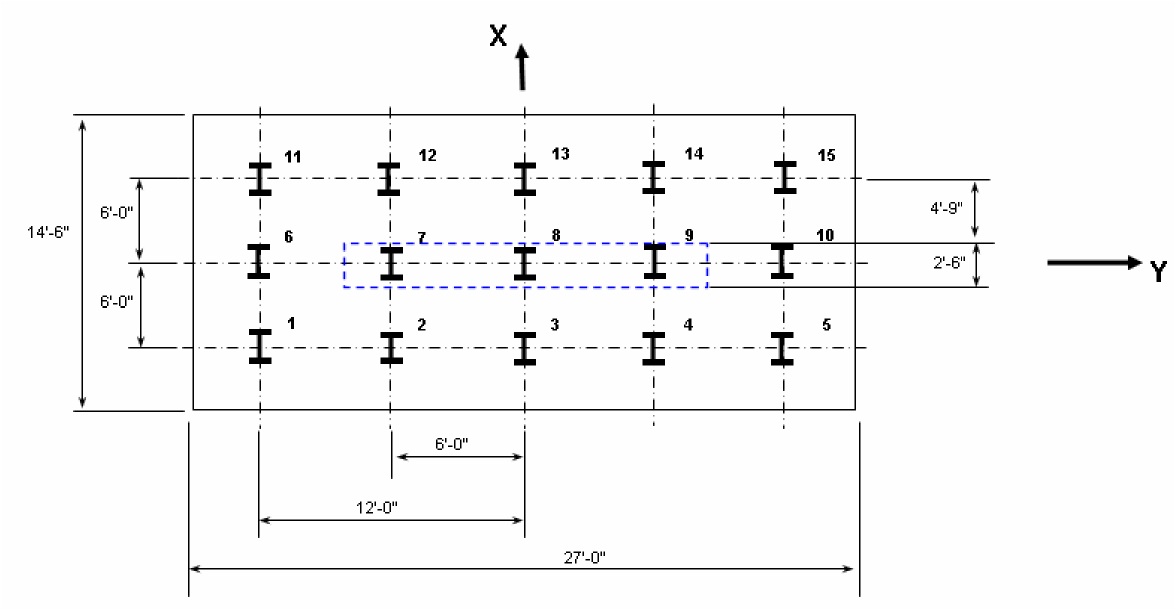
$d_{aver} = 42$ in -Average depth of the footing slab (from top to bottom reinforcement)

$\Phi := 0.9$ - for flexure

$h := 46$ in -Thickness of the pile cap

$b_{pc} := 27.0 \cdot 12$ in - Length of pile cap $b_{pc} = 324$ in

$w_{pc} := 14.5 \cdot 12$ in - Width of pile cap $w_{pc} = 174$ in



$M_{ult} := n \cdot F_{uplift} \cdot Marm \cdot 1000 \cdot 12$

$M_{ult} = 4.104 \times 10^7$ lb – in

$f_r := 7.5 \cdot \sqrt{f_c}$

$f_r = 443.706$ psi

$$I_g := \frac{b_{pc} \cdot \left[\left(d_{aver} \right)^3 \right]}{12}$$

$I_g = 2 \times 10^6$ in^4

$y_t := \frac{h}{2}$ $y_t = 23$ in

$M_{cr} := \frac{f_r \cdot \left(I_g \right)}{y_t}$ $M_{cr} = 3.859 \times 10^7$ lb – in

$M_{nom} := \frac{M_{ult}}{\Phi}$

$\Phi \cdot M_{nom} = 4.104 \times 10^7$ lb – in

$1.2 \cdot M_{cr} = 4.631 \times 10^7$ lb – in

$\Phi \cdot M_{nom} < 1.2 \cdot M_{cr}$ - provide minimum reinforcement (shrinkage reinforcement)

$A_{g1} := b_{pc} \cdot h$ $A_{g1} = 1.49 \times 10^4$ in^2

$A_{smin1} := 0.0018 \cdot A_{g1}$ $A_{smin1} = 26.827$ in^2

$N_{short} := \frac{A_{smin1}}{a_{bar8}}$ $N_{short} = 33.958$ Provide 34 # 8 bars at 9.75 in (on center) along short direction.

$A_{g2} := w_{pc} \cdot h \qquad A_{g2} = 8.004 \times 10^{3} \quad in^{2}$

$A_{smin2} := 0.0018 \cdot A_{g2}$

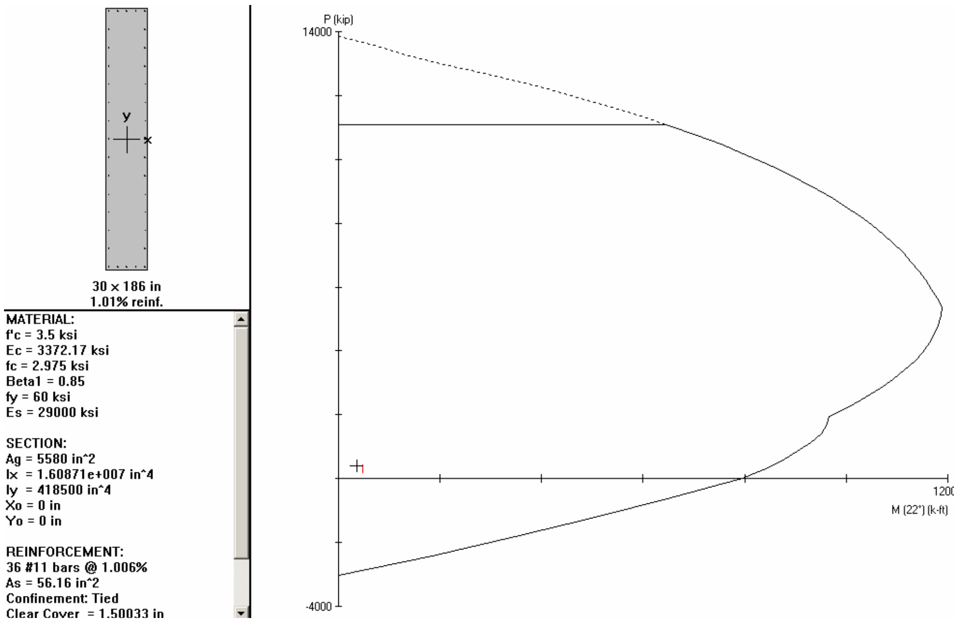
$A_{smin2} = 14.407 \qquad in^{2}$

$N_{long} := \dfrac{A_{smin2}}{a_{bar8}} \qquad N_{long} = 18.237$

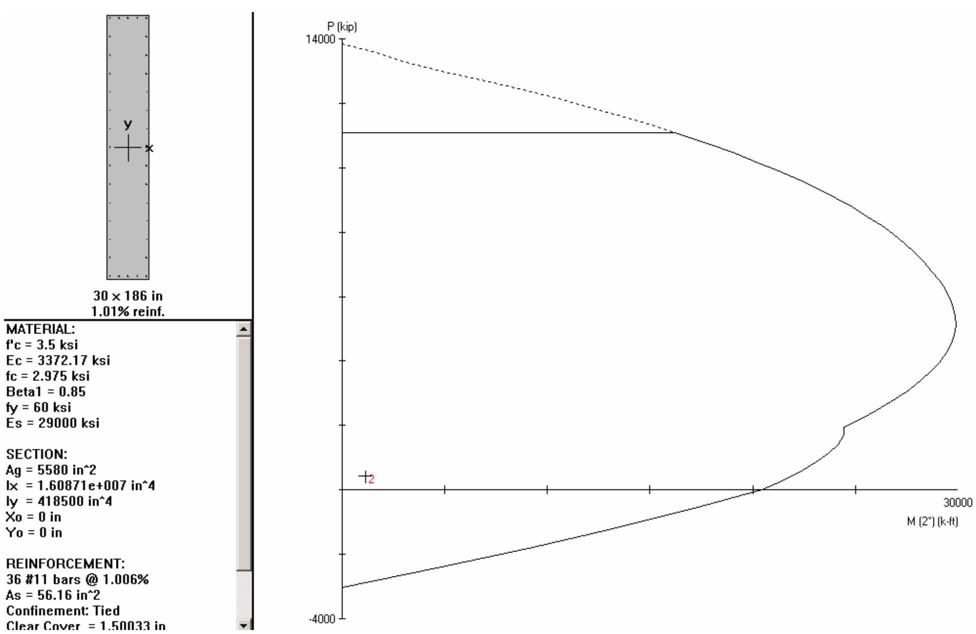
Provide 19 # 8 bars at 9.5 in (on center) along short direction.

PCACOL Output

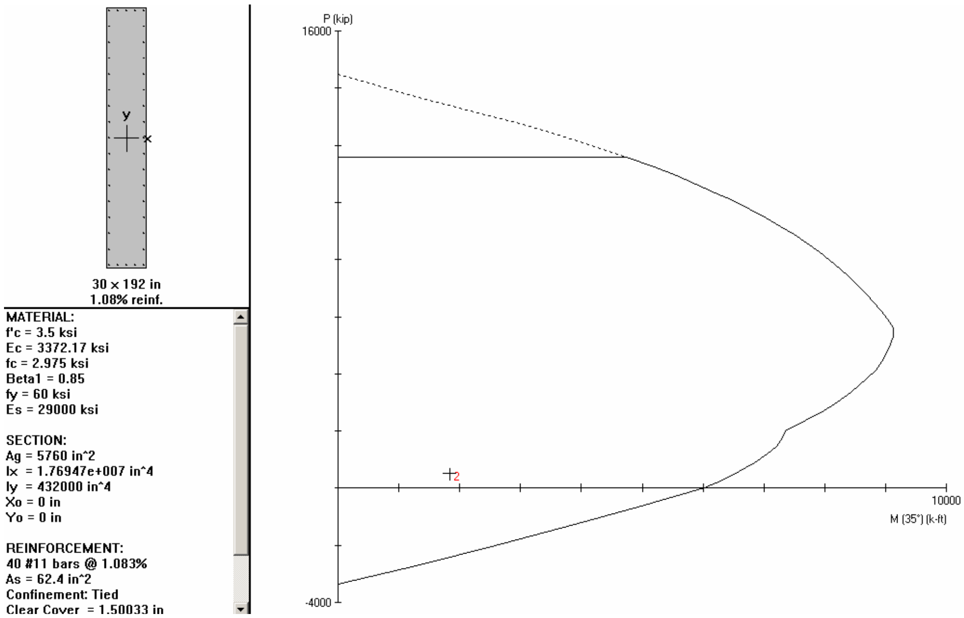
Piers # 1 & # 3 (fixed). At base for load combination COMB1



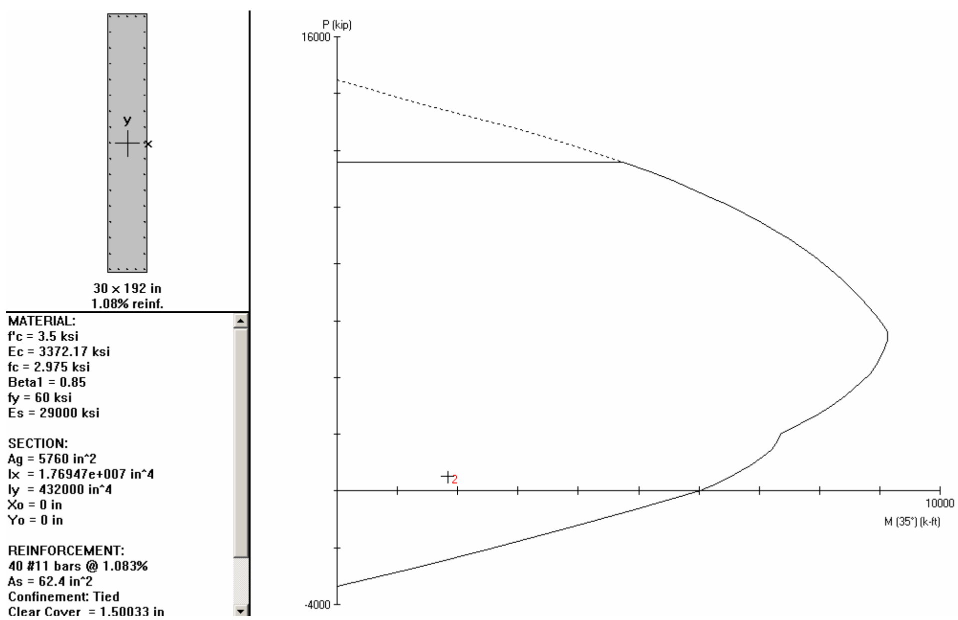
Piers # 1 & # 3 (fixed). At base for load combination COMB2



Pier # 2 (fixed). At base for load combination COMB1



Pier # 2 (fixed). At base for load combination COMB2



Minimum seat width

For Seismic Performance Category C

$L := 236.5$ ft - Length of bridge

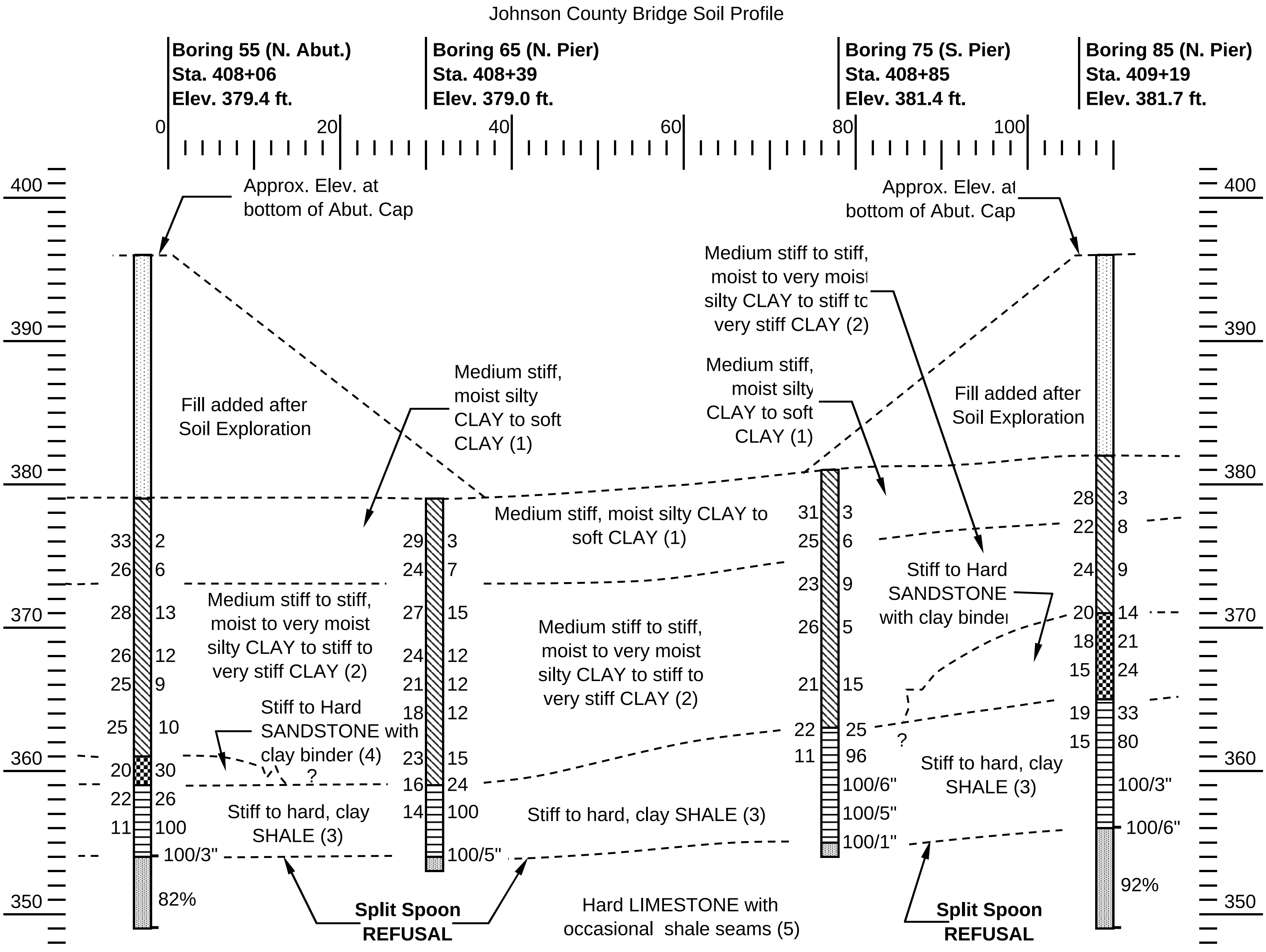
$H := 23$ ft - Height of the tallest pier

$S := 0$ - Angle of skew

$$N := (12 + 0.03 \cdot L + 0.12 \cdot H) \cdot \left(1 + 0.000125 \cdot S^2\right)$$

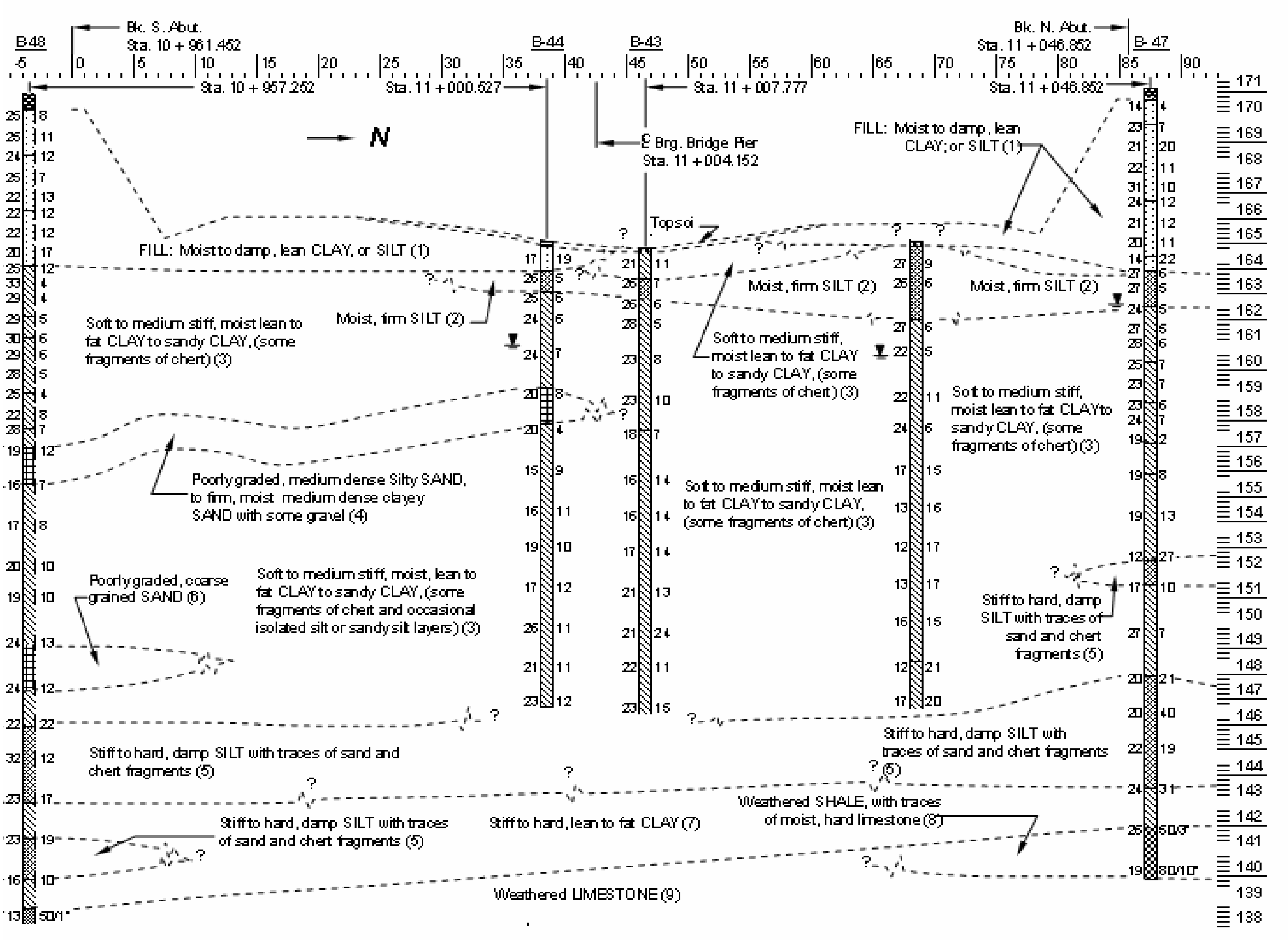
$N = 21.855$ in - Minimum width of support

APPENDIX G - SOIL PROFILES AT ILLINOIS BRIDGE SITES

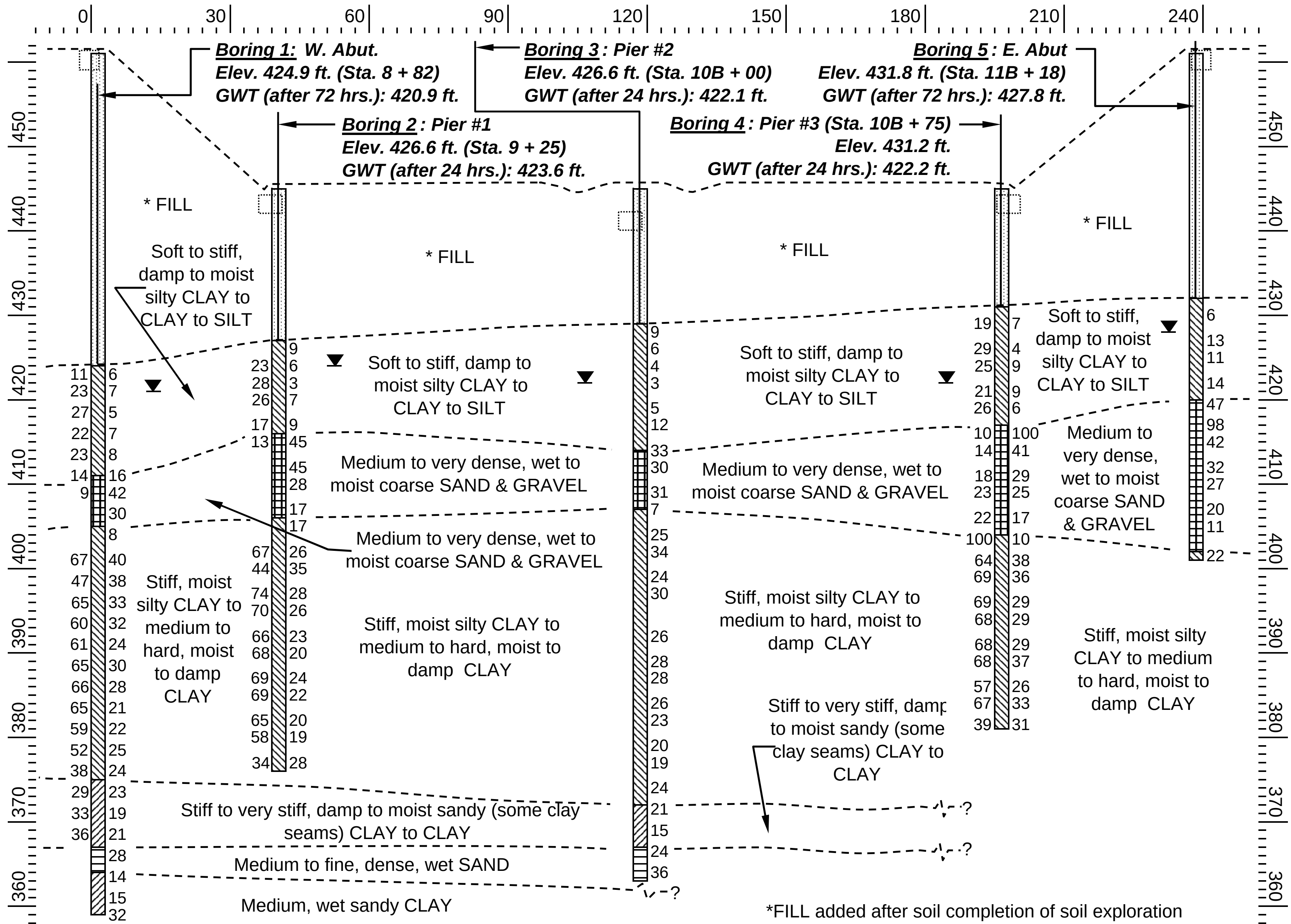


Johnson County Bridge Soil Profile
Boring 55 (N. Abut.)
Sta. 408+06
Elev. 379.4 ft.
Boring 65 (N. Pier)
Sta. 408+39
Elev. 379.0 ft.
Boring 75 (S. Pier)
Sta. 408+85
Elev. 381.4 ft.
Boring 85 (N. Pier)
Sta. 409+19
Elev. 381.7 ft.
0
20
40
60
80
100
400
390
380
370
360
350
Approx. Elev. at bottom of Abut. Cap
Fill added after Soil Exploration
Medium stiff, moist silty CLAY to soft CLAY (1)
Medium stiff to stiff, moist to very moist silty CLAY to stiff to very stiff CLAY (2)
Stiff to Hard SANDSTONE with clay binder (4)
Stiff to hard, clay SHALE (3)
Hard LIMESTONE with occasional shale seams (5)
Split Spoon REFUSAL
Stiff to Hard SANDSTONE with clay binder
33 2
26 6
28 13
26 12
25 9
25 10
20 30
22 26
11 100
100/3"
82%
29 3
24 7
27 15
24 12
21 12
18 12
23 15
16 24
14 100
100/5"
31 3
25 6
23 9
26 5
21 15
22 25
11 96
100/6"
100/5"
100/1"
28 3
22 8
24 9
20 14
18 21
15 24
19 33
15 80
100/3"
100/6"
92%
?

St. Clair County Bridge Soil Profile



Pulaski County Bridge Soil Profile



Madison-Pulaski County Bridge Soil Profile