MCEER / ATC 49-2

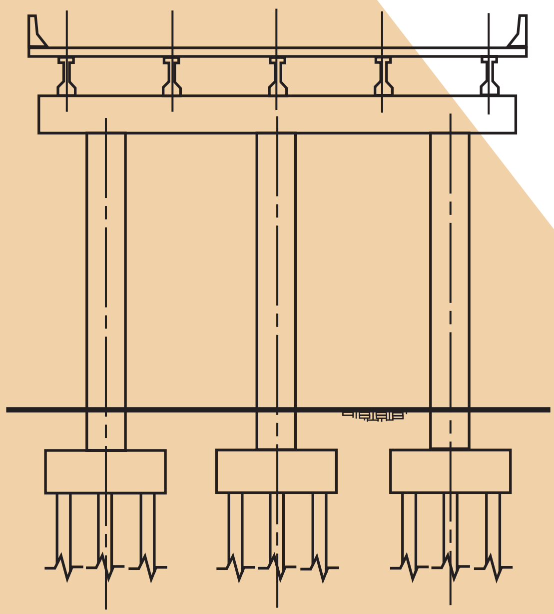
ATC/MCEER Joint Venture

A Partnership of:
Applied Technology Council and
Multidisciplinary Center for Earthquake Engineering Research

MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
A National Center of Excellence in Advanced Technology Applications

Design Examples

Recommended LRFD guidelines for the seismic design of highway bridges



Applied Technology Council

The Applied Technology Council (ATC) is a nonprofit, tax-exempt corporation established in 1971 through the efforts of the Structural Engineers Association of California. ATC's mission is to develop state-of-the-art, user-friendly engineering resources and applications for use in mitigating the effects of natural and other hazards on the built environment. ATC also identifies and encourages needed research and develops consensus opinions on structural engineering issues in a non-proprietary format. ATC thereby fulfills a unique role in funded information transfer.

ATC is guided by a Board of Directors consisting of representatives appointed by the American Society of Civil Engineers, the National Council of Structural Engineers Associations, the Structural Engineers Association of California, the Western Council of Structural Engineers Associations, and four at-large representatives concerned with the practice of structural engineering. Each director serves a three-year term.

Project management and administration are carried out by a full-time Executive Director and support staff. Project work is conducted by a wide range of highly qualified consulting professionals, thus incorporating the experience of many individuals from academia, research, and professional practice who would not be available from any single organization. Funding for ATC projects is obtained from government agencies and from the private sector in the form of tax-deductible contributions.

Multidisciplinary Center for Earthquake Engineering Research

The Multidisciplinary Center for Earthquake Engineering Research (MCEER) is a national center of excellence in advanced technology applications that is dedicated to the reduction of earthquake losses nationwide. Headquartered at the University at Buffalo, State University of New York, the Center was originally established by the National Science Foundation (NSF) in 1986, as the National Center for Earthquake Engineering Research (NCEER).

Comprising a consortium of researchers from numerous disciplines and institutions throughout the United States, the Center's mission is to reduce earthquake losses through research and the application of advanced technologies that improve engineering, pre-earthquake planning and post-earthquake recovery strategies. Toward this end, the Center coordinates a nationwide program of multidisciplinary team research, education and outreach activities.

Funded principally by NSF, the State of New York and the Federal Highway Administration (FHWA), the Center derives additional support from the Federal Emergency Management Agency (FEMA), other state governments, academic institutions, foreign governments and private industry.

ATC/MCEER Joint Venture

The ATC/MCEER Joint Venture, a partnership of the Applied Technology Council (ATC) and the Multidisciplinary Center for Earthquake Engineering Research (MCEER), was established to conduct the NCHRP 12-49 project, Development of Comprehensive Specifications for the Seismic Design of Bridges, which was funded by the Transportation Research Board of the National Research Council.

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Notice

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MCEER/ATC-49-2

Design Examples

Recommended LRFD Guidelines for the Seismic Design of Highway Bridges

Prepared under

NCHRP Project 12-49, FY '98

"Comprehensive Specification for the Seismic Design of Bridges"

National Cooperative Highway Research Program

Prepared by

ATC/MCEER JOINT VENTURE

A partnership of the

Applied Technology Council

(www.ATCouncil.org)

and the

Multidisciplinary Center for Earthquake Engineering Research

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2003

PREFACE

In 2003 the ATC/MCEER Joint Venture, a partnership of the Applied Technology Council (ATC) and the Multidisciplinary Center for Earthquake Engineering Research (MCEER), University at Buffalo, published the set of documents, *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges, Part I, Specifications,* and *Part II, Commentary and Appendices* (MCEER/ATC-49 Report). These documents are reformatted versions of the seismic design provisions (specifications and commentary) for highway bridges developed under NCHRP (National Cooperative Highway Research Program) Project 12-49, a recently completed project to develop seismic design provisions that would be compatible with the AASHTO *LRFD Bridge Design Specifications.* The reformatting effort, which was carried out to facilitate immediate use of the Project 12-49 provisions by bridge design professionals, was funded as a task under the MCEER Highway Project, which is sponsored by the Federal Highway Administration (FHWA).

NCHRP Project 12-49 also included a companion study to investigate the effects of liquefaction and an effort to develop design examples using the NCHRP 12-49 recommended provisions. The design examples are provided in this MCEER/ATC-49-2 Report, *Design Examples, Recommended LRFD Guidelines for the Seismic Design of Highway Bridges*, and the liquefaction study is documented in the companion MCEER/ATC-49-1 Report, *Liquefaction Study Report*, *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges.*

The two design examples contained in this document, which illustrate use of the *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges*, are the eighth and ninth design examples in a series originally developed for FHWA to illustrate the use of the American Association of State Highway and Transportation Officials (AASHTO) Division 1-A *Standard Specifications for Highway Bridges*. Each of the nine design examples, including the seven previously developed, were carried out and reported on in a consistent manner, using the same calculation and report formatting procedures. Design Example 8 was performed on a five-span continuous cast-in-place concrete box girder bridge and the ninth design example (Design Example 2LRFD) was performed on a three-span continuous steel girder bridge.

A broad array of engineering expertise was engaged by the ATC/MCEER Joint Venture to develop the original NCHRP 12-49 seismic design provisions, companion liquefaction study, and design examples. Ian Friedland of ATC (and formerly MCEER) served as the Project Principal Investigator and Ronald Mayes (Simpson Gumpertz & Heger, Inc.) served as the Project Technical Director. The NCHRP Project 12-49 team consisted of Donald Anderson (CH2M Hill, Inc.), Michel Bruneau (University at Buffalo), Gregory Fenves (University of California at Berkeley), John Kulicki (Modjeski and Masters, Inc.), John Mander (University of Canterbury, formerly University at Buffalo), Lee Marsh (BERGER/ABAM Engineers), Ronald Mayes (Simpson, Gumpertz & Heger, Inc.), Geoffrey Martin (University of Southern California), Andrzej Nowak (University of Michigan), Richard Nutt (bridge consultant), Maurice Power (Geomatrix Consultants, Inc.), and Andrei Reinhorn (University at Buffalo).

The project also included an advisory Project Engineering Panel; Ian Buckle, of the University of Nevada at Reno, co-chaired this committee with Christopher Rojahn of ATC, who also served as the Project Administrative Officer. Other members included Serafim Arzoumanidis (Steinman Engineers), Mark Capron (Sverdrup Civil Inc.), Ignatius Po Lam (Earth Mechanics), Paul Liles (Georgia DOT), Brian Maroney (California DOT), Joseph Nicoletti (URS Greiner Woodward Clyde), Charles Roeder (University of Washington), Frieder Seible (University of California at San Diego), and Theodore Zoli (HNTB Corporation).

NCHRP Project Panel C12-49, under the direction of NCHRP Senior Program Officer David Beal and chaired by Harry Capers of the New Jersey Department of Transportation (DOT), also provided a significant amount of input and guidance during the conduct of the project. The other members of the NCHRP Project Panel were D.W. Dearasaugh (Transportation Research Board), Gongkang Fu (Wayne State University), C. Stewart Gloyd (Parsons Brinckerhoff), Manoucher Karshenas (Illinois DOT), Richard Land (California DOT), Bryan Millar (Montana DOT), Amir Mirmirman (University of

Central Florida), Charles Ruth (Washington State DOT), Steven Starkey (Oregon DOT), and Phillip Yen (FHWA).

Three drafts of the Project 12-49 specifications and commentary were prepared and reviewed by the ATC Project Engineering Panel, NCHRP Project Panel 12-49, and the AASHTO Highway Subcommittee on Bridges and Structures seismic design technical committee (T-3), which was chaired by James Roberts of Caltrans.

Lee Marsh led the development of the design examples provided in this volume and ATC and MCEER staff provided publishing services.

Michel Bruneau, MCEER
Christopher Rojahn, ATC

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DESIGN EXAMPLE 8

DESIGN EXAMPLE NO. 8

PURPOSE OF DESIGN EXAMPLE

This is the eighth in a series of seismic design examples originally developed for the FHWA. The original seven examples were developed to illustrate the use of the AASHTO Division I-A Specification for seismic design. The eighth and ninth examples illustrate the use of the *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges*, MCEER/ATC 49 (2003) for seismic design, which is a comprehensive revision of the AASHTO seismic design provisions. Each example emphasizes different features that must be considered in the seismic analysis and design process. The matrix below is a summary of the features of the nine examples.

DESIGN EXAMPLE NO.	DESIGN EXAMPLE DESCRIPTION	SEISMIC CATEGORY	PLAN GEOMETRY	SUPER-STRUCTURE TYPE	PIER TYPE	ABUTMENT TYPE	FOUNDATION TYPE	CONNECTIONS AND JOINTS
1	Two-Span Continuous	SPC - C	Tangent Square	CIP Concrete Box	Three-Column Integral Bent	Seat Stub Base	Spread Footings	Monolithic Joint at Pier Expansion Bearing at Abutment
2	Three-Span Continuous	SPC - B	Tangent Skewed	Steel Girder	Wall Type Pier	Tall Seat	Spread Footings	Elastomeric Bearing Pads (Piers and Abutments)
3	Single-Span	SPC - C	Tangent Square	AASHTO Precast Concrete Girders	(N/A)	Tall Seat (Closed-In)	Spread Footings	Elastomeric Bearing Pads
4	Three-Span Continuous	SPC - C	Tangent Skewed	CIP Concrete	Two-Column Integral Bent	Seat	Spread Footings	Monolithic at Col. Tops Pinned Column at Base Expansion Bearings at Abutments
5	Nine-Span Viaduct with Four-Span and Five-Span Continuous Structs.	SPC - B	Curved Square	Steel Girder	Single-Column (Variable Heights)	Seat	Steel H-Piles	Conventional Steel Pins and PTFE Sliding Bearings
6	Three-Span Continuous	SPC - C	Sharply-Curved Square	CIP Concrete Box	Single Column	Monolithic	Drilled Shaft at Piers, Steel Piles at Abutments	Monolithic Concrete Joints
7	12-Span Viaduct with (3) Four-Span Structures	SPC - B	Tangent Square	AASHTO Precast Concrete Girders	Pile Bents (Battered and Plumb)	Seat	Concrete Piles and Steel Piles	Pinned and Expansion Bearings

SECTION I INTRODUCTION

DESIGN EXAMPLE NO.	DESIGN EXAMPLE DESCRIPTION							
8	Five-Span Continuous	SDAP E	Tangent Square	CIP Concrete Box Girder	Two-Column Integral Bent	Stub Abutment with Over-hanging Diaphragm	CIP Concrete Piles with Steel Casings	Monolithic at Interior Piers Expansion Bearings at Abutments
2LRFD	Three-Span Continuous	SDAP A2 and C	Tangent Skewed	Steel Girder	Four-Column Bent and Wall Type Pier	Tall Seat	Spread Footings	Conventional and Elastomeric Bearing Pads (Piers and Abutments)

REFERENCE AASHTO SPECIFICATIONS

Example Nos. 1 through 7 conform to the following specifications.

AASHTO Division I (herein referred to as "Division I")

Standard Specifications for Highway Bridges, American Association of State Highway and Transportation Officials, Inc., 15th Edition, as amended by the Interim Specifications-Bridges-1993 through 1995.

AASHTO Division I-A (herein referred to as "Division I-A" or the "Specification")

Standard Specifications for Highway Bridges, Division I-A, Seismic Design, American Association of State Highway and Transportation Officials, Inc., 15th Edition, as amended by the Interim Specifications-Bridges-1995.

Example Nos. 8 and 2LRFD conform to the following.

***Recommended LRFD Guidelines for the Seismic Design of Highway Bridges*, MCEER/ATC 49 (2003) (herein referred to as the Guide Specification)**

Additionally, these examples cross reference the original NCHRP Specification that is the source document of the Guide Specification.

NCHRP 12-49 *Comprehensive Specification for the Seismic Design of Bridges*, Revised LRFD Design Specifications, Third Draft, March 2001.

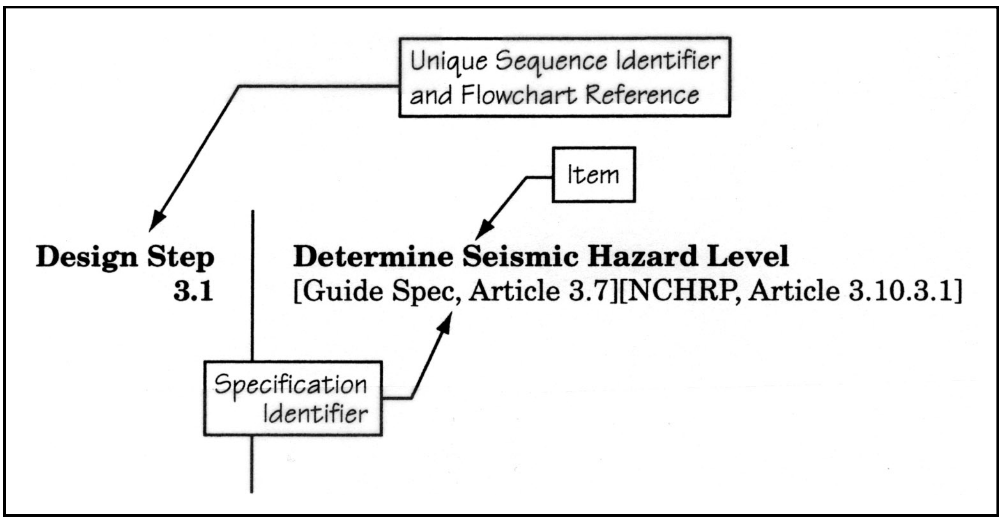
SECTION I INTRODUCTION

FLOWCHARTS AND DESIGN STEPS

This eighth example follows the outline given in detailed flowcharts presented in Section II, Flowcharts. The flowcharts include a main chart, which generally follows the one currently used in the proposed seismic Guide Specification.

The purpose of Design Steps is to present the information covered by the example in a logical and sequential manner that allows for easy referencing within the example itself. Each Design Step has a unique number in the left margin of the calculation document. The title is located to the right of the Design Step number. Where appropriate, a reference to both the Guide Specification and the NCHRP Specification follows the title.

An example is shown below.

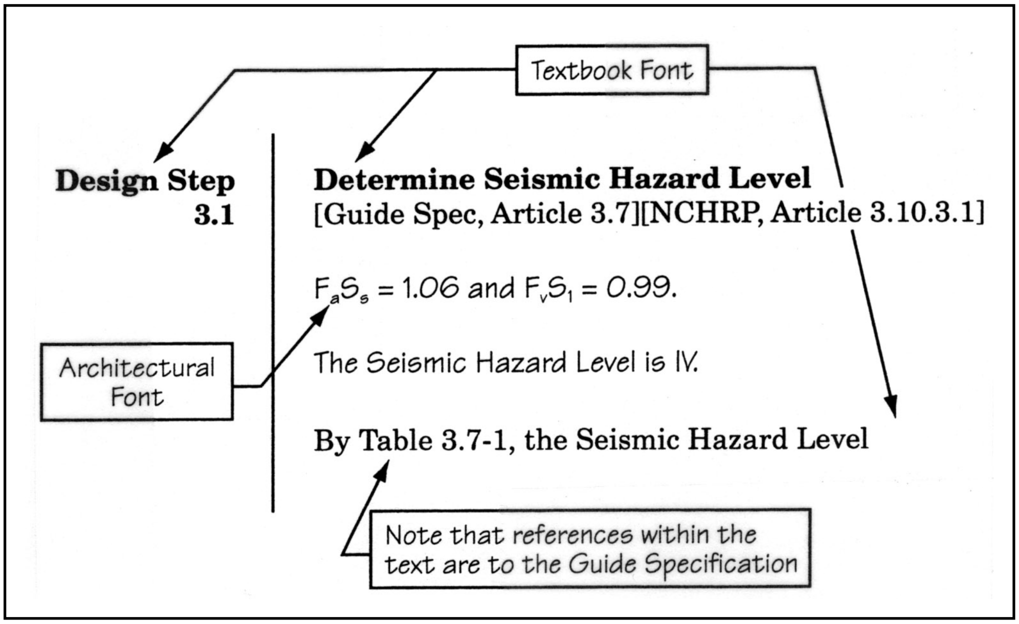


SECTION I INTRODUCTION

USE OF DIFFERENT TYPE FONTS

In the example, two primary type fonts have been used. One font, similar to the type used for textbooks, is used for all section headings and for commentary. The other, an architectural font that appears hand printed, is used for all primary calculations. The material in the architectural font is the essential calculation material and essential results.

An example of the use of the fonts is shown below.



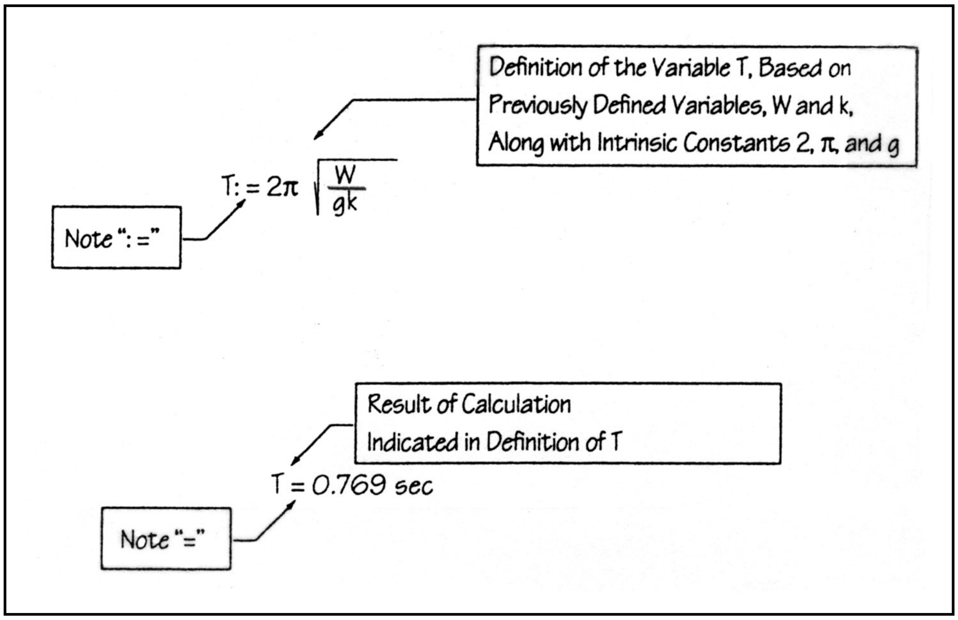
SECTION I INTRODUCTION

USE OF MATHCAD®

To provide consistent results and quality control, all calculations have been performed using the program Mathcad®.

The variables used in equations calculated by the program are defined before the equation, and the **definition** of either a variable or an equation is distinguished by a ':=' symbol. The **echo** of a variable or the result of a calculation is distinguished by a '=' symbol, i.e., no colon is used.

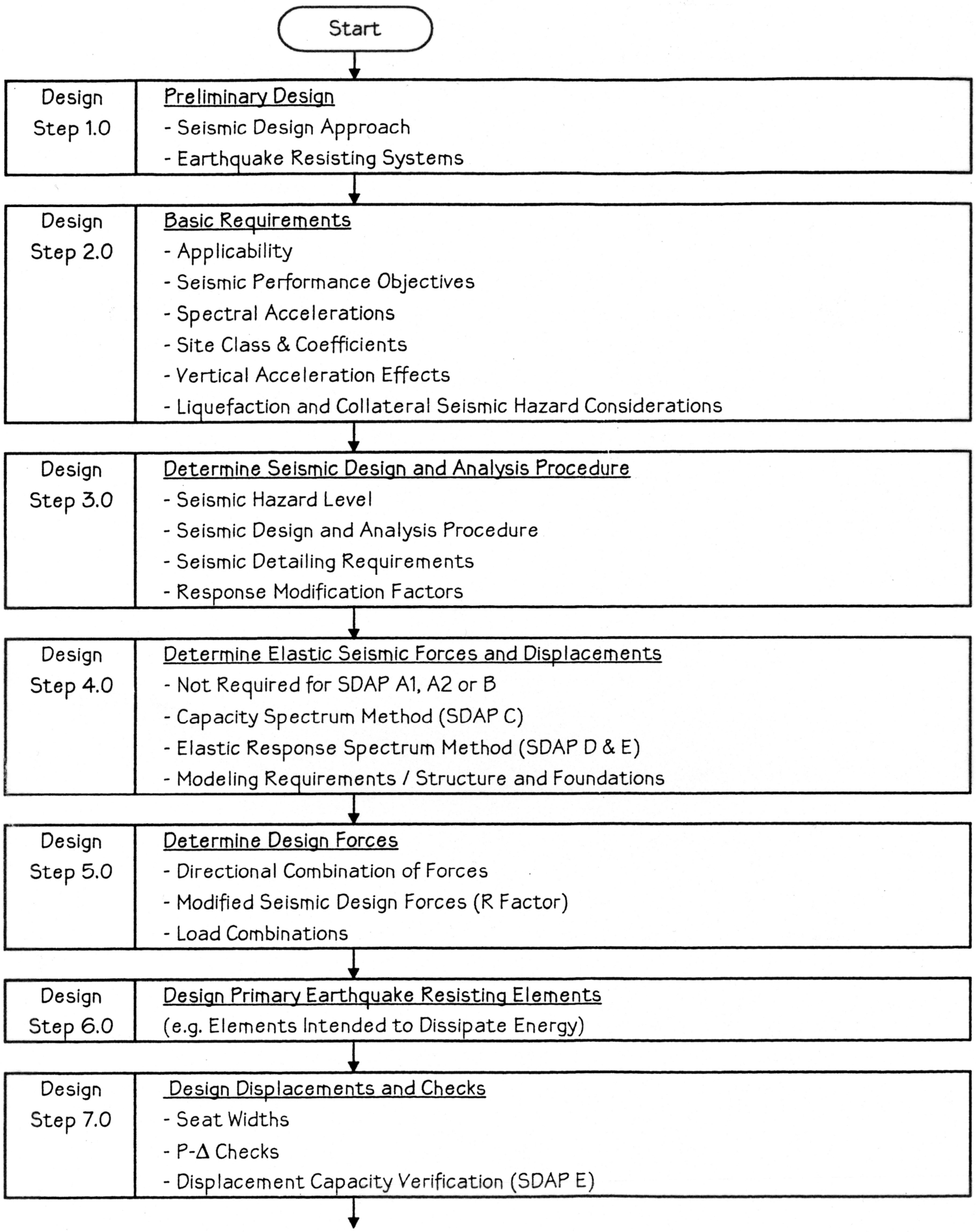
An example is shown below.



Note that Mathcad® carries the full precision of the variables throughout the calculations, even though the listed result of a calculation is rounded off. Thus, hand-calculated checks made using intermediate rounded results may not yield the same result as the number being checked.

Also, Mathcad® does not allow the superscript " ´ " to be used in a variable name. Therefore, the specified compressive strength of concrete is defined as $f`_c$ in this example (not $f´_c$).

SECTION II FLOWCHARTS



SECTION II FLOWCHARTS

Design Step 8.0	Design Structural Components - Seismic Detailing Requirements SDRs - Transverse Steel in Columns and Walls - Connections, Shear Keys, Joint Designs, Restrainers, Bearings - Superstructure Checks / Design Requirements - Capbeams, Diaphragms
Design Step 9.0	Design Foundations - Seismic Detailing Requirements SDRs - Footings, Piles, Shafts - Connections, Joints
Design Step 10.0	Design Abutments - Seismic Detailing Requirements SDRs - Shear Keys, Connections - Footings, Piles, Shafts
Design Step 11.0	Consideration of Liquefaction-Induced Flow or Spread - Evaluation of Foundation Displacement Demands / Capacities - Ground Improvement / Structural Improvement
Design Step 12.0	Seismic Design Complete ? - Revise As Necessary to Meet Criteria

SECTION III BRIDGE WITH TWO-COLUMN BENTS

SECTION III ANALYSIS AND DESIGN

DATA

The bridge is to be built in the western United States in the southern part of the Puget Sound region of Washington State. The site latitude is 47.0 degrees north, and the longitude is 122.9 degrees west.

Latitude and longitude now define the location for development of the earthquake acceleration data. Two earthquake loadings will be considered in the design, one for a rare event, called the maximum considered earthquake (MCE), and one for a frequent or expected event. The rare event has a 3 percent chance of exceedence in 75 years, and the frequent event has a 50 percent chance of exceedence in 75 years. Seventy-five years is the nominal “design life” of a bridge as defined by the LRFD Specifications.

The five-span bridge is 500 feet long with five equivalent spans of 100 feet. All substructure elements are square to a line perpendicular to a straight bridge centerline alignment. Figure 1a shows a plan and elevation of the bridge. The superstructure is a cast-in-place concrete box girder with two interior webs. The intermediate bents have a cross beam integral with the box girder and two round columns that are integral with the cap on the pile combined foundations. Figure 1b shows a cross section through the bridge with an elevation of an intermediate bent. The stub-type abutments with overhanging superstructure diaphragm are on pile foundations, as shown in Figure 1c; and the intermediate bents are all cast-in-place concrete. The pile foundations at all piers are 24-inch-diameter, cast-in-place concrete piles with steel casings. Framing of the box girder superstructure is shown in Figure 1d.

The subsurface conditions consist of 10 feet of soft clay overlying approximately 90 of loose to medium dense alluvial sands, with a thin clay layer at about 50 feet of depth. These subsurface conditions are uniform across the site. The site has several liquefiable layers, one between –10 and –20 feet and the other from –45 to about –55 feet. Appendix A contains key geotechnical information for the site.

The focus of this design example is not how to design for liquefiable conditions. Therefore, the design information contained in this example focuses entirely on the design of the structure for the nonliquefied conditions for both the MCE and frequent earthquake events. Extensive discussion of the design for liquefaction and the associated site-specific geotechnical engineering is contained in the “Liquefaction Study Report,”

SECTION III BRIDGE WITH TWO-COLUMN BENTS

NCHRP(b) 2001, developed as part of the NCHRP 12-49 project. The reader is referred to that volume for specific information regarding liquefaction.

REQUIRED

Design the bridge for seismic loading, exclusive of liquefaction, using the *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges*, MCEER/ATC 49 (2003).

FEATURES

ISSUES EMPHASIZED FOR THIS EXAMPLE

Proposed LRFD Seismic Guide Specification, including

- Basic Application of the Provisions
- Foundation Springs for Pile Foundations
- Two-Column Bent Behavior
- Displacement Capacity Verification – Push-Over Analysis
- Consideration of Passive Abutment Soil Resistance

SECTION III BRIDGE WITH TWO-COLUMN BENTS

BRIDGE DATA
(continued)

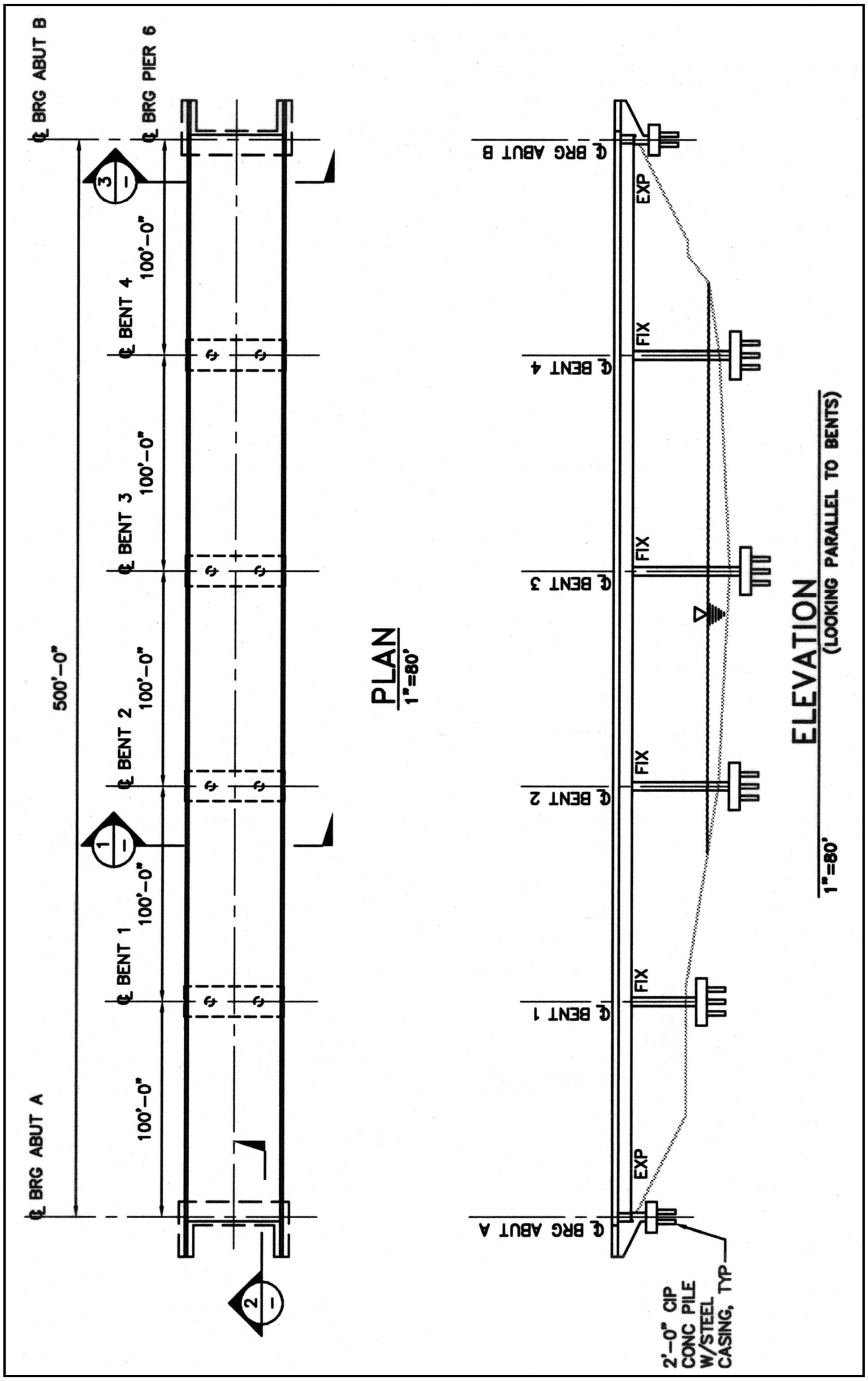


Figure 1a — Bridge No. 8 - Plan and Elevation

SECTION III BRIDGE WITH TWO-COLUMN BENTS

BRIDGE DATA
(continued)

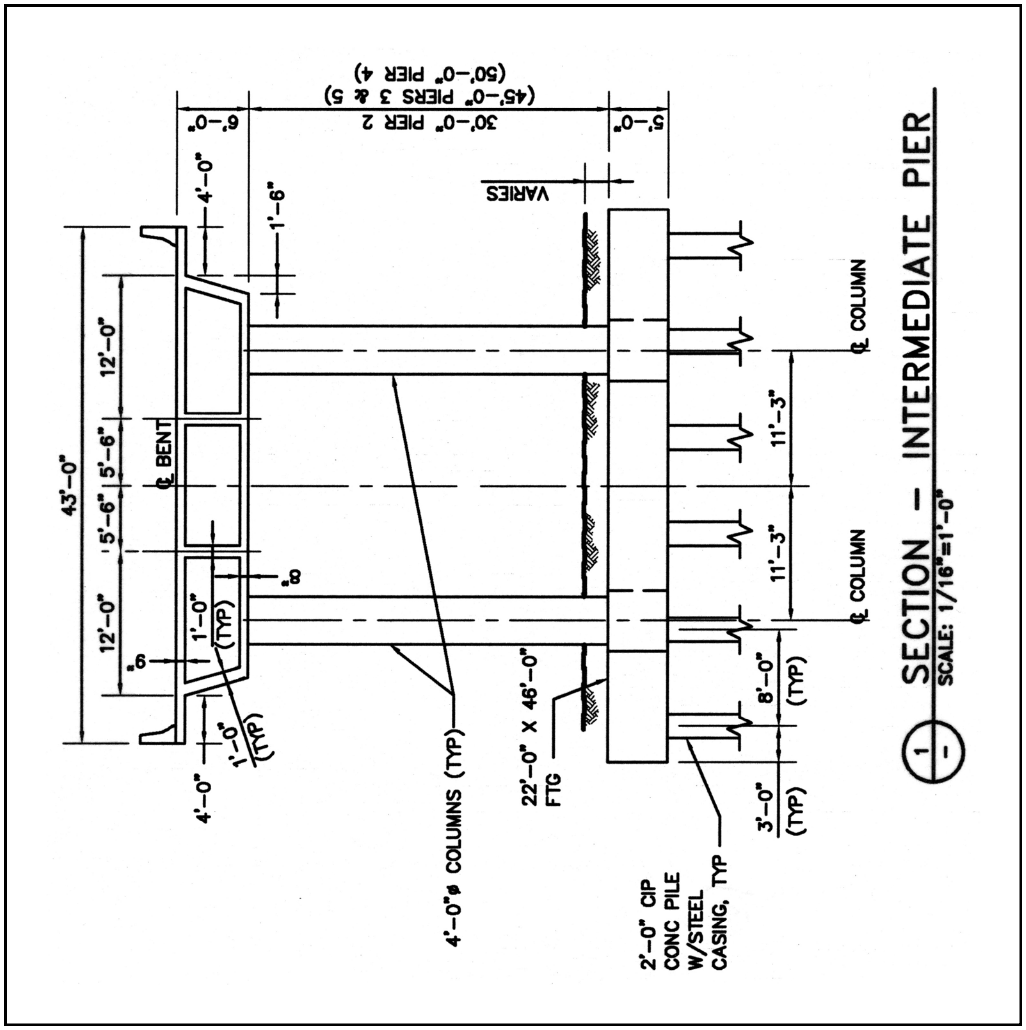


Figure 1b — Bridge No. 8 - Typical Cross Section

SECTION III BRIDGE WITH TWO-COLUMN BENTS

BRIDGE DATA
(continued)

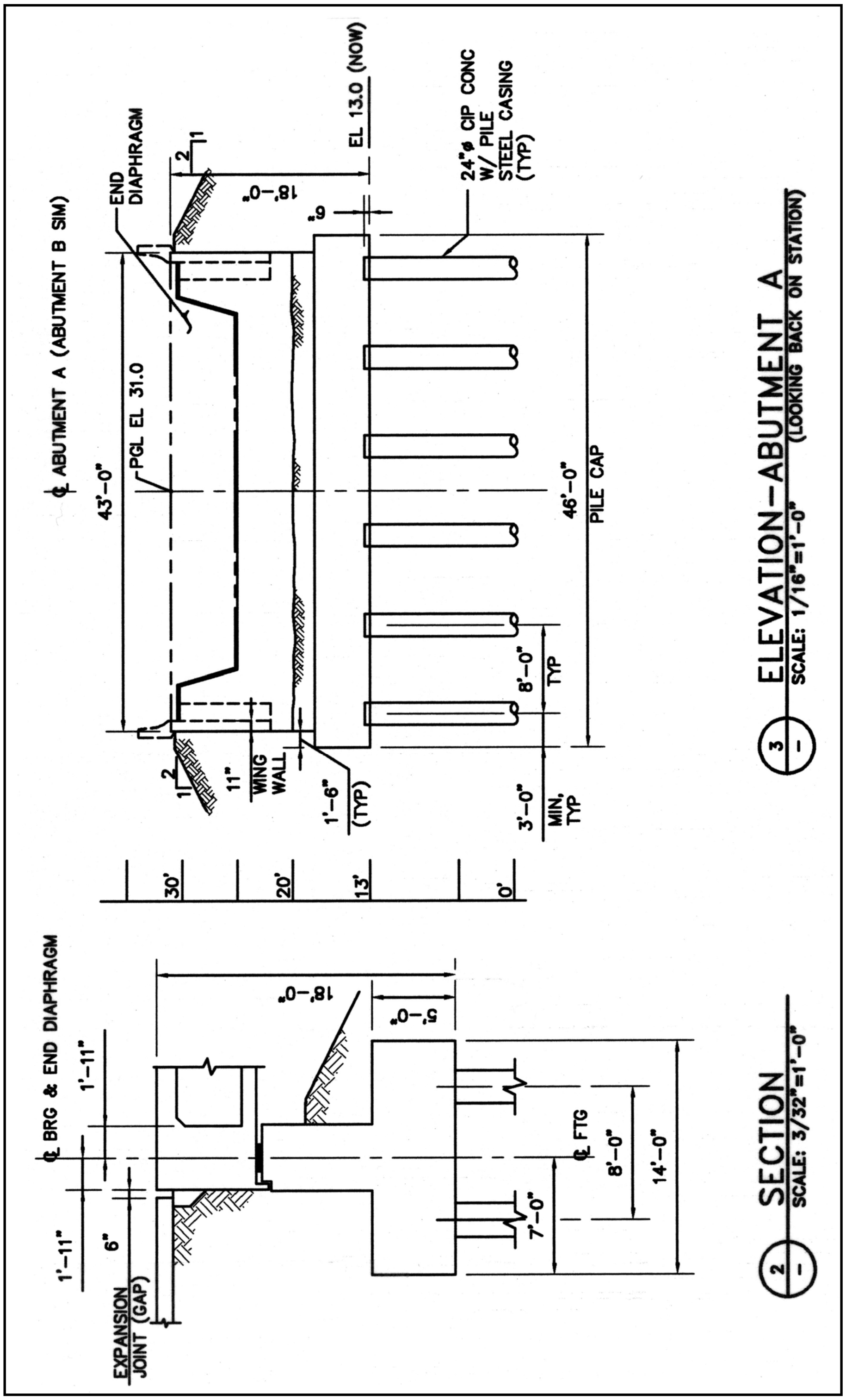


Figure 1c — Bridge No. 8 - Stub-Type Abutment

SECTION III BRIDGE WITH TWO-COLUMN BENTS

BRIDGE DATA
(continued)

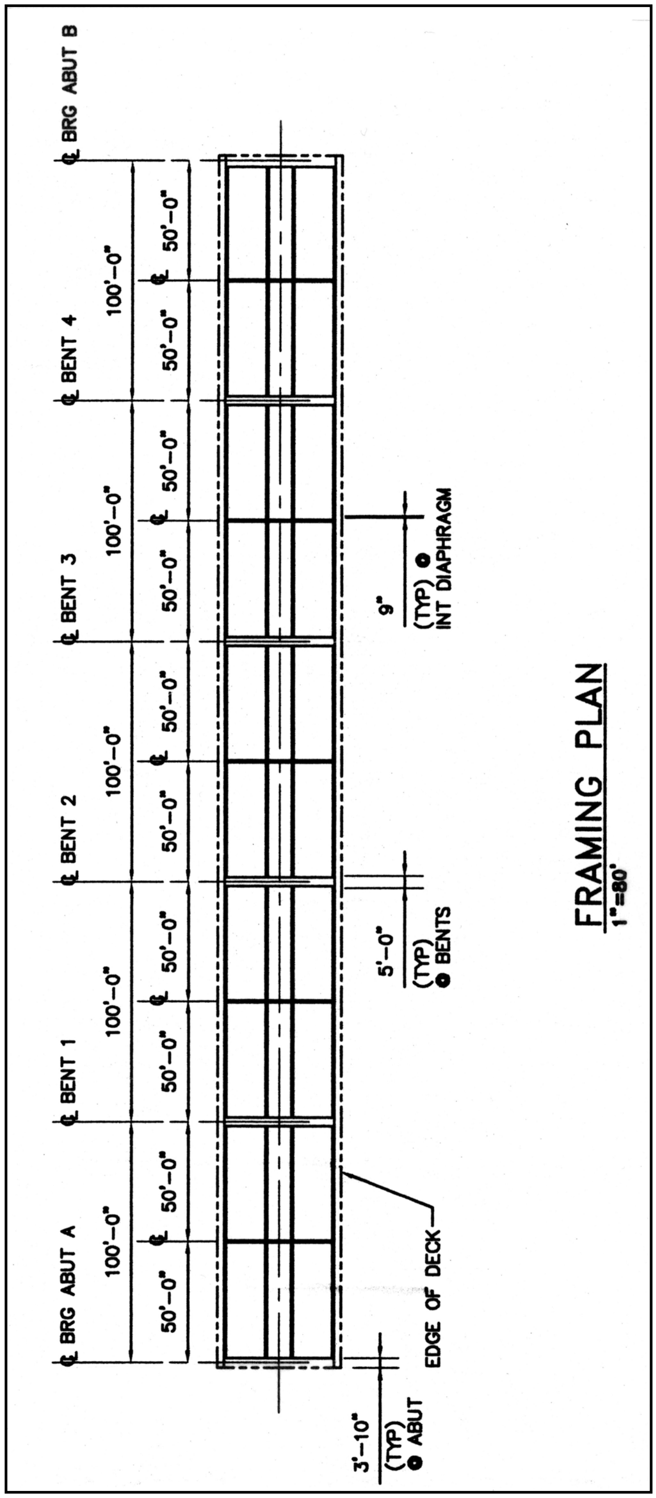


Figure 1d — Bridge No. 8 - Box Girder Framing Plan

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 1, Preliminary Design

SOLUTION

DESIGN STEP 1 **PRELIMINARY DESIGN**

A static load design (live and dead loads) and a preliminary seismic design of the bridge have been completed. The initial configuration of the superstructure and preliminary sizes of substructure elements are as shown in Figure 1 (a to d).

Design Step 1.1 **Seismic Design Objectives**
[Guide Spec, Article 3.3] [NCHRP, Article 2.5.6]

Section 3.3 of the LRFD Guide Specification requires that a "clearly identifiable earthquake resisting system (ERS)" be selected to achieve the appropriate performance objectives defined in Table 3.2-1.

In this example, the ERS includes conventional inelastic action (plastic hinging) in the columns and reliance upon the abutment backfill to "passively" resist longitudinal forces.

The overall concepts for the lateral force resistance of this bridge are described below.

The initial iterative process of preliminary design, which resulted in the sizes of the bent columns and footings, is not shown in this example. However, the assumed seismic behavior of the structure used for preliminary design is described below.

For preliminary design, the bases of the bent columns are considered fixed by the pile caps in both the transverse and longitudinal directions. The moments of inertia of the structural elements are using effective properties (i.e., cracked cross section properties).

In the longitudinal direction, the intermediate bent columns, in addition to the abutment backfill, resist the longitudinal seismic force. The abutments with the overhanging end diaphragm are in direct contact with the backfill and thus the soil is effective in resisting longitudinal force in any magnitude displacement. This behavior is illustrated in Figure 2.

In the transverse direction, the superstructure acts as a combination of a simply supported beam spanning laterally between the abutments and individual piers resisting tributary load. The maximum transverse displacement will occur somewhere near the center, depending on the

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 1, Preliminary Design

DESIGN STEP 1 (continued)

relative stiffnesses of the intermediate piers. This behavior is illustrated in Figure 3. The intermediate bents and the abutments are assumed to participate in resisting the transverse seismic force along with the superstructure. There is no skew effect because the piers are perpendicular to the bridge centerline.

At the abutments, transverse restraint will be provided by a girder stop or shear key to enable transfer of superstructure transverse seismic forces to the abutment. Transverse shear in the abutment, itself, is transferred to the soil via two rows of piling as shown in the figures above.

All the foundations are supported by piling, which primarily acts as friction piling. At the intermediate piers, the resistance to lateral loads is comprised of a combination of piling lateral resistance and passive resistance of the soil adjacent to the foundations and seals. At the abutments, in the longitudinal direction, passive soil resistance is counted upon for lateral resistance, as has been discussed above. However, in the transverse direction, passive resistance of the abutment foundation acting against the soil is not counted upon due to the proximity of the foundation to the edge of the fill slope. Thus, two distinct behaviors have been used at the intermediate piers and abutments.

Design Step 1.2

Earthquake Resisting Systems
[Guide Spec, Article 3.3.1] [NCHRP, Article 2.5.6.1]

Section 3.3.1 of the LRFD Guide Specification introduces the concept of Earthquake Resisting Systems (ERS) and Earthquake Resisting Elements (ERE). This concept is new and it organizes commonly occurring systems and elements into three categories: 1) Permissible, 2) Permissible with Owner's Approval, and 3) Not Recommended for New Bridges. Examples of common systems and elements are included in Figures C3.3.1-1a and 1b, C3.3.1-2, and C3.3.1-3 of the commentary to the provisions. The provisions encourage the designer to decide, early in the design process, what ERS and EREs will be used, and they encourage designers to use Permissible systems.

The Permissible with Owner's Approval category covers situations that either require special consideration by the owner or are generally not desirable, but often cannot be avoided. An example of the former is using the full capacity of the backfill behind an abutment to resist the longitudinal movement of the superstructure. This requires consideration and specification of backfill material the extends beyond that typical of most bridges. The latter includes in-ground hinging that cannot be

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 1, Preliminary Design

DESIGN STEP 1
(continued)

avoided. This means that inelastic demands may occur in the foundations during a major earthquake, and these may not be inspectable.

In this example, the bridge is classified as "Permissible with Owner's Approval" because the full prescriptive passive capacity of the soil backfill behind the abutments has been counted on to resist longitudinal lateral forces. Otherwise the EREs used in this bridge are all in the "Permissible" category.

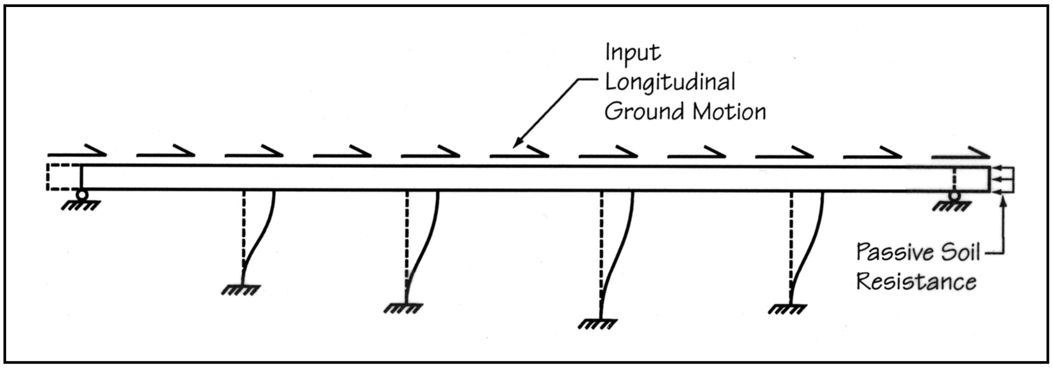


Figure 2 — Longitudinal Seismic Behavior

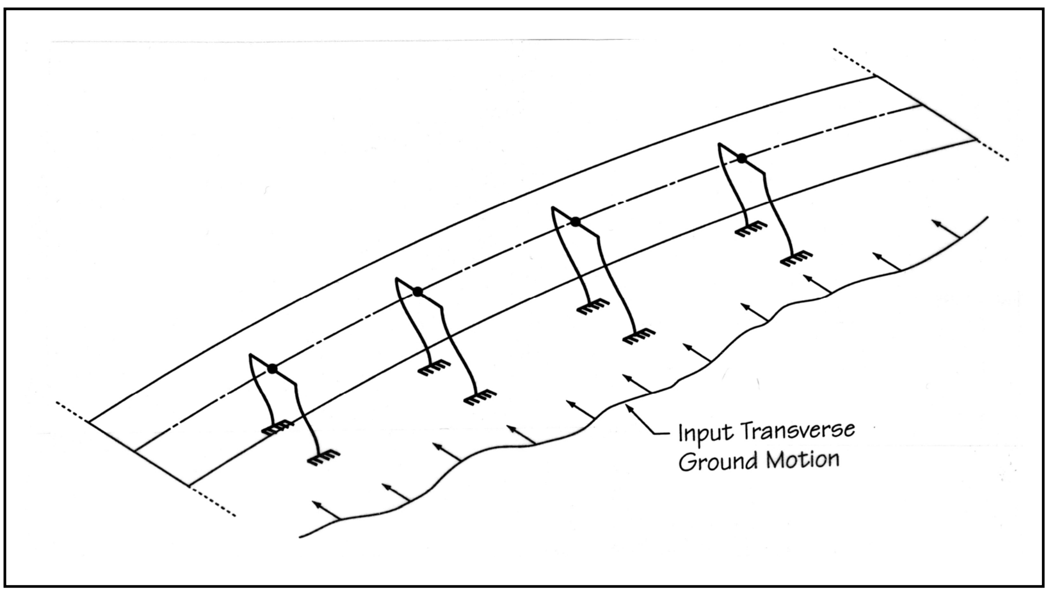


Figure 3 — Transverse Seismic Behavior

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 2, Basic Requirements

DESIGN STEP 2 BASIC REQUIREMENTS

Design Step 2.1 Applicability of Specification

[Guide Spec, Article 3.1] [NCHRP, Article 3.10.1.1]

The bridge has five spans that total 500 feet and is a cast-in-place concrete box girder with a reinforced concrete substructure. Thus, because this bridge is conventional and regular, the specification applies.

The potential for soil liquefaction and slope movements are considered in a separate report.

Design Step 2.2 Seismic Performance Objectives

[Guide Spec, Article 3.2] [NCHRP, Article 3.10.1.2]

For this example, the selected performance level is "Life Safety," the minimum required for all bridges. This is the case for both the rare and the frequent earthquake.

Table 3.2-1 defines the performance levels for service and damage the bridge is to be designed for. In this case, the choice of Life Safety as the performance level implies that for the frequent earthquake minimal damage is expected and the structure is expected to fully open to normal traffic following an inspection of the bridge. The Life Safety choice also implies that in the rare earthquake significant damage is expected, and the bridge will likely not be available to full traffic following an earthquake. The bridge may, in fact, be damaged to the point where it needs to be replaced following the rare event. Displacement limits are established by the provisions to guide the designer in assessing geometrically what is implied by the specified service levels. Per the Proposed LRFD Specification, displacements should be checked "to satisfy geometric, structural, and foundation constraints on performance" as outlined in Table C3.2-1 of the Specification.

Design Step 2.3 Spectral Acceleration Parameters

[Guide Spec, Article 3.4.1] [NCHRP, Article 3.10.2.1]

The site in this example is located at latitude 47.0 and longitude -122.9, which is near Olympia, Washington. Using these coordinates, the national ground motion maps for the MCE designate the interpolated, short-period (0.2 second) acceleration, S_S, as 1.175g and the 1.0-second acceleration, S_1, as 0.411g.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 2, Basic Requirements

DESIGN STEP 2
(continued)

The spectral accelerations for the frequent earthquake were determined by the geotechnical engineer, and likewise are based on national ground motion maps. The short-period (0.2 second) acceleration, S_S, is 0.261g and the 1.0-second acceleration, S_1, as 0.081g.

New to the provisions is the concept of using spectral accelerations taken directly from maps. This differs from the existing I-A provisions in that a peak ground acceleration, PGA, or acceleration coefficient, A, is never used. The national maps include both a short- and a long-period spectral acceleration, and these two quantities are then used to construct a full-design spectrum. This approach is the one developed by the NEHRP efforts in the late 1990s and is that which most of the building codes are now using. The user is cautioned that the mapped accelerations, particularly the short-period accelerations, appear to be much larger than the PGA values that one is used to seeing. This is in part due to the longer return period that is being used in these provisions, but also to an even greater extent due to the fact that the accelerations are spectral accelerations. Thus the accelerations represent the accelerations of the structure, which are amplified above those of the ground.

These accelerations values are based on the horizontal component of ground motion for rock, specifically site Class B. They need to be modified for the site class determined for the example site if it is different than B.

Figures 3.4.1-1(a) and 1(b) show the spectral response contours, shown in percent of gravitational acceleration, for the MCE developed by U.S. Geological Survey (USGS). A CD-ROM is available from USGS (Frankel and Leyendecker, 2000) that contains large-scale ground motion maps for the United States and will provide interpolated accelerations given specific latitude/longitude coordinates or Zip Code.

For this example, the prescribed spectral acceleration parameters will be used to develop the response spectra using the general procedure. A site-specific response spectra is not required by the provisions for this site per the conditions stated in Article 3.4. A site investigation by a qualified geotechnical engineer or seismic hazard assessment specialist may be used to develop more accurate acceleration data. Such an investigation is required if Site Class F soils are present at the site and they have a significant effect on the bridge response, the bridge is considered to be a major or very important structure, or if the site is within 10 km of an active fault.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 2, Basic Requirements

Design Step 2.4

Site Class
[Guide Spec, Article 3.4.2.1] [NCHRP, Article 3.10.2.2.1]

The site class for the Olympia, Washington, site is E based on the shear wave velocity, V_s, which was provided by the geotechnical engineer. The soil profile, including properties for the nonliquefied condition, was also generated by the geotechnical engineer and is shown in Figure 4. The site has a 10-foot-deep clay layer and the upper 100 feet (approximately 30 meters) have an average shear velocity of 600 ft/s.

A single soil profile is being used to represent the soil conditions at this site. However, the soil conditions at a site will generally be characterized by several soil profiles, as many as one for each pier location. For demonstration purposes, the soil conditions have been simplified. Also, in an actual design study, all locations would normally have to be considered in the liquefaction assessment.

The site class can be established by either using shear wave velocity data, standard penetration test (SPT) data, or undrained shear strength data. The class depends on a weighted average for the upper 30 meters (roughly 100 feet) of the site.

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 2, Basic Requirements

SITE DATA

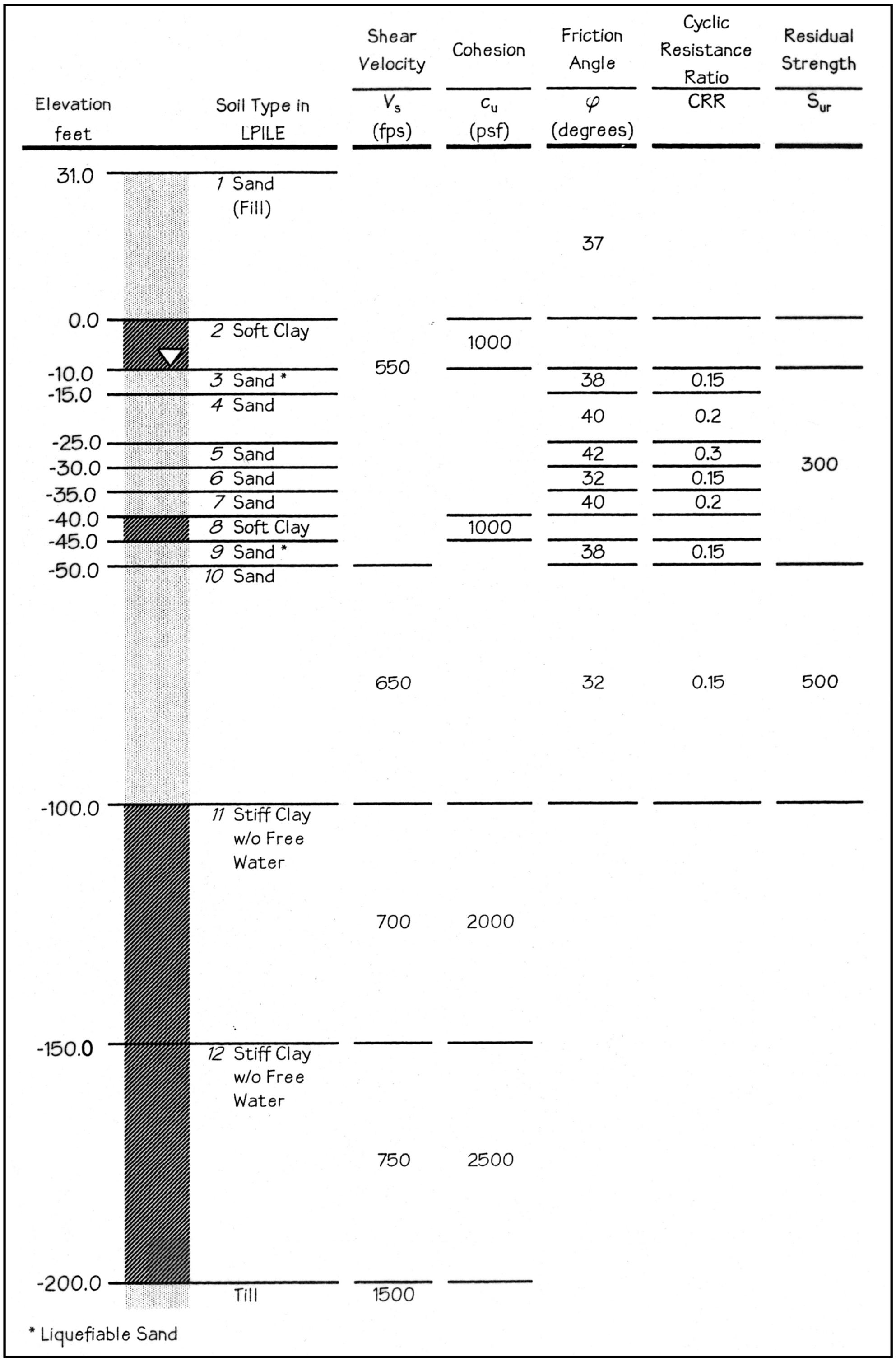


Figure 4 — Soil Profile

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 2, Basic Requirements

Design Step 2.4 (continued)

At the site, the material at depths less than 150 feet are generally alluvial deposits. At greater depths, some estuarine materials exist; and below about 200 feet, dense glacial materials are found. This then produces a site with the potential for deep liquefiable soils.

Figure 4 also includes relevant properties of the soil layers that have been used for the seismic response assessments and bridge design. Shear wave velocity (Vs), undrained shearing strength (c_u), soil friction angle (f), and residual soil strength (S_{ur}) were interpreted from the field and laboratory data. The cyclic resistance ratio (CRR) was obtained by conducting simplified liquefaction analyses using both the SPT and CPT methods to obtain CRR values. For a complete analysis of the geotechnical aspect covered by the proposed provisions, refer to the *Liquefaction Study Report* prepared as a part of the NCHRP 12-49 project.

Design Step 2.5

Site Coefficients
[Guide Spec, Article 3.4.2.3] [NCHRP, Article 3.10.2.2.3]

Maximum Considered Earthquake (3% in 75 years)

The site coefficient for the short-period range, F_a, is 0.9 the 0.2-second spectral acceleration, S_S = 1.175g, and site Class E.

The site coefficient for the long-period range, F_v, is 2.4 the 1.0-second spectral acceleration, S_1 = 0.411g, and site Class E.

Frequent Earthquake (50% in 75 years)

The interpolated site coefficient for the short-period range, F_a, is 2.46 for the 0.2-second spectral acceleration, S_S = 0.261g, and site Class E.

The site coefficient for the long-period range, F_v, is 3.5 for the 1.0-second spectral acceleration, S_1 = 0.081g, and site Class E.

Note that the site coefficients determined from Table 3.4.2.3-1 and Table 3.4.2.3-2 shall be linearly interpolated for intermediate values of S_S and S_1.

A geotechnical investigation may be made by qualified professionals to establish site-specific seismic response information (e.g., site-specific response spectra). This investigation is typically done on a site-by-site basis. In some cases, State Departments of Transportation (DOTs) may develop representative spectra for soil types and seismic hazards in their

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 2, Basic Requirements

jurisdictions. These spectra might then be used in lieu of the information in Article 3.4. Lacking such specific information, the structural engineer should decide whether to have site-specific information generated or use the approach described in this section. In most cases, a site-specific study would not be required.

Design Step 2.6 **Design Earthquake Response Spectra**
[Guide Spec, Article 3.4.1] [NCHRP, Article 3.10.2.1]

Figure 3.4.1-1 illustrates the computed values needed to define the design response spectrum, which will be computed for both the MCE and the frequent earthquake. Also, because the site in this design example has liquefiable layers, the liquefied condition of the soils must be considered. Therefore, different foundation springs and response spectra are developed for the nonliquefied and the liquefied conditions when designing for the MCE. The full spectra is used for the nonliquefied case, and a reduced response spectra may be used when liquefied conditions are considered in the dynamic model, as described below.

A two-thirds reduced spectra is allowed when a site-specific analysis indicates that the ground motions may be reduced by at least that amount. If the site specific analysis does not support a one-third reduction, then the site-specific value or the full spectral value shall be used. The reduction may be considered to be a function of period. For instance, the one-third reduction may govern for short periods and the site-specific value may govern at longer periods. A reduction greater than one-third is not allowed for conservatism.

In this case, a site-specific study was conducted for liquefaction; therefore, there are three complete spectra to be developed.

Design Response Spectrum Development - MCE/Nonliquefied

$S_s := 1.175$

$S_1 := 0.411$

$F_a := 0.9$

$F_v := 2.4$

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 2, Basic Requirements

Design Step 2.6
(continued)

$S_{DS} := F_a \cdot S_s \qquad S_{DS} = 1.058$

$S_{D1} := F_v \cdot S_1 \qquad S_{D1} = 0.986$

$0.40 \cdot S_{DS} = 0.423$

$$T_s := \frac{S_{D1}}{S_{DS}} \cdot sec$$

$T_s = 0.933\ s$

$T_o := 0.2 \cdot T_s$

$T_o = 0.187\ s$

Construct spectrum:

$T1 := 0, 0.001 \cdot s .. T_o$

$$S_{a1}(T1) := \left(\frac{0.6 \cdot S_{DS}}{T_o}\right) \cdot T1 + 0.40 \cdot S_{DS}$$ (Eqn 3.4.1-3)

$T2 := T_o, T_o + 0.001 \cdot sec .. T_s$

$S_{a2}(T2) := S_{DS}$ (Eqn 3.4.1-4)

$T3 := T_s, T_s + 0.001 \cdot sec .. 3 \cdot s$

$$S_{a3}(T3) := \frac{S_{D1}}{T3}$$ (Eqn 3.4.1-5)

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 2, Basic Requirements

Design Step 2.6
(continued)

Design Response Spectrum Development - Frequent EQ

$S_s := 0.261$

$S_1 := 0.081$

$F_a := 2.46$

$F_v := 3.5$

$S_{DS} := F_a \cdot S_s$ $\quad S_{DS} = 0.642$

$S_{D1} := F_v \cdot S_1$ $\quad S_{D1} = 0.284$

$0.40 \cdot S_{DS} = 0.257$

$T_s := \dfrac{S_{D1}}{S_{DS}} \cdot sec$ $\quad T_s = 0.442\ s$

$T_o := 0.2 \cdot T_s$ $\quad T_o = 0.088\ s$

Construct spectrum:

$T1F := 0, 0.001 \cdot s .. T_o$

$$S_{a1F}(T1F) := \left(\frac{0.6 \cdot S_{DS}}{T_o}\right) \cdot T1F + 0.40 \cdot S_{DS} \qquad \text{(Eqn 3.4.1-3)}$$

$T2F := T_o, T_o + 0.001 \cdot sec .. T_s$

$$S_{a2F}(T2F) := S_{DS} \qquad \text{(Eqn 3.4.1-4)}$$

$T3F := T_s, T_s + 0.001 \cdot sec .. 3 \cdot s$

$$S_{a3F}(T3F) := \frac{S_{D1}}{T3F} \qquad \text{(Eqn 3.4.1-5)}$$

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 2, Basic Requirements

Design Step 2.6 (continued)

Design Earthquake Response Spectra
[Guide Spec, Article 3.4.1] [NCHRP, Article 3.10.2.1]

The constructed design response spectra for the MCE, nonliquefied and liquefied soil cases, and the Frequent earthquake are shown in Figure 5. The period, T, and spectral acceleration, S, values from these curves will be entered into the SAP2000 model. Thus, the dynamic analysis will be run three times, once for each of the design response spectra.

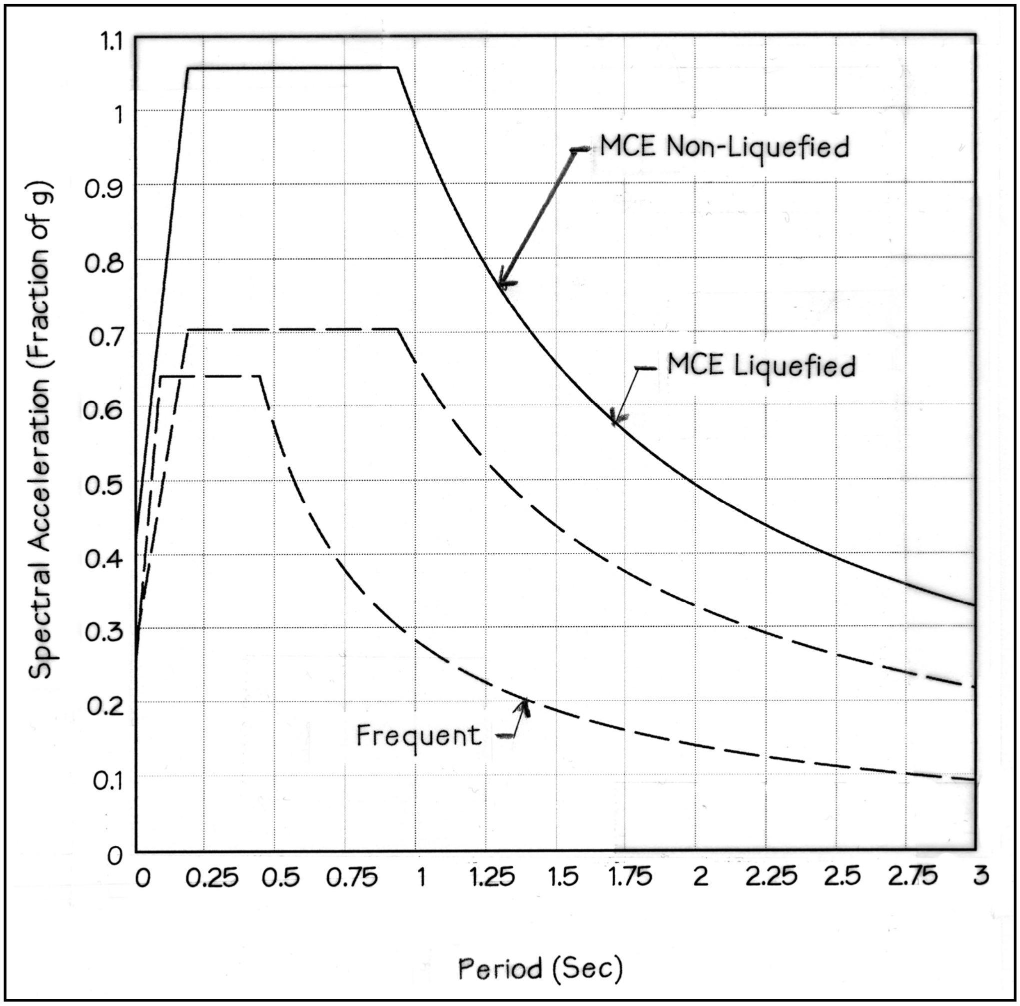


Figure 5 — Design Response Spectra

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 2, Basic Requirements

Design Step 2.7

Vertical Acceleration Effects
[Guide Spec, Article 3.4.5] [NCHRP, Article 3.10.2.6]

The bridge site is effectively more than 50 km from an active fault. Therefore, vertical acceleration effects are not required to be considered in the design.

The provisions covering vertical effects were developed based on strike-slip-type faulting typical of California. They were not developed to consider deep faulting such as that present in subduction zones. Thus the near-fault provisions only apply for those situations where surface or near-surface faulting occurs. Because no surface faults are within 50 km of the example bridge, no account of vertical effects is required.

Design Step 2.8

Liquefaction and Collateral Seismic Hazard Considerations
[Guide Spec, Article 8.6 and Appendix D]
[NCHRP, Article 3.10.4 and Appendix 3B]

Collateral seismic hazards, such as liquefaction, lateral spreading, landslides, fault rupture, or other earthquake-induced ground movement phenomena, shall be investigated for the higher seismic categories, for instance SDAP D and E (note that these are defined in the next section).

As indicated above, liquefaction effects are considered in a separate document, Liquefaction Study Report, and are not discussed further in this example.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 3, Determine Seismic Design and Analysis Procedure

DESIGN STEP 3 DETERMINE SEISMIC DESIGN AND ANALYSIS PROCEDURE

Design Step 3.1 Determine Seismic Hazard Level

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3.1]

$F_aS_s = 1.06$ and $F_vS_1 = 0.99$.

The Seismic Hazard Level is IV.

By Table 3.7-1, the Seismic Hazard Level is IV because F_vS_1 exceeds 0.4, and the Seismic Hazard Level is IV because F_aS_s exceeds 0.6. The controlling value is the more restrictive of the two values. In this case, both spectral accelerations lead to Level IV.

The short- and long-period design spectral accelerations are given in the previous design step. It will be seen later that the fundamental period of the structure is greater than 1.0 second; thus, according to the commentary, the F_vS_1 definition of the seismic hazard level is more appropriate. However, either definition gives the same seismic hazard level in this case.

Design Step 3.2 Determine Seismic Design and Analysis Procedure (SDAP)

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3.1]

The required SDAP is E.

Table 3.7-2 of the Specification gives the requirements for determining what Seismic Design and Analysis Procedure (SDAP) should be used. The table suggests either C, D, or E can be used for the Life-Safety performance level in Seismic Hazard Level IV. The table notes, however, further restrict which SDAP can be used for this structure. Although Notes 1 and 2 would allow SDAP C or D to be used based on the performance criteria and regularity of the structure, Note 3 and especially Note 4 restrict the SDAP to procedure E, the elastic response spectrum method with displacement capacity verification, because liquefaction potential at the site will likely cause inelastic deformations of the pile foundations. Furthermore, because the full passive resistance behind the abutment is relied upon, SDAP D or E is required per Figure 3.3.1-2 of the proposed provisions. Finally, if the largest of the available response modification factors, R, is used, then the displacement verification calculation that is part of SDAP E must be used. Therefore for this bridge, the minimum SDAP allowed is E.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 3, Determine Seismic Design and Analysis Procedure

Design Step 3.3 **Determine Seismic Detailing Requirements (SDR)**
[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3.1]

SDR 4 is applicable to this structure.

Because the structure is classified for Life-Safety Performance and Seismic Hazard Level IV, Table 3.7-2 requires SDR 4. The detailing provisions for various components of the structure will be discussed in more detail in future design steps in this design example.

In these new provisions, the single design categories that cover both analysis methods and detailing have been eliminated in favor of categories for both analysis and detailing. This was done because a variety of analysis procedures may be used even for the higher seismic hazard levels. It was felt that the detailing should be essentially the same at these higher hazard levels while the analysis procedure could vary widely primarily based on regularity and simplicity of the bridge.

Design Step 3.4 **Determine Response Modification Factors**
[Guide Spec, Article 4.7] [NCHRP, Article 3.10.3.7]

The structural details must satisfy the provisions of Article 8.8 if the response modification factors are applied. These provisions will be satisfied once the components are designed for seismic load combinations.

In this case, Table 4.7-1 of the Specification gives the following R_B factors for the substructure.

<u>MCE</u>

$R_B = 6$ For the columns (Multiple-column bents are used. The SDAP is E, and the performance objective is Life Safety.)

$R = 0.8$ For the superstructure to abutment connection (bearings and girder stops), the connection of the bent columns to the cap beam or superstructure, and the connection of the bent columns to the foundations. This factor only applies if elastic design forces are used in lieu of capacity design of the connections.

<u>Frequent</u>

$R_B = 1.3$ For all elements

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 3, Determine Seismic Design and Analysis Procedure

Design Step 3.4
(continued)

These factors will be used to ensure that inelastic effects are restricted to elements that can be designed to provide reliable, ductile response, that can be inspected after an earthquake to assess damage, and that can be repaired relatively easily. The foundations and column connections do not fit this constraint, and thus will be designed not to experience inelastic effects. For bridges classified as SDAP D or E, it is recommended that the connections of the bent columns to the superstructure and foundation be designed for the maximum forces capable of being developed by plastic hinging of the bent column. These forces will often be significantly less than those obtained using an R_B factor of 1. If the inelastic (plastic) hinge forces govern, that is, are less than the elastic forces, then an overstrength factor must be applied to the column strength for design of the foundations.

This approach is known as capacity design whereby a distinct plastic mechanism is postulated and then the structure is designed to ensure that only that mechanism occurs. Structural elements that are not intended to yield are designed to accommodate the forces attendant with the formation of the plastic mechanism. In fact, overstrength factors are applied to the yield forces such that the structure is then capable of withstanding forces that are somewhat greater than the yield forces, alone. Article 4.8 discusses the topic of capacity design in more detail.

For the frequent earthquake, essentially elastic response is required of the structure to ensure that little or no damage occurs. If the performance objective is Life Safety, then an R_B of 1.3 is allowed. This permits the seismic demands to push the structure just beyond the point of first yield, although no significant damage would be expected. If the performance objective is operational, then the R_B is 0.9. This value ensures that no damage occurs, and the structure stays within its basic yield limit.

New in these provisions is a modifier that accounts for the observation that the inelastic demands in a short-period structure are larger than predicted by the assumption of equal displacements between the elastic and yielding structures. Therefore, the base response modification factor, R_B, is adjusted for this phenomenon. Because the adjustment is period dependent, this modification is discussed in Design Step 5 after the fundamental periods of the structure are determined.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

DESIGN STEP 4 DETERMINE ELASTIC SEISMIC FORCES AND DISPLACEMENTS

Design Step 4.1 Seismic Analysis

[Guide Spec, Section 5] [NCHRP, Article 4.8]

Design Step 4.1.1 General

[Guide Spec, Article 5.1.1] [NCHRP, Article 4.8.1]

As discussed in the previous design step, the Seismic Design and Analysis Procedure, SDAP, that is to be used is E. This means that an elastic multimode response spectrum analysis must be executed, and the design of the structure must be assessed using the Displacement Capacity Verification (pushover) procedure. Thus the modal analysis will be used to obtain the forces with which to enter the design procedure, and it will be used to obtain target displacements for the pushover procedure.

Design Step 4.1.2 Seismic Lateral Load Distribution

[Guide Spec, Article 5.2] [NCHRP, Article 4.8.3]

The transverse and longitudinal behavior of this bridge under seismic loading was described in Design Step 1 and shown in Figures 2 and 3. The load paths of the structure are to be as follows.

1. Transverse Direction

   Inertial loads originating in the box girder superstructure are carried via flexure to the integral cap beams or integral end diaphragms. At the intermediate bents, the cap beams transfer the loads to the columns, which transfer the load to the pile caps, which then transfer load directly to the soil, to the seals and then soil, and to the piles and then soil. At the abutments, the end diaphragms transfer lateral load to the abutment shear keys, which transfer the load to the abutment stem wall. The load is then transferred directly to the soil and to the piles and then soil.

2. Longitudinal Direction

   Inertial loads originating in the box girder superstructure are carried through axial drag strut action primarily to the end diaphragm that is in compression against the soil. That diaphragm then transfers the forces to the soil, which provides passive resistance. Secondarily, loads are transferred to the cap beams via the upper and lower slabs and webs. The cap beams transfer the loads to the columns. The loads then make their way to the column foundations as described for the transverse direction.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

The provisions require that a clearly defined load path be identified and designed for seismic loading. The above is simply a description of the load paths for loading in the two principal directions. This vision of the load path then guides the designer in identifying the elements that require seismic design and proportioning them for the expected actions.

Design Step 4.2 **Description of Model**
[Guide Spec, Article 5.1] [NCHRP, Article 4.8]

Design Step 4.2.1 General
[Guide Spec, Article 5.1.1] [NCHRP, Article 4.8.1]

The structural analysis program SAP2000 Nonlinear Version 7.40 (CSI, 2000) was used for the analyses. The model used is shown in Figure 6, and includes a single line of frame elements, or spine elements, for the superstructure and individual elements for the cap beam, columns, pile cap, and cap seal of the intermediate bents. A copy of the SAP2000 input file for the analyses is provided in Appendix B.

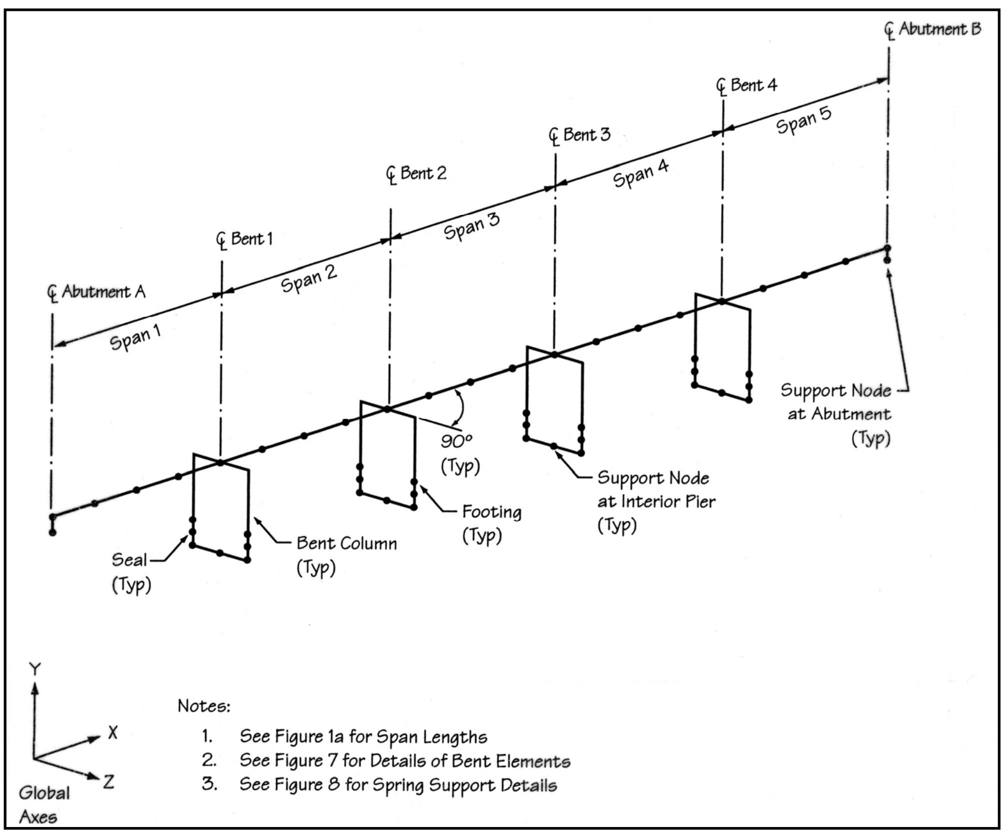


Figure 6 — Structural Model of Bridge

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.2.2

Superstructure
[Guide Spec, Article 5.3] [NCHRP, Article 4.8.4]

The superstructure has been modeled with four elements per span and the work lines of the elements are located along the centroid of the superstructure.

The properties of the elements used for the model are for the structure configuration shown in Design Step 1, Preliminary Design. The superstructure density used for the modal analysis has been adjusted to include additional dead loads from traffic barriers and wearing surface overlay. The total weight of these additional dead loads is 2.35 kips per lineal foot of superstructure. The properties of the structure used in the seismic model (both superstructure and substructure) are shown in Table 1.

Table 1
Section Properties for Model

	Model Element				
	CIP Box Superstructure	Bent Cap Beam	Bent Columns (Each Column)	Pile Caps	Seals (4)
Area (ft^2)	72.18	27.00	12.57	506.0	196.0
Ix – Torsion (ft^4)	1,177	10,000 (1)	10.0	109634	6403
Iy (ft^4)	9,697	10,000 (2)	5.0	20409	20409
Iz (ft^4)	401	10,000 (3)	5.0	89225	89225
Density (lb / ft^3)	180	150	150	150	140

(1) This value has been increased for force distribution to bent columns. Actual value is Ix = 139 ft^4.
(2) This value has been increased for force distribution to bent columns. Actual value is Iy = 90 ft^4.
(3) This value has been increased for force distribution to bent columns. Actual value is Iz = 63 ft^4.
(4) The seals have been included in the model to account for their stiffening effect on the piles. They may conservatively be ignored at the designer's discretion. The seal concrete typically will not be of the same quality as that of the cap.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.2.2

As shown in Figure 6, the superstructure has been collapsed into a single line of 3-D frame elements that follow the horizontal geometry of the bridge centerline. This "stick" model is used solely for the determination of seismic forces for this example. Such a model does not give exactly the same forces for other loadings (for instance, dead loads) because the weight of the superstructure is not distributed uniformly across the cap beam. However, because weight or mass is an important parameter in dynamic analysis, the total weight of the structure should be close to that obtained from an accurate dead load analysis or check.

Enough nodes must be used along the length of the superstructure to accurately characterize the response and forces bearing in mind that SAP2000 and most other programs lump mass at the nodes. For a bridge, such as this one, with uniform cross section and a straight alignment, nodes at the quarter points are sufficient. Determination of moments of inertia and torsional stiffness of the superstructure is based on uncracked cross-sectional properties.

The end diaphragm of the box girder is in contact with the soil behind because a stub-type abutment is used. Therefore, a foundation spring is used to model the passive resistance of the backfill that will carry a portion of the forces resulting from the longitudinal earthquake. Two important characteristics of this spring are 1) to use half the spring value at each end diaphragm, and 2) to determine whether the model response is yielding the soil. The development and iterative process associated with this spring value is discussed in Design Step 4.3, Foundation Stiffnesses.

Design Step 4.2.3

Substructure
[Guide Spec, Article 5.3] [NCHRP, Article 4.8.4]

The bents are modeled with 3-D frame elements that represent the cap beam, individual columns, pile cap, and cap seal. (There are no elements to model the abutments, only support nodes as shown in Figure 4). Figure 7 shows the relationship between the actual bent and the "stick" model of 3-D frame elements. A single element was used for each column between the top of pile cap and the soffit of the box girder superstructure. The connection of the column top at the soffit of the box girder to the center of gravity of the cap (at the superstructure centroid) beam is made with rigid link elements. Foundation springs representing the piles are connected to the node (2xx) at the base of the seal. For this model, the moments of inertia properties of the columns are based on cracked sections. Although the torsional properties are based on uncracked sections, this value is typically based on cracked sections.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**

Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.2.3 (continued)

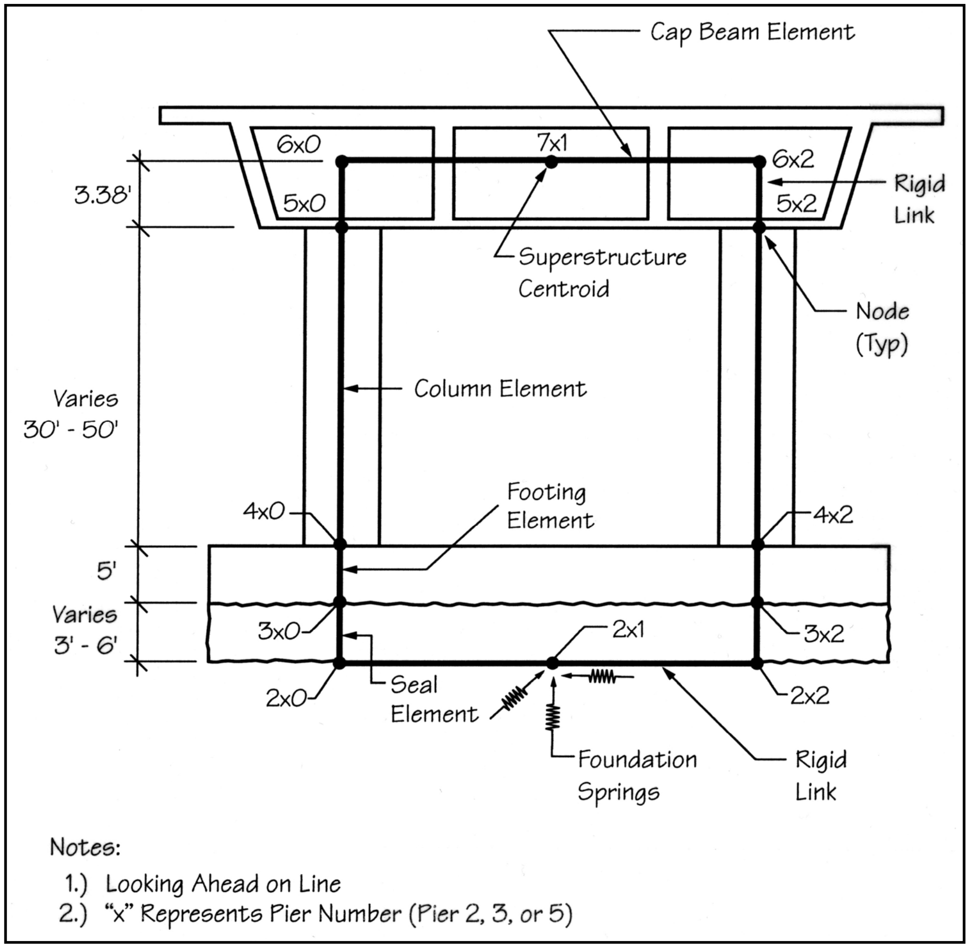


Figure 7 — Details of Bent Elements

In the actual structure, internal forces are transferred between the superstructure and the bent almost uniformly along the cap beam. In the seismic model, the superstructure forces are transferred at the single point where the superstructure and bent intersect. Therefore, in the seismic model the forces in the cap beam are not representative of actual forces, and the distribution of forces to the columns may not be accurate. For this example, the torsional stiffness and moments of inertia of the model's cap beams were increased in order to provide a more representative distribution of forces to the columns. These adjusted properties are shown in Table 1, along with the actual calculated properties. The determination of foundation spring stiffnesses to model the foundations is discussed in Design Step 4.3.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.2.3 (continued)

During seismic events, cracking along the height of the column will occur and will reduce the stiffness from the gross value to some effective stiffness value, resulting in larger displacements of the structure. Therefore in this example, cracked section properties have been used for the column elements. Values for effective bridge column moments of inertia related to axial load and reinforcing percentages have been developed by Priestley, Seible, and Calvi (1996); and FHWA, *Seismic Retrofitting Manual* (1995) recommends their use in evaluating structure displacements.

Design Step 4.3

Foundation Stiffnesses
[Guide Spec, Articles 5.3.4 and 8.4] [NCHRP, Articles 4.8.4.4 and 10.7]

Design Step 4.3.1

Bent Foundations

For SDAP E, the new provisions require that the foundation stiffness be included in the mathematical model. Thus, the intermediate bent foundations and the abutment foundations were modeled with equivalent spring stiffnesses for the pile foundations. Figure 8 shows details of the spring supports. Figure 9 shows the layout of the structure relative to the soil profile.

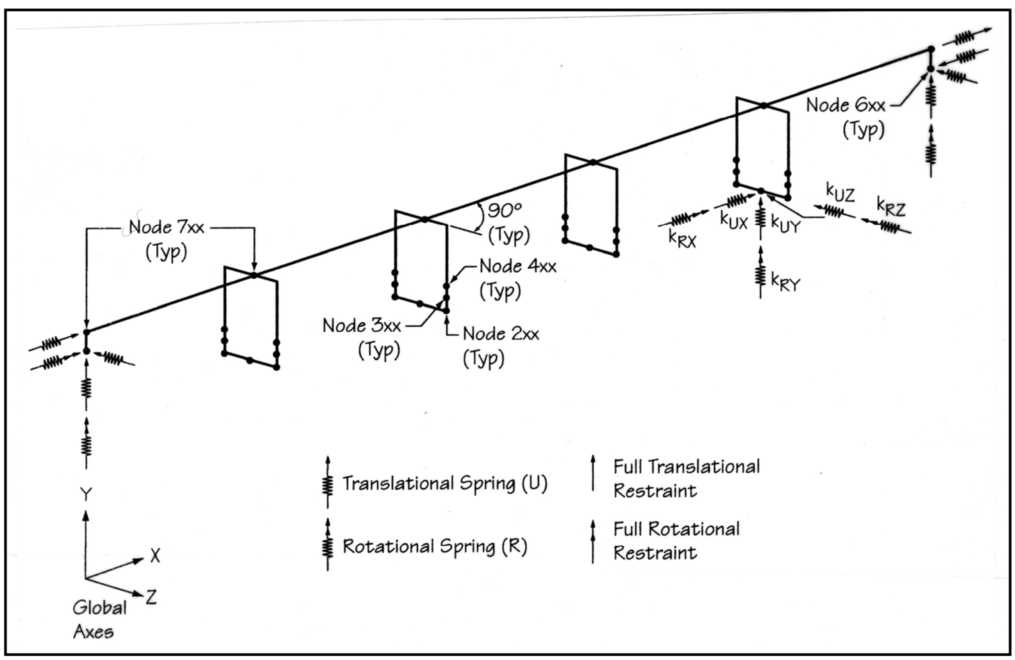


Figure 8 — Details of Spring Supports

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.3.1 (continued)

The spring stiffnesses are developed for the local bent support coordinate geometry but are input into the SAP2000 model with the same orientation as the global axes. Because the bridge is straight and square, the designation from local to global coordinates requires merely a change in the subscripts, such as k_{UX}, K_{RX}, etc. SAP2000 can accommodate local coordinate geometry, which would change the spring subscripts to numerical terms instead, such as k_{11}, k_{44}, etc. For a program that can only accommodate global directions for spring releases, the local stiffnesses computed would require transformation from local to global coordinate geometry for input into the model if the bridge is skewed or has a significant horizontal curve.

Establishing meaningful soil stiffnesses for bridge foundations is a complex problem that is often simplified to linear springs for static or modal analyses. There are several methods available for establishing pile foundation spring constants for use in a seismic analysis. The complexity of the methods varies widely, as does the input information required. Generally, any reasonable estimate of foundation stiffness will produce satisfactory results for dynamic analysis. The use of springs computed by some rational method, or by modifying substructure stiffness with an equivalent length to fixity, will provide better results than no foundation stiffness considerations at all. Article 8.4.3.4 provides guidance for developing lateral stiffnesses of piles, and it includes simplified charts that provide estimates of pile stiffnesses. These charts require the pile flexural stiffness and the soil subgrade modulus. These charts are most useful for preliminary design and checking. For detailed design, programs such as LPILE, Reese, et al. (1998), are recommended.

The spring stiffnesses of the pile groups at Bents 1, 2, 3, and 4 are calculated for the 6 degrees of freedom. A nonlinear lateral pile computer program, LPILE, is used to determine the head deflection of a single pile due to an applied shear force. The behavior is nonlinear because the soil has nonlinear response to applied loads. The soil data entered into the program includes group effects for the pile group. The method used here for the group effects is from the Washington State Department of Transportation (WSDOT), *Bridge Design Manual, M23-50* (including interims through September 2000). Also, the pile head is assumed fixed because the connection into the pile cap will develop the flexural strength of the reinforced concrete portion of the pile. A spring force can be computed based on the applied shear force and the resulting pile head deflection determined by LPILE. The following computation illustrates the computations done for the springs at Bent 1 (Pier 2). The springs at the remaining intermediate piers are computed similarly.

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.3.1 (continued)

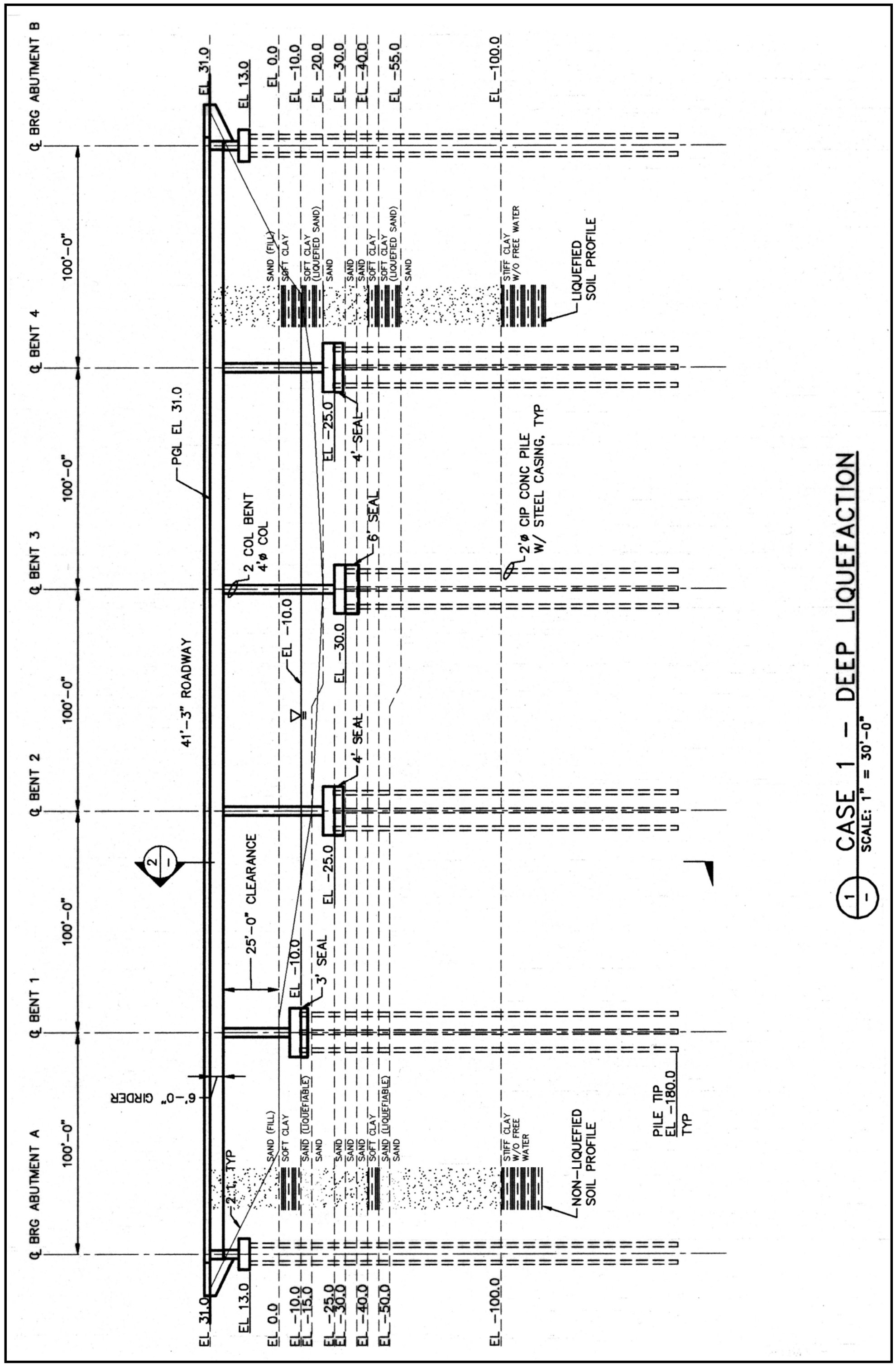


Figure 9 — Bridge Elevation Superimposed on Soil Profile

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.3.1 (continued)

The translational and rotational springs are computed below. The subscripts on the spring values are according to the global axes as opposed to local coordinates of the column. A plan view of the pile cap and pile arrangement for a typical intermediate bent is shown in Figure 10. The computations for each degree of freedom is outlined. Design Example Nos. 5, 6, and 7 illustrate the development of the foundation springs in more detail.

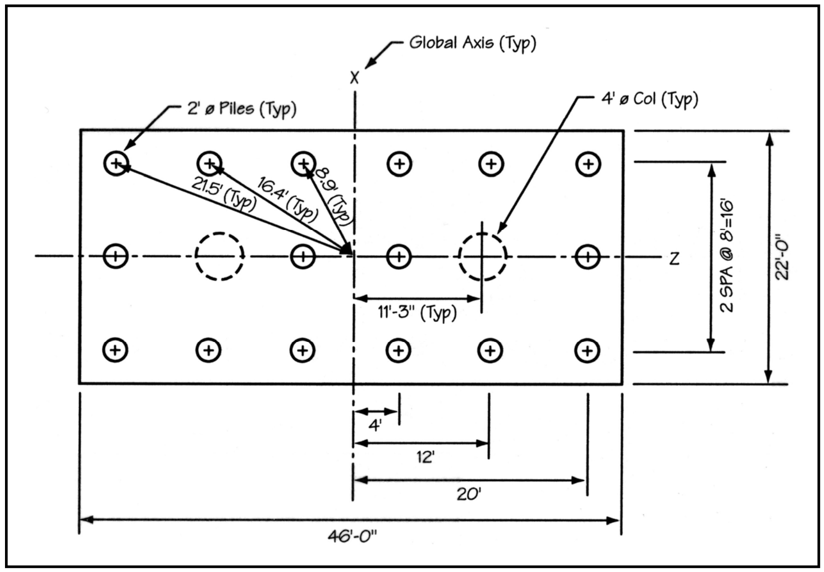


Figure 10 — Plan View of Typical Intermediate Bent Pile Cap

Compute foundation springs at Pier 2, referenced as Bent 1 in the figures.

Assume the following pile properties.

$A_{pile} := 673 \cdot in^2$ Reinforced concrete pile area, including transformed area of steel casing

$E_c := 3830 \cdot ksi$ Young's Modulus of Elasticity for concrete

$L_{pile} := 167 \cdot ft$ Length of pile at Pier 2

$N_{piles} := 16$ Number of piles at Pier 2

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.3.1 (continued)

Compute the axial stiffness of a single pile at Pier 2.

$k_{axial} := \dfrac{A_{pile} \cdot E_c}{L_{pile}}$ $\qquad k_{axial} = 15435 \dfrac{kips}{ft}$

Compute the axial spring for the pile group at Pier 2.

$k_{UY} := N_{piles} \cdot k_{axial}$ $\qquad k_{UY} = 246955 \dfrac{kips}{ft}$

Compute the lateral translational springs for an individual pile at Pier 2.

$V_{applied} := 50 \cdot kips$ Shear force applied to a single, fixed-head pile in LPILE. The soil properties entered into the program represent reduced properties for group effect.

$\Delta_{head} := 0.031 \cdot in$ Pile head deflection from LPILE results.

$k_{pile} := \dfrac{V_{applied}}{\Delta_{head}}$ $\qquad k_{pile} = 19355 \dfrac{kips}{ft}$

The lateral translational springs consist of two components:

1) the passive soil resistance against the pile cap, and
2) the lateral resistance of the pile

1) Compute the passive soil resistance from pile cap in both the transverse and longitudinal directions of the bridge.

$H_{cap} := 5 \cdot ft$ The height of the pile cap

$W_{capUX} := 46 \cdot ft$ The width of the pile cap, for longitudinal spring

$W_{capUZ} := 22 \cdot ft$ The width of the pile cap, for transverse spring

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.3.1 (continued)

The surface area of the pile face is now computed.

$$A_{capUX} := H_{cap} \cdot W_{capUX}$$

$$A_{capUX} = 230.00 \cdot ft^2$$ Area of pile cap face in longitudinal direction

$$A_{capUZ} := H_{cap} \cdot W_{capUZ}$$

$$A_{capUZ} = 110.00 \cdot ft^2$$ Area of pile cap face in transverse direction

The passive resistance of the soil is determined from Figure 11 assuming the following soil properties at the pile head.

Enter chart with: $\phi := 37 \cdot deg$

Using curve of: $\delta := -0.5 \cdot \phi$

Read off value of passive pressure coefficient: $K_p := 10$

The passive pressure of the soil is computed as:

$$p_p := 10 \cdot ksf$$

$$P_{pUX} := p_p \cdot A_{capUX}$$

$$P_{pUX} = 2300 \text{ kips}$$

The passive soil force on the longitudinal face of the pile cap.

$$P_{pUZ} := p_p \cdot A_{capUZ}$$

$$P_{pUZ} = 1100 \text{ kips}$$

The passive soil force on the transverse face of the pile cap.

$$\Delta := 0.02 \cdot H_{cap}$$

$$\Delta = 0.10 \text{ ft}$$

The displacement required to mobilize the passive soil force (Guide Spec 8.5.2.2)

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**

Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.3.1 (continued)

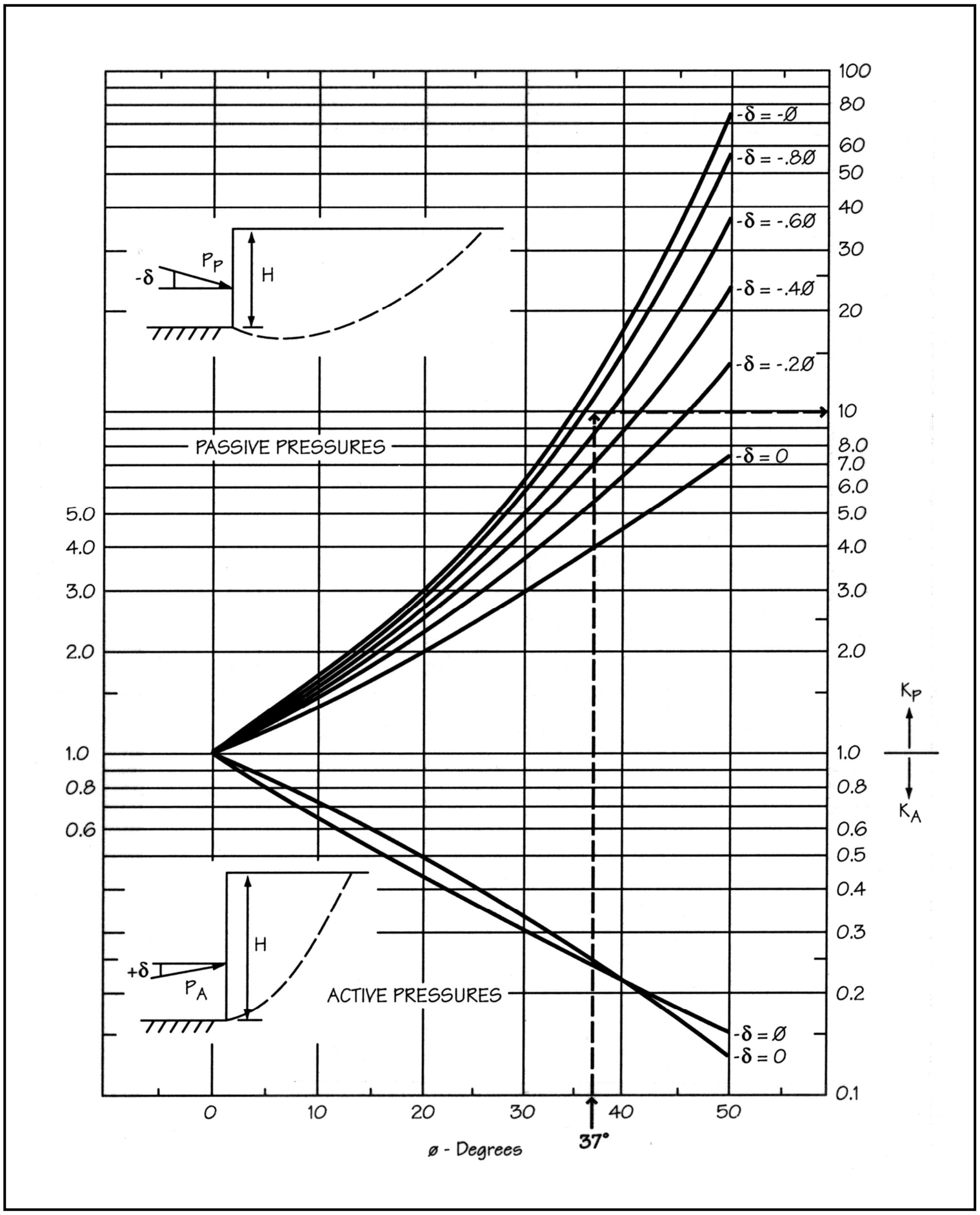


Figure 11 — Active and Passive Pressure Coefficients for Vertical Wall and Horizontal Backfill Based on Log Spiral Failure Surfaces

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.3.1 (continued)

The component of the spring from the passive soil resistance in both lateral directions is computed.

$$k_{UXcap} := \frac{P_{pUX}}{\Delta}$$

$$k_{UXcap} = 23000 \cdot \frac{kips}{ft}$$

Passive soil on pile cap component of the longitudinal translational spring

$$k_{UZcap} := \frac{P_{pUZ}}{\Delta}$$

$$k_{UZcap} = 11000 \cdot \frac{kips}{ft}$$

Passive soil on pile cap component of the transverse translational spring

2) The component of the spring from the lateral displacement of the piles is simply the number of piles times the individual pile lateral spring, which is the same for either direction of movement.

$$k_{16piles} := N_{piles} \cdot k_{pile}$$

$$k_{16piles} = 3.10 \cdot 10^5 \cdot \frac{kips}{ft}$$

Lateral pile component of the transverse and longitudinal translational springs

The complete translational springs are computed by summing the two components.

$$k_{UX} := k_{16piles} + k_{UXcap}$$

Longitudinal translational springs

$$k_{UX} = 3.33 \cdot 10^5 \cdot \frac{kips}{ft}$$

$$k_{UZ} := k_{16piles} + k_{UZcap}$$

$$k_{UZ} = 3.21 \cdot 10^5 \cdot \frac{kips}{ft}$$

Transverse translational springs

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.3.1 (continued)

The rotational springs are now computed for Pier 2. Figure 10 illustrates the dimensions of each pile from the axis of rotation, which in this case is the vertical axis. These distances are summed up using the "parallel axis theorem" and multiplied by the lateral spring of an individual pile. This computation is analogous to the term "Ad^2" when computing moment of inertia, following the parallel axis theorem.

$$k_{RY} := k_{pile} \cdot \left[2 \cdot \left[(4 \cdot ft)^2 + (20 \cdot ft)^2 \right] + 4 \cdot \left[(8.9 \cdot ft)^2 + (14.4 \cdot ft)^2 + (21.5 \cdot ft)^2 \right] \right]$$

$$k_{RY} = 7.41 \cdot 10^7 \circ \frac{ft \cdot kips}{rad}$$ Torsional Spring at Pier 2

This is done similarly for the rotation about the horizontal axis, except the axial spring of the individual pile is used instead of the lateral spring. This is because the rotation of the pile cap causes the piles to be extended or shortened. Again, Figure 10 illustrates the distance of the piles from the axis of rotation.

$$k_{RX} := k_{axial} \cdot \left[6 \cdot (4 \cdot ft)^2 + 4 \cdot (12 \cdot ft)^2 + 6 \cdot (20 \cdot ft)^2 \right]$$

$$k_{RX} = 4.74 \cdot 10^7 \circ \frac{ft \cdot kips}{rad}$$ Rotational Spring about the longitudinal axis at Pier 2

$$k_{RZ} := k_{axial} \cdot \left[12 \cdot (8 \cdot ft)^2 \right]$$

$$k_{RZ} = 1.19 \cdot 10^7 \circ \frac{ft \cdot kips}{rad}$$ Rotational Spring about the transverse axis at Pier 2

The following is a summary of the foundation springs calculated for Pier 2.

$$k_{UX} = 3.33 \cdot 10^5 \circ \frac{kips}{ft}$$ Translation, x axis

$$k_{UY} = 2.47 \cdot 10^5 \circ \frac{kips}{ft}$$ Translation, y (vertical) axis

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.3.1 (continued)

$$k_{UZ} = 3.21 \cdot 10^{5} \cdot \frac{kips}{ft}$$ Translation, z axis

$$k_{RX} = 4.74 \cdot 10^{7} \cdot \frac{ft \cdot kips}{rad}$$ Rotation, x axis

$$k_{RY} = 7.41 \cdot 10^{7} \cdot \frac{ft \cdot kips}{rad}$$ Rotation, y (vertical) axis

$$k_{RZ} = 1.19 \cdot 10^{7} \cdot \frac{ft \cdot kips}{rad}$$ Rotation, z axis

Use these springs to model the foundation stiffnesses at Pier 2 in the Multimode Spectral Analysis. The foundation springs at the remaining intermediate bents can be computed similarly and are given in Table 2. These details are input into the SAP2000 model in the local bent support node coordinate system, as shown in Figure 6.

Care should be taken to obtain the correct orientation for input of the springs into the model. This precision is especially important for foundations that have significantly different stiffnesses for each of its orthogonal directions, such as the case with this structure.

Design Step 4.3.2

Abutments

The abutments were also modeled with equivalent spring stiffnesses (transverse translation), as shown in Figure 6. Some degrees of freedom are released between the superstructure and the abutment foundation. These include the rotation about the vertical and transverse axes and translation along the longitudinal axis. The model allows longitudinal translational response that is unrestrained at the stub-type abutment (see Figure 12).

An unrestrained longitudinal response also assumes that the bearings are free to translate in the longitudinal direction, which may not be exactly the case. The actual stiffness and movement characteristics of the bearings should be assessed. However, because the intent of this example is to provide all of the longitudinal resistance at the bents and at the back of the superstructure against the soil, the assumption of "free bearings" in the longitudinal direction is conservative and desirable for design of the bents.

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.3.2 (continued)

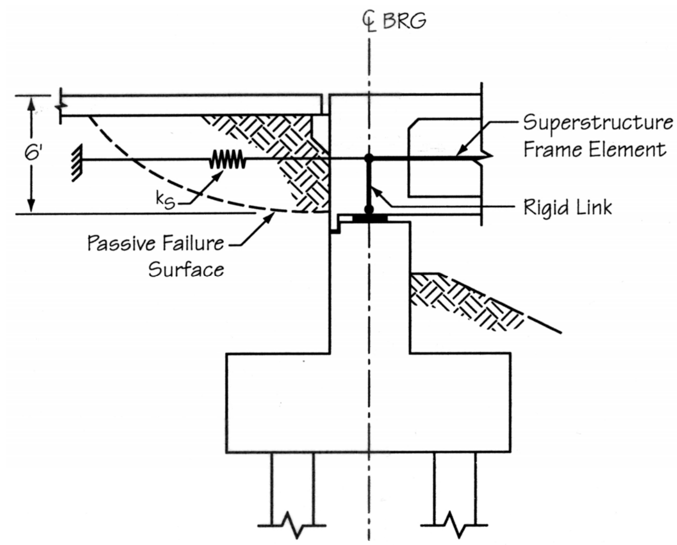


Figure 12 – Longitudinal Superstructure Passive Soil Spring

The ends of the superstructure are restrained against translation in the transverse direction at the abutments by girder stops at each side of the bridge. The transverse force resulting from this restraint is taken through the girder stops into the abutment, and is then resisted by the soil acting against the abutment and its wingwalls. The transverse stop and wingwall elevation are shown in Figure 1c. A translational spring stiffness for the transverse direction, based upon the stiffness of soil against the wingwalls that resist translation in the transverse direction, will not be addressed in this example. Refer to Design Example No. 4 for computing this value. In this example, the soil against the wingwall will be ignored.

Table 2
Washington Bridge Foundation Springs

	Axial	Longitudinal	Lateral	Axial	Longitudinal	Lateral
Global	UY	UX	UZ	RY	RX	RZ
Pier	K_{11} k/ft	K_{22} k/ft	K_{33} k/ft	K_{44} k-ft/rad	K_{55} k-ft/rad	K_{66} k-ft/rad
1	1.60E+05	0.00E+00	7.30E+04	0.00E+00	2.99E+07	0.00E+00
2	2.47E+05	3.33E+05	3.21E+05	1.19E+09	4.74E+07	1.19E+07
3	2.73E+05	3.43E+05	3.31E+05	1.23E+09	5.24+07	1.31E+07
4	2.86E+05	4.59E+05	4.47E+05	1.68E+09	5.50E+07	1.37E+07
5	2.73E+05	3.43E+05	3.31E+05	1.23E+09	5.24E+07	1.31E+07
6	1.60E+05	0.00E+00	7.30E+04	0.00E+00	2.99E+07	0.00E+00

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.3.3

Passive Soil Resistance Behind Superstructure End Diaphragm
[Guide Spec, 8.5.2.2] [NCHRP, Article 11.6.5.1.1]

The soil behind the superstructure end diaphragm is considered in the longitudinal movement of the bridge during a seismic event. Therefore, a soil spring is developed to represent the passive soil resistance the backfill provides.

The specification prescribes the passive pressure and displacement required to mobilize the passive soil force. The following computation illustrates the recommended method.

Longitudinal Superstructure Spring

1) Compute the passive soil resistance from diaphragm in both the transverse and longitudinal directions of the bridge.

Assume the surface area of the diaphragm face is:

$A_{diaph} := 208.1 \cdot ft^2$ Area of diaphragm face against soil longitudinal direction

$H_{diaph} := 6 \cdot ft$ Height of diaphragm face against soil

The passive resistance of the soil is prescribed in the Specification (Guide Spec 8.5.2.2) by the following computation.

$$p_p := \frac{2}{3} \cdot H_{diaph} \cdot \left(\frac{ksf}{ft}\right)$$

$p_p = 4.00\,ksf$

Passive pressure of backfill is 2/3 of the height of the backwall, or diaphragm, in contact with the soil, in ksf per foot of backwall

$P_p := p_p \cdot A_{diaph}$

$P_p = 832\,kips$

Passive soil force on diaphragm

$\Delta := 0.02 \cdot H_{diaph}$

$\Delta = 0.12\,ft$

The displacement required to mobilize the passive soil force

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.3.3 (continued)

The longitudinal spring force on the superstructure diaphragm is the force divided by the displacement that mobilizes it.

$$k_s := \frac{P_p}{\Delta}$$

$$k_s = 6937 \cdot \frac{kips}{ft}$$

Longitudinal superstructure spring. Half of this value is applied to each end of the model.

Note that the stiffness of the longitudinal abutment spring is essentially an initial stiffness. If the structure longitudinal movement is larger than the calculated displacement at which the passive resistance is mobilized, then the stiffness of the spring used in the analytical model should be reduced. The reduction is typically done iteratively until reasonable results are obtained. The objective is to end up with a 'secant stiffness' for the spring that just produces the passive soil resistance at the calculated maximum displacement. Article 8.5.2.2 of the provisions describes this process and suggests that the precision need not be less than 30 percent, although it is relatively easy to obtain results much closer than 30 percent. Figure 8.5.2.2-2 of the provisions also illustrates the secant stiffness concept, and in the figure this stiffness is denoted, K_{eff2}.

Looking ahead to the modal analysis, the final secant stiffness for the longitudinal springs was 750 kips/ft. Half of this value was distributed to each end of the analytical model. The longitudinal displacement, as will be seen later in Design Step 4, is about 1.1 feet. Thus with a stiffness of 750 and a displacement of 1.1, the passive resistance calculated is about 830 kips, which is the passiver resistance listed above. It is apparent that the results converged much closer than 30 percent.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**

Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.4

Multimode Spectral Analysis - General
[Guide Spec, Article 5.4.2.3] [NCHRP, Article 4.8.5.3.4]

Design Step 4.4.1

Mode Shapes and Periods
[Guide Spec, Article 5.4.2.3] [NCHRP, Article 4.8.5.3.4]

The structure has been discretized using four elements per span and elements at each bent cap, column, pile cap, and seal, as discussed previously. Twenty vibration modes have been used in the multimodal spectral analyses for both the MCE and Frequent earthquakes, which involve the superposition of individual modal responses to estimate the overall structural seismic response.

The SAP2000 program (or any other dynamic spectral analysis program) lumps the tributary mass of each element at the adjacent nodes. Spring elements, which provide foundation flexibility, are massless. SAP2000 determines the vibration periods and shapes for each of the vibration modes of the structure. The number of modes is dependent on the number of masses, the number of constrained degrees of freedom, and the number of foundation restraints for the system. Enough modes have to be specified so that the modal superposition to determine forces and displacements is sufficiently accurate. Typically, the modes are numbered sequentially from the longest period to the shortest.

The natural periods of vibration for the bridge and mass participation for the first 20 modes are shown in Table 3 for the MCE event, and Table 4 for the Frequent event.

Results are shown for both the MCE and Frequent events, which ordinarily should have the same vibration periods and modes. However, in this case, the two events have slightly different models because the longitudinal springs at the abutments are different for the two earthquakes. Therefore, the longitudinal periods are slightly different, as well. This difference can be seen by closely comparing the tables. As would be expected, the transverse periods and mass are not affected. This would not be the case for a bridge with skewed abutments.

Figures 13 and 14 show two selected modes for the structure. Figure 13 shows the first mode, which is associated with the fundamental period in the transverse direction. The transverse period for this mode is 1.62 seconds. Figure 14 shows the second mode, which is the mode associated with the fundamental period in the longitudinal direction. The period for the second mode is 1.38 seconds for the MCE event.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.4.1 (continued)

Table 3
Modal Periods and Participating Mass
MCE Earthquake

```
Program SAP2000 Nonlinear Version 7.10                              File:WA2500N.OUT
                                                                                Page
7NCHRP 12-49 WASHINGTON SITE / 2500-YR EQ / NON-LIQUEFIED / FOUNDATION             5

M O D A L   P A R T I C I P A T I N G   M A S S   R A T I O S
```

MODE	PERIOD	INDIVIDUAL MODE (PERCENT) UX	UY	UZ	CUMULATIVE SUM (PERCENT) UX	UY	UZ
1	1.622421	0.0000	0.0000	45.0088	0.0000	0.0000	45.0088
2	1.377229	65.1140	0.0000	0.0000	65.1140	0.0000	45.0088
3	1.042341	0.0000	0.0000	2.3105	65.1140	0.0000	47.3193
4	0.676841	0.0000	0.0000	10.3288	65.1140	0.0000	57.6482
5	0.448192	0.0000	0.0000	0.1441	65.1140	0.0000	57.7923
6	0.278303	0.0000	0.0000	1.7982	65.1140	0.0000	59.5905
7	0.208264	0.0000	0.0000	0.0032	65.1140	0.0000	59.5937
8	0.154739	0.0000	0.0000	1.0421	65.1140	0.0000	60.6358
9	0.132195	0.0000	79.0529	0.0000	65.1140	79.0529	60.6358
10	0.126373	0.0000	2.8171	0.0000	65.1140	81.8700	60.6358
11	0.120127	0.0000	0.0000	0.0013	65.1140	81.8700	60.6371
12	0.112612	0.0000	9.8091	0.0000	65.1141	91.6791	60.6371
13	0.110311	0.0009	0.0001	0.0000	65.1150	91.6791	60.6371
14	0.096714	0.0000	0.0000	0.9737	65.1150	91.6791	61.6107
15	0.091491	0.0004	0.0012	0.0000	65.1154	91.6804	61.6107
16	0.078145	0.0000	0.0000	0.0001	65.1154	91.6804	61.6108
17	0.073143	0.0000	0.0000	18.1599	65.1154	91.6804	79.7708
18	0.073140	0.0000	0.0000	0.2067	65.1154	91.6804	79.9775
19	0.072549	0.0000	0.0000	2.3118	65.1154	91.6804	82.2893
20	0.070581	0.0000	0.0000	7.3554	65.1154	91.6804	89.6447

SECTION III BRIDGE WITH TWO-COLUMN BENTS

Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.4.1 (continued)

Table 4
Modal Periods and Participating Mass
Frequent Earthquake

```
Program SAP2000 Nonlinear Version 7.40                           File:Wa100n.OUT
                                                                            Page
NCHRP 12-49 WASHINGTON SITE / 100-YR EQ / NON-LIQUEFIED / FOUNDATION S         5

M O D A L   P A R T I C I P A T I N G   M A S S   R A T I O S
```

MODE	PERIOD	INDIVIDUAL MODE (PERCENT) UX	UY	UZ	CUMULATIVE SUM (PERCENT) UX	UY	UZ
1	1.622421	0.0000	0.0000	45.0088	0.0000	0.0000	45.0088
2	1.147991	65.1142	0.0000	0.0000	65.1142	0.0000	45.0088
3	1.042341	0.0000	0.0000	2.3105	65.1142	0.0000	47.3193
4	0.676841	0.0000	0.0000	10.3288	65.1142	0.0000	57.6482
5	0.448192	0.0000	0.0000	0.1441	65.1142	0.0000	57.7923
6	0.278303	0.0000	0.0000	1.7982	65.1142	0.0000	59.5905
7	0.208264	0.0000	0.0000	0.0032	65.1142	0.0000	59.5937
8	0.154739	0.0000	0.0000	1.0416	65.1142	0.0000	60.6354
9	0.132195	0.0000	79.0543	0.0000	65.1143	79.0543	60.6354
10	0.126373	0.0000	2.8166	0.0000	65.1143	81.8709	60.6354
11	0.120127	0.0000	0.0000	0.0012	65.1143	81.8709	60.6366
12	0.112612	0.0000	9.8078	0.0000	65.1143	91.6788	60.6366
13	0.109998	0.0010	0.0001	0.0000	65.1153	91.6788	60.6366
14	0.096714	0.0000	0.0000	0.9735	65.1153	91.6788	61.6101
15	0.091491	0.0004	0.0012	0.0000	65.1157	91.6801	61.6101
16	0.078145	0.0000	0.0000	0.0001	65.1157	91.6801	61.6102
17	0.073143	0.0000	0.0000	18.2320	65.1157	91.6801	79.8423
18	0.073140	0.0000	0.0000	0.1345	65.1157	91.6801	79.9768
19	0.072549	0.0000	0.0000	2.3127	65.1157	91.6801	82.2894
20	0.070581	0.0000	0.0000	7.3554	65.1157	91.6801	89.6448

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.4.1 (continued)

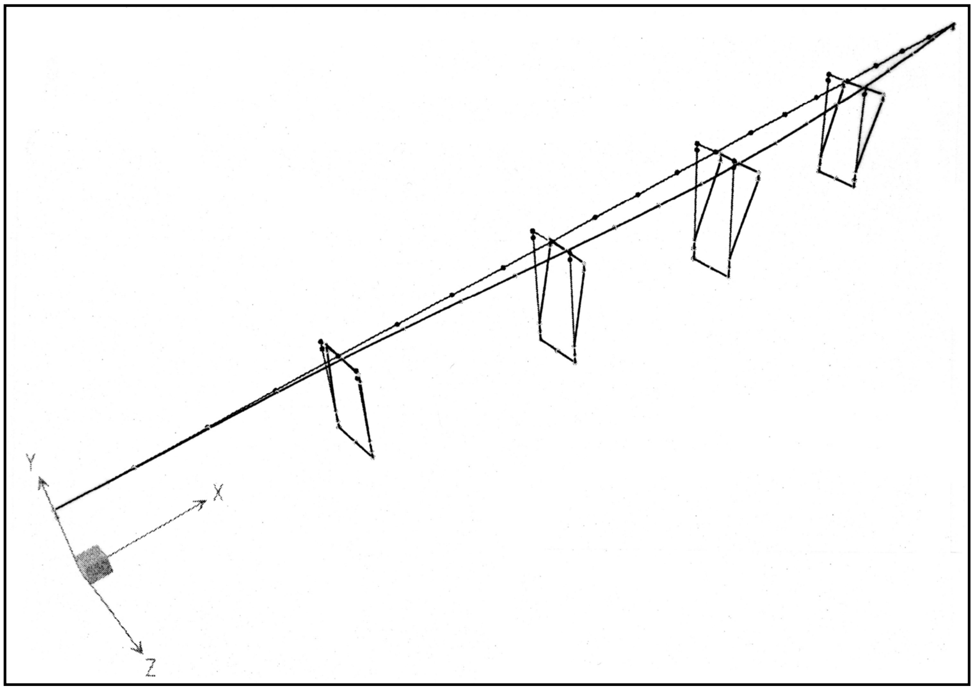


Figure 13 — Deformed Shape for MCE Mode 1

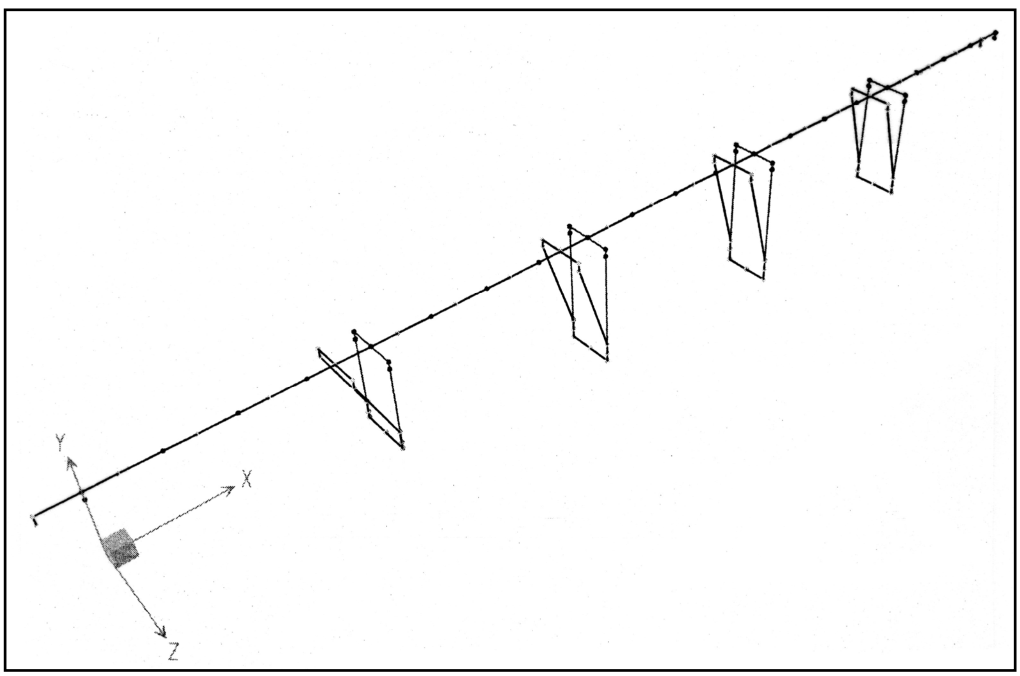


Figure 14 — Deformed Shape for MCE Mode 2

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.4.1 (continued)

Note that the cumulative mass participation in the longitudinal (X) and transverse (Z) directions is less than the 90 percent value suggested by Article C5.4.2.3 of the provisions. In this analytical model, the mass of the pile caps and the seals have been included. These masses are not generally required, unless one is designing using only the elastic forces. In this example, capacity design will be used; therefore, the elastic analyses are used primarily to obtain design forces for the columns and displacements of the superstructure.

The modal analyses were rerun using 40 modes instead of 20 and the mass participation ratios increased to 99 percent in all three directions. The column forces and superstructure displacements were also compared with the results for 20 modes and no differences were apparent. The reason is that the additional modes required to increase the mass participation were all associated with movement of the foundation elements. Thus the 20 mode results reported herein are valid, even though nominally the mass participation is less than 90 percent.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.5

Determine Forces and Displacements in Transverse Direction
[Guide Spec, Article 5.3.1] [NCHRP, Article 4.8.4.1]

Using the Multimode Dynamic Method, perform a transverse analysis. Transverse analysis means that the input response spectrum was assigned to the transverse direction; and along with this transverse load case, no longitudinal or vertical spectra were used. For the longitudinal analysis, only the longitudinal input spectrum was used (i.e., no transverse or vertical spectra were simultaneously applied). The longitudinal direction is along a straight line parallel to the centerline of the bridge (global X). The transverse direction is applied at 90 degrees to the longitudinal direction (global Z). These directions are shown in Figure 15. In most cases, when the same model is used for both directions of loading, both the transverse and longitudinal analyses are performed in the same computer run, as is the case for this example.

The analysis program handles all the calculations, including the modal combinations. In this case, 20 modes were used to characterize the response. This number was kept constant for all the analyses.

The results are given in Table 5. The SAP2000 input file for this analysis is 2500N (represents MCE, 2,475-year return period, and nonliquefied foundation stiffnesses). Shown in the table are forces and moments. Directions for forces and moments at the bents are shown in Figure 15, and are oriented along the local coordinate system for the bent elements. For bent columns, the transverse direction is parallel to the plane of the bent frame (global Z direction), and the longitudinal direction is 90 degrees to the plane of the bent frame (global X direction). Abutment transverse forces are oriented in the global coordinate system (global Z direction) as shown in Figure 15.

Displacements for both transverse and longitudinal analyses are given in Table 6. Directions for the displacements are in the global coordinate directions which are shown in Figure 15.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.5
(continued)

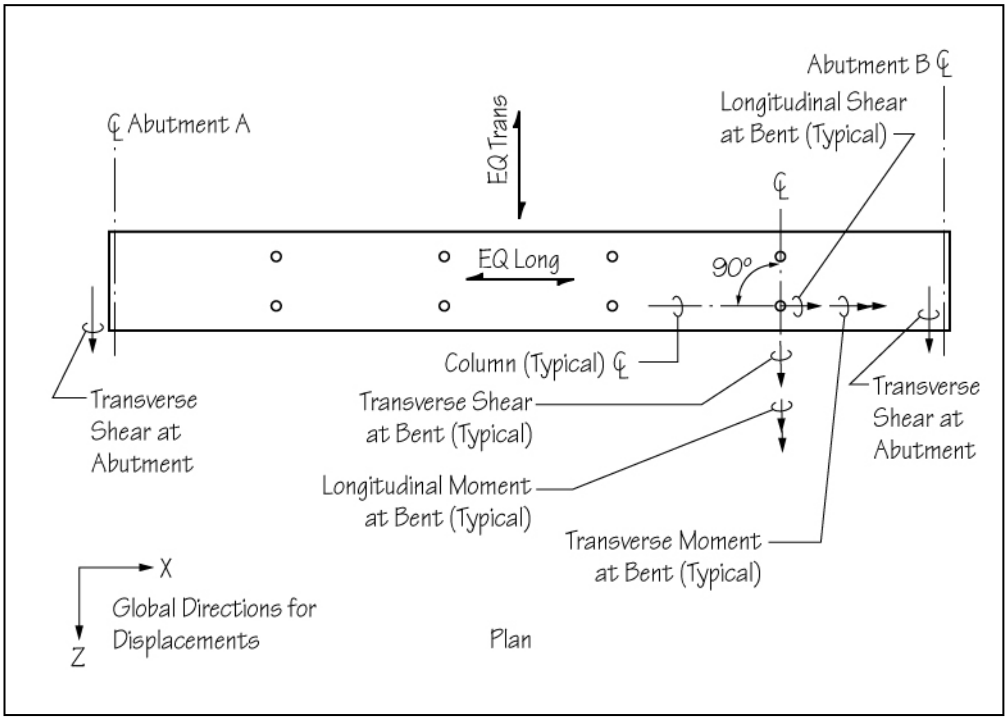


Figure 15 — Key to Force, Moment, and Displacements Directions

SECTION III BRIDGE WITH TWO-COLUMN BENTS

Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.5 (continued)

Table 5
Response for Transverse Direction (EQ_{trans})

MCE Earthquake		Forces and Moments - EQ_{trans}				
		Longitudinal		Transverse		
Support/Location		Shear X (kips)	Moment Z (kip-ft)	Shear Z (kips)	Moment X (kip-ft)	Axial (kips)
Abutment A		0	0	437	1871	0
Bent 1	Top	84	1265	513	7676	795
Typical Col	Bottom	84	1265	513	7733	795
Bent 2	Top	40	911	415	9295	922
Typical Col	Bottom	40	911	415	9368	922
Bent 3	Top	12	301	448	11153	1070
Typical Col	Bottom	12	301	448	11235	1070
Bent 4	Top	40	906	446	10006	957
Typical Col	Bottom	40	905	446	10077	957
Abutment B		0	0	492	2930	0

Frequent Earthquake		Forces and Moments - EQ_{trans}				
		Longitudinal		Transverse		
Support/Location		Shear X (kips)	Moment Z (kip-ft)	Shear Z (kips)	Moment X (kip-ft)	Axial (kips)
Abutment A		0	0	205	822	0
Bent 1	Top	25	370	170	2545	259
Typical Col	Bottom	25	370	170	2566	259
Bent 2	Top	12	281	119	2670	265
Typical Col	Bottom	12	281	119	2691	265
Bent 3	Top	4	102	129	3212	308
Typical Col	Bottom	4	102	129	3236	308
Bent 4	Top	12	265	129	2902	277
Typical Col	Bottom	12	265	129	2923	277
Abutment B		0	0	215	1003	0

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.5
(continued)

Table 6
Displacements

MCE EQ	Displacements of CGC of Superstructure			
	EQtrans		EQlong	
Location	Global X (ft)	Global Z (ft)	Global X (ft)	Global Z (ft)
Abutment A	0.00	0.00	1.11	0.00
Bent 1	0.00	0.43	1.11	0.00
Bent 2	0.00	1.17	1.11	0.00
Bent 3	0.00	1.72	1.11	0.00
Bent 4	0.00	1.25	1.12	0.00
Abutment B	0.00	0.01	1.16	0.00

Frequent EQ	Displacements of CGC of Superstructure			
	EQtrans		EQlong	
Location	Global X (ft)	Global Z (ft)	Global X (ft)	Global Z (ft)
Abutment A	0.00	0.00	0.27	0.00
Bent 1	0.00	0.14	0.27	0.00
Bent 2	0.00	0.33	0.27	0.00
Bent 3	0.00	0.50	0.27	0.00
Bent 4	0.00	0.36	0.27	0.00
Abutment B	0.00	0.00	0.27	0.00

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.6

Determine Forces and Displacements in Longitudinal Direction
[Guide Spec, Article 5.3.1] [NCHRP, Article 4.8.4.1]

Perform the analysis for loading in the longitudinal direction.

The resulting forces and moments at the intermediate piers for the spectral analysis in the longitudinal direction are given in Table 7. The SAP2000 input file for this analysis is 2500N. Displacements for both transverse and longitudinal analyses are given in Table 6. Directions for displacements are in the global coordinate system, which is shown in Figure 15.

Table 7
Response for Longitudinal Direction (EQ_{long})

MCE Earthquake		Forces and Moments - EQ_{long}				
		Longitudinal (1)		Transverse (2)		
Support/Location		Shear X (kips)	Moment Z (kip-ft)	Shear Z (kips)	Moment X (kip-ft)	Axial (kips)
Abutment A		416	0	0	0	230
Bent 1	Top	1329	19953	0	0	72
Typical Col	Bottom	1329	19909	0	0	72
Bent 2	Top	400	9013	0	0	95
Typical Col	Bottom	400	8999	0	0	95
Bent 3	Top	293	7328	0	0	1
Typical Col	Bottom	293	7318	0	0	1
Bent 4	Top	401	9033	0	0	47
Typical Col	Bottom	401	9024	0	0	47
Abutment B		416	0	0	0	92

(1) For bent columns, the longitudinal direction is 90 degrees to the plane of the bent frame.
(2) For bent columns, the transverse direction is parallel to the plane of the bent frame.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**

Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.6 (continued)

Table 7
Response for Longitudinal Direction (EQ_{long})
(continued)

Frequent Earthquake		Forces and Moments - EQ_{long}				
		Longitudinal (1)		Transverse (2)		
Support/Location		Shear X (kips)	Moment Z (kip-ft)	Shear Z (kips)	Moment X (kip-ft)	Axial (kips)
Abutment A		398	0	0	0	55
Bent 1 Typical Col	Top	318	4782	0	0	17
	Bottom	318	4773	0	0	17
Bent 2 Typical Col	Top	96	2163	0	0	23
	Bottom	96	2159	0	0	23
Bent 3 Typical Col	Top	70	1758	0	0	1
	Bottom	70	1756	0	0	1
Bent 4 Typical Col	Top	96	2165	0	0	11
	Bottom	96	2163	0	0	11
Abutment B		401	0	0	0	22

(1) For bent columns, the longitudinal direction is 90 degrees to the plane of the bent frame.
(2) For bent columns, the transverse direction is parallel to the plane of the bent frame.

Note that the longitudinal shear at the abutments is taken from SAP as the longitudinal spring force acting against the end diaphragm. In this case, the force is 416 kips, which is equal to 1.11-foot displacement times 375 kip/ft stiffness. These forces should be doubled for design of the end diaphragm, because the abutment compression secant spring has been split in two and assigned to each end of the model. Additionally, the axial force from the end superstructure element in the SAP model cannot be used because the inertial force at the end of the superstructure is applied to the node between the spring and the end member. Because this force includes the inertial effect of the end diaphragm, it is a relatively large force, and to not account for it will introduce a large error into the calculations.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 5, Determine Design Forces

DESIGN STEP 5 DETERMINE DESIGN FORCES

INTRODUCTION

The designations for the load combinations in the LRFD Specification are different from those used in the Standard Specifications. The reference to Group loads, for instance for seismic loading, Group VII no longer applies. In the LRFD provisions, Article 3.5 covers load factors and load combinations. Table 3.5-1 of the provisions gives the load combinations and factors for each 'Limit State.'

The load combinations that apply to earthquake are those for 'Extreme Event I.' While the table makes no reference to the two-level approach that the proposed provisions include, Extreme Event I covers both events. Thus the load combination factors for both the MCE and Frequent events are those given for Extreme Event I. This is reasonable because both earthquake return periods exceed the nominal 75-year design life assumed for new bridges.

Design Step 5.1 Determine Nonseismic Forces

[Guide Spec, Article 3.5] [NCHRP, Article 3.4.1]

The nonseismic loads included with the Extreme Event I load combination are all 'Permanent Loads,' such as dead load, earth pressure, and any locked in loads from the sequence of construction. Also included are water loads and friction loads. Finally, some portion of the live load should be considered; but at this time, a specific amount has not been established.

In addition to the basic nonseismic loads, the LRFD provisions contain a high and a low load factor for the permanent loads. This is referred to as, γ_p. Thus, the earthquake load cases would consider the worse of the maximum and minimum factored load conditions. In the current draft of the proposed provisions, these factors have been taken as 1.0 for the Extreme Event I combinations.

Thus for this example, the primary nonseismic load is the dead load. Additional loads, for instance water loads, are considered in the capacity design of the foundations, but water loads do not affect the modal analysis or basic load combinations used to design the columns.

Design Step 5.1.1 Determine Dead Load Forces

The dead load forces obtained from a previously performed static analysis are summarized in Table 8.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 5, Determine Design Forces

Design Step 5.1.1 (continued)

Table 8
Dead Load Forces

MCE and Frequent Events		Forces and Moments - Dead Load				
		Longitudinal		Transverse		
Support / Location		Shear X (kips)	Moment Z (ft-kips)	Shear Z (kips)	Moment X (ft-kips)	Axial (kips)
Abutment A		0	0	0	0	583
Bent 1 Columns	Top	0.6	12	0.2	1	706
	Bottom	0.6	5	0.2	5	763
Bent 2 Columns	Top	0	0.7	0.1	0.7	720
	Bottom	0	1	0.1	4	805
Bent 3 Columns	Top	0	0.3	0.1	0.9	718
	Bottom	0	0.4	0.1	4	812
Bent 4 Columns	Top	0.5	13	0.1	0.7	698
	Bottom	0.5	8	0.1	3	783
Abutment B		0	0	0	0	593

Design Step 5.2

Determine Seismic Forces
[Guide Spec, Article 3.6] [NCHRP, Article 3.10.2.4]

Design Step 5.2.1

Summary of Elastic Seismic Forces
[Guide Spec, Article 3.6] [NCHRP, Article 3.10.2.4]

The Multimode Spectral Method results are used to determine the modified design forces. These are summarized for both seismic events in Table 9.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**

Design Step 5, Determine Design Forces

Design Step 5.2.1 (continued)

Table 9
Full Elastic Seismic Forces

MCE Event		Full Elastic Seismic Forces and Moments				
		Longitudinal		Transverse		
Seismic Direction	Location*	Shear, x (kips)	Moment, z (ft-kips)	Shear, z (kips)	Moment, x (ft-kips)	Axial (kips)
EQlong, L	Abutment A	416	0	0	0	230
	B1 Column	1329	19909	0	0	72
	B2 Column	400	8999	0	0	95
	B3 Column	293	7318	0	0	1
	B4 Column	401	9024	0	0	47
	Abutment B	419	0	0	0	92
EQtrans, T	Abutment A	0	0	437	1871	0
	B1 Column	84	1265	513	7733	795
	B2 Column	40	911	415	9368	922
	B3 Column	12	301	448	11235	1070
	B4 Column	40	906	446	10077	957
	Abutment B	0	0	492	2930	0

Frequent Event		Full Elastic Seismic Forces and Moments				
		Longitudinal		Transverse		
Seismic Direction	Location*	Shear, x (kips)	Moment, z (ft-kips)	Shear, z (kips)	Moment, x (ft-kips)	Axial (kips)
EQlong, L	Abutment A	398	0	0	0	55
	B1 Column	318	4773	0	0	17
	B2 Column	96	2159	0	0	23
	B3 Column	70	1756	0	0	0.6
	B4 Column	96	2163	0	0	11
	Abutment B	401	0	0	0	22
EQtrans, T	Abutment A	0	0	205	822	0.2
	B1 Column	25	370	170	2566	259
	B2 Column	12	281	119	2691	265
	B3 Column	4	102	129	3236	308
	B4 Column	12	265	129	2923	277
	Abutment B	0	0	215	1003	0.3

*The column moment at bottom is used in this design example. However, the top and bottom column moments are typically evaluated separately.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 5, Determine Design Forces

Design Step 5.2.2

Combination of Orthogonal Seismic Forces
[Guide Spec, Article 3.6] [NCHRP, Article 3.10.2.4]

Before the seismic forces are combined with the dead load to create the modified design forces, the seismic forces along the two principal axes must be combined (without dead load).

The specification allows the seismic force effects from two or three orthogonal directions to be combined using one of two methods. The SRSS ("Square-Root of the Sum of the Squares") is the method of choice according to the commentary, especially if vertical analysis is significant. However, either method is permitted. For this design example, the 100 - 40 percent rule was adopted, although both methods were used to develop the seismic forces for comparison. See Table 10 for a summary of the seismic forces resulting from the SRSS combination rule for both the MCE and frequent events. Similarly, see Table 11 for a summary of the seismic forces resulting from the 100 - 40 percent combination rule.

The SRSS combination rule is computed as follows, from Guide Spec, Article 3.6.

COMBINATION OF SEISMIC FORCE EFFECTS

Pier 2 (Bent 1) Results - 2500N for MCE Event

$M_{xT} := 7733 \cdot ft \cdot kips$

$M_{xL} := 0 \cdot ft \cdot kips$

$M_{zT} := 1265 \cdot ft \cdot kips$

$M_{zL} := 19909 \cdot ft \cdot kips$

SRSS Combination Rule

("x" and "z" refer to global axes)

$M_x := \sqrt{M_{xT}^2 + M_{xL}^2}$ $\qquad M_x = 7733\ ft \cdot kips$

$M_z := \sqrt{M_{zT}^2 + M_{zL}^2}$ $\qquad M_z = 19949\ ft \cdot kips$

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 5, Determine Design Forces

Design Step 5.2.2 (continued)

For biaxial design, the maximum vector moment is the maximum of the following.

$$M_1 := \sqrt{M_x^{\,2} + (0.4M_z)^2} \qquad M_1 = 11112\ \text{ft}\cdot\text{kips}$$

$$M_2 := \sqrt{(0.4M_x)^2 + M_z^{\,2}} \qquad M_2 = 20188\ \text{ft}\cdot\text{kips}$$

$$M_{SRSSmax} := \max(M_1, M_2) \qquad M_{SRSSmax} = 20188\ \text{ft}\cdot\text{kips}$$

Note that all of the forces in the SRSS combination are the full elastic seismic forces.

Table 10
Orthogonal Seismic Force Combinations
SRSS Combination Rule

MCE Event

Location	Longitudinal Shear (kips)	Longitudinal Moment (ft-kips)	Transverse Shear (kips)	Transverse Moment (ft-kips)	Axial (kips)
Abutment A	416	0	437	1871	230
B1 Column	1332	19949	513	7733	798
B2 Column	402	9045	415	9368	927
B3 Column	293	7324	448	11235	1070
B4 Column	403	9069	446	10077	958
Abutment B	419	0	492	2930	92

Frequent Event

Location	Longitudinal Shear (kips)	Longitudinal Moment (ft-kips)	Transverse Shear (kips)	Transverse Moment (ft-kips)	Axial (kips)
Abutment A	398	0	205	822	55
B1 Column	319	4787	170	2566	260
B2 Column	97	2177	119	2691	266
B3 Column	70	1759	129	3236	308
B4 Column	97	2179	129	2923	277
Abutment B	401	0	215	1003	22

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 5, Determine Design Forces

Design Step 5.2.2 (continued)

The definition of LC1 and LC2 for the 100 – 40 percent combination rule is as follows, from Guide Spec, Article 3.6.

LC1 = 40 percent of the Longitudinal Analysis Results + 100 percent of the Transverse Analysis Results

LC2 = 100 percent of the Longitudinal Analysis Results + 40 percent of the Transverse Analysis Results

Note that the definitions of LC1 and LC2 are reversed from the definitions used in Division I-A of the Standard Specifications. LC1 is now primarily transverse loading and LC2 is primarily longitudinal loading. Also, the contribution from the orthogonal earthquake component has been increased from 30 to 40 percent in the proposed provisions. This provides better accuracy in predicting elastic forces and displacements. Studies have indicated that the 40 percent contribution provides a better match with actual time history results than does the 30 percent value.

100%-40% Combination Rule

("x" and "z" refer to global axes)

$M_{xLC1} := 1.0 \cdot M_{xT} + 0.4 \cdot M_{xL}$ $M_{xLC1} = 7733\ \text{ft} \cdot \text{kips}$

$M_{xLC2} := 0.4 \cdot M_{xT} + 1.0 \cdot M_{xL}$ $M_{xLC2} = 3093\ \text{ft} \cdot \text{kips}$

$M_{zLC1} := 1.0 \cdot M_{zT} + 0.4 \cdot M_{zL}$ $M_{zLC1} = 9229\ \text{ft} \cdot \text{kips}$

$M_{zLC2} := 0.4 \cdot M_{zT} + 1.0 \cdot M_{zL}$ $M_{zLC2} = 20415\ \text{ft} \cdot \text{kips}$

To compare the differences between the two load combination methods for this bent, develop the elastic vector moment for the column and form the quotient of the vector moment obtained from the SRSS and 100-40 rules.

$M_{max\_LC1} := \sqrt{M_{xLC1}^2 + M_{zLC1}^2}$ $M_{max\_LC1} = 12040\ \text{ft} \cdot \text{kips}$

$M_{max\_LC2} := \sqrt{M_{xLC2}^2 + M_{zLC2}^2}$ $M_{max\_LC2} = 20648\ \text{ft} \cdot \text{kips}$

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 5, Determine Design Forces

Design Step 5.2.2 (continued)

Comparison of SRSS and 100%-40% Combination Rules

$$\frac{M_{SRSSmax}}{M_{max\_LC2}} = 0.98$$

For this design example, all seismic forces will be computed using the 100%-40% combination rule.

Table 11
Orthogonal Seismic Force Combinations
100% - 40% Rule / LC1 and LC2

MCE Event		Forces and Moments				
		Longitudinal		Transverse		
Location	Load Case	Shear, x (kips)	Moment, z (ft-kips)	Shear, z (kips)	Moment, x (ft-kips)	Axial (kips)
Abutment	LC1	166	0	437	1871	92
	LC2	416	0	175	748	230
Bent 1 Column	LC1	616	9229	513	7733	824
	LC2	1363	20415	205	3093	390
Bent 2 Column	LC1	200	4511	415	9368	960
	LC2	416	9363	166	3747	464
Bent 3 Column	LC1	129	3228	448	11235	1070
	LC2	298	7438	179	4494	429
Bent 4 Column	LC1	200	4516	446	10077	976
	LC2	417	9386	178	4031	430
Abutment	LC1	168	0	492	2930	37
	LC2	419	0	197	1172	92

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 5, Determine Design Forces

Design Step 5.2.2 (continued)

Table 11
Orthogonal Seismic Force Combinations
100% - 40% Rule / LC1 and LC2
(continued)

Frequent Event		Forces and Moments				
		Longitudinal		Transverse		
Location	Load Case	Shear, x (kips)	Moment, z (ft-kips)	Shear, z (kips)	Moment, x (ft-kips)	Axial (kips)
Abutment	LC1	159	0	205	822	22
	LC2	398	0	82	329	55
Bent 1 Column	LC1	152	2279	170	2566	266
	LC2	328	4921	68	1026	121
Bent 2 Column	LC1	50	1145	119	2691	274
	LC2	101	2271	48	1076	129
Bent 3 Column	LC1	32	804	129	3236	308
	LC2	72	1797	52	1294	124
Bent 4 Column	LC1	50	1130	129	2923	281
	LC2	101	2269	52	1169	122
Abutment	LC1	160	0	215	1003	9
	LC2	401	0	86	401	22

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 5, Determine Design Forces

Design Step 5.2.2 (continued)

These forces are combinations using the full elastic seismic results, and have not yet been modified by the R factor. (The R factor is discussed in Design Step 3.4.) At this stage, the designer could elect to compare these forces (as Extreme Event I when combined with dead load) with other load cases for the substructure design, to see if they control. If other load cases, such as stream flow or temperature control, the seismic design forces given in Table 10 could be used without further modification. However, in the spirit of capacity design, the seismic plastic mechanism should still be identified, even though its size is controlled by nonseismic loadings. Then the elements connecting with the likely yielding elements would still be designed to withstand the plastic hinging effects.

Design Step 5.3 Determine Modified Design Forces

For design of the primary members of the earthquake resisting system (i.e., those members that will experience inelastic action) modified design forces are developed. These forces are the elastic seismic forces 'modified' by the R factor combined with the other required loads of the Extreme Event I combination. These modified forces, along with the forces associated with plastic hinging in the columns, are used in the seismic design of the various components of the bridge.

The modified design forces use the R Factor in modifying the elastic seismic forces. Viewing the entire bridge as a system, the intent of the specification is to force the plastic hinging to occur in the columns. Therefore, inelastic action is prevented from occurring in the cap beam or foundation, where damage may not be detectable by visual inspection and may be very difficult or costly to repair.

Design Step 5.3.1

Modified Design Forces for Structural Members – MCE Event
[Guide Spec, Article 3.5] [NCHRP, Article 3.4.1]

$$\begin{aligned}\text{Extreme Event I Load} = \gamma_p{}^* &(DC+DD+DW+EH+EV+ES)+\\ &\gamma_{EQ}{}^*(LL+IM+CE+BR+PL+LS+EL)+\\ &1{}^*WA+1{}^*FR+1{}^*EQ\end{aligned}$$

For this example, forces DD, DW, EL, EH, ES, EV, WA, and FR are assumed zero, and only DC and EQ forces are combined. Additionally, as discussed above, γ_p is taken as 1.0 and γ_{EQ} is taken as 0. Making these substitutions, the equation reduces to

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 5, Determine Design Forces

Design Step 5.3.1 (continued)

Extreme Event I Load = 1.0 (DC + EQ)

where

EQ = (LC1 or LC2 forces) divided by R

a) Response Modification Reduction Factor, R
[Guide Spec, Article 4.7, Table 4.7-1]
[NCHRP, Article 3.10.3.7, Table 3.10.3.7.1-1]

In this example, R reduces the seismic column moments, but increases the seismic lateral shear force on the connection of the superstructure to the abutment. Recall that R_B was determined in Design Step 3.4.

The base value, R_B, is adjusted to obtain a final R value that is used in design. The adjustment accounts for the observation that structures with short periods tend to experience higher inelastic demands than the 'equal displacement' method of predicting inelastic demands indicates. To account for this increase, the R_B factor is decreased for periods shorter than $T_{star,}$, where this period is based on the break point in the response spectrum.

Determine the R factor to use in design. The base R factor from Table 4.7-1 is adjusted to account for short-period effects.

Consider the MCE case

$T_s := 0.933 \cdot sec$ Corner of spectrum from Design Step 2

$R_B := 6$ Basic R factor for SDAP E and Life Safety

$T := 1.38 \cdot sec$ Shorter period of longitudinal and transverse directions from Design Step 4.4

$T_{star} := 1.25 \cdot T_s$ $T_{star} = 1.17\ s$

$$R := 1 + (R_B - 1) \cdot \frac{T}{T_{star}} \qquad R = 6.9$$

However, R must be less than R_B; thus <u>R is 6</u>.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 5, Determine Design Forces

Design Step 5.3.1 (continued)

For the MCE Event:

R = 6 For moments in columns when SDAP E is used and the performance objective is Life Safety

b) Calculate the Modified Design Forces with EQ

Once the R values have been established, the value of EQ can be calculated.

Table 12 summarizes the modified design forces. The R values used for specific forces are shown.

For example, the Bent 1 longitudinal column moment using LC1 is derived as follows.

M = (DC + EQ/R)
M = (5 + 9229/6) = 1543 k-ft

All other forces in Table 12 are calculated similarly.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 5, Determine Design Forces

Design Step 5.3.1 (continued)

Table 12
Modified Design Forces for MCE Earthquake

R =	6	Column Moments
R =	1	Abutments, Column P & V

DL + (100%-40%)/R		Forces and Moments				
		Longitudinal		Transverse		
Location	Load Case	Shear (kips)	Moment (ft-kips)	Shear (kips)	Moment (ft-kips)	Axial (kips)
Abutment	LC1	166	0	414	1815	675
	LC2	416	0	166	726	813
Bent 1 Column	LC1	616	1543	513	1294	1587
	LC2	1363	3408	205	521	1153
Bent 2 Column	LC1	200	753	415	1565	1765
	LC2	416	1562	166	629	1269
Bent 3 Column	LC1	129	538	448	1877	1882
	LC2	298	1240	179	753	1241
Bent 4 Column	LC1	201	761	446	1683	1759
	LC2	418	1572	179	675	1213
Abutment	LC1	168	0	473	2897	630
	LC2	419	0	189	1159	685

Design Step 5.3.2

Modified Design Forces for Structural Members – Frequent Event
[Guide Spec, Article 3.5] [NCHRP, Article 3.4.1]

The same procedure as used for the MCE event is used for the Frequent event, the only exception is that a different R factor is used.

a) Recall the Response Modification Reduction Factor, R
[Guide Spec, Article 4.7] [NCHRP, Article 3.10.3.7]

For the Frequent Event:

$R = 1.3$ For moments in columns

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 5, Determine Design Forces

Design Step 5.3.2 (continued)

b) Calculate the Modified Design Forces with EQ

Table 13 summarizes the values of EQ modified design forces for the bent columns.

For example, the longitudinal Bent 1 column moment using LC1 is derived as follows.

M = (DC + EQ/R)
M = (5 + 2297/1.3) = 1758 kip-ft

All other forces in Table 13 are calculated similarly.

Table 13
Modified Design Forces for Frequent Earthquake

R = 1.3 All Elements

DL + (100%-40%)/R

Location	Load Case	Longitudinal Shear (kips)	Longitudinal Moment (ft-kips)	Transverse Shear (kips)	Transverse Moment (ft-kips)	Axial (kips)
Abutment	LC1	122	0	158	632	600
	LC2	306	0	63	253	625
Bent 1 Column	LC1	118	1758	131	1979	967
	LC2	253	3790	53	795	856
Bent 2 Column	LC1	39	881	92	2074	1016
	LC2	78	1748	37	832	904
Bent 3 Column	LC1	25	619	99	2493	1049
	LC2	55	1383	40	1000	907
Bent 4 Column	LC1	39	877	99	2251	999
	LC2	78	1753	40	902	877
Abutment	LC1	123	0	165	772	600
	LC2	308	0	66	309	610

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 6, Design Primary Earthquake Resisting Elements

DESIGN STEP 6 DESIGN PRIMARY EARTHQUAKE RESISTING ELEMENTS

This step includes the design of those elements that are intended to provide the energy dissipation for the structure during an earthquake. The objective of this design step is to develop enough of the system design that the capacity design forces, which will be required for the rest of the structure, can be developed.

For this example, the columns of the bents (intermediate piers) are the primary energy dissipation elements. In the longitudinal direction, the soil behind the abutments also is used to dissipate energy, but no design is required of that beyond the end diaphragm and the specification of the backfill material. Thus, this chapter deals only with the design of the flexural reinforcement of the columns.

Design Step 6.1 Preliminary Column Design

[Guide Spec, Articles 4.6, 8.8.2.1, and 8.8.2.2]
[NCHRP, Articles 3.10.3.6, 5.10.11.4.1a, and 5.10.11.4.1b]

The flexural design of the columns of Bent 3 (Pier 4) will be considered in this step. The forces listed are based on the 100-40 percent combination rule.

Below is a summary of the controlling Modified Design Forces for the preliminary column design, taken from Tables 12 and 13. On inspection , it can be seen that LC1 controls.

For MCE Non-Liquefied Condition:

$Pmax_u := 1882 \cdot kip$ Maximum axial load

$Pmin_u := -258 \cdot kip$ Minimum axial load

$M_L := 538 \cdot kip \cdot ft$ Longitudinal moment

$M_T := 1877 \cdot kip \cdot ft$ Transverse moment

For a circular column, the modified biaxial bending moment can be converted to a moment about a single axis by calculating that

$$M_u := \sqrt{{M_L}^2 + {M_T}^2} \qquad M_u = 1953\,kip \cdot ft$$

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**

Design Step 6, Design Primary Earthquake Resisting Elements

Design Step 6.1
(continued)

For Frequent Non-Liquefied Condition:

$Pmax_u := 1049 \cdot kip$ Maximum axial load

$Pmin_u := 575 \cdot kip$ Minimum axial load

$M_L := 619 \cdot kip \cdot ft$ Longitudinal moment

$M_T := 2493 \cdot kip \cdot ft$ Transverse moment

For a circular column, the modified biaxial bending moment can be converted to a moment about a single axis by calculating that

$$M_u := \sqrt{{M_L}^2 + {M_T}^2} \qquad M_u = 2569\,kip \cdot ft$$

The above forces will be used in the design of the longitudinal reinforcement in the column.

Try a 48-inch-diameter column with 20 #10 bars (1.4 percent reinforcement).

The column capacity curve in Figure 16 graphs the nominal capacity of Pn versus Mn. The forces for the four load cases calculated above are plotted on the curve in the figure.

Because the forces for both load cases plot inside the capacity curve for a column with 20 #10 bars, this reinforcement is sufficient. The 1.4 percent reinforcement provided is between 0.08 and 4 percent allowed (Article 8.8.2.1).

Note that the load combinations for the MCE nonliquefied case and those from the Frequent case actually both plot very near the interaction diagram. Even though the overall moment from the MCE case is less than that for the Frequent case, the MCE minimum case plots closer to the interaction diagram. It can be seen that the strength supplied is slightly greater than that required, and in fact the longitudinal steel could be reduced if necessary. It will not be for this example. Also, the axial forces used to select the longitudinal steel have not been reduced by the R factor; and, therefore, they are a bit larger than those that will actually occur when the bent reaches its plastic mechanism. This will be seen in Design Step 7 when the displacement capacity verification (pushover) is performed.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 6, Design Primary Earthquake Resisting Elements

Design Step 6.1 (continued)

Article 4.6 allows the design forces to be reduced to a minimum of 70 percent of the original design moments if a pushover is executed and the structure can meet the pushover displacement limits. This reduction is not taken in this example, but it may be useful to invoke if the capacity design of any elements becomes a problem. In other words, this reduction can reduce the capacity design forces and thereby reduce steel congestion.

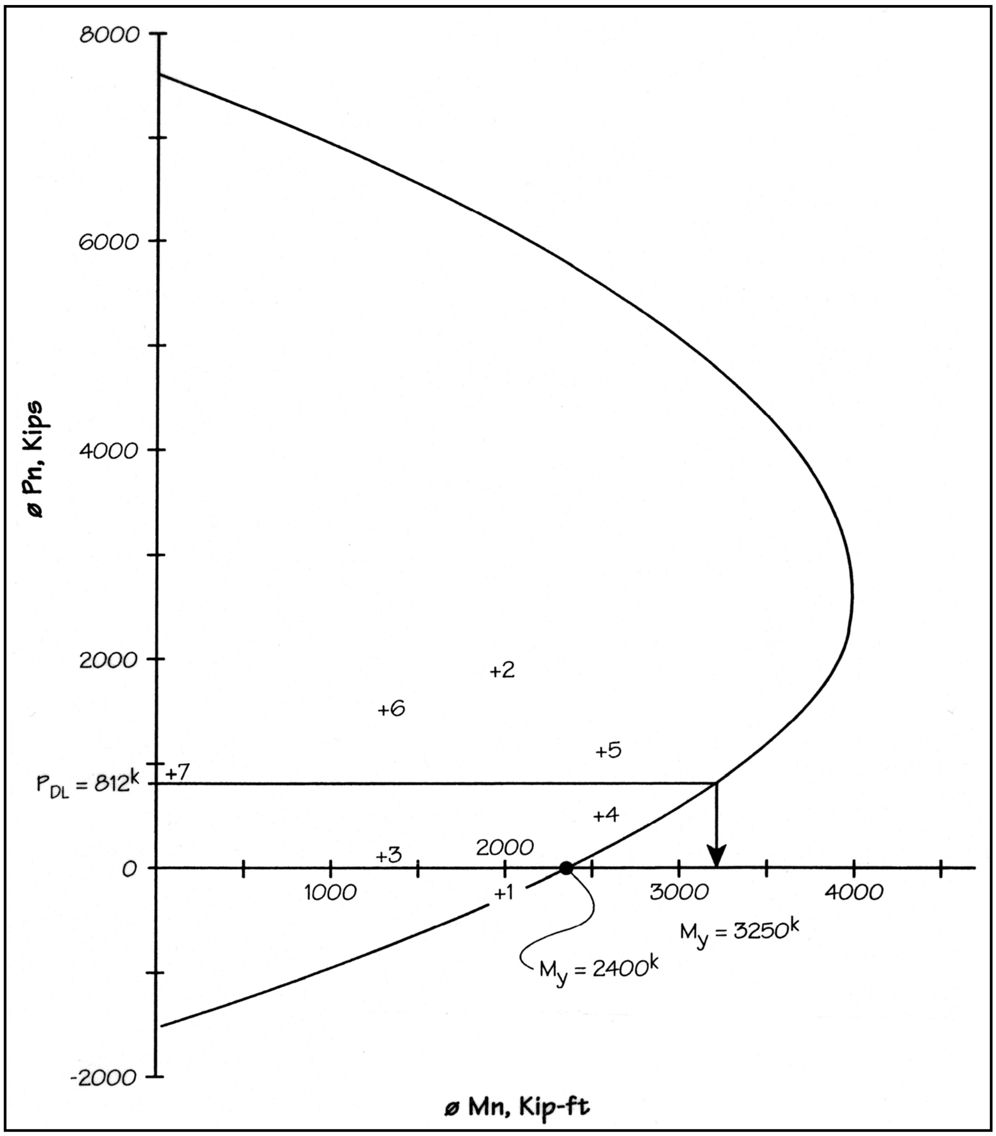


Figure 16 – Column Interaction Capacity Curve

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 7, Design Displacements and Checks

DESIGN STEP 7 DESIGN DISPLACEMENTS AND CHECKS

[Guide Spec, Article 8.3] [NCHRP, Article 3.10.3.10]

Design Step 7.1 Seat Widths

[Guide Spec, Article 8.3.2] [NCHRP, Article 3.10.3.10.2]

Compute Minimum Seat Width Required at the Abutment
[Guide Spec 8.3.2]

Data from "2500N" Sap2000 Model

$\Delta_L := 1.16 \cdot ft$ Longitudinal displacement demand at Abutment superstructure CG for longitudinal EQ (from Table 6)

$T_s := 0.933 \cdot sec$ Period of vibration at end of short period plateau from design response spectrum

$T_L := 1.37 \cdot sec$ Longitudinal period of vibration of SAP2000 bridge model

$R := 6$ Response modification factor for 2-column bent

First, determine short period modifier as defined by Eqn 8.3.4-3:

$T_{star} := 1.25 \cdot T_s$ Note: Tstar = T* within MathCad computations.

$T_{star} = 1.17\ sec$

Because $T^* < T_L$, R_d is unity.

$R_d := 1.0$ Short Period Modifier

Compute displacement at the seat:

$\Delta_{mL} := R_d \cdot \Delta_L$

$\Delta_{mL} = 1.16\ ft$

$1.5 \cdot R_d \cdot \Delta_{mL} = 1.74\ ft$ minimum seat width

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 7, Design Displacements and Checks

Design Step 7.1 (continued)

Seat width shall not be less than 1.74 ft

$L := 500 \cdot ft$ $L = 152.4\,m$ $L := 152.4$ distance btwn joints

$H := 50 \cdot ft$ $H = 15.24\,m$ $H := 15.24$ tallest pier btwn joints

$B := 43 \cdot ft$ $B = 13.11\,m$ $B := 13.11$ width of superstructure

$F_v := 2.4$ from Design Step 2.6

$S_1 := 0.411$ from Design Step 2.6

$\alpha := 0 \cdot deg$ skew angle

$$N := \left[0.10 + 0.0017 \cdot L + 0.007 \cdot H + 0.05\sqrt{H} \cdot \sqrt{1 + \left(\frac{2 \cdot B}{L}\right)^2}\right] \cdot \frac{(1 + 1.25 \cdot F_v \cdot S_1)}{\cos(\alpha)}$$

$N = 1.48$ meters

$N := 4.86$ feet minimum seat width

Seat width shall not be less than 4.86 ft

Therefore the minimum seat width is 58 inches.

Per Figure 1c, the abutment seat width provided is less than this. The abutment must be widened from 46 inches to 58 inches or the overhang must be extended.

Design Step 7.2 **Displacement Capacity Verification (SDAP E)**
[Guide Spec, 4.6, 8.3.5, and 5.4.3] [NCHRP, 3.10.3.6, 3.10.3.10.5, and 4.8.5.4]

For this example, only Bent 3 (Pier 4) will be checked using the Displacement Capacity Verification.

Design Step 7.2.1 Compute Modified Seismic Displacement Demand for MCE Event
[Guide Spec, Article 8.3.4] [NCHRP, Article 3.10.3.10.4]

Data from "2500N" Sap2000 Model

$\Delta_T := 1.72 \cdot ft$ Transverse displacement demand at Pier 4 superstructure CG for transverse EQ (from Table 6)

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 7, Design Displacements and Checks

Design Step 7.2.1 (continued)

$\Delta_L := 1.11 \cdot ft$ Longitudinal displacement demand at Pier 4 superstructure CG for longitudinal EQ (from Table 6)

$T_s := 0.933 \cdot sec$ Period of vibration at end of short period plateau from design response spectrum

$T_T := 1.62 \cdot sec$ Transverse period of vibration of SAP2000 bridge model

$T_L := 1.37 \cdot sec$ Longitudinal period of vibration of SAP2000 bridge model

$R := 6$ Response modification factor for 2-column bent

First, determine short period modifier as defined by Eqn 8.3.4-3:

$T_{star} := 1.25 \cdot T_s$

$T_{star} = 1.17\ sec$

Note: Tstar = T* within MathCad computations.

Because $T^* < T_T$ and $T^* < T_L$, R_d is unity for both the transverse and longitudinal analyses. Let $R_d = R_{dT} = R_{dL}$.

$R_d := 1.0$ Short Period Modifier

Compute modified displacements:

$\Delta_{mT} := R_d \cdot \Delta_T$

$\Delta_{mT} = 1.72\ ft$

Modified Seismic Displacement Demand for Transverse Earthquake

$\Delta_{mL} := R_d \cdot \Delta_L$

$\Delta_{mL} = 1.11\ ft$

Modified Seismic Displacement Demand for Longitudinal Earthquake

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 7, Design Displacements and Checks

Design Step 7.2.2

Minimum Displacement Requirement for Lateral Load Resisting Piers and Bents
[Guide Spec, Article 8.3.5] [NCHRP, Article 3.10.3.10.5]

For SDAP E, the displacement capacity must be greater than or equal to the following computed minimum displacement. The transverse displacement capacity is then determined using an approximate method and by a simple pushover analysis using SAP2000. The longitudinal direction is evaluated using the approximate method only.

$1.5 \cdot \Delta_{mT} = 2.58\ \text{ft}$ Minimum Displacement Requirement for the Transverse Earthquake

$1.5 \cdot \Delta_{mL} = 1.67\ \text{ft}$ Minimum Displacement Requirement for the Longitudinal Earthquake

Design Step 7.2.3

Plastic Rotational Capacity for Life-Safety Performance
[Guide Spec, Article 8.8.6] [NCHRP, Article 5.16]

First, estimate plastic hinge length:

$\varepsilon_y := 0.00207$ Yield Strain of the Column Longitudinal Reinforcement

$d_b := 1.41 \cdot \text{in}$ Diameter of Column Longitudinal Reinforcement (#11 bars)

$H_{col} := 50 \cdot \text{ft}$ Clear Height of Column

In the following equation for the effective plastic hinge length, the term for the "shear span" of the column, M/V, is replaced by $H_{col}/2$. For this example, a conservative assumption of a fixed-fixed column results in this simplification of the shear span, which is nearly the case.

$$L_p := 0.08 \cdot \frac{H_{col}}{2} + 4400 \cdot \varepsilon_y \cdot d_b$$

$L_p = 3.07\ \text{ft}$ Effective Plastic Hinge Length

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 7, Design Displacements and Checks

Design Step 7.2.3 (continued)

Now compute plastic rotational capacity of the hinges.

Recall the fundamental periods of vibration of the structure:

$T_T = 1.62\ \text{sec}$ Transverse period of vibration of SAP2000 bridge model

$T_L = 1.37\ \text{sec}$ Longitudinal period of vibration of SAP2000 bridge model

$$N_{fT} := 3.5 \cdot \left(\frac{T_T}{1 \cdot \text{sec}}\right)^{\frac{-1}{3}}$$

$N_{fT} = 2.98$

Estimated Number of Cycles of Loading Expected at the Maximum Displacement Amplitude for the Transverse Earthquake

[Note: The unit of seconds for the period is removed in the radical expression to get a unitless result for N_f.]

$$N_{fL} := 3.5 \cdot \left(\frac{T_L}{1 \cdot \text{sec}}\right)^{\frac{-1}{3}}$$

$N_{fL} = 3.15$

Estimated Number of Cycles of Loading Expected at the Maximum Displacement Amplitude for the Longitudinal Earthquake

Both N_{fT} and N_{fL} are within the acceptable range, which is between 2 and 10 cycles.

$\text{cover} := 2 \cdot \text{in}$ Concrete Clear Cover on Column

$d_{col} := 4 \cdot \text{ft}$ Diameter of Column

$d_b = 1.41 \cdot \text{in}$ Recall the Diameter of Column Longitudinal Reinforcement (#11 bars)

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 7, Design Displacements and Checks

Design Step 7.2.3 (continued)

$$D' := d_{col} - 2 \cdot cover - d_b$$

$$D' = 3.55 \cdot ft$$

Distance between the Outer Layers of the Column Longitudinal Reinforcement, equal to the center-to-center pitch

$$\Theta_{pT} := 0.11 \cdot \frac{L_p}{D'} \cdot \left(N_{fT}^{-0.5} \right)$$

Plastic Rotational Capacity of Hinges in the Transverse Direction

$$\Theta_{pT} = 0.0551 \cdot rad$$

$$\Theta_{pL} := 0.11 \cdot \frac{L_p}{D'} \cdot \left(N_{fL}^{-0.5} \right)$$

Plastic Rotational Capacity of Hinges in the Longitudinal Direction

$$\Theta_{pL} = 0.0536 \cdot rad$$

Design Step 7.2.4

Approximate Check of Maximum Transverse and Longitudinal Displacements

The plastic rotational capacities can now be used to estimate the overall translational capacity of the pier column hinges. This is an approximate check that can be performed by hand to check the actual pushover analysis results that will be generated in the next step.

The plastic translational capacity can be determined assuming the fixed-fixed end condition of the columns, which results in the following formulation used for typical plastic hinge framing of the two-column pier.

Recall: $H_{col} = 50.00\ ft$ $L_p = 3.07\ ft$

$$\Delta_{pT} := \Theta_{pT} \cdot \left[H_{col} - 2 \cdot \left(\frac{L_p}{2} \right) \right]$$

Plastic Translational Capacity for the Transverse Earthquake

$$\Delta_{pT} = 2.587\ ft$$

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 7, Design Displacements and Checks

Design Step 7.2.4 (continued)

$$\Delta_{pL} := \Theta_{pL} \cdot \left[H_{col} - 2 \cdot \left(\frac{L_p}{2} \right) \right]$$

Plastic Translational Capacity for the Longitudinal Earthquake

$$\Delta_{pL} = 2.516\ ft$$

Now estimate the elastic translational capacity.

$M_y := 2400 \cdot ft \cdot kips$ Yield Moment of Column, taken from Interaction Diagram where P = 0 kips.

$E_c := 3830 \cdot ksi$ Young's Modulus of Elasticity for Concrete [LRFD 5.4.2.4]

$I_{cr} := 5 \cdot ft^4$ Moment of Inertia of Column based on Cracked Section

$H_{col} = 50.00\ ft$ Recall: Clear Height of Column

$$\Delta_y := \frac{M_y \cdot H_{col}^{\ 2}}{6 \cdot E_c \cdot I_{cr}}$$

Approximate Yield Displacement

$$\Delta_y = 0.363\ ft$$

$$\Delta_{capacityT} := \Delta_y + \Delta_{pT}$$

$$\Delta_{capacityT} = 2.949\ ft$$

Approximate Maximum Displacement Capacity in Transverse Direction

$\Delta_{capacityT} > 1.5 \cdot \Delta_{mT} = 2.58\ ft$ therefore OK

$$\Delta_{capacityL} := \Delta_y + \Delta_{pL}$$

$$\Delta_{capacityL} = 2.878\ ft$$

Approximate Maximum Displacement Capacity in Longitudinal Direction

$\Delta_{capacityL} > 1.5 \cdot \Delta_{mL} = 1.67\ ft$ therefore ok

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 7, Design Displacements and Checks

Design Step 7.2.5

Pushover Analysis – Seismic Displacement Capacity Verification
[Guide Spec, Article 5.4.3] [NCHRP, Article 4.8.5.4]

The pushover analysis was performed on Bent 3, isolated directly from the elastic seismic model, to determine the transverse lateral load-displacement behavior of the bent. SAP2000 was used to perform the pushover analysis, because it has the capability to perform such analysis relatively simply.

The plastic hinge lengths, Lp, were calculated as shown above and additional nodes were placed at Lp/2 from the top and bottom of the columns. Figure 17 shows the pushover model that was isolated from the elastic model.

The axial load-moment interaction diagram for the column with 1.4 percent reinforcement was input as the yield property for the hinges. The only members allowed to yield were the columns at the nodes located at the center of the plastic hinges. Because capacity design procedures will be used to design the cap beams and foundations, these elements are not allowed to yield. This simplifies the input data required for the pushover.

Recall that the target displacement demand for the transverse pushover analysis was

$1.5 \cdot \Delta_{mT} = 2.58\ \text{ft}$ Minimum Displacement Requirement for the Transverse Earthquake

Figure 18 shows the pushover behavior of the bent, up to the target displacement. The target displacement is reached at Step 11 of the analysis. Selected output tables from the analysis are given in Appendix C. The following data was extracted from the SAP2000 pushover analysis.

At the Pmin column, at Step 11:

$\Theta_{ptop} := .0441 \cdot \text{rad}$ $\Theta_{pbot} := .0446 \cdot \text{rad}$

$V_{e1} := 121.6 \cdot \text{kips}$ column plastic shear

At the Pmax column, at Step 11:

$\Theta_{ptop} := .0419 \cdot \text{rad}$ $\Theta_{pbot} := .0429 \cdot \text{rad}$

$V_{e2} := 145.6 \cdot \text{kips}$ column plastic shear

SECTION III BRIDGE WITH TWO-COLUMN BENTS

Design Step 7, Design Displacements and Checks

Design Step 7.2.5 (continued)

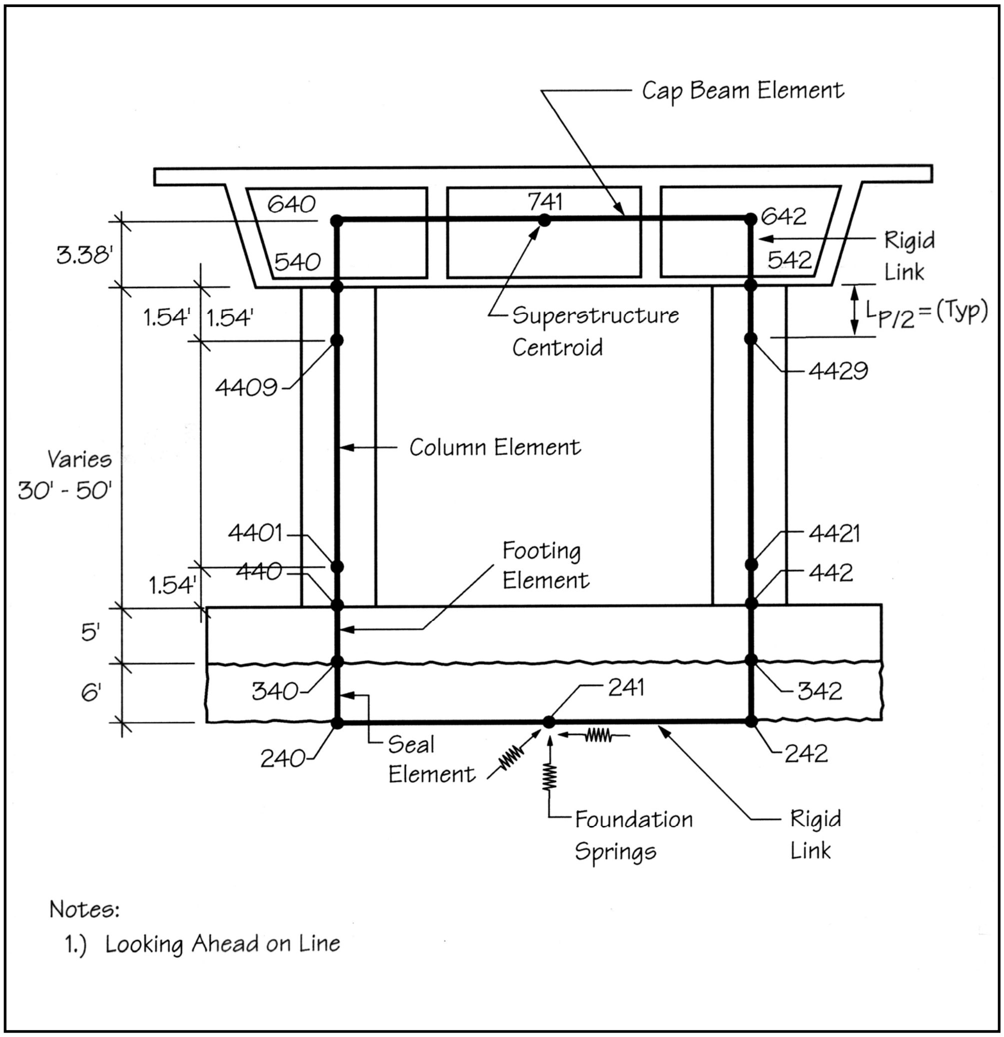


Figure 17 – Details of Pushover Model Elements

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 7, Design Displacements and Checks

Design Step 7.2.5 (continued)

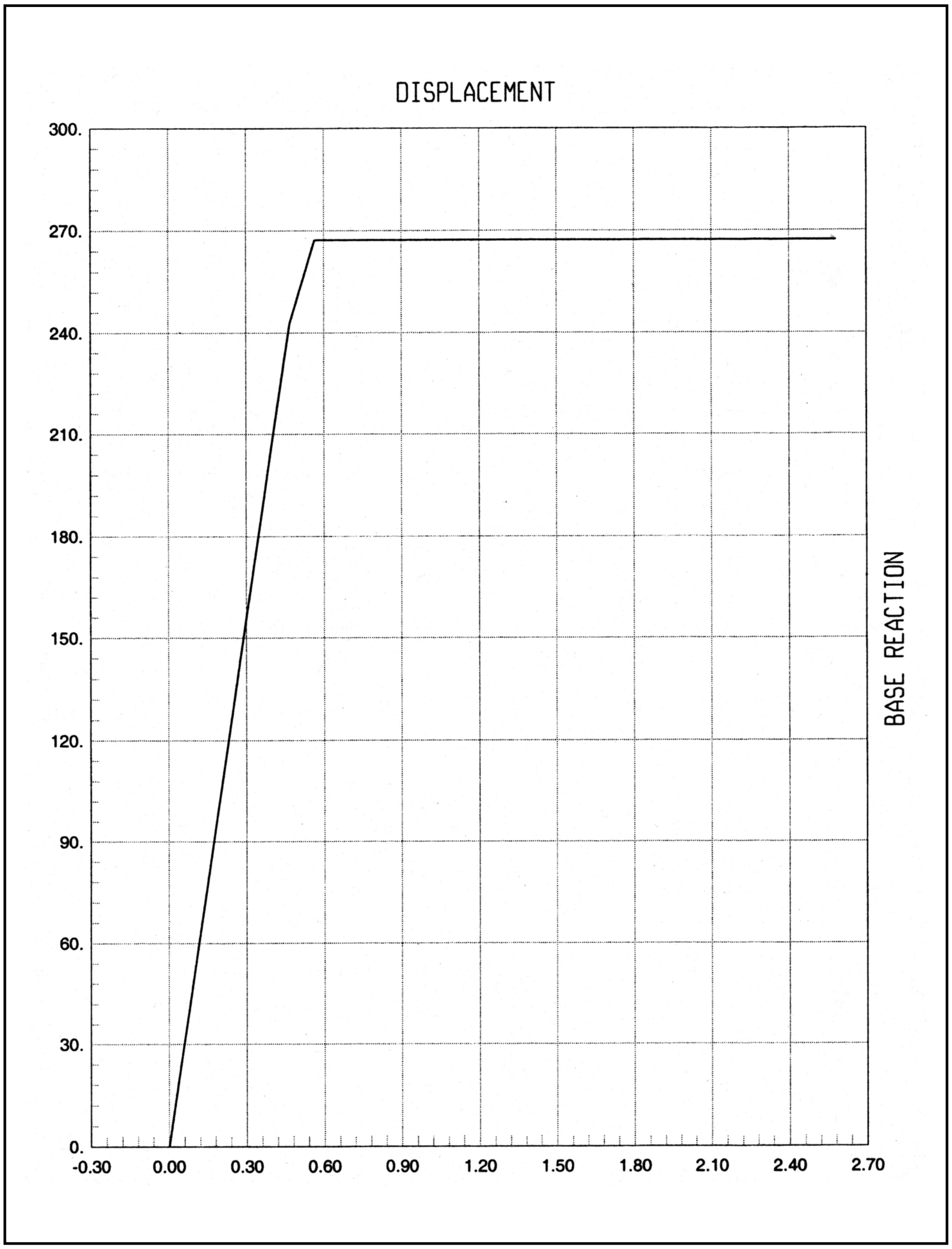


Figure 18 – Pushover Curve

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 7, Design Displacements and Checks

Design Step 7.2.5 (continued)

$\Theta_{pT} = 0.0551\,rad$ Recall Plastic Hinge Rotational Capacity

Predicted plastic hinge rotations are less than plastic rotational capacities calculated in Step 7.2.3. Therefore, the design is acceptable.

Note that in lieu of the more precise analysis above, the designer can assume a conservative value of the θ_p= 0.035 rad for the life-safety performance category. In the case of this example, a 50 percent increase in capacity is gained by using the computed method over the flat value assumption. As the pushover model shows, the value of the computed method is needed to pass the minimum displacement demand.

Design Step 7.3

P-Δ Requirements
[Guide Spec, Article 8.3.4] [NCHRP, Article 3.10.3.10.4]

Check the limit for Modified Seismic Displacement Demands.

$W := 1530 \cdot kips$ Weight of Participating Mass in the Response of the Pier (taken as the average of the column top and bottom axial forces for dead load).

$P := W$ Participating mass is approximately equal to the tributary weight for this bridge.

$V_{supplied} := 267.2 \cdot kips$ Actual Plastic Shear Developed in the Pier

$H_{col} := 50 \cdot ft$ Clear Height of Pier 4 Column

$$C := \frac{V_{supplied}}{W}$$ Seismic Coefficient Based on Lateral Strength

$C = 0.17$ Note that we are not using overstrength to calculate C.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 7, Design Displacements and Checks

Design Step 7.3
(continued)

$$\Delta_{m\_limit} := 0.25 \cdot C \cdot \left(\frac{W}{P}\right) \cdot H_{col}$$

$$\Delta_{m\_limit} = 2.18 \text{ ft}$$ Modified Seismic Displacement Limit

The modified seismic displacements for the transverse and longitudinal earthquakes, 1.72 and 1.11 feet, respectively, are less that the limit. Therefore, use of computed modified displacements is appropriate.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 8, Design Structural Components

DESIGN STEP 8 DESIGN STRUCTURAL COMPONENTS

This step includes the design of the structural (i.e., nonfoundation or abutment) components. Most of these elements will be designed or checked against the capacity design forces developed for each bent. If the design were being done for a lower seismic hazard zone, then the capacity design process may not be required. For this example, it is required.

Design Step 8.1 Seismic Detailing Requirements

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3]

Article 3.7 provides requirements for detailing as a function of the seismic hazard level. The detailing becomes more comprehensive as the hazard level increases. For SDR 4, the material design articles in Chapter 8 include specific and often prescriptive requirements to assure adequate detailing. In addition, the detailing provisions include such items as when capacity design is required versus only suggested.

For this example, the transverse reinforcing steel for the columns of Bent 3 will be designed, as will the connection reinforcement for the integral cap beam of Bent 3.

Design Step 8.2 Transverse Steel in Columns and Walls

[Guide Spec, Article 8.8.2] [NCHRP, Article 5.10.11.4.1]

The transverse steel will be designed by the two approaches included in 8.8.2.3 of the proposed provisions. The first method is an implicit method where no direct calculation of the plastic shear demand is required. This method is new to these provisions, and it is included here for comparison and demonstration purposes. Method 2, which is the explicit approach and does require a direct shear demand calculation, is the primary method for use with this example. This is because Method 2 is required when SDAP E is used.

The design of the transverse steel in columns and walls includes three parts: 1) shear strength, 2) confinement, and 3) anti-buckling restraint. Shear and confinement requirements have traditionally been part of the provisions, while the anti-buckling provisions are new. All the provisions have been made more comprehensive than those used previously, and therefore, they appear more complex.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 8, Design Structural Components

Design for Bent 3

$L := 50 \cdot ft$ Column Height

$D := 4 \cdot ft$ Column Diameter

$\phi := 0.90$ Strength Reduction Factor for Shear

Design Step 8.2.1

Method 1: Implicit Shear Detailing Approach
[Guide Spec, Article 8.8.2.3] [NCHRP, Article 5.10.11.4.1c]

In potential plastic hinge zones:

$K_{shape} := 0.32$ Circular Section

$\rho_t := 0.014$ Longitudinal Steel Content

$D' := 41.48 \cdot in$ Circle Diameter of Longitudinal Reinforcment

$D'' := 43.375 \cdot in$ Spiral Diameter

$\alpha := \frac{D'}{L}$ $\alpha = 3.961\,deg$

$f_{yh} := 60 \cdot ksi$ Yield Strength of Spiral

$f_{su} := 1.5 \cdot f_{yh}$ $f_{su} = 90\,ksi$

$\Lambda := 2$ Fixity Factor = 2 for Fixed-Fixed Conditions

$A_g := \frac{\pi \cdot D^2}{4}$ Cross-sectional Area of Column

$A_v := 0.8 \cdot A_g$ Shear Area of Concrete $A_v = 1448\,in^2$

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 8, Design Structural Components

Design Step 8.2.1 (continued)

The transverse steel content is obtained by solving simultaneous equations in terms of the steel ratio, ρ_v, and the crack angle, θ. Because these include a trigonometric function for θ, it is easier to solve these by trial and error.

As specified in this section, the maximum spiral spacing shall not exceed 10 inches.

Guess $s := 10 \cdot in$ and $A_{bh} := 0.31 \cdot in^2$

$$\rho_v := \frac{2 \cdot A_{bh}}{s \cdot D''} \qquad \rho_v = 0.00143$$

$$\theta := \text{atan}\left[\left(\frac{1.6 \cdot \rho_v \cdot A_v}{\Lambda \cdot \rho_t \cdot A_g}\right)^{0.25}\right]$$

$\theta = 26.8\,deg$ — θ calculated must be greater than or equal to 25 deg.

Recalculate ρ_v based on θ

$$\rho_v := K_{shape} \cdot \Lambda \cdot \left(\frac{\rho_t \cdot f_{su} \cdot A_g}{\phi \cdot f_{yh} \cdot A_v}\right) \cdot \tan(\theta) \cdot \tan(\alpha)$$

$$\rho_v = 0.000654$$

$$A_{bh} := \rho_v \cdot s \cdot \frac{D''}{2} \qquad A_{bh} = 0.1417\,in^2$$

Area of spiral req'd for shear.

#5 spiral at 10-inch pitch is adequate in the potential plastic hinge zone.

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 8, Design Structural Components

Design Step 8.2.1 (continued)

The transverse steel content is obtained by solving simultaneous equations in terms of the steel ratio, ρ_v, and the crack angle, θ. Because these include a trigonometric function for θ, it is easier to solve these by trial and error. Outside the potential plastic hinge zone:

$$f'c := 4000 \cdot psi$$

$$v_c := 2 \cdot \sqrt{f'c} \qquad v_c := .126 \cdot ksi$$

Outside the plastic hinge zone, the amount of transverse reinforcement can be reduced to account for some contribution of the concrete in shear resistance.

$$\rho_{vstar} := \rho_v - \frac{v_c}{f_{yh}} \qquad \rho_{vstar} = -0.0014$$

Because the amount is negative, the contribution of the concrete is more than sufficient to carry the shear.

No spiral is needed outside the plastic hinge zone.

A #5 spiral at a pitch of 10 inches would be adequate to satisfy the implicit detailing in the plastic hinge zone only. As will be seen, the confinement and anti-buckling provisions will control over the shear requirements.

Design Step 8.2.2

Method 2: Explicit Shear Detailing Approach – Pmax Column
[Guide Spec, Article 8.8.2.3] [NCHRP, Article 5.10.11.4.1c]

This method is required because SDAP E is used.

$$f'c := 4000 \cdot psi$$

Check Shear and Transverse Reinforcement for Pmax Column (i.e., column with higher compression):

Inside potential plastic hinge zones:

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 8, Design Structural Components

Design Step 8.2.2 (continued)

From the Displacement Capacity Verification, which was conducted for plastic moments not amplified by the overstrength factor,

$P_d := 812 \cdot kip$

$P_e := 1146 \cdot kip$

$M_{p\_top} := 3619 \cdot ft \cdot kip$ $\qquad M_{p\_bot} := 3662 \cdot ft \cdot kip$

Approximate the overstrength effects simply as 1.5 times the forces from the verification.

$OS := 1.5$ Overstrength Factor

$P_e := P_d + OS \cdot (P_e - P_d)$ $\qquad P_e = 1313\, kip$

$M_{p\_top} := OS \cdot M_{p\_top}$ $\qquad M_{p\_top} = 5429\, ft \cdot kip$

$M_{p\_bot} := OS \cdot M_{p\_bot}$ $\qquad M_{p\_bot} = 5493\, ft \cdot kip$

$V_u := \dfrac{(M_{p\_top} + M_{p\_bot})}{L}$ $\qquad V_u = 218\, kip$

For shear resistance in the end regions,

$V_c := 0.6 \cdot \sqrt{f'c} \cdot A_v$ $\qquad V_c := 54.9 \cdot kip$

$V_p := \dfrac{\Lambda \cdot P_e \cdot \tan(\alpha)}{2}$ $\qquad V_p = 91\, kip$

$V_s := \dfrac{V_u}{\phi} - V_c - V_p$ $\qquad V_s = 96.9\, kip$

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 8, Design Structural Components

Design Step 8.2.2 (continued)

Guess $s := 18 \cdot in$ and $A_{bh} := 0.31 \cdot in^2$

(We will neglect the spacing limit of 10 inches given in the implicit section for now)

$$\rho_v := \frac{2 \cdot A_{bh}}{s \cdot D''} \qquad \rho_v = 0.00079$$

$$\theta := atan\left[\left(\frac{1.6 \cdot \rho_v \cdot A_v}{\Lambda \cdot \rho_t \cdot A_g}\right)^{0.25}\right] \qquad \theta = 23.6\,deg$$

$\tan(\theta) = 0.436$ $\tan(\alpha) = 0.069$

Because $\tan(\theta)$ is greater than $\tan(\alpha)$, use $\tan(\theta)$ to calculate A_{bh}.

For a circular section:

$$A_{bh} := \left(\frac{2}{\pi}\right) \cdot \left(\frac{V_s \cdot s}{f_{yh} \cdot D''}\right) \cdot \left(\frac{1}{\cot(\theta)}\right) \qquad A_{bh} = 0.1862\,in^2$$

A #5 spiral with a pitch of 18 inches is more than is required for shear in the end region of the Pmax column.

Outside the plastic hinge zone:

$$V_c := 2.0 \cdot \sqrt{f'c} \cdot A_v \qquad V_c := 183 \cdot kip$$

Thus, the spiral spacing can be much greater than 18 inches outside the plastic hinge zone.

Per LRFD, Article 5.10.6.2, the spiral spacing for a compression member shall not exceed 6 inches. Therefore, #5 spiral at a pitch of 6 inches will be used for shear throughout the column height.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 8, Design Structural Components

Design Step 8.2.3

Transverse Reinforcement for Confinement at Plastic Hinges – Pmax Column
[Guide Spec, Article 8.8.2.4] [NCHRP, Article 5.10.11.4.1d]

$f_y := 60 \cdot ksi$

$U_{sf} := 15.95 \cdot ksi$ strain energy capacity (modulus of toughness) of transverse reinforcement = 110 MPa.

$A_{bh} := 0.31 \cdot in^2$

$$A_c := \frac{\pi \cdot (D'')^2}{4}$$

Spacing per (8.8.2.6): $s := 6 \cdot in$ maximum (150 mm)

For #5 spiral at 6 in: $\rho_s := \frac{4 \cdot A_{bh}}{s \cdot D''}$ $\rho_s = 0.0048$ provided

Check requirements for volumetric ratio of spiral reinforcement:

$$\rho_s := 0.008 \cdot \frac{f'c}{U_{sf}} \cdot \left[12 \cdot \left(\frac{P_e}{f'c \cdot A_g} + \frac{\rho_t \cdot f_y}{f'c} \right)^2 \cdot \left(\frac{A_g}{A_c} \right)^2 - 1 \right]$$

$\rho_s = 0.0035$ min'm

$s := \frac{4 \cdot A_{bh}}{D'' \cdot \rho_s}$ $s = 8.1104\,in$ maximum

A #5 spiral with a pitch of 6 inches is adequate for confinement in the end region of the Pmax column.

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 8, Design Structural Components

Design Step 8.2.4

Method 2: Explicit Shear Detailing Approach – Pmin Column
[Guide Spec, Article 8.8.2.3] [NCHRP, Article 5.10.11.4.1c]

Check Shear and Transverse Reinforcement for Pmin Column (i.e., column with lower compression):

Inside the potential plastic hinge zone:

From the Displacement Capacity Verification, which was conducted for plastic moments not amplified by the overstrength factor,

$P_d := 812 \cdot kip$

$P_e := 385 \cdot kip$

$M_{p\_top} := 2992 \cdot ft \cdot kip$ $\quad M_{p\_bot} := 3088 \cdot ft \cdot kip$

Approximate the overstrength effects simply as 1.5 times the forces from the verification.

$OS := 1.5$ Overstrength Factor

$P_e := P_d + OS \cdot (P_e - P_d)$ $\quad P_e = 171.5\, kip$ (C)

$M_{p\_top} := OS \cdot M_{p\_top}$ $\quad M_{p\_top} = 4488\, ft \cdot kip$

$M_{p\_bot} := OS \cdot M_{p\_bot}$ $\quad M_{p\_bot} = 4632\, ft \cdot kip$

$$V_u := \frac{(M_{p\_top} + M_{p\_bot})}{L} \qquad V_u = 182.4\, kip$$

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 8, Design Structural Components

Design Step 8.2.4 (continued)

For shear resistance in the end regions,

$$V_c := 0.6\cdot\sqrt{f'c}\cdot A_v \qquad V_c := 54.9\cdot kip$$

$$V_p := \frac{\Lambda\cdot P_e\cdot\tan(\alpha)}{2} \qquad V_p = 12\ kip$$

$$V_s := \frac{V_u}{\phi} - V_c - V_p \qquad V_s = 135.89\ kip$$

Guess $s := 18\cdot in$ and $A_{bh} := 0.31\cdot in^2$

$$\rho_v := \frac{2\cdot A_{bh}}{s\cdot D''} \qquad \rho_v = 0.00079$$

$$\theta := atan\left[\left(\frac{1.6\cdot\rho_v\cdot A_v}{\Lambda\cdot\rho_t\cdot A_g}\right)^{0.25}\right] \qquad \theta = 23.58\ deg$$

$$\tan(\theta) = 0.436 \qquad \tan(\alpha) = 0.069$$

$$A_{bh} := \left(\frac{2}{\pi}\right)\cdot\left(\frac{V_s\cdot s}{f_{yh}\cdot D''}\right)\cdot\left(\frac{1}{\cot(\theta)}\right) \qquad A_{bh} = 0.261\ in^2$$

A #5 spiral with a pitch of 18 inches is more than is required for shear in the end region of the Pmin column. As already shown, the spiral spacing can be much greater than 18 inches outside the plastic hinge zone.

Per LRFD, Article 5.10.6.2, the spiral spacing for a compression member shall not exceed 6 inches. Therefore, #5 spiral at a pitch of 6 inches will be used for shear throughout the column height.

SECTION III BRIDGE WITH TWO-COLUMN BENTS

Design Step 8, Design Structural Components

Design Step 8.2.5

Transverse Reinforcement for Confinement at Plastic Hinges – Pmin Column

[Guide Spec, Article 8.8.2.4] [NCHRP, Article 5.10.11.4.1d]

$f_y := 60 \cdot ksi$

$U_{sf} := 15.95 \cdot ksi$ strain energy capacity (modulus of toughness) of transverse reinforcement = 110 MPa.

$A_{bh} := 0.31 \cdot in^2$

$$A_c := \frac{\pi \cdot (D'')^2}{4}$$

Spacing per (8.8.2.6): $s := 6 \cdot in$ maximum (150 mm)

For #5 spiral at 6 in: $\rho_s := \frac{4 \cdot A_{bh}}{s \cdot D''}$ $\rho_s = 0.0048$ provided

Check requirements for volumetric ratio of spiral reinforcement:

$$\rho_s := 0.008 \cdot \frac{f'c}{U_{sf}} \cdot \left[12 \cdot \left(\frac{P_e}{f'c \cdot A_g} + \frac{\rho_t \cdot f_y}{f'c} \right)^2 \cdot \left(\frac{A_g}{A_c} \right)^2 - 1 \right]$$

$\rho_s = -0.00003$ min'm

A #5 spiral with a pitch of 6 inches is required for confinement in the end region of the Pmin column.

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 8, Design Structural Components

Design Step 8.2.6

Anti-Buckling Steel
[Guide Spec, Article 8.8.2.5] [NCHRP, Article 5.10.11.4.1e]

Transverse Reinforcement for Longitudinal Bar Restraint in Plastic Hinges - anti-buckling steel (8.8.2.5) for both columns

$d_b := 1.25 \cdot in$ bar diameter of #10 longitudinal steel

$s := (6 \cdot d_b)$ $s = 7.5\,in$ does not control because shear requires a smaller s

$s := 6 \cdot in$

Check requirement for volumetric ratio of spiral reinforcement:

Where global buckling is required to be inhibited to ensure post earthquake repairability (not required for this example):

$$\rho_s := 0.024 \cdot \frac{D}{s} \cdot \frac{s}{d_b} \cdot \rho_t \cdot \frac{f_y}{f_{yh}} \qquad \rho_s = 0.0129$$

$$s := \frac{4 \cdot A_{bh}}{D'' \cdot \rho_s} \qquad s = 2.2157\,in$$

to prevent global buckling (buckling over several spiral pitches) of longitudinal reinforcement

Where some global buckling of the longitudinal bars is tolerated but the yield force of the longitudinal bar is to be maintained for life-safety:

$$\rho_s := 0.016 \cdot \frac{D}{s} \cdot \frac{s}{d_b} \cdot \rho_t \cdot \frac{f_y}{f_{yh}} \qquad \rho_s = 0.0086$$

controls

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 8, Design Structural Components

Design Step 8.2.6 (continued)

$$s := \frac{4 \cdot A_{bh}}{D'' \cdot \rho_s} \qquad s = 3.3236\,in \qquad \text{controls}$$

A #5 spiral with a pitch of 3.25 inches is required for confinement and anti-buckling in the end regions of both columns. Repairability assumed not to be a priority for Life-Safety Performance.

At the shortest pier, Bent 1, the longitudinal steel ratio is larger, because the columns are stiffer. The pitch will be even less per this criteria. This column has been designed with ρ_t of 2.4 percent (28 #11). Therefore, the pitch will be

$$s := s \cdot \frac{1.56}{1.27} \cdot \frac{.014}{.024} \qquad s = 2.3814\,in$$

A #5 spiral with a pitch of 2.25 inches is required for confinement and anti-buckling in the end regions of these columns. If we use #6 spiral, the pitch will be 3.38 inches, slightly more reasonable.

Bundled spiral would provide some additional space. However, for reinforcement ratios greater than 2.5 percent, this requirement will become nearly impossible to accommodate for life safety - repairability will be worse yet.

Because anti-buckling reinforcement is directly proportional to ρ_t, it will be important in design of the columns to provide only enough longitudinal steel to just meet the design forces. Excess steel will penalize the spiral spacing.

Design Step 8.2.7

Extent of Shear Steel, Confinement, and Anti-Buckling Steel
[Guide Spec, Articles 8.8.2.6 and 4.9]
[NCHRP, Articles 5.10.11.4.1f and 3.10.3.9]

Extent of end region from the top and bottom of column shall be a distance taken as the greater of:

$D = 48\,in$ Maximum cross-sectional dimension of column

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 8, Design Structural Components

Design Step 8.2.7 (continued)

$\frac{L}{6} = 100\,\text{in}$ One-sixth the column clear height

$450\cdot\text{mm} = 17.7\,\text{in}$

$D\cdot\left(\cot(\theta) + \frac{\tan(\theta)}{2}\right) = 120.4\,\text{in}$ $117.6\cdot\text{in} = 9.8\,\text{ft}$

$\frac{M}{V}\cdot\left(1 - \frac{M_y}{M_{po}}\right)$ can be taken as

$M_{p\_bot} := 5493\cdot\text{kip}\cdot\text{ft}$

$$\frac{M_{p\_bot}}{V_u}\cdot\left[1 - \frac{0.85\cdot\left(\frac{M_{p\_bot}}{1.5}\right)}{M_{p\_bot}}\right] = 13\,\text{ft}$$

$1.5\cdot\left(4400\cdot\varepsilon_y\cdot d_b + 0.08\cdot\frac{M}{V}\right)$ can be taken as

$\varepsilon_y := .00207$

$$1.5\cdot\left(4400\cdot\varepsilon_y\cdot d_b + 0.08\cdot\frac{M_{p\_bot}}{V_u}\right) = 5.04\,\text{ft}$$

Therefore, end regions are 13 feet long at each end of each column.

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 8, Design Structural Components

Design Step 8.2.8

Summary of Transverse Steel Design

A summary of the transverse steel design of the Bent 3 columns is shown in Table 14 and Figure 19. The end regions of the column extend 13 feet from the top and bottom of the column. The anti-buckling restraint requires a #5 spiral at a 3.25-inch pitch over the end regions. In the center region, the spiral pitch is set at 6 inches, the maximum limit for a spiral reinforced compression member.

Table 14
Column Transverse Steel Design Summary

Bent 3	Spiral	Location
Implicit Shear Detailing	#5 @ 10 inches	Inside the potential plastic hinge zone
	None	Outside the potential plastic hinge zone
Explicit Shear – Pmax	#5 @ 18 inches Vc = 55 kips, Vp = 91 kips, Vs = 97 kips	L_{end} = 13 feet
Confinement – Pmax	#5 @ 6 inches	L_{end} = 13 feet
Explicit Shear – Pmin	#5 @ 18 inches Vc = 55 kips, Vp = 12 kips, Vs = 136 kips	L_{end} = 13 feet
Confinement – Pmin	#5 @ 6 inches	L_{end} = 13 feet
Maximum Spiral Spacing in Compression Member	#5 @ 6 inches	
Anti-Buckling Steel	#5 @ 3.25 inches	$\rho_t = 1.4\%$ #10 bars

Bent 1	Spiral	Location
Anti-Buckling Steel	#5 @ 2.25 inches or #6 @ 3.25 inches	$\rho_t = 2.4\%$ #11 bars

SECTION III BRIDGE WITH TWO-COLUMN BENTS

Design Step 8, Design Structural Components

Design Step 8.2.8 (continued)

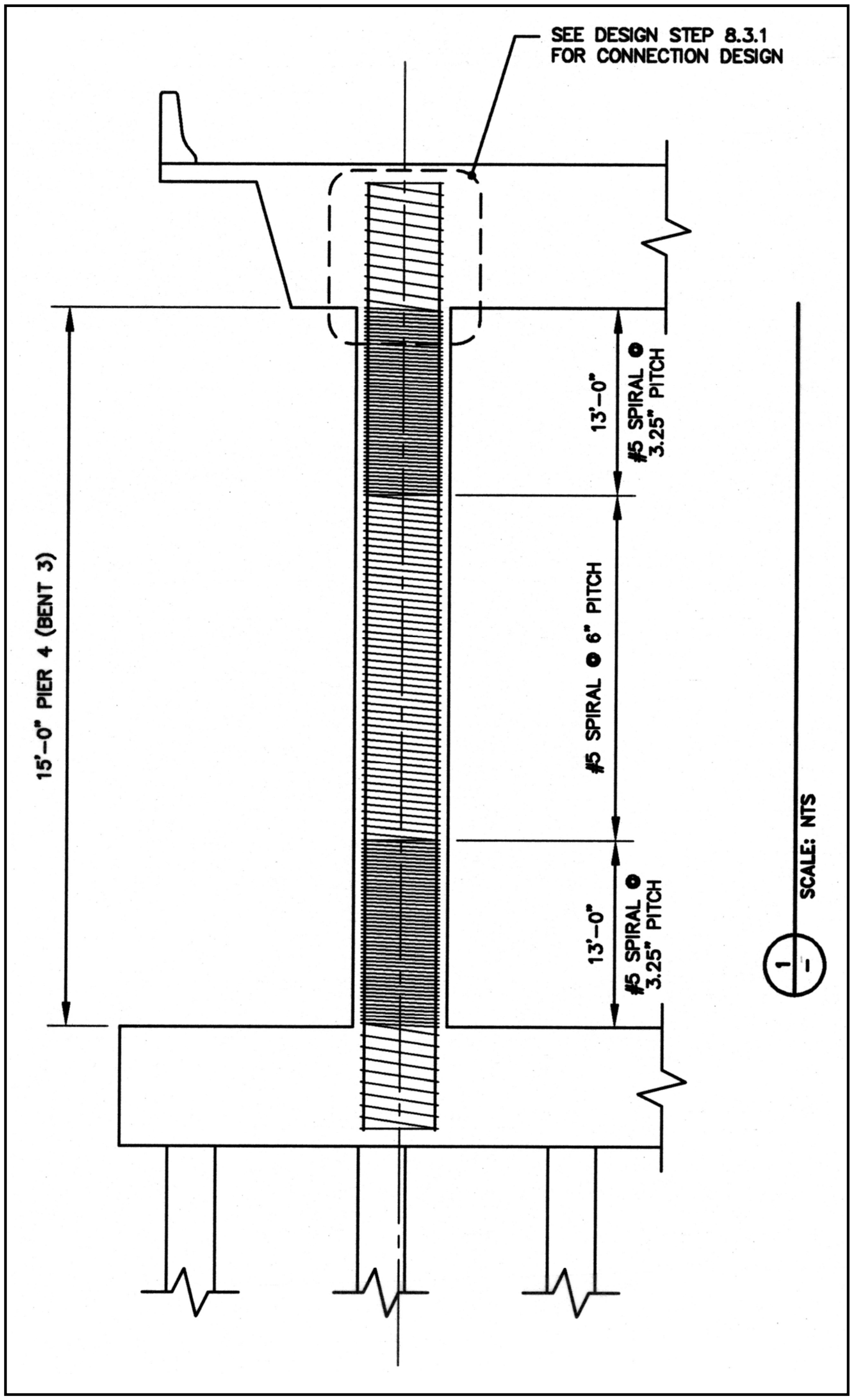


Figure 19 – Column Transverse Reinforcement Summary

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 8, Design Structural Components

Design Step 8.3

Connections, Shear Keys, Joint Designs, Restrainers, and Bearings
[Guide Spec, Article 8.8.4 for RC joint design]
[NCHRP, Article 5.12 for RC joint design]

In this portion of the example, the joint between the columns of Bent 3 and the cap beam will be designed. The joints of the other columns and of the column connections with the pile caps are similar.

As with the design of the transverse steel in the columns, there now is an implicit and an explicit design procedure. The implicit procedure is easier to use, and unlike the design of the transverse steel in the column, the choice of whether to use the implicit or explicit is not dependent on the SDR or seismic hazard level. Instead, it is simply the designer's choice. If the easier implicit method gives a result that is too difficult to construct, then the explicit method may be used to reduce the steel congestion in the joint. We begin this example with the explicit method.

Design Step 8.3.1

Implicit Approach for Joint Design
[Guide Spec, Article 8.8.4.1] [NCHRP, Article 5.12.1]

Because we are in SDR 4, joint connections must be designed for these provisions.

Design for Bent 3

Implicit Approach: Direct Design Guide Spec (8.8.4.1)

a) Confinement reinforcement per Guide Spec 8.8.2.4 -
See Transverse Steel Design calculation

#5 spiral @ 6 inches max spacing

b) Antibuckling reinforcement per Guide Spec 8.8.2.5 -
See Transverse Steel Design calculation

5 spiral @ 3.25 in max spacing

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 8, Design Structural Components

Design Step 8.3.1 (continued)

c) Shear reinforcement per Guide Spec 8.8.2.3 - where

$f'c := 4 \cdot ksi$

$D := 48 \cdot in$ Column Diameter

$D'' := 43.375 \cdot in$ Spiral Diameter

$H_c := 6 \cdot ft$ Height of the joint

$\alpha := atan\left(\frac{D}{H_c}\right)$ $\alpha = 0.588$

$V_s := 136 \cdot kip$ Recall that the shear calc for the Pmin column requires the largest shear steel (see Transverse Design calculation).

For a circular column with #5 spiral,

$A_{bh} := 0.31 \cdot in^2$

$f_{yh} := 60 \cdot ksi$ Yield Strength of Spiral

$s := A_{bh} \cdot \frac{\pi}{2} \cdot \frac{f_{yh} \cdot D''}{V_s} \cdot cot(\alpha)$ $s = 13.98\,in$

$\rho_t := .014$ Longitudinal Steel Content

$f_{su} := 1.5 \cdot f_{yh}$ $f_{su} = 90\,ksi$

$A_g := \frac{\pi \cdot D^2}{4}$ Cross-sectional Area of Column

$A_c := \frac{\pi \cdot D''^2}{4}$ Cross-sectional Area of Column Core

$\phi := 0.90$ Strength Reduction Factor for Shear

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 8, Design Structural Components

Design Step 8.3.1 (continued)

For a circular column, minimum ratio:

$$\rho_s := 0.76 \cdot \frac{\rho_t}{\phi} \cdot \frac{f_{su}}{f_{yh}} \cdot \frac{A_g}{A_c} \cdot (\tan(\alpha))^2 \qquad \rho_s = 0.0097$$

$$s := \frac{4 \cdot A_{bh}}{D'' \cdot \rho_s} \qquad s = 2.96\,\text{in} \qquad \text{maximum}$$

#5 spiral @ 3 in max spacing is needed within the height of the joint. This is very tight. We will use the explicit approach instead.

Design Step 8.3.2

Explicit Approach for Joint Design
[Guide Spec, Article 8.8.4.2] [NCHRP, Article 5.12.2]

Explicit Detailed Approach Guide Spec (8.8.4.2)

Design Forces and Applied Stresses Guide Spec (8.8.4.2.1)

There are 3 cases to consider:

Case 1: From the transverse Displacement Capacity Verification with overstrength:

Calculate principal tension stress:

$f_h := 0 \cdot \text{ksi}$ average axial stress in the horizontal direction

$P_{max} := 1313 \cdot \text{kip}$

$M_p := 5429 \cdot \text{kip} \cdot \text{ft}$

at mid-depth of joint:

$b_b := 60 \cdot \text{in}$ width of cap beam

$L_{mid\_depth\_jt} := D + H_c \qquad L_{mid\_depth\_jt} = 120\,\text{in}$

$A_{mid\_depth\_jt} := b_b \cdot L_{mid\_depth\_jt} \qquad A_{mid\_depth\_jt} = 7200\,\text{in}^2$

$$f_v := \frac{P_{max}}{A_{mid\_depth\_jt}} \qquad f_v = 0.1824\,\text{ksi}$$

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**

Design Step 8, Design Structural Components

Design Step 8.3.2 (continued)

$h_b := 72 \cdot in$ joint depth

$h_c := 48 \cdot in$ diameter for circular column

$b_{je} := D \cdot \sqrt{2}$ effective joint width for circular column

$b_{je} := b_b$ effective joint width less than or equal to the width of the cross beam - controls

$$v_{hv} := \frac{M_p}{h_b \cdot h_c \cdot b_{je}}$$ joint shear stress

$v_{hv} = 0.3142\,ksi$

$$p_t := \frac{(f_h + f_v)}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + {v_{hv}}^2} \qquad p_t = -0.236\,ksi$$

$p_{t\_max} := 3.5 \cdot \sqrt{f'c} \cdot ksi$ $\qquad p_{t\_max} := .221 \cdot ksi$

therefore, must use Guide Spec 8.8.4.3

$$p_c := \frac{(f_h + f_v)}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + {v_{hv}}^2} \qquad p_c = 0.4183\,ksi$$

Maximum Allowable Compression Stresses Guide Spec (8.8.4.2.3)

$p_{c\_max} := 0.25 \cdot f'c$ $\qquad p_{c\_max} = 1\,ksi$ OK, we are less

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 8, Design Structural Components

Design Step 8.3.2 (continued)

Case 2: From the transverse Displacement Capacity Verification with overstrength:

$P_{min} := 172 \cdot kip$ $\quad M_p := 4488 \cdot kip \cdot ft$

By inspection, this will produce smaller principal stresses than Case 1.

Case 3: In the longitudinal direction, with overstrength:

$P_{DL} := 812 \cdot kip$ $\quad M_p := 2400 \cdot 1.5 \cdot kip \cdot ft$

By inspection, this will produce smaller principal stresses than Case 1.

Design Reinforcement for Joint Force Transfer Guide Spec (8.8.4.3)

Stirrups Guide Spec (8.8.4.3.2):

Column has 20 #10 vertical bars

$A_{bar} := 1.27 \cdot in^2$ $\quad N_{bar} := 20$

$A_{ST} := A_{bar} \cdot N_{bar}$

$A_{jv} := 0.16 \cdot A_{ST}$

Using #5 stirrups, $A_{stirrup\_leg} := 0.31 \cdot in^2$

$$\frac{A_{jv}}{A_{stirrup\_leg}} = 13.1097$$

13 #5 legs required in each quadrant (see Figures 20 & 21)

Clamping Guide Spec (8.8.4.3.2):

$A_{clamp} := 0.08 \cdot A_{ST}$

$$\frac{A_{clamp}}{A_{stirrup\_leg}} = 6.5548$$

7 #5 legs required in joint core (see Figures 20 & 21)

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**

Design Step 8, Design Structural Components

Design Step 8.3.2 (continued)

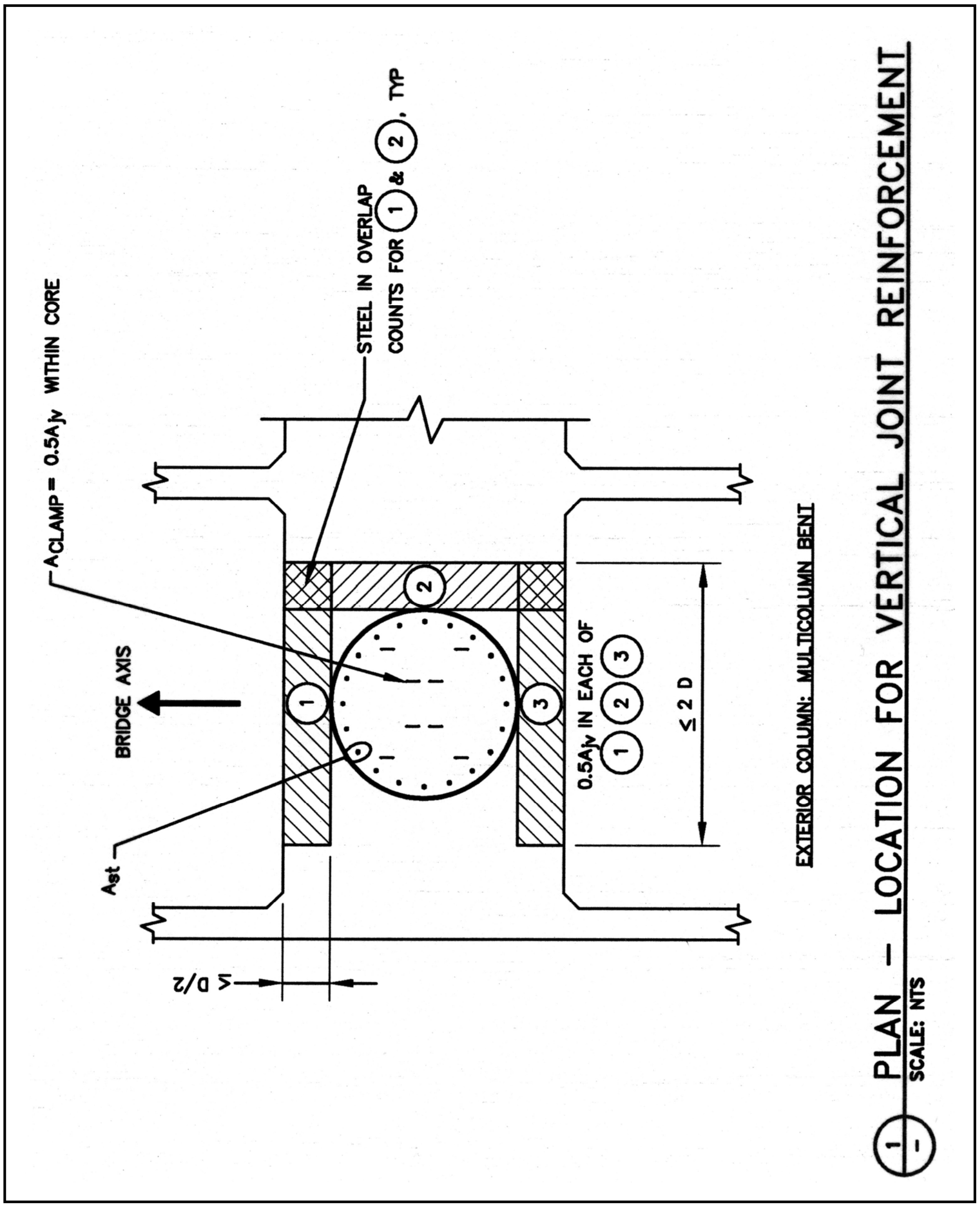


Figure 20 – Plan Layout of Joint Reinforcement

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 8, Design Structural Components

Design Step 8.3.2 (continued)

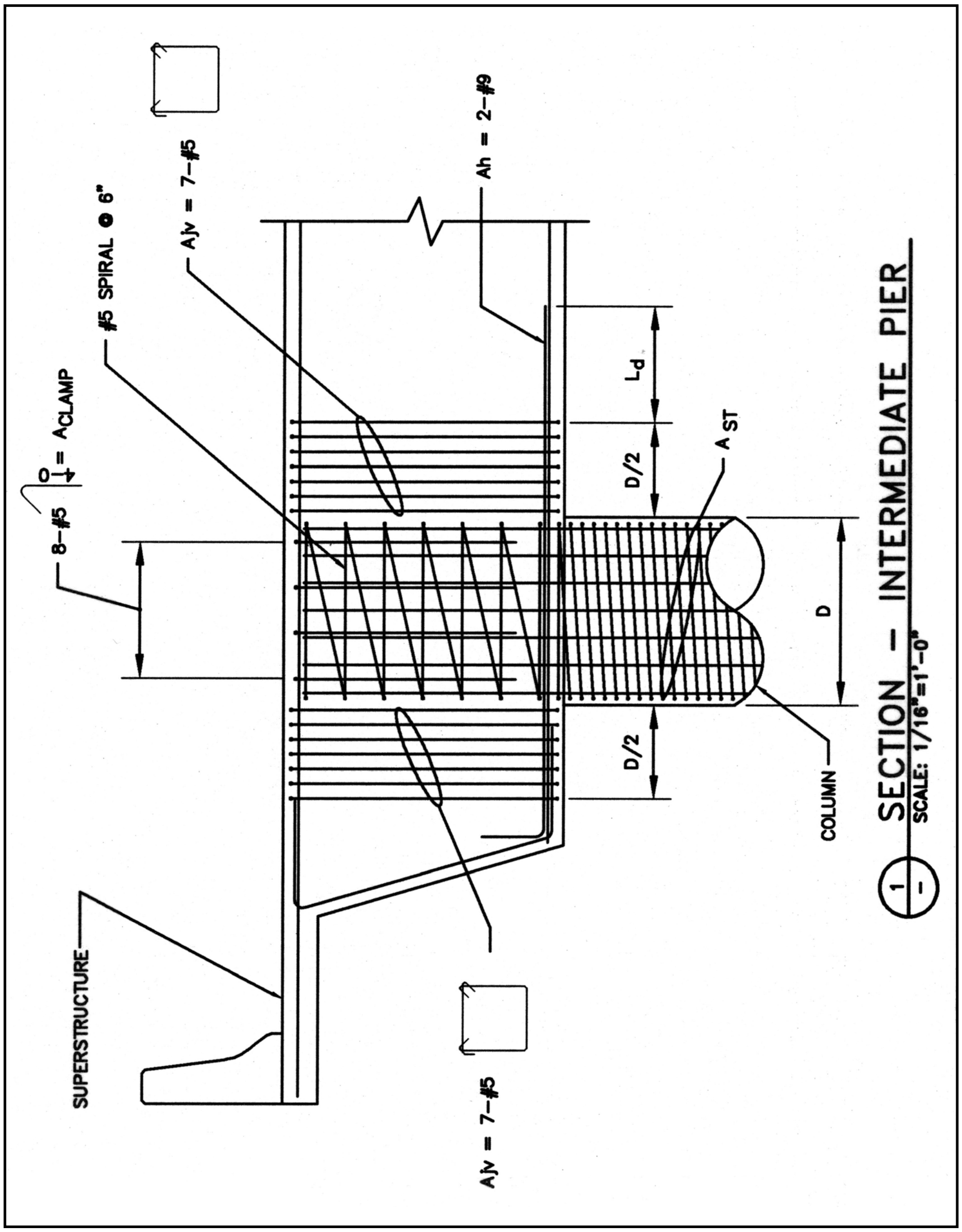


Figure 21 – Elevation of Joint Reinforcement

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 8, Design Structural Components

Design Step 8.3.2 (continued)

Horizontal reinforcement Guide Spec (8.8.4.3.3)

$A_h := 0.08 \cdot A_{ST}$

Using #9 bars, $A_{bar} := 1.0 \cdot in^2$

$\frac{A_h}{A_{bar}} = 2.032$ 2 #9 bars required in bottom of cap beam

Spiral reinforcement Guide Spec (8.8.4.3.4)

$l_{ac} := 48 \cdot in$ development length of #10 per LRFD 5.11.2.1

$\rho_s := 0.4 \cdot \frac{A_{ST}}{l_{ac}^2}$ $\rho_s = 0.0044$

$A_{spiral} := 0.31 \cdot in^2$

$D'' := 43.375 \cdot in$ Spiral Diameter

$s := 4 \cdot \frac{A_{spiral}}{\rho_s \cdot D''}$ $s = 6.4829\, in$

#5 spiral at 6 in pitch required in the joint

Design Step 8.3.3

Summary of Joint Reinforcement

Figure 21 shows the joint reinforcement required for the condition where the explicit method is used to design the joint.

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 8, Design Structural Components

Design Step 8.4 Superstructure Checks/Design Requirements
[Guide Spec, Article 8.11] [NCHRP, Article 3.10.3.12]

The superstructure check has not been included in this design example.

Design Step 8.5 Cap Beams and Diaphragms
[Guide Spec, Articles 4.8, 8.8.4, and 8.5.1.1]
[NCHRP, Articles 3.10.3.8, 5.12, and 11.6.5.1]

The cap beam and diaphragm designs have not been included in this design example.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 9, Design Foundations

DESIGN STEP 9 DESIGN FOUNDATIONS

In this example, a portion of the foundation design for Bent 3 (Pier 4) will be illustrated. Shown will be the calculation of the capacity design forces acting on the pile cap and the piles of Bent 3. A combined pile cap that is connected to both columns of the bent has been selected in the preliminary design. This was done because the construction of the foundations will require a cofferdam, and due to the proximity of the columns to one another, it was felt that a single excavation and cofferdam will be used. Therefore, a combined cap, with its ability to better mobilize the pile axial forces, was selected.

Design Step 9.1 Seismic Detailing Requirements

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3]

Article 3.7 outlines the detailing requirements categories (SDRs) based on seismic hazard level. The detailing requirements increase in complexity with increasing hazard level. For SDR 4, the provisions for detailing are included in Chapter 8. For the foundations of Bent 3, the detailing requirements will cover the forces used in the design of the caps and piles in addition to prescriptive details for the connections and longitudinal and transverse steel in the piles.

Design Step 9.2 Footings, Piles, and Shafts

[Guide Spec, Articles 4.8, 8.8.5, and 8.4.3]
[NCHRP, Articles 3.10.3.8, 5.14.4, and 10.7.4]

Design/Check of the Pile Group.

The following ultimate pile capacities are assumed. Geotechnical information is provided in Appendix A.

$$C_{ult} := 800 \cdot kip$$

$$Tult := -900 \cdot kip$$

Use plastic overstrength values from the Seismic Displacement Capacity Verification for the transverse direction.

Note also that the strength reduction factor for piles is 1.0 for the seismic loading cases, per Article 8.4.3.1.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 9, Design Foundations

Design Step 9.2.1

Determine Axial Forces in Piles
[Guide Spec, Article 8.4.3] [NCHRP, Article 10.7.4]

Refer to Figure 10 for pile layout and Figure 22 for forces acting on the foundation.

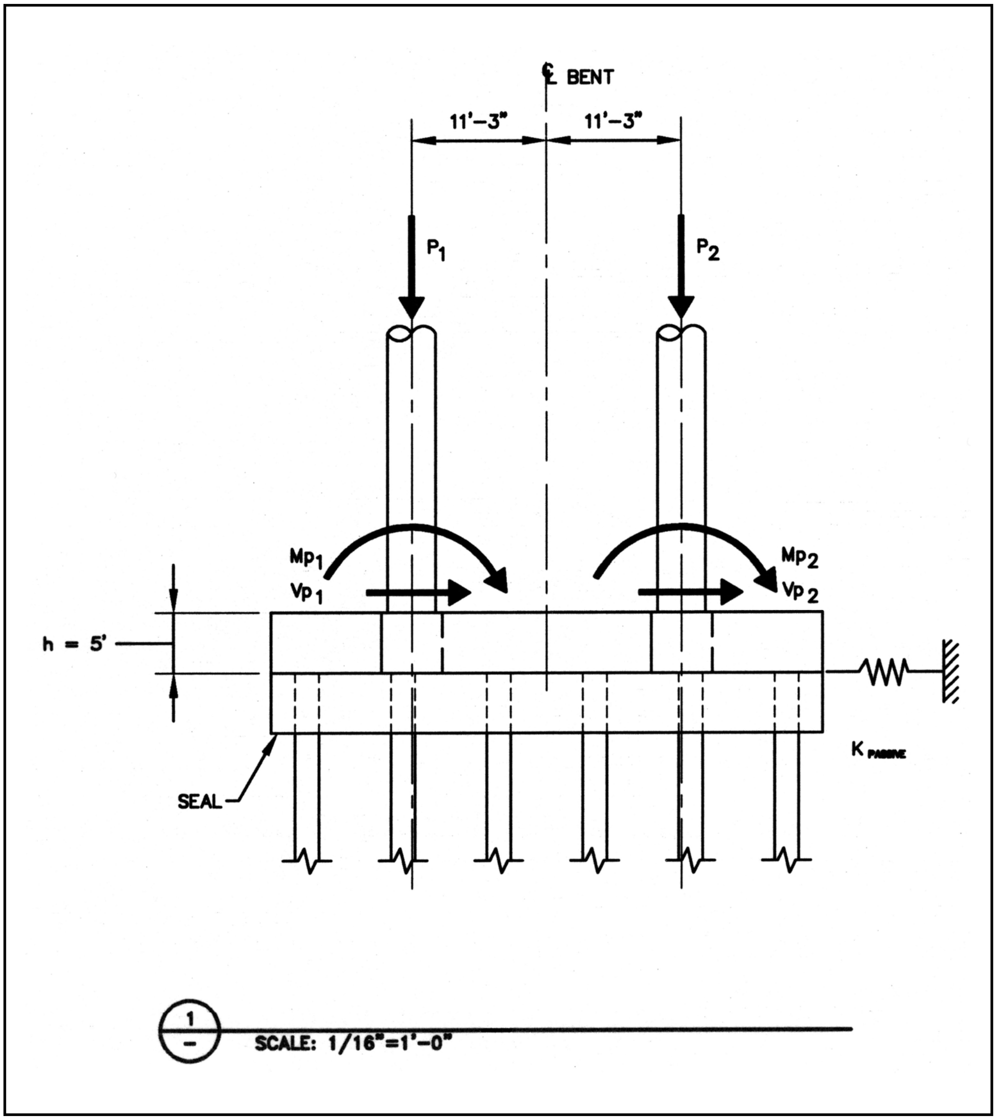


Figure 22 – Foundation Forces

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 9, Design Foundations

Design Step 9.2.1 (continued)

$P_1 := 172 \cdot kip$ $P_2 := 1313 \cdot kip$

$M_{p1} := 4632 \cdot kip \cdot ft$ $M_{p2} := 5493 \cdot kip \cdot ft$

$V_{p1} := 182 \cdot kip$ $V_{p2} := 218 \cdot kip$

$h_{cap} := 5 \cdot ft$

a) Compute Pile Axial Loads from Vertical Forces

$w_{soil} := 2 \cdot ft \cdot 22 \cdot ft \cdot 46 \cdot ft \cdot .120 \cdot \frac{kip}{ft^3}$ $w_{soil} = 243 \circ kip$

$P_u := P_1 + P_2 + w_{soil}$

$N_p := 16$ Number of piles in group

$$P_{pile} := \frac{P_u}{N_p}$$

$P_{pile} = 108 \circ kip$

b) Include Effects from Moment

$$M_u := M_{p1} + M_{p2} + \left(V_{p1} + V_{p2}\right) \cdot h_{cap} + \left(P_2 - P_1\right) \cdot 11.25 \cdot ft$$

$M_u = 24961 \circ kip \cdot ft$

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 9, Design Foundations

Design Step 9.2.1 (continued)

Assume that the pile cap is rigid. Use the "parallel axis theorem," as was done for computing the rotational springs for the foundation

Distance to pile row i from group center # pile per row

$x_1 := 20 \cdot ft$ $N_1 := 3$

$x_2 := 12 \cdot ft$ $N_2 := 2$

$x_3 := 4 \cdot ft$ $N_3 := 3$

$x_4 := -4 \cdot ft$ $N_4 := 3$

$x_5 := -12 \cdot ft$ $N_5 := 2$

$x_6 := -20 \cdot ft$ $N_6 := 3$

$$x_{sum} := x_1^2 \cdot N_1 + x_2^2 \cdot N_2 + x_3^2 \cdot N_3 + x_4^2 \cdot N_4 + x_5^2 \cdot N_5 + x_6^2 \cdot N_6$$

$x_{sum} = 3072\ ft^2$

c) Compute Pile Combined Axial Load

$P_{x1} := \dfrac{M_u \cdot x_1}{x_{sum}} + P_{pile}$ $P_{max} := P_{x1}$

$P_{max} = 271 \cdot kip$

$P_{x6} := \dfrac{M_u \cdot x_6}{x_{sum}} + P_{pile}$ $P_{min} := P_{x6}$

$P_{min} = -55 \cdot kip$

$C_{ult} > P_{max}$ OK

$T_{ult} > P_{min}$ OK

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 9, Design Foundations

Design Step 9.2.2

Determine Transverse Forces on Piles
[Guide Spec, Article 8.4.3] [NCHRP, Article 10.7.4]

a) Recall Maximum Pile Shears and Moments from Pushover Analysis, including overstrength

$$V_u := V_{p1} + V_{p2}$$

$$V_u = 400 \cdot kip$$

Determine portion of shear resisted by the passive soil pressure acting against the pilecap and portion of shear resisted by the piles.

$$K_T := 447000 \cdot \frac{kip}{ft}$$

$$K_{passive} := 11000 \cdot \frac{kip}{ft}$$

$$K_{piles} := K_T - K_{passive}$$

$$\Delta_u := \frac{V_u}{K_T} \qquad \Delta_u = 0.001\ ft \qquad \text{ok, less than } 0.02h = 0.01\ ft$$

$$V_{passive} := \Delta_u \cdot K_{passive} \qquad V_{passive} = 10 \cdot kip$$

$$V_{piles} := \Delta_u \cdot K_{piles} \qquad V_{piles} = 390 \cdot kip$$

$$V_{passive} + V_{piles} = 400 \cdot kip$$

checks, since $V_{passive} + V_{piles} = V_u$

$$V_{pile} := \frac{V_{piles}}{N_p}$$

$$V_{pile} = 24 \cdot kip$$

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 9, Design Foundations

Design Step 9.2.2 (continued)

Recall that LPILE run for pile lateral stiffness for V = 50 kip produced a maximum moment in the pile of 94 kip-ft. See Appendix A for moment vs. depth plot for Bent 3 from LPILE, therefore

$$M_{pile} := \frac{V_{pile}}{50 \cdot kip} \cdot 94 \cdot kip \cdot ft$$

$$M_{pile} = 46 \cdot kip \cdot ft$$

Pile moment capacity is okay per the interaction diagram for the pile section at the base of the pile cap. See PCA column interaction diagram in Figure 23.

Design Step 9.2.3

Check the Longitudinal Direction
[Guide Spec, Article 8.4.3] [NCHRP, Article 10.7.4]

Determine Axial Forces in Piles

$$P_{DL} := 812 \cdot kip$$

$$M_p := 2400 \cdot kip \cdot ft$$

$$M_{po} := M_p \cdot 1.5$$

$$L := 50 \cdot ft$$

$$V_p := 2 \cdot \frac{M_p}{L}$$

$$h_{cap} := 5 \cdot ft$$

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 9, Design Foundations

Design Step 9.2.3 (continued)

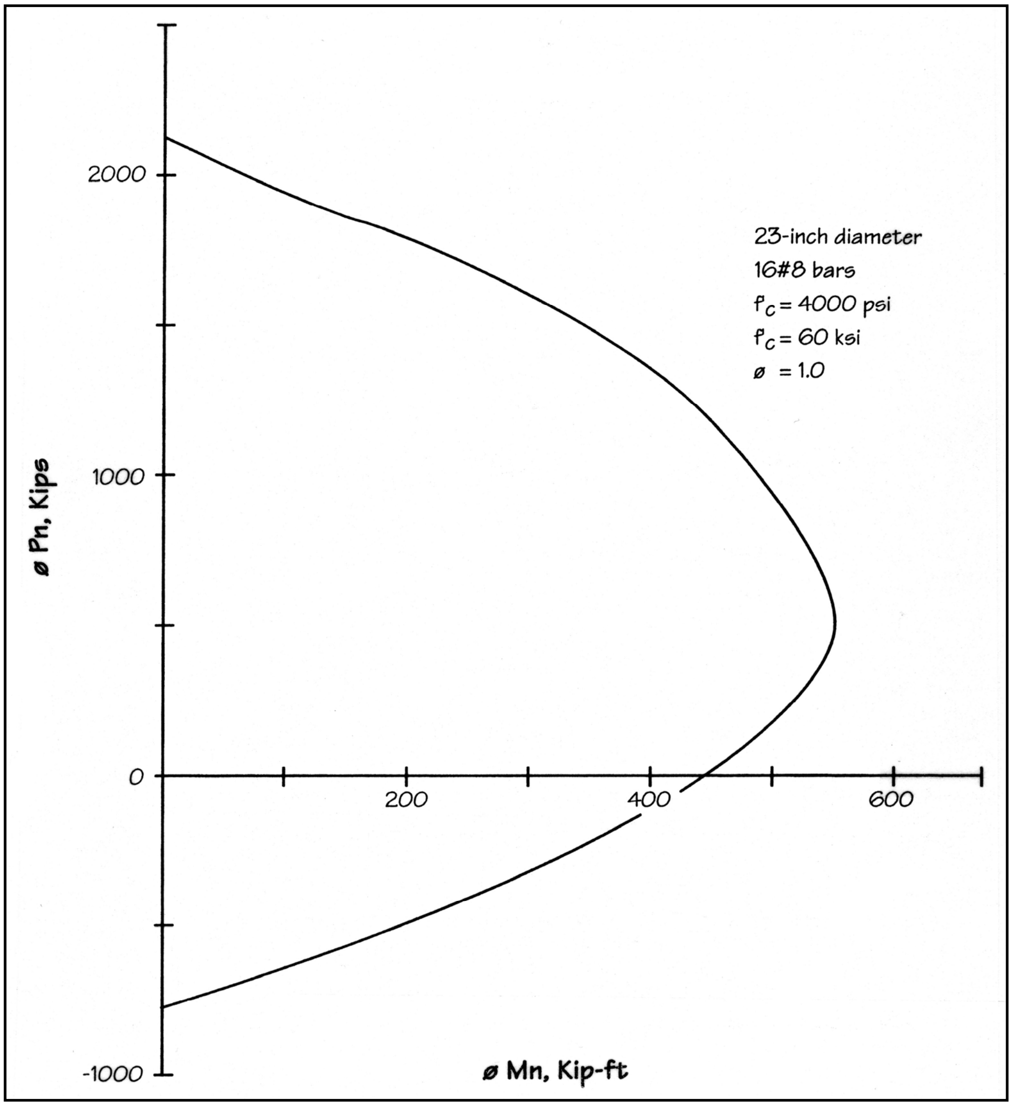


Figure 23 – Pile Top Interaction Diagram

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 9, Design Foundations

Design Step 9.2.3 (continued)

a) Compute Pile Axial Loads from Vertical Forces

$w_{soil} := 2 \cdot ft \cdot 22 \cdot ft \cdot 46 \cdot ft \cdot .120 \cdot \frac{kip}{ft^3}$ $w_{soil} = 243 \circ kip$

$P_u := 2 \cdot P_{DL} + w_{soil}$

$N_p := 16$ Number of piles in group

$P_{pile} := \frac{P_u}{N_p}$

$P_{pile} = 117 \circ kip$

b) Include Effects from Moment

$M_u := 2 \cdot M_{po} + 2 \cdot V_p \cdot h_{cap}$

$M_u = 8160 \circ kip \cdot ft$

Distance to pile row i from group center	# pile per row
$x_1 := 8 \cdot ft$	$N_1 := 6$
$x_2 := 0 \cdot ft$	$N_2 := 4$
$x_3 := -8 \cdot ft$	$N_3 := 6$

$x_{sum} := x_1^2 \cdot N_1 + x_2^2 \cdot N_2 + x_3^2 \cdot N_3$

$x_{sum} = 768\ ft^2$

SECTION III BRIDGE WITH TWO-COLUMN BENTS
Design Step 9, Design Foundations

Design Step 9.2.3 (continued)

c) Compute Pile Combined Axial Load

$$P_{x1} := \frac{M_u \cdot x_1}{x_{sum}} + P_{pile} \qquad P_{max} := P_{x1}$$

$$P_{max} = 202 \cdot kip$$

$$P_{x6} := \frac{M_u \cdot x_6}{x_{sum}} + P_{pile} \qquad P_{min} := P_{x6}$$

$$P_{min} = -96 \cdot kip$$

$C_{ult} > P_{max}$ OK

$T_{ult} > P_{min}$ OK

Determine Transverse Pile Forces

a) Determine Maximum Pile Shears and Moments

$$V_u := V_p$$

$$V_u = 96 \cdot kip$$

Determine portion of shear resisted by the passive soil pressure acting against the pilecap and portion of shear resisted by the piles.

$$K_T := 459000 \cdot \frac{kip}{ft}$$

$$K_{passive} := 23000 \cdot \frac{kip}{ft}$$

$$K_{piles} := K_T - K_{passive}$$

$$\Delta_u := \frac{V_u}{K_T} \qquad \Delta_u = 0.0002\ ft$$

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**

Design Step 9, Design Foundations

Design Step 9.2.3 (continued)

$$V_{passive} := \Delta_u \cdot K_{passive}$$

$$V_{piles} := \Delta_u \cdot K_{piles}$$

$$V_{passive} + V_{piles} = 96 \cdot kip$$

$$V_{pile} := \frac{V_{piles}}{N_p}$$

$$V_{pile} = 6 \cdot kip$$

Recall that LPILE run for pile lateral stiffness for V = 50 kip produced a maximum moment in the pile of 94 kip-ft. See Appendix for moment vs. depth plot for Bent 3 from LPILE, therefore

$$M_{pile} := \frac{V_{pile}}{50 \cdot kip} \cdot 94 \cdot kip \cdot ft$$

$M_{pile} = 11 \cdot kip \cdot ft$ Pile moment capacity OK

Pile design must also satisfy the detailing requirements of Article 8.8.5 for SDR 4. In this example, the piles must also satisfy the requirements for the liquefied soil condition and any lateral spreading demands arising from liquefaction. The piles are 24-inch steel pipes with 1/2-inch-thick walls that are in turn filled with reinforced concrete. The longitudinal reinforcement is 16 #8 bars that extend over the upper roughly one-third of the pile length. A nominal cage also extends to the bottom of the pile, which is closed off with a steel plate to prevent a soil plug from forming during driving.

The detailing provisions will require that the connection of the pile to the cap be adequate to transfer the expected forces. The longitudinal steel from the reinforcement cage in the pile should extend to the top of the pile cap so that a proper load path exists for transferring tension from the pile to cap.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 9, Design Foundations

A nominal spiral will be required below the plastic hinge zone that exists in the pile adjacent to the base of the pile cap. The nominal spiral will extend to the bottom of the pile and serve to hold the cage together for handling. A heavier spiral is not required in this case because the steel pipe can provide confinement to the concrete core and shear resistance. The exception is the upper portion of the pile (upper 1.5 diameters) where a spiral that meets Article 5.14.4.6.2b should be included.

Design Step 9.3 **Connections, Joint Designs**
[Guide Spec, Article 8.8.4] [NCHRP, Article 5.12]

The connection design of the column to the pile cap is handled similarly to the design of the column-cap beam connection. Because that connection design was illustrated in Design Step 8, the similar design for the foundation will not be repeated here.

The design of the pile cap for the capacity design forces input by the columns at their overstrength is fairly straightforward once the forces are obtained. Because capacity design is being used in this example for the foundations, the resistance factor (or strength reduction factor) for flexure is taken as 1.0 per Article 8.8.2.2; and for shear, it is taken as the normal 0.9 for normal weight concrete.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 10, Design Abutments

DESIGN STEP 10 DESIGN ABUTMENTS

The detailed design of the abutments is not included in this design example. The backfill passive resistance in the longitudinal direction has been relied upon for seismic loading in this example. Details of considering this effect were discussed in Chapter 4. Thus the force values have been developed, and they are consistent with the prescriptive provisions included in Chapter 8.5 of the proposed LRFD provisions. From this point, the end diaphragm must be designed to accommodate the passive forces expected from the soil. This is a straightforward design.

The transverse forces and associated load path must also be considered in the abutment design. In general, the abutment design forces will be the elastic forces associated with the design earthquakes. This bridge is relatively long and slender; thus the transverse forces at the abutments are not expected to pose any problem in design. The design would be handled in the conventional manner.

The active forces from the soil would be considered in the static design; however, the status of applying the seismic active forces, particularly to the stub abutment itself, has not been resolved yet in the provisions.

Design Step 10.1 Seismic Detailing Requirements

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3]

As with the rest of the structure and foundations, specific detailing requirements are included in the provisions. The complexity and rigor of these increase as a function of seismic hazard level. The requirements control the design force level, i.e., whether and how capacity design principles are applied, and they provide prescriptive detailing requirements that are aimed at ensuring integrity and ductility in the structure.

Design Step 10.2 Shear Keys and Connections

[Guide Spec, Article 8.11.4] [NCHRP, Article 3.10.3.12.4]

The abutment shear keys have not been designed in this example.

Design Step 10.3 Footings, Piles, and Shafts

[Guide Spec, Articles 8.4.3 and 8.5] [NCHRP, Articles 10.7 and 11.6.5]

The design of the piles and cap/stub abutment have not been discussed in this example.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 11, Consideration of Liquefaction Induced Flow or Lateral Spreading

DESIGN STEP 11 CONSIDERATION OF LIQUEFACTION-INDUCED FLOW OR LATERAL SPREADING

As discussed previously, this design example only focused on the design of the bridge for the nonliquefied conditions. The *Liquefaction Study Report* discusses the design for liquefied and lateral spread conditions.

It is recognized in the provisions that two phenomena are likely with liquefaction, one is the reduced soil strength and stiffness associated with the rise in pore water pressure, and the second is the potential lateral movement of layers or blocks of soil as a result of the loss of strength and stiffness.

The approach taken in the provisions is to design the structure for vibration of the structure, including both nonliquefied and liquefied conditions. This basically addresses the first phenomenon. Then assess the structure for the effects of lateral movement of the soil and determine whether the structure can meet the performance objective as designed for vibration or whether some type of structural or ground improvement will be required.

The assessment should include all potential beneficial factors in reducing likely ground displacements. Such factors include the pinning or passive pile effects of the structure foundations. The overall objective is to extract as much as possible out of the structure, as designed for vibration. This also includes allowing substantial plastic deformation of the foundations if the performance objective is life safety in the MCE event. Such practice is a departure from the traditional objective of preventing damage to the foundation.

Design Step 11.1 Evaluation of Foundation Displacement Demands and Capacities

[Guide Spec, Appendix D] [NCHRP, Appendix 3B.4.2.2]

Although not discussed in this example, Appendix D, of the Guide Specification, includes a detailed procedure for assessing the foundation elements under loading induced by lateral soil movements. This procedure was applied in the design of the bridge foundations, even though it is not discussed herein.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 11, Consideration of Liquefaction Induced Flow or Lateral Spreading

Design Step 11.2 **Ground Improvement and Structural Improvement**
[Guide Spec, Appendix D4.3] [NCHRP, Appendix 3B.4.2.2]

This particular structure would likely require some ground improvements to meet the life-safety performance objective in the MCE event. The foundation pile systems are subject to lateral soil movement-induced loads that cause substantial plastic deformation of the piles. While this is permitted by the new provisions, limits are set for these deformations. In this case, ground improvement will substantially reduce these demands. Envisioned for the bridge are stone columns that will serve to reduce the liquefaction and lateral spread potential. Such ground improvements, then provide the additional benefit of reducing the inelastic demands on the foundations.

The reader is referred to the *Liquefaction Study Report* (NCHRP b, 2001) for more details.

SECTION III **BRIDGE WITH TWO-COLUMN BENTS**
Design Step 12, Seismic Design Complete?

DESIGN STEP 12 SEISMIC DESIGN COMPLETE?

The seismic design process is by nature an iterative one, and one that may not always be applied in a purely sequential fashion. For instance, one may determine that a preselected configuration, member size, or layout may not work, and at that time the design is revised and the process iterated. Thus, one may not work through the entire process before a decision to iterate is made. The layout and flowchart for this example are meant to be illustrative of the design process, and they do not need to be followed rigorously. They are meant only to provide a general framework for the seismic design process.

SECTION IV CLOSING STATEMENT

The design of this five-span bridge has focused on the nonliquefied site condition, and has not directly addressed the liquefied and the lateral spreading loadings. The process of applying the proposed LRFD provisions has been illustrated for selected portions of the design of the bridge. A single iteration through the design process has been included in this example, and thus some refinements would be possible upon further iteration.

A companion document, Liquefaction Study Report, which was also developed as part of the NCHRP 12-49 project, includes discussion of the liquefied site condition and the lateral spread loading condition. It should, therefore, be recognized that some of the member sizes are controlled by conditions associated with the liquefied state. For instance, the pile forces calculated as part of this example were shown to be quite low, and they would be expected to be somewhat larger for the other load conditions.

No attempt was made in the design example to refine the column longitudinal steel design as a result of the Displacement Capacity Verification or pushover. Per the provisions, a limited reduction in the design moment demands is allowed if the pushover indicates that there exists reserve displacement capacity in the columns. In this example, some such reserve was available; thus some reduction in the longitudinal steel content at Bent 3 would be permitted. With the addition of requirements for anti-buckling reinforcement, which is directly proportional to ρ of the column, the designer may wish to optimize on ρ in order to minimize the spiral requirements.

Although the new provisions appear more complex than Division I-A, they are also more comprehensive and include more alternatives for seismic design It is felt that the additional effort required beyond that of Division I-A will be offset by the savings in design time provided by the more comprehensive nature of the specification. This should ultimately help the designer in applying the provisions by alleviating the need for undo interpretation and debate on what should be done.

SECTION V REFERENCES

SECTION V REFERENCES

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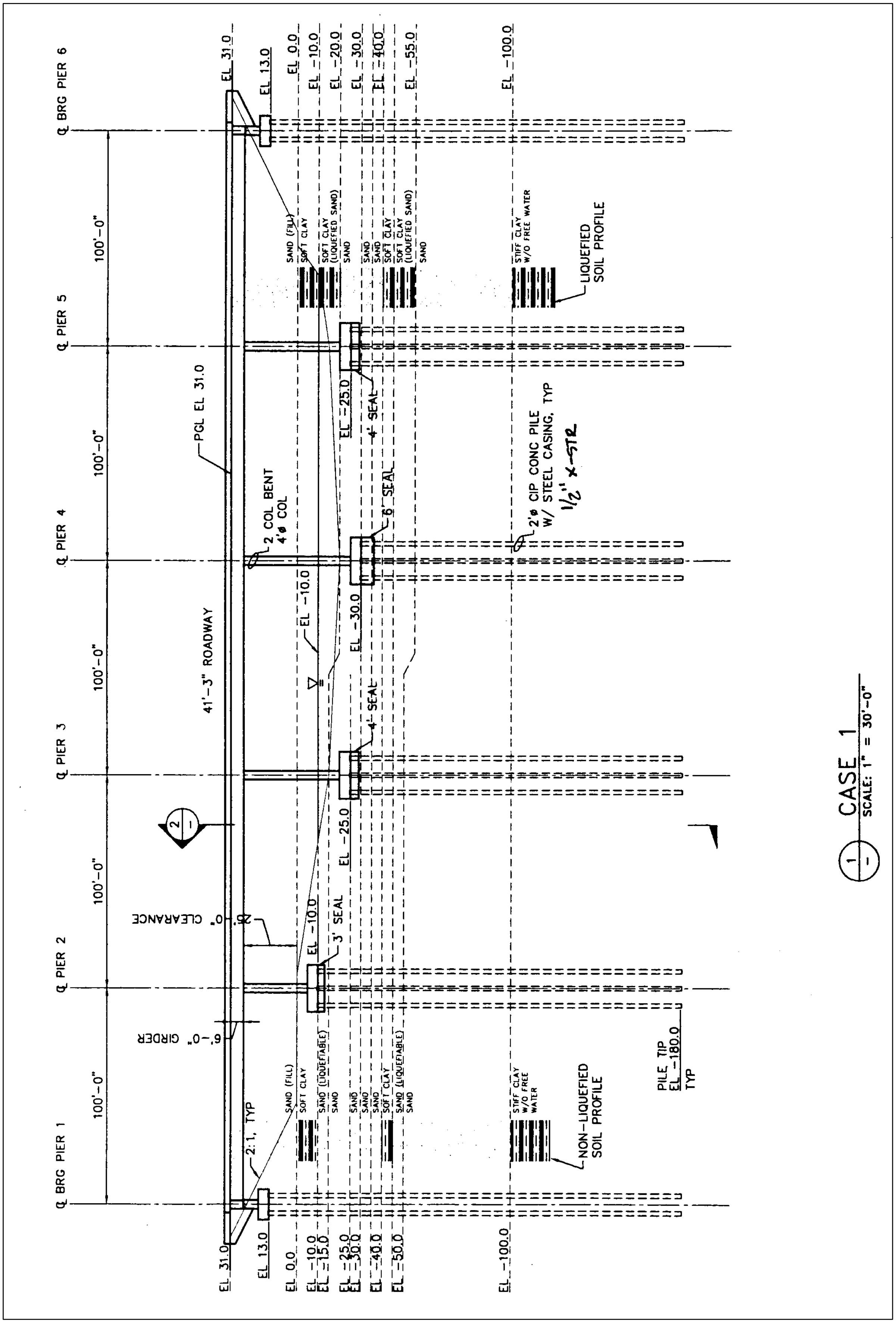
Appendix A
Geotechnical Data



℄ BRG PIER 1
℄ PIER 2
℄ PIER 3
℄ PIER 4
℄ PIER 5
℄ BRG PIER 6
100'-0"
100'-0"
100'-0"
100'-0"
100'-0"
PGL EL 31.0
41'-3" ROADWAY
2 COL BENT
4'ø COL
6'-0" GIRDER
25'-0" CLEARANCE
2:1, TYP
3' SEAL
4' SEAL
6' SEAL
4' SEAL
2'ø CIP CONC PILE
W/ STEEL CASING, TYP
1/2" x-STR
LIQUEFIED
SOIL PROFILE
NON-LIQUEFIED
SOIL PROFILE
PILE TIP
EL -180.0
TYP
EL 31.0
EL 13.0
EL 0.0
EL -10.0
EL -15.0
EL -25.0
EL -30.0
EL -40.0
EL -50.0
EL -55.0
EL -100.0
SAND (FILL)
SOFT CLAY
SAND (LIQUEFIABLE)
SAND
SOFT CLAY (LIQUEFIED SAND)
STIFF CLAY W/O FREE WATER
CASE 1
SCALE: 1" = 30'-0"

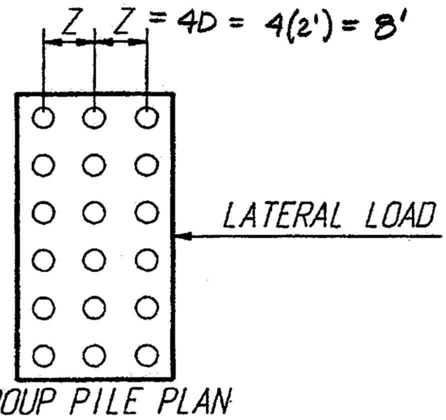
Elevation feet	Soil Type in LPILE	Density γ (pcf)	Subgrade Modulus k (pci)	Strain ε_{50}	Including Group Effects: Cohesion c_u (psf)	Including Group Effects: Friction Angle φ (degrees)	Including Group Effects: Subgrade Modulus k (pci)
31.0	1 Sand (Fill)	110	75			31	75
0.0	2 Soft Clay	110	5	0.01	500		5
-10.0	3 Sand *	120	50			32	50
-15.0	4 Sand	120	75			34	75
-25.0	5 Sand	120	100			36	100
-30.0	6 Sand	120	25			28	25
-35.0	7 Sand	120	75			34	75
-40.0	8 Soft Clay	110	5	0.01	500		5
-45.0	9 Sand *	120	50			32	50
-50.0	10 Sand	120	60			28	30
-100.0	11 Stiff Clay w/o Free Water						
-150.0	12 Stiff Clay w/o Free Water						
-200.0	Till						

* Liquefiable Sand

ALL PIERS

Z	EFFICIENCY (REDUCTION) FACTOR
8D	1.0
6D	0.8
4D	0.5
3D	0.4

EFFICIENCY FACTOR



GROUP PILE PLAN

Efficiency Factor
Table 4.4.3-1

INPUT INTO LPILE 0.5K

For driven piles, the following factors apply:

Contact the Olympia Service Center Materials Lab to verify any assumptions.

The LPILE1 computer program will generate P-Y curves, or the user can input them. To obtain generated curves, input a modulus of subgrade reaction (K) and a soil shear strength (C) which are the values taken from the soils report multiplied by the efficiency factor. To figure P-Y curves for input, multiply the P-Y values from the soils report by the efficiency factor.

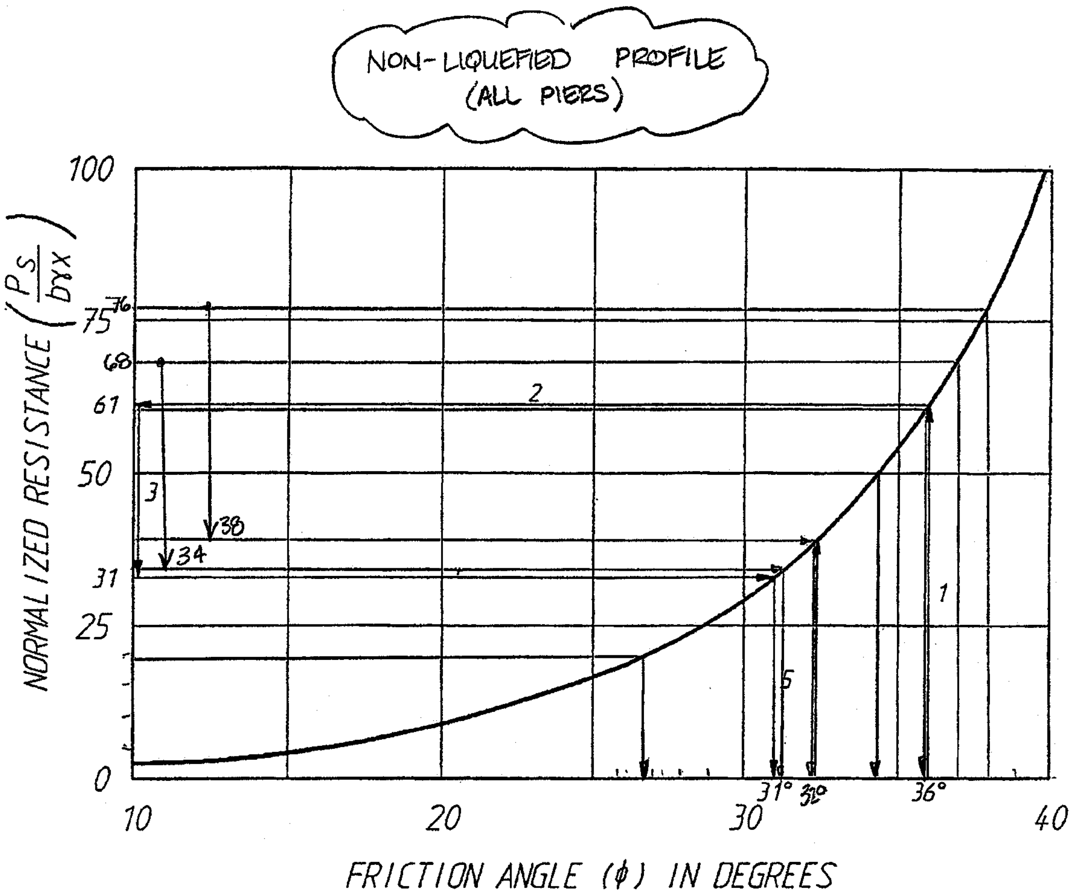
$0.5C_u$

For a typical soil, the relationship between its normalized resistance value and friction angle is defined by the curve in Figure 4.4.3-1. The friction angle could be adjusted for efficiency and input to LPILE1 by following these steps:

$\phi \Rightarrow$
1. Begin at the coordinate of the natural friction angle (36°).
2. Read across to the normalized resistance (61).
3. Multiply the resistance by the efficiency reduction factor, i.e., 61 (0.5) = 31.
4. Read across from the reduced value to obtain the adjusted friction angle (31°).
5. Input the ϕ value to LPILE1.

$Z = 8'$ $\quad D = 2'$

$\frac{Z}{D} = 4 \Rightarrow 4D$



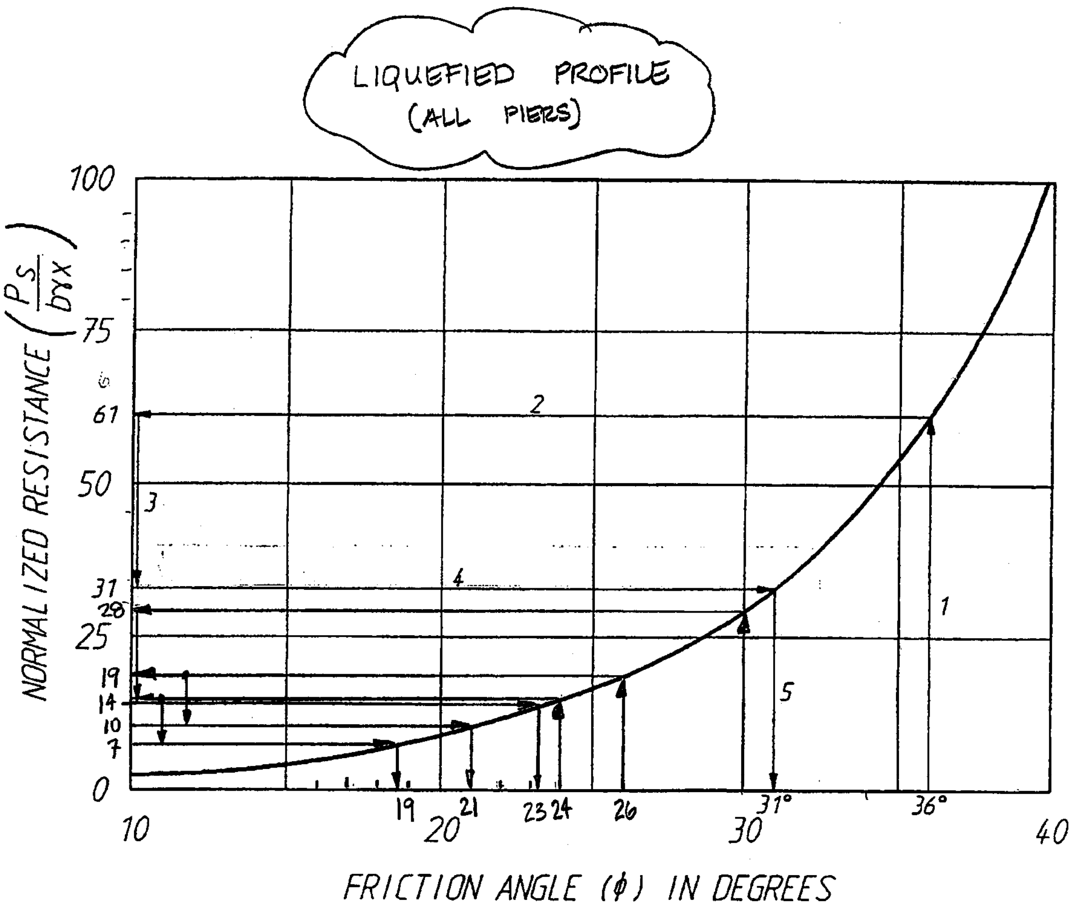
EFFICIENCY FACTOR = 0.5

LAYER	ϕ_1	NORM. RESIS.[2]	N. RESIST, x0.5	ϕ_{LPILE}
2	37°	68	34	31°
3 9	38°	76	38	32°
4 7	40°	100	50	34°
5	42°	SAY 120	60	36°
6 10	32°	38	19	28°

Friction Angle (φ)

$$\frac{PS}{b\gamma x} = K_a(\tan^8 B - 1) + K_o \tan\phi \tan^4 B$$

P_s	=	Soil Resistance on Pile Element
b	=	Pile Width
g	=	Soil Unit Weight
X	=	Depth to Pile Element
N	=	Step in Example
B	=	$45° + \phi/2$
K_a	=	$\tan 2(45° - \phi/2)$
K_o	=	$1 - \sin\phi$



Friction Angle (ϕ)

$$\frac{PS}{b\gamma x} = K_a (\tan^8 B - 1) + K_o \tan\phi \tan^4 B$$

P_s	=	Soil Resistance on Pile Element
b	=	Pile Width
g	=	Soil Unit Weight
X	=	Depth to Pile Element
N	=	Step in Example
B	=	$45° + \phi/2$
K_a	=	$\tan 2(45° - \phi/2)$
K_o	=	$1 - \sin\phi$

EFFICIENCY FACTOR = 0.5

LAYER	ϕ_1	NORM. RESIS.[2]	N. RESIST. × 0.5	$\emptyset_{LPILE}$
4	30°	28	14	23
5	24°	14	7	19
6	30°	28	14	23
9	26°	19	10	21

1 - FROM GEOFF MARTIN

2 - FROM CHART

INPUT INTO LPILE

LPILE COORDINATES

WASHINGTON SITE

NON-LIQUEFIED SOIL PROFILE

Layer No.	Soil Type	Top of Soil Layer Elevations	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6
		Pile Head Elevation	13	-13	-29	-36	-29	13
1	Sand (Fill)	23	-120					-120
2	Soft Clay	0	156	-156				156
3	Sand (Liquefiable)	-10	276	-36				276
4	Sand	-15	336	24	-168	-192	-168	336
5	Sand	-25	456	144	-48	-132	-48	456
6	Sand	-30	516	204	12	-72	12	516
7	Sand	-35	576	264	72	-12	72	576
8	Soft Clay	-40	636	324	132	48	132	636
9	Sand (Liquefiable)	-45	696	384	192	108	192	696
10	Sand	-50	756	444	252	168	252	756
	Pile Tip Elevation	-180	2316	2004	1812	1728	1812	2316

LIQUEFIED SOIL PROFILE

Layer No.	Soil Type	Top of Soil Layer Elevations	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6
		Pile Head Elevation	13	-13	-29	-36	-29	13
1	Sand (Fill)	23	-120					-120
2	Soft Clay	0	156	-96				156
3	Soft Clay	-10	276	-36	-168		-168	276
4	Sand	-20	396	84	-108	-132	-108	396
5	Sand	-30	516	204	12	-72	12	516
6	Sand	-35	576	264	72	-12	72	576
7	Soft Clay	-40	636	324	132	48	132	636
8	Soft Clay	-45	696	384	192	108	192	696
9	Sand	-55	816	504	312	228	312	816
10	Stiff Clay w/o Free Water	-100	1356	1044	852	768	852	1356
	Pile Tip Elevation	-180	2316	2004	1812	1728	1812	2316

LPILE COORDINATES

WASHINGTON SITE

NON-LIQUEFIED SOIL PROFILE

Layer No.	Soil Type	Top of Soil Layer Elevations
1	Sand (Fill)	23
2	Soft Clay	0
3	Sand (Liquefiable)	-10
4	Sand	-15
5	Sand	-25
6	Sand	-30
7	Sand	-35
8	Soft Clay	-40
9	Sand (Liquefiable)	-45
10	Sand	-50
	Pile Tip Elevation	-180

P-Y Curve Depths at Mid-Depth of Each Soil Layer

Pier					
1	2	3	4	5	6
18					18
216	-96				216
306	-6				306
396	84	-108	-162	-108	396
486	174	-18	-102	-18	486
546	234	42	-42	42	546
606	294	102	18	102	606
666	354	162	78	162	666
726	414	222	138	222	726
1536	1224	1032	948	1032	1536

LIQUEFIED SOIL PROFILE

Layer No.	Soil Type	Top of Soil Layer Elevations
1	Sand (Fill)	23
2	Soft Clay	0
3	Soft Clay	-10
4	Sand	-20
5	Sand	-30
6	Sand	-35
7	Soft Clay	-40
8	Soft Clay	-45
9	Sand	-55
10	Stiff Clay w/o Free Water	-100
	Pile Tip Elevation	-180

P-Y Curve Depths at Mid-Depth of Each Soil Layer

Pier					
1	2	3	4	5	6
18					18
216	-66				216
336	24	-138		-138	336
456	144	-48	-102	-48	456
546	234	42	-42	42	546
606	294	102	18	102	606
666	354	162	78	162	666
756	444	252	168	252	756
1086	774	582	498	582	1086
1836	1524	1332	1248	1332	1836

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NCHRP 12-49 Washington Site  - Pier 4

UNITS--ENGLISH UNITS


                   I N P U T   I N F O R M A T I O N
                   *********************************


THE LOADING IS STATIC
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PILE GEOMETRY AND PROPERTIES
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    PILE LENGTH                    =     1728.00 IN
     2 POINTS

        X          DIAMETER    MOMENT OF       AREA       MODULUS OF
                                INERTIA                   ELASTICITY
        IN            IN         IN**4         IN**2       LBS/IN**2
        .00         24.000     .232E+05      .673E+03     .383E+07
    1728.00         24.000     .232E+05      .673E+03     .383E+07


SOILS INFORMATION
-----------------

    X AT THE GROUND SURFACE           =     -708.00 IN

    SLOPE ANGLE AT THE GROUND SURFACE     =        .00 DEG.

    * LAYER(S) OF SOIL

    LAYER  1
    THE SOIL IS A SAND - P-Y CRITERIA BY REESE ET AL, 1974
    X AT THE TOP OF THE LAYER         =     -708.00 IN
    X AT THE BOTTOM OF THE LAYER      =     -432.00 IN
    MODULUS OF SUBGRADE REACTION      =    .750E+02 LBS/IN**3

    LAYER  2
    THE SOIL IS A SOFT CLAY
    X AT THE TOP OF THE LAYER         =     -432.00 IN
    X AT THE BOTTOM OF THE LAYER      =     -312.00 IN
    MODULUS OF SUBGRADE REACTION      =    .500E+01 LBS/IN**3

    LAYER  3
    THE SOIL IS A SAND - P-Y CRITERIA BY REESE ET AL, 1974
    X AT THE TOP OF THE LAYER         =     -312.00 IN
    X AT THE BOTTOM OF THE LAYER      =     -252.00 IN
    MODULUS OF SUBGRADE REACTION      =    .500E+02 LBS/IN**3

    LAYER  4
    THE SOIL IS A SAND - P-Y CRITERIA BY REESE ET AL, 1974
    X AT THE TOP OF THE LAYER         =     -252.00 IN
    X AT THE BOTTOM OF THE LAYER      =     -132.00 IN
    MODULUS OF SUBGRADE REACTION      =    .750E+02 LBS/IN**3
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LAYER  5
THE SOIL IS A SAND - P-Y CRITERIA BY REESE ET AL, 1974
X AT THE TOP OF THE LAYER          =     -132.00 IN
X AT THE BOTTOM OF THE LAYER       =      -72.00 IN
MODULUS OF SUBGRADE REACTION       =    .100E+03 LBS/IN**3

LAYER  6
THE SOIL IS A SAND - P-Y CRITERIA BY REESE ET AL, 1974
X AT THE TOP OF THE LAYER          =      -72.00 IN
X AT THE BOTTOM OF THE LAYER       =      -12.00 IN
MODULUS OF SUBGRADE REACTION       =    .250E+02 LBS/IN**3

LAYER  7
THE SOIL IS A SAND - P-Y CRITERIA BY REESE ET AL, 1974
X AT THE TOP OF THE LAYER          =      -12.00 IN
X AT THE BOTTOM OF THE LAYER       =       48.00 IN
MODULUS OF SUBGRADE REACTION       =    .750E+02 LBS/IN**3

LAYER  8
THE SOIL IS A SOFT CLAY
X AT THE TOP OF THE LAYER          =       48.00 IN
X AT THE BOTTOM OF THE LAYER       =      108.00 IN
MODULUS OF SUBGRADE REACTION       =    .500E+01 LBS/IN**3

LAYER  9
THE SOIL IS A SAND - P-Y CRITERIA BY REESE ET AL, 1974
X AT THE TOP OF THE LAYER          =      108.00 IN
X AT THE BOTTOM OF THE LAYER       =      168.00 IN
MODULUS OF SUBGRADE REACTION       =    .500E+02 LBS/IN**3

LAYER 10
THE SOIL IS A SAND - P-Y CRITERIA BY REESE ET AL, 1974
X AT THE TOP OF THE LAYER          =      168.00 IN
X AT THE BOTTOM OF THE LAYER       =     1750.00 IN
MODULUS OF SUBGRADE REACTION       =    .300E+02 LBS/IN**3


DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                 20 POINTS
          X,IN      WEIGHT,LBS/IN**3
       -708.00         .64E-01
       -432.00         .64E-01
       -432.00         .64E-01
       -312.00         .64E-01
       -312.00         .34E-01
       -252.00         .34E-01
       -252.00         .34E-01
       -132.00         .34E-01
       -132.00         .34E-01
        -72.00         .34E-01
        -72.00         .34E-01
        -12.00         .34E-01
        -12.00         .34E-01
         48.00         .34E-01
         48.00         .28E-01
        108.00         .28E-01
        108.00         .34E-01
        168.00         .34E-01
        168.00         .34E-01
       1750.00         .34E-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
                 20 POINTS
          X,IN        C,LBS/IN**2      PHI,DEGREES      E50
       -708.00         .000E+00         .310E+02       -----
       -432.00         .000E+00         .310E+02       -----
       -432.00         .347E+01             .000       .100E-01
       -312.00         .347E+01             .000       .100E-01
       -312.00         .000E+00         .320E+02       -----
       -252.00         .000E+00         .320E+02       -----
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-252.00	.000E+00	.340E+02	-----
-132.00	.000E+00	.340E+02	-----
-132.00	.000E+00	.360E+02	-----
-72.00	.000E+00	.360E+02	-----
-72.00	.000E+00	.280E+02	-----
-12.00	.000E+00	.280E+02	-----
-12.00	.000E+00	.340E+02	-----
48.00	.000E+00	.340E+02	-----
48.00	.347E+01	.000	.100E-01
108.00	.347E+01	.000	.100E-01
108.00	.000E+00	.320E+02	-----
168.00	.000E+00	.320E+02	-----
168.00	.000E+00	.280E+02	-----
1750.00	.000E+00	.280E+02	-----

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BOUNDARY AND LOADING CONDITIONS
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    LOADING NUMBER   1

    BOUNDARY CONDITION CODE                   =         2
    LATERAL LOAD AT THE PILE HEAD             =   .500E+05 LBS
    SLOPE AT THE PILE HEAD                    =   .000E+00 IN/IN
    AXIAL LOAD AT THE PILE HEAD               =   .000E+00 LBS


FINITE-DIFFERENCE PARAMETERS
    NUMBER OF PILE INCREMENTS                                   =       100
    DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE            =   .100E-04 IN
    MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS      =       100
    MAXIMUM ALLOWABLE DEFLECTION                                =    .10E+03 IN


OUTPUT CODES
    KOUTPT =  1
    KPYOP  =  1
    INC    =  1
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DEPTH IN	DIAM IN	PHI	GAMMA AVG LBS/IN**3	A	B	PST	PSD
138.00	24.00	31.0	.640E-01	.88	.50	.314E+04	.689E+04

Y IN	P LBS/IN
.000E+00	.000E+00
.333E-01	.345E+03
.667E-01	.528E+03
.100E+00	.676E+03
.133E+00	.805E+03
.167E+00	.922E+03
.200E+00	.103E+04
.233E+00	.113E+04
.267E+00	.123E+04
.300E+00	.132E+04
.333E+00	.141E+04
.367E+00	.149E+04
.400E+00	.157E+04
.900E+00	.276E+04
.249E+02	.276E+04
.489E+02	.276E+04
.729E+02	.276E+04

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA AVG LBS/IN**3	E50
336.00	24.000	3.4700	.0640	.0100

Y,IN	P,LBS/IN
.000E+00	.000E+00
.480E-02	.750E+02
.150E+00	.236E+03
.300E+00	.297E+03
.450E+00	.340E+03
.600E+00	.375E+03
.750E+00	.404E+03
.900E+00	.429E+03
.105E+01	.452E+03
.120E+01	.472E+03
.135E+01	.491E+03
.150E+01	.509E+03
.165E+01	.525E+03
.180E+01	.540E+03
.480E+01	.750E+03
.900E+01	.750E+03
.120E+02	.750E+03

DEPTH IN	DIAM IN	PHI	GAMMA AVG LBS/IN**3	A	B	PST	PSD
426.00	24.00	32.0	.619E-01	.88	.50	.275E+05	.233E+05

Y IN	P LBS/IN
.000E+00	.000E+00
.333E-01	.710E+03
.667E-01	.142E+04
.100E+00	.213E+04
.133E+00	.284E+04
.167E+00	.355E+04
.200E+00	.426E+04
.233E+00	.497E+04
.267E+00	.568E+04
.300E+00	.639E+04
.333E+00	.710E+04
.367E+00	.781E+04
.400E+00	.852E+04
.900E+00	.192E+05
.249E+02	.205E+05
.489E+02	.205E+05
.729E+02	.205E+05

DEPTH IN	DIAM IN	PHI	GAMMA AVG LBS/IN**3	A	B	PST	PSD
516.00	24.00	34.0	.570E-01	.88	.50	.436E+05	.334E+05

Y IN	P LBS/IN
.000E+00	.000E+00
.333E-01	.129E+04
.667E-01	.258E+04
.100E+00	.387E+04
.133E+00	.516E+04
.167E+00	.645E+04
.200E+00	.774E+04
.233E+00	.903E+04
.267E+00	.103E+05
.300E+00	.116E+05
.333E+00	.129E+05
.367E+00	.142E+05
.400E+00	.155E+05
.900E+00	.294E+05
.249E+02	.294E+05
.489E+02	.294E+05
.729E+02	.294E+05

DEPTH	DIAM	PHI	GAMMA AVG	A	B	PST	PSD

IN	IN		LBS/IN**3				
606.00	24.00	36.0	.536E-01	.88	.50	.667E+05	.477E+05

Y IN	P LBS/IN
.000E+00	.000E+00
.333E-01	.202E+04
.667E-01	.404E+04
.100E+00	.606E+04
.133E+00	.808E+04
.167E+00	.101E+05
.200E+00	.121E+05
.233E+00	.141E+05
.267E+00	.162E+05
.300E+00	.182E+05
.333E+00	.202E+05
.367E+00	.222E+05
.400E+00	.239E+05
.900E+00	.420E+05
.249E+02	.420E+05
.489E+02	.420E+05
.729E+02	.420E+05

DEPTH IN	DIAM IN	PHI	GAMMA AVG LBS/IN**3	A	B	PST	PSD
666.00	24.00	28.0	.518E-01	.88	.50	.388E+05	.187E+05

Y IN	P LBS/IN
.000E+00	.000E+00
.333E-01	.555E+03
.667E-01	.111E+04
.100E+00	.167E+04
.133E+00	.222E+04
.167E+00	.278E+04
.200E+00	.333E+04
.233E+00	.388E+04
.267E+00	.444E+04
.300E+00	.500E+04
.333E+00	.555E+04
.367E+00	.611E+04
.400E+00	.666E+04
.900E+00	.150E+05
.249E+02	.164E+05
.489E+02	.164E+05
.729E+02	.164E+05

DEPTH IN	DIAM IN	PHI	GAMMA AVG LBS/IN**3	A	B	PST	PSD
726.00	24.00	34.0	.504E-01	.88	.50	.751E+05	.415E+05

Y IN	P LBS/IN
.000E+00	.000E+00
.333E-01	.182E+04
.667E-01	.363E+04
.100E+00	.545E+04
.133E+00	.726E+04
.167E+00	.908E+04
.200E+00	.109E+05
.233E+00	.127E+05
.267E+00	.145E+05
.300E+00	.163E+05
.333E+00	.182E+05
.367E+00	.197E+05
.400E+00	.208E+05
.900E+00	.366E+05
.249E+02	.366E+05
.489E+02	.366E+05

.729E+02	.366E+05

DEPTH IN	DIAM IN	C LBS/IN**2	GAMMA AVG LBS/IN**3	E50
786.00	24.000	3.4700	.0489	.0100

Y,IN	P,LBS/IN
.000E+00	.000E+00
.480E-02	.750E+02
.150E+00	.236E+03
.300E+00	.297E+03
.450E+00	.340E+03
.600E+00	.375E+03
.750E+00	.404E+03
.900E+00	.429E+03
.105E+01	.452E+03
.120E+01	.472E+03
.135E+01	.491E+03
.150E+01	.509E+03
.165E+01	.525E+03
.180E+01	.540E+03
.480E+01	.750E+03
.900E+01	.750E+03
.120E+02	.750E+03

DEPTH IN	DIAM IN	PHI	GAMMA AVG LBS/IN**3	A	B	PST	PSD
846.00	24.00	32.0	.476E-01	.88	.50	.806E+05	.356E+05

Y IN	P LBS/IN
.000E+00	.000E+00
.333E-01	.141E+04
.667E-01	.282E+04
.100E+00	.423E+04
.133E+00	.564E+04
.167E+00	.705E+04
.200E+00	.846E+04
.233E+00	.987E+04
.267E+00	.113E+05
.300E+00	.127E+05
.333E+00	.141E+05
.367E+00	.155E+05
.400E+00	.169E+05
.900E+00	.313E+05
.249E+02	.313E+05
.489E+02	.313E+05
.729E+02	.313E+05

DEPTH IN	DIAM IN	PHI	GAMMA AVG LBS/IN**3	A	B	PST	PSD
1656.00	24.00	28.0	.410E-01	.88	.50	.184E+06	.367E+05

Y IN	P LBS/IN
.000E+00	.000E+00
.333E-01	.166E+04
.667E-01	.331E+04
.100E+00	.497E+04
.133E+00	.662E+04
.167E+00	.828E+04
.200E+00	.994E+04
.233E+00	.116E+05
.267E+00	.132E+05
.300E+00	.149E+05
.333E+00	.164E+05
.367E+00	.174E+05
.400E+00	.183E+05

.900E+00	.323E+05
.249E+02	.323E+05
.489E+02	.323E+05
.729E+02	.323E+05

O U T P U T   I N F O R M A T I O N
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*************************************************
* COMPUTE LOAD-DISTRIBUTION AND LOAD-DEFLECTION *
* CURVES FOR LATERAL LOADING                    *
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LOADING NUMBER 1

BOUNDARY CONDITION CODE	=	2
LATERAL LOAD AT THE PILE HEAD	=	.500E+05 LBS
SLOPE AT THE PILE HEAD	=	.000E+00 IN/IN
AXIAL LOAD AT THE PILE HEAD	=	.000E+00 LBS

X IN	DEFLECTION IN	MOMENT LBS-IN	SHEAR LBS	SLOPE RAD.	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2	SOIL REACTION LBS/IN
*****	*********	*********	*********	*********	*********	*********	*********
.0	.216E-01	-.113E+07	.500E+05	-.100E-18	.582E+03	.888E+11	-.114E+04
17.3	.197E-01	-.433E+06	.309E+05	-.152E-03	.224E+03	.888E+11	-.107E+04
34.6	.163E-01	-.592E+05	.138E+05	-.199E-03	.306E+02	.888E+11	-.909E+03
51.8	.128E-01	.431E+05	.502E+04	-.201E-03	.223E+02	.888E+11	-.104E+03
69.1	.937E-02	.114E+06	.331E+04	-.186E-03	.591E+02	.888E+11	-.937E+02
86.4	.636E-02	.158E+06	.179E+04	-.159E-03	.815E+02	.888E+11	-.823E+02
103.7	.387E-02	.176E+06	.477E+03	-.127E-03	.912E+02	.888E+11	-.697E+02
121.0	.197E-02	.174E+06	-.833E+03	-.927E-04	.900E+02	.888E+11	-.818E+02
138.2	.665E-03	.147E+06	-.178E+04	-.615E-04	.763E+02	.888E+11	-.281E+02
155.5	-.150E-03	.112E+06	-.197E+04	-.362E-04	.581E+02	.888E+11	.652E+01
172.8	-.587E-03	.793E+05	-.178E+04	-.176E-04	.410E+02	.888E+11	.155E+02
190.1	-.757E-03	.508E+05	-.147E+04	-.492E-05	.263E+02	.888E+11	.204E+02
207.4	-.757E-03	.285E+05	-.111E+04	.279E-05	.147E+02	.888E+11	.208E+02
224.6	-.661E-03	.123E+05	-.775E+03	.676E-05	.637E+01	.888E+11	.185E+02
241.9	-.523E-03	.168E+04	-.487E+03	.812E-05	.870E+00	.888E+11	.149E+02
259.2	-.380E-03	-.450E+04	-.263E+03	.785E-05	.233E+01	.888E+11	.110E+02
276.5	-.252E-03	-.740E+04	-.103E+03	.669E-05	.383E+01	.888E+11	.744E+01
293.8	-.149E-03	-.807E+04	-.157E+00	.519E-05	.417E+01	.888E+11	.447E+01
311.0	-.728E-04	-.740E+04	.577E+02	.368E-05	.383E+01	.888E+11	.222E+01
328.3	-.216E-04	-.607E+04	.827E+02	.237E-05	.314E+01	.888E+11	.669E+00
345.6	.920E-05	-.454E+04	.860E+02	.134E-05	.235E+01	.888E+11	-.292E+00
362.9	.247E-04	-.310E+04	.766E+02	.595E-06	.160E+01	.888E+11	-.794E+00
380.2	.298E-04	-.190E+04	.614E+02	.109E-06	.981E+00	.888E+11	-.972E+00
397.4	.285E-04	-.981E+03	.448E+02	-.170E-06	.507E+00	.888E+11	-.944E+00
414.7	.239E-04	-.347E+03	.297E+02	-.300E-06	.180E+00	.888E+11	-.804E+00
432.0	.181E-04	.463E+02	.174E+02	-.329E-06	.239E-01	.888E+11	-.620E+00
449.3	.125E-04	.255E+03	.830E+01	-.300E-06	.132E+00	.888E+11	-.434E+00
466.6	.777E-05	.333E+03	.218E+01	-.242E-06	.172E+00	.888E+11	-.274E+00
483.8	.414E-05	.330E+03	-.147E+01	-.178E-06	.171E+00	.888E+11	-.148E+00
501.1	.162E-05	.282E+03	-.326E+01	-.118E-06	.146E+00	.888E+11	-.586E-01
518.4	.460E-07	.217E+03	-.378E+01	-.699E-07	.112E+00	.888E+11	-.163E-02
535.7	-.797E-06	.152E+03	-.353E+01	-.340E-07	.785E-01	.888E+11	.298E-01
553.0	-.113E-05	.951E+02	-.291E+01	-.100E-07	.492E-01	.888E+11	.427E-01
570.2	-.114E-05	.512E+02	-.216E+01	.420E-08	.265E-01	.888E+11	.438E-01
587.5	-.984E-06	.204E+02	-.145E+01	.112E-07	.105E-01	.888E+11	.383E-01
604.8	-.757E-06	.997E+00	-.864E+00	.132E-07	.516E-03	.888E+11	.298E-01

622.1	-.527E-06	-.947E+01	-.424E+00	.124E-07	.490E-02	.888E+11	.210E-01
639.4	-.328E-06	-.137E+02	-.128E+00	.102E-07	.707E-02	.888E+11	.133E-01
656.6	-.176E-06	-.139E+02	.489E-01	.748E-08	.719E-02	.888E+11	.718E-02
673.9	-.697E-07	-.120E+02	.136E+00	.497E-08	.619E-02	.888E+11	.288E-02
691.2	-.393E-08	-.919E+01	.162E+00	.291E-08	.476E-02	.888E+11	.162E-03
708.5	.309E-07	-.636E+01	.152E+00	.140E-08	.329E-02	.888E+11	-.131E-02
725.8	.443E-07	-.393E+01	.125E+00	.396E-09	.203E-02	.888E+11	-.191E-02
743.0	.446E-07	-.206E+01	.914E-01	-.186E-09	.106E-02	.888E+11	-.194E-02
760.3	.379E-07	-.769E+00	.602E-01	-.461E-09	.398E-03	.888E+11	-.167E-02
777.6	.286E-07	.212E-01	.347E-01	-.534E-09	.110E-04	.888E+11	-.128E-02
794.9	.195E-07	.430E+00	.161E-01	-.490E-09	.223E-03	.888E+11	-.877E-03
812.2	.117E-07	.578E+00	.391E-02	-.392E-09	.299E-03	.888E+11	-.534E-03
829.4	.591E-08	.566E+00	-.306E-02	-.281E-09	.293E-03	.888E+11	-.272E-03
846.7	.201E-08	.472E+00	-.623E-02	-.180E-09	.244E-03	.888E+11	-.937E-04
864.0	-.299E-09	.350E+00	-.692E-02	-.998E-10	.181E-03	.888E+11	.142E-04
881.3	-.143E-08	.233E+00	-.620E-02	-.430E-10	.120E-03	.888E+11	.684E-04
898.6	-.179E-08	.136E+00	-.487E-02	-.717E-11	.703E-04	.888E+11	.861E-04
915.8	-.168E-08	.646E-01	-.342E-02	.123E-10	.334E-04	.888E+11	.819E-04
933.1	-.136E-08	.178E-01	-.213E-02	.203E-10	.919E-05	.888E+11	.670E-04
950.4	-.979E-09	-.907E-02	-.113E-02	.212E-10	.469E-05	.888E+11	.487E-04
967.7	-.628E-09	-.214E-01	-.439E-03	.182E-10	.111E-04	.888E+11	.316E-04
985.0	-.349E-09	-.242E-01	-.125E-04	.138E-10	.125E-04	.888E+11	.177E-04
1002.2	-.152E-09	-.218E-01	.208E-03	.931E-11	.113E-04	.888E+11	.777E-05
1019.5	-.273E-10	-.170E-01	.287E-03	.554E-11	.882E-05	.888E+11	.141E-05
1036.8	.397E-10	-.119E-01	.282E-03	.273E-11	.614E-05	.888E+11	-.208E-05
1054.1	.669E-10	-.731E-02	.233E-03	.861E-12	.378E-05	.888E+11	-.354E-05
1071.4	.695E-10	-.381E-02	.171E-03	-.220E-12	.197E-05	.888E+11	-.371E-05
1088.6	.592E-10	-.141E-02	.111E-03	-.728E-12	.731E-06	.888E+11	-.319E-05
1105.9	.443E-10	.287E-04	.626E-04	-.862E-12	.149E-07	.888E+11	-.241E-05
1123.2	.294E-10	.750E-03	.278E-04	-.787E-12	.388E-06	.888E+11	-.162E-05
1140.5	.171E-10	.988E-03	.559E-05	-.618E-12	.511E-06	.888E+11	-.948E-06
1157.8	.809E-11	.943E-03	-.652E-05	-.430E-12	.488E-06	.888E+11	-.453E-06
1175.0	.225E-11	.763E-03	-.115E-04	-.264E-12	.395E-06	.888E+11	-.127E-06
1192.3	-.103E-11	.545E-03	-.121E-04	-.137E-12	.282E-06	.888E+11	.587E-07
1209.6	-.248E-11	.344E-03	-.104E-04	-.504E-13	.178E-06	.888E+11	.143E-06
1226.9	-.277E-11	.185E-03	-.777E-05	.103E-14	.959E-07	.888E+11	.161E-06
1244.2	-.244E-11	.751E-04	-.515E-05	.264E-13	.388E-07	.888E+11	.143E-06
1261.4	-.186E-11	.746E-05	-.296E-05	.344E-13	.386E-08	.888E+11	.110E-06
1278.7	-.125E-11	-.274E-04	-.137E-05	.325E-13	.142E-07	.888E+11	.746E-07
1296.0	-.737E-12	-.399E-04	-.344E-06	.259E-13	.207E-07	.888E+11	.443E-07
1313.3	-.356E-12	-.393E-04	.225E-06	.182E-13	.203E-07	.888E+11	.216E-07
1330.6	-.108E-12	-.322E-04	.469E-06	.113E-13	.166E-07	.888E+11	.659E-08
1347.8	.325E-13	-.231E-04	.509E-06	.588E-14	.119E-07	.888E+11	-.201E-08
1365.1	.953E-13	-.146E-04	.440E-06	.222E-14	.754E-08	.888E+11	-.593E-08
1382.4	.109E-12	-.786E-05	.330E-06	.386E-16	.406E-08	.888E+11	-.685E-08
1399.7	.967E-13	-.318E-05	.218E-06	-.103E-14	.165E-08	.888E+11	-.611E-08
1417.0	.734E-13	-.329E-06	.125E-06	-.138E-14	.170E-09	.888E+11	-.468E-08
1434.2	.491E-13	.112E-05	.568E-07	-.130E-14	.582E-09	.888E+11	-.315E-08
1451.5	.285E-13	.164E-05	.136E-07	-.103E-14	.846E-09	.888E+11	-.185E-08
1468.8	.135E-13	.159E-05	-.999E-08	-.716E-15	.825E-09	.888E+11	-.880E-09
1486.1	.379E-14	.129E-05	-.198E-07	-.436E-15	.667E-09	.888E+11	-.249E-09
1503.4	-.158E-14	.912E-06	-.210E-07	-.222E-15	.472E-09	.888E+11	.105E-09
1520.6	-.388E-14	.564E-06	-.179E-07	-.783E-16	.292E-09	.888E+11	.259E-09
1537.9	-.428E-14	.294E-06	-.131E-07	.513E-17	.152E-09	.888E+11	.289E-09
1555.2	-.370E-14	.110E-06	-.847E-08	.445E-16	.570E-10	.888E+11	.251E-09
1572.5	-.275E-14	.143E-08	-.467E-08	.553E-16	.738E-12	.888E+11	.188E-09
1589.8	-.179E-14	-.513E-07	-.199E-08	.505E-16	.265E-10	.888E+11	.123E-09
1607.0	-.100E-14	-.672E-07	-.319E-09	.389E-16	.348E-10	.888E+11	.696E-10
1624.3	-.442E-15	-.623E-07	.550E-09	.263E-16	.322E-10	.888E+11	.309E-10
1641.6	-.919E-16	-.482E-07	.873E-09	.156E-16	.249E-10	.888E+11	.646E-11
1658.9	.965E-16	-.322E-07	.870E-09	.778E-17	.166E-10	.888E+11	-.687E-11
1676.2	.177E-15	-.182E-07	.701E-09	.289E-17	.939E-11	.888E+11	-.127E-10
1693.4	.196E-15	-.792E-08	.470E-09	.351E-18	.410E-11	.888E+11	-.141E-10
1710.7	.189E-15	-.191E-08	.229E-09	-.606E-18	.990E-12	.888E+11	-.137E-10
1728.0	.175E-15	.000E+00	.000E+00	-.792E-18	.000E+00	.888E+11	-.128E-10

OUTPUT VERIFICATION

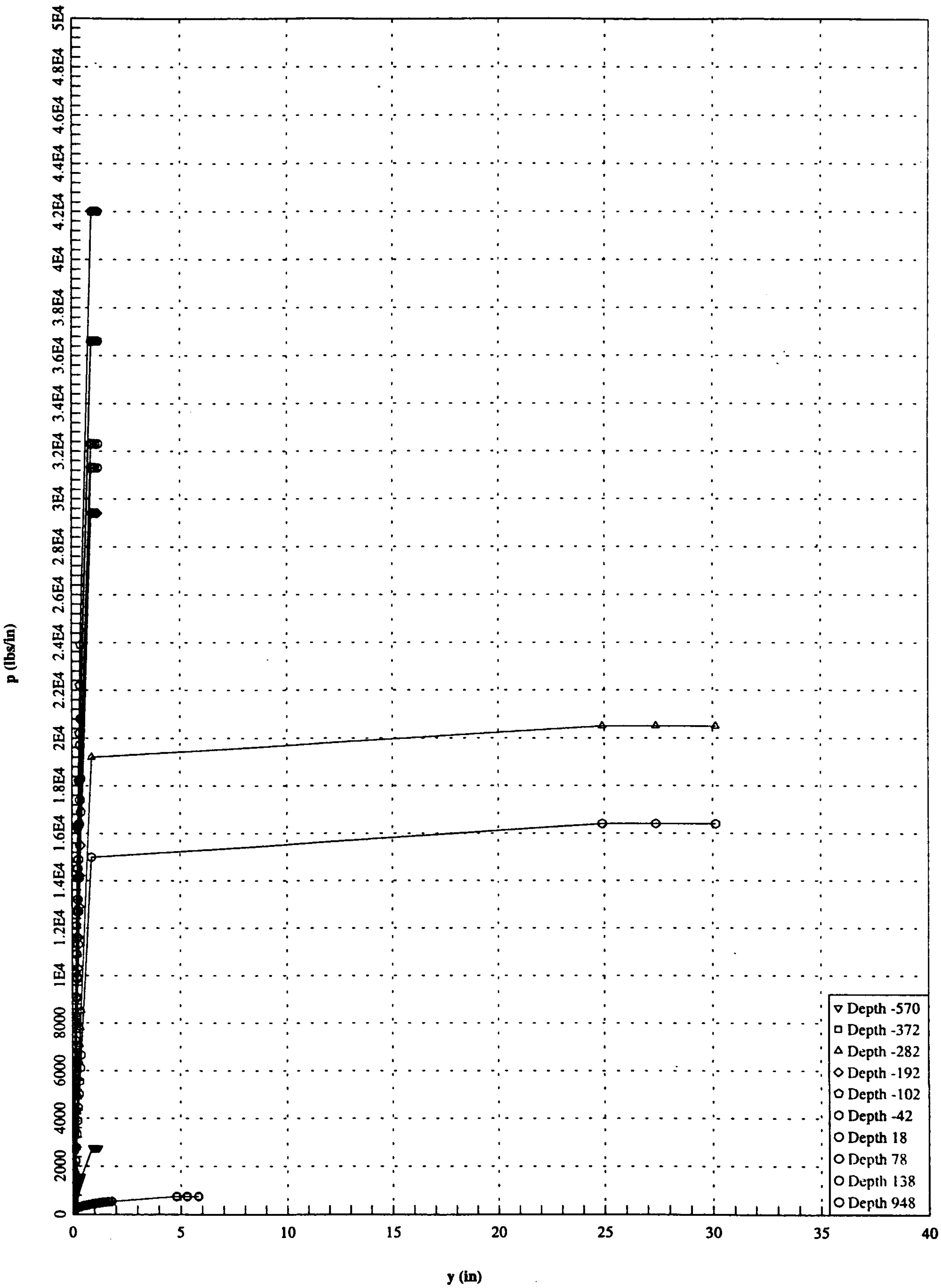
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.896E-08 IN-LBS

```
    THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT =   .207E-09 LBS

OUTPUT SUMMARY

    PILE-HEAD DEFLECTION          =  .216E-01 IN
    COMPUTED SLOPE AT PILE HEAD   = -.100E-18
    MAXIMUM BENDING MOMENT        = -.113E+07 LBS-IN
    MAXIMUM SHEAR FORCE           =  .500E+05 LBS
    NO. OF ITERATIONS             =        6
    NO. OF ZERO DEFLECTION POINTS =       10
```

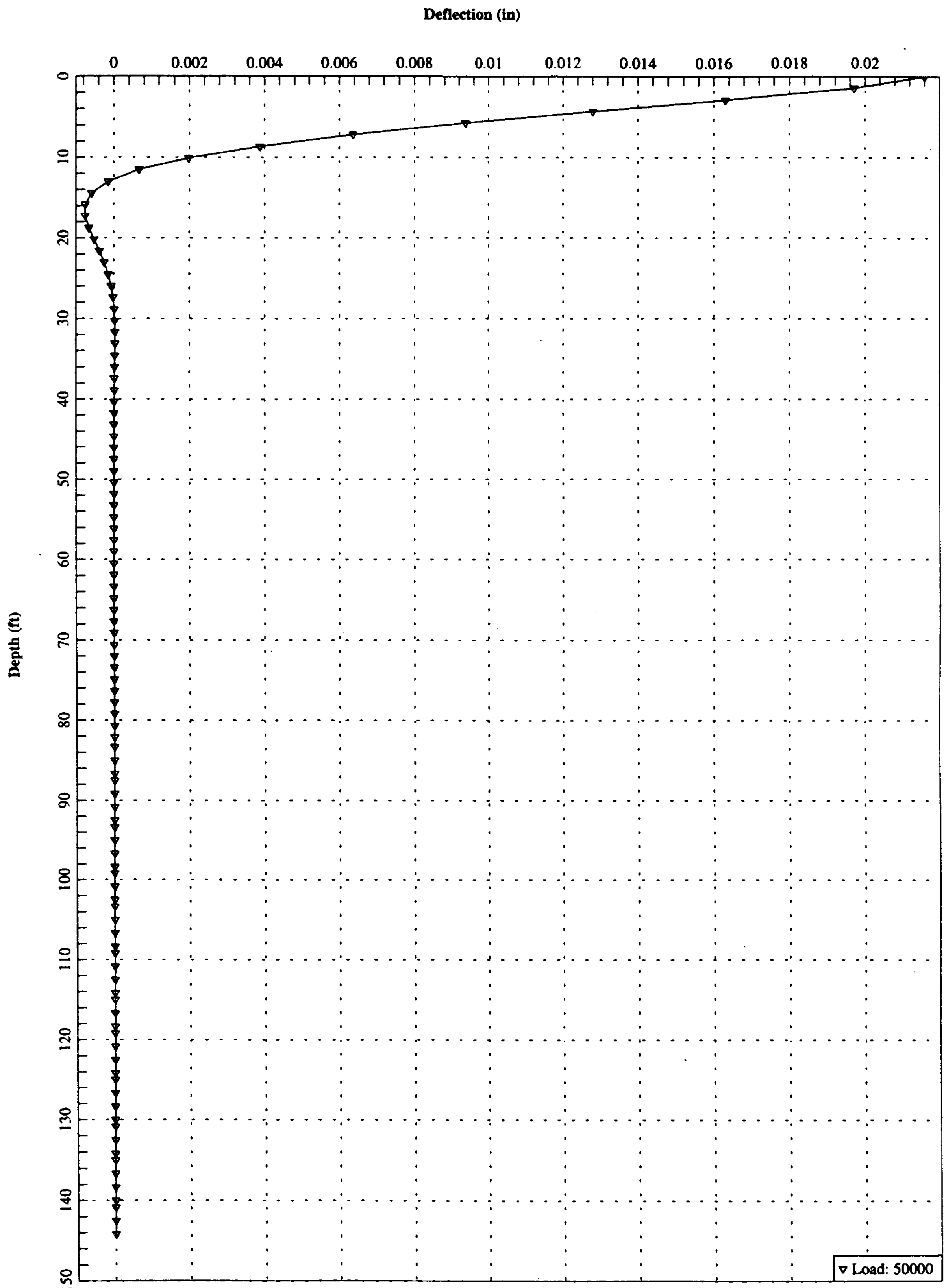
S U M M A R Y   T A B L E
****

BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
.5000E+05	.0000E+00	.0000E+00	.2156E-01	-.1126E+07	.5000E+05

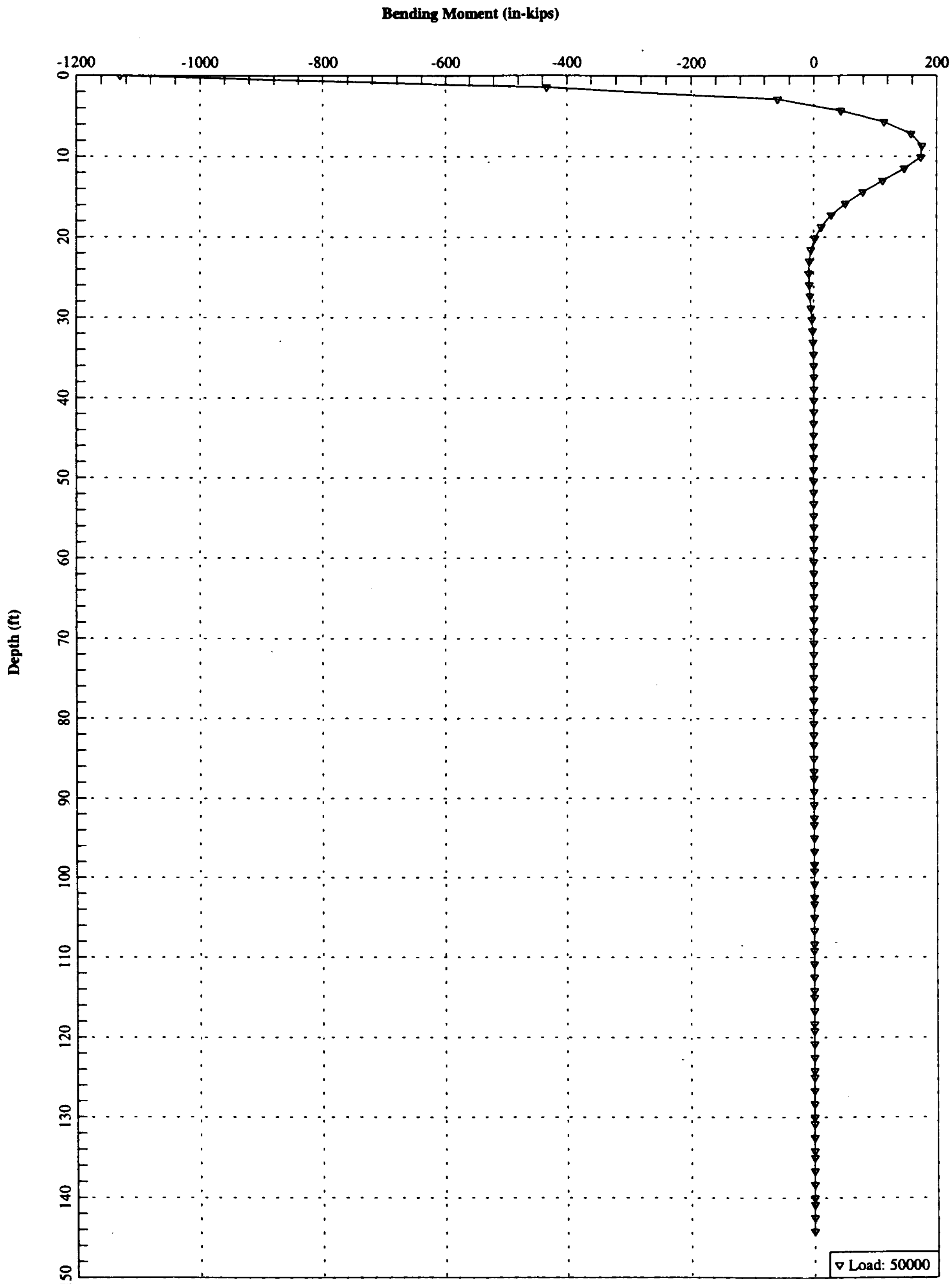
Pier 4 (Bent 3)


p (lbs/in)
y (in)
0
2000
4000
6000
8000
1E4
1.2E4
1.4E4
1.6E4
1.8E4
2E4
2.2E4
2.4E4
2.6E4
2.8E4
3E4
3.2E4
3.4E4
3.6E4
3.8E4
4E4
4.2E4
4.4E4
4.6E4
4.8E4
5E4
0
5
10
15
20
25
30
35
40
Depth -570
Depth -372
Depth -282
Depth -192
Depth -102
Depth -42
Depth 18
Depth 78
Depth 138
Depth 948

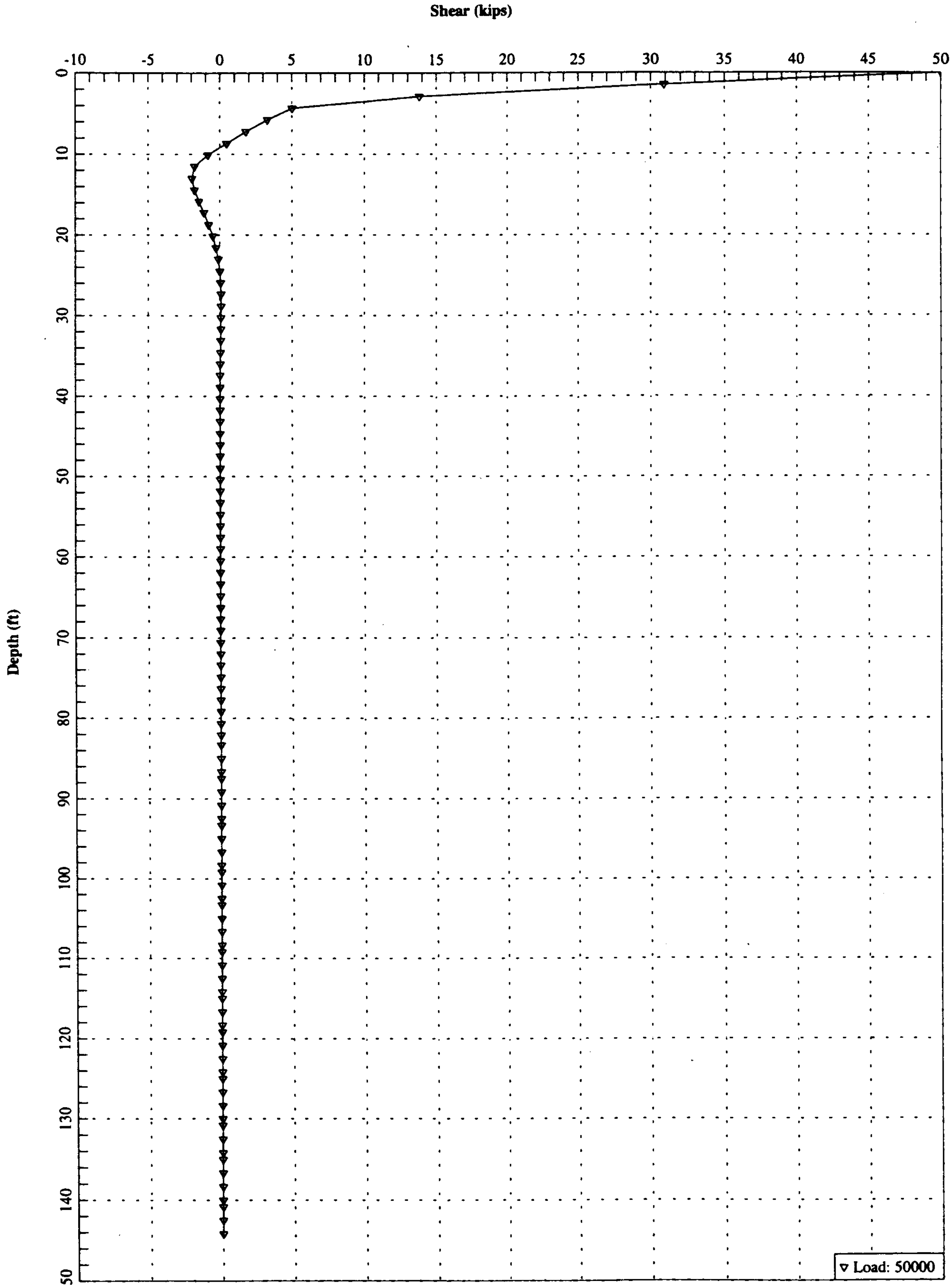
Pier 4 (Bent 3)


Deflection (in)
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
0.018
0.02
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
▿ Load: 50000

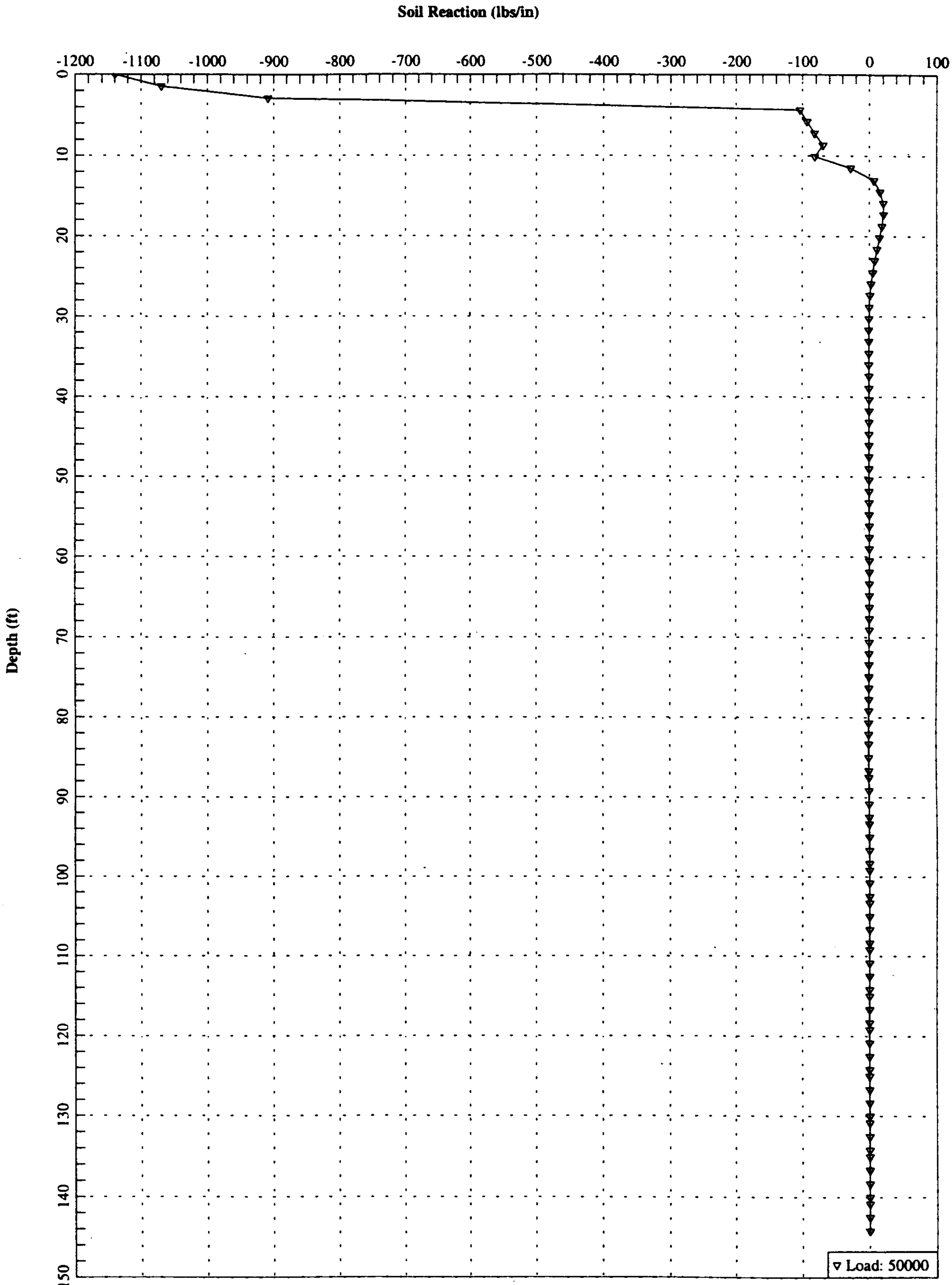
Pier 4 (Bent 3)


Bending Moment (in-kips)
-1200
-1000
-800
-600
-400
-200
0
200
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
▽ Load: 50000

Pier 4 (Bent 3)


Shear (kips)
-10
-5
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
▽ Load: 50000

Pier 4 (Bent 3)


Soil Reaction (lbs/in)
-1200
-1100
-1000
-900
-800
-700
-600
-500
-400
-300
-200
-100
0
100
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
▽ Load: 50000

PILE PULL-OUT CAPACITY

(ASSUME GROUP EFFECT IS 0.5 FOR 4D FOR UPLIFT)

☐ TENSION CAPACITY OF PILE

- ASSUME CONCRETE IS CRACKED & STEEL IS YIELDING (SAY, PIPE & REBAR YIELD AT SAME TIME)

$$T_{Py} = f_y A_{pipe} + f_y A_{rebar} = (36\,ksi)(36.9\,in^2) + (60\,ksi)(8.7\,in^2)$$

2.1% reinf

$$= 1328^k + 522^k \quad \Rightarrow \underline{\underline{T_{Py} = 1850^k}} \quad \rho = 2.1\%$$

If $A_{rebar} = 12.64\,in^2$ then $(T_{Py} = 2087^k)$ $\rho = 3.0\%$

☐ PULLOUT CAPACITY OF PILE

- ULTIMATE PULLOUT CAPACITY $= T_u = \Sigma A_s f_s + W_p$

WHERE $\Sigma A_s f_s$ = skin resistance capacity

W_p = weight of pile being pulled (no group factor)

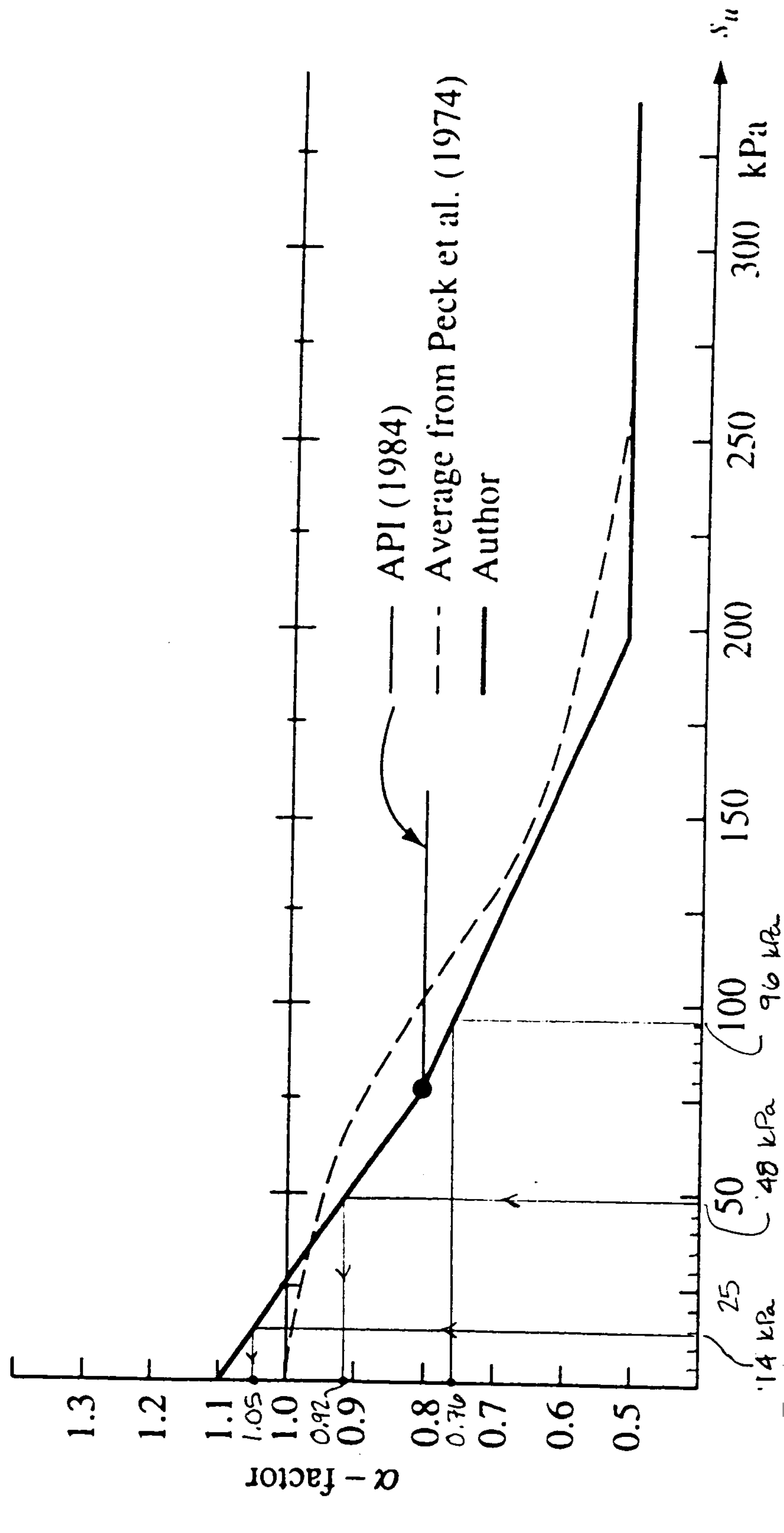
$$W_p = (13' - (-180'))\left[(0.490\,kcf)(36.9\,in^2) + (0.150\,kcf)(415.5\,in^2)\right]/144\,in^2/ft^2$$

$$= 193'(0.126\,klf + 0.433\,klf) = 24.2^k + 83.5^k$$

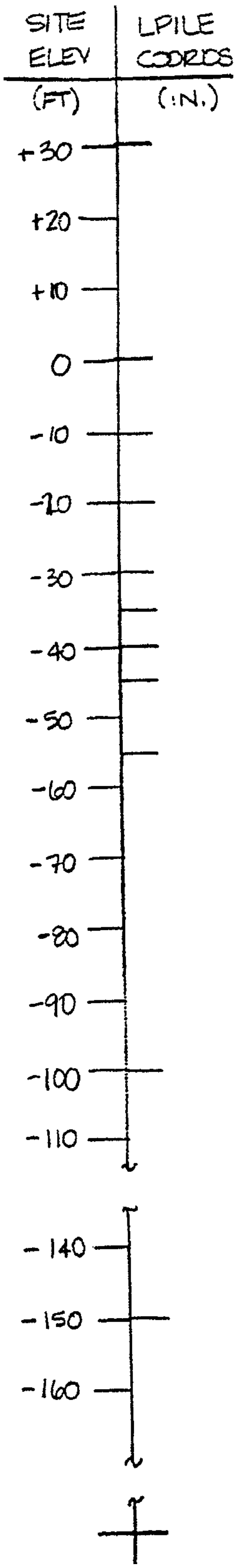
$$\Rightarrow \underline{\underline{W_p = 107.8^k}}$$

$A_s = C_{pile}\,\Delta L$, C_{pile} = perimeter & ΔL = soil layer thickness

$$C_{pile} = \pi d = \pi(24") = 75.4" \quad \Rightarrow \underline{\underline{C_{pile} = 6.28\,ft}}$$



Relationship between the adhesion factor α and undrained shear strength s_u (from sources noted).



PIER PILE

LIQUEFIED SOIL PROFILE

① SAND (FILL) γ = 110 pcf = 0.064 pci
k = 150 pci → 75 pci for Group Effect (GE)
Ø = 37° → 31° (GE)

→ 3.47 psi (GE), typ

② SOFT CLAY γ = 120 pcf = 0.069 pci k = 10 pci → 5 pci (GE)
C_u = 1000 psf = 6.94 psi ε_{50} = 0.010

③ SOFT CLAY γ = 60 pcf = 0.035 pci k = 3 pci → 1.5 pci (GE)
C_u = 300 psf = 2.08 psi → 1.04 (GE) ε_{50} = 0.020

④ SAND γ = 60 pcf Ø = 30°
k = 40 pci → 20 pci (GE)

⑤ SAND γ = 60 pcf k = 40 pci Ø = 24°

⑥ SAND γ = 60 pcf k = 40 pci Ø = 30°

⑦ SOFT CLAY γ = 60 pcf K = 10 pci C_u = 1000 psf ε_{50} = 0.01

⑧ SOFT CLAY γ = 60 pcf k = 3 pci
C_u = 300 psf ε_{50} = 0.020

⑨ SAND γ = 60 pcf
k = 50 pci → 25 pci (GE)
Ø = 26° → 21° (GE)

⑩ STIFF CLAY W/O FREE WATER
γ = 60 pcf
k = 200 pci → 100 pci (GE)
C_u = 2000 psf = 13.89 psi → 6.95 psi C=
ε_{50} = 0.005

☐ PILE SKIN RESISTANCE CAPACITY (Bowles, *Foundation Analysis and Design*, 5th Ed., pp 885-905)

• RECOMMENDED METHODS:

- α METHOD → NORMALLY CONSOLIDATED CLAY
- λ METHOD → OVERCONSOLIDATED CLAY
- β METHOD → COHESIONLESS SOILS

• ASSUME LIQUEFIED SAND LAYERS WHICH ARE MODELED AS "SOFT CLAY" ARE NORMALLY CONSOLIDATED CLAYS FOR WHICH THE α METHOD IS USED.

• SOFT CLAY LAYERS (α METHOD)

$\tan\delta = 0$ if use $\delta = 0$ when $\phi = 0$

$$f_s = \alpha C + \cancel{\bar{q} K \tan\delta} \qquad \text{GENERAL FORM (NOT USED MUCH)}$$

$$f_s = \alpha C \text{ or } \alpha S_u \qquad \text{TYPICAL FORM}$$

GROUP EFFECT

LAYER	C_u/S_u	THKN. ΔL	α (FIG. 16-14)	$f_s = \alpha C_u$	$f_s C_{pile}$	$f_s A_s$	$T_u = 0.5(f_s A_s)$	
2	1.0 ksf = 47.9 kPa	10 ft	0.92	0.920 ksf	5.78 klf	57.8^k	28.9	
3	0.3 ksf = 14.4 kPa	10 ft	1.05	0.315 ksf	1.98 klf	19.8^k	9.9	(L)
7	1.0 ksf = 47.9 kPa	5 ft	0.92	0.920 ksf	5.78 klf	28.9^k	14.5	
8	0.3 ksf = 14.4 kPa	10 ft	1.05	0.315 ksf	1.98 klf	19.8^k	9.9	(L)
10	2.0 ksf = 95.8 kPa	80 ft	0.76	1.52 ksf	9.55 klf	764^k	382	

$$\Sigma = T_{u_{clay}} = 445^k$$

THIS VALUE IS TOO HIGH BECAUSE LAYERS 2, 3, 7 & 8 ARE LIQUEFIED, SO ASSUME NO SKIN RESISTANCE EXISTS. ASSUME ONLY LAYER 10 PROVIDES ANY SKIN RESISTANCE

$$\boxed{T_{u_{clay}} = 425^k}$$

- SAND LAYERS (β METHOD)

$$f_s = K\bar{q}\tan\delta = \beta\bar{q}, \quad \text{where} \quad \beta = K\tan\delta$$

- SOLVE FOR β

$K = K_o = 1 - \sin\phi$ IS COMMONLY USED

$\delta = \phi'$

ASSUME $\phi \approx \phi'$

LAYER	$\phi\ (=\delta)$	$K = 1 - \sin\phi$	$\beta = K\tan\delta$
1	31°	0.48	0.29
4	23°	0.61	0.26
5	19°	0.67	0.23
6	23°	0.61	0.26
9	21°	0.64	0.25

ADJUSTED FOR GROUP EFFECT

GROUP EFFECT CONSIDERED IN β

LAYER	γ'	ΔL LAYER THICKNESS	$\bar{q}$	$f_s = \beta\bar{q}$	$A_s = C_{pile}\Delta L$	$T_u = f_s A_s$
1	0.110 kcf	13'	0.715 ksf	0.208 ksf	81.7 ft²	17.0
4	0.120 kcf	10'	2.03 ksf	0.525 ksf	62.8 ft²	33.0
5	0.060 kcf	5'	2.78 ksf	0.646 ksf	31.4 ft²	20.3
6	0.060 kcf	5'	3.08 ksf	0.797 ksf	31.4 ft²	25.0
9	0.060 kcf	45'	4.58 ksf	1.128 ksf	282.7 ft²	318.9

$C_{pile} = 6.28$ ft

$$\Sigma = \boxed{414^k = T_{U_{sand}}}$$

Sample Calc

$\bar{q}_{1} = (.110\text{kcf})\left(\frac{13'}{2}\right) = 0.715$

$\bar{q}_{4} = (0.11)(13') + (0.12)\left(\frac{10'}{2}\right) = 1.43 + 0.6 = 2.03$

$\bar{q}_{5} = (1.43) + (0.12)(10') + (0.06)(2.5') = 1.43 + 1.20 + 0.15 = 2.78$

∴ PILE PULLOUT CAPACITY (ULTIMATE)
= SKIN FRICTION OF SOIL LAYERS + PILE WEIGHT

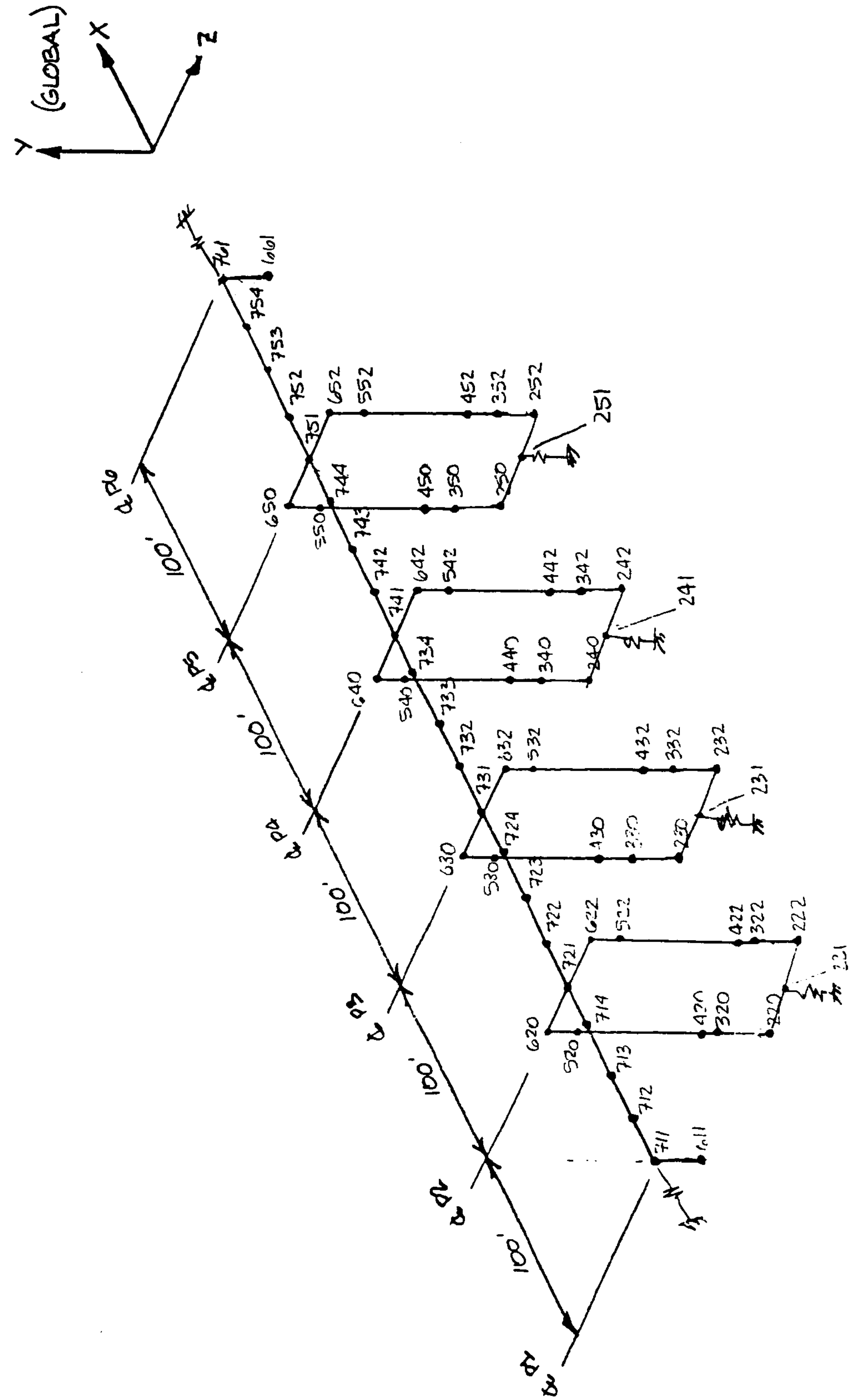
$$T_u = T_{u_{clay}} + T_{u_{sand}} + W_p \qquad (T_{u_{clay}} = \text{liquefied sand})$$

$$= 382^k + 414^k + 108^k$$

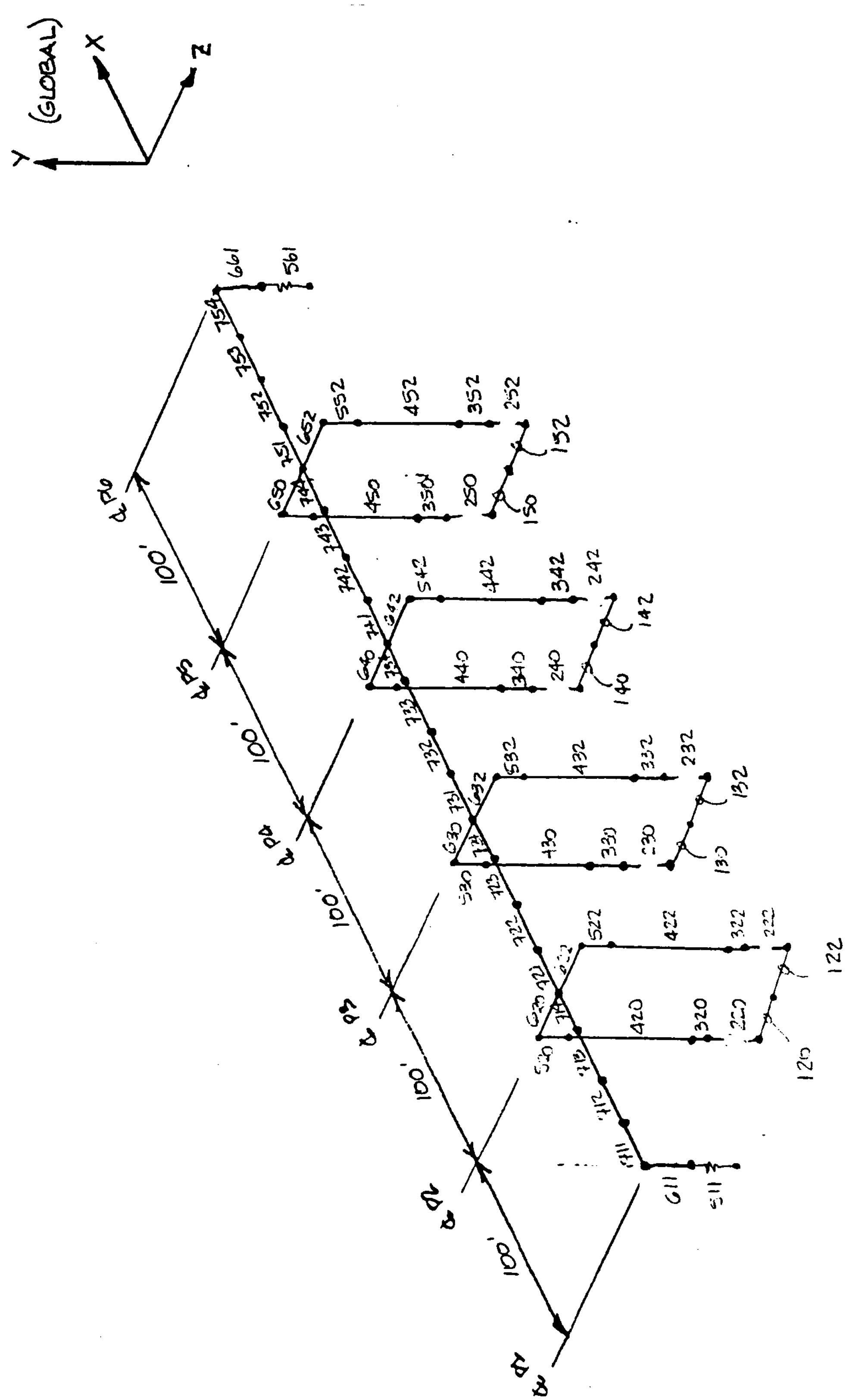
$$\Rightarrow \boxed{T_u = 904^k}$$

[SINCE PILE TENSILE CAPACITY IS 1850^k, THEN PULLOUT WILL OCCUR FIRST.]

Appendix B
SAP2000 Input



Joint Numbers



Member Numbers

NCHRP 12-49 DESIGN EXAMPLE 8/ 2500 YR EQ / NON-LIQUEFIED / FOUNDATION SPRINGS

```
; NCHRP 12-49
; WASHINGTON SITE, DESIGN EXAMPLE NO. 8
;
; STRUCTURE DESCRIPTION
; SUPERSTRUCTURE = FIVE 100-FT SPANS, CIP POST-TENSIONED BOX GIRDER
;                  WITH 9-INCH MIDSPAN DIAPHRAGMS
;                  AND FOUR 12-INCH WEBS
; SUBSTRUCTURE   = 2-COLUMN 48-IN DIAMETER PIERS, NO SKEW,
;                  5-FT WIDE CROSSBEAMS
; FOUNDATIONS    = END AND INT PIERS SUPPORTED BY PILE CAP WITH CIP
;                  CONCRETE PILES WITH 24-IN STEEL CASINGS
;                = END DIAPHRAGMS IN CONTACT W/SOIL FOR LONGITUDINAL
;                  RESISTANCE
;                = SEALS UNDER INT PIER PILE CAPS, ASSUME 3KSI & 140PCF
;                  3FT AT PIER 2, 4FT AT PIERS 3&5, 6FT AT PIER4
;
; MODEL DESCRIPTION
;                = INTERMEDIATE PIERS ON FOUNDATION SPRINGS
;                = ABUTMENTS ON FOUNDATION SPRINGS
;                = LONGITUDINAL SPRING AT SUPERSTRUCTURE ENDS
;
;
; File O:\1999\A99067\ENGR\WASHINGTON\SAP2000\2500YR N\WA2500N.SDB

SYSTEM

DOF=UX,UY,UZ,RX,RY,RZ  LENGTH=FT  FORCE=KIP  PAGE=SECTIONS

JOINT
; SUPERSTRUCTURE AT CG OF BOX GIRDER
711  X=   0.00  Y= 28.38  Z=  0.00
712  X=  25.00  Y= 28.38  Z=  0.00
713  X=  50.00  Y= 28.38  Z=  0.00
714  X=  75.00  Y= 28.38  Z=  0.00
721  X= 100.00  Y= 28.38  Z=  0.00
722  X= 125.00  Y= 28.38  Z=  0.00
723  X= 150.00  Y= 28.38  Z=  0.00
724  X= 175.00  Y= 28.38  Z=  0.00
731  X= 200.00  Y= 28.38  Z=  0.00
732  X= 225.00  Y= 28.38  Z=  0.00
733  X= 250.00  Y= 28.38  Z=  0.00
734  X= 275.00  Y= 28.38  Z=  0.00
741  X= 300.00  Y= 28.38  Z=  0.00
742  X= 325.00  Y= 28.38  Z=  0.00
743  X= 350.00  Y= 28.38  Z=  0.00
744  X= 375.00  Y= 28.38  Z=  0.00
751  X= 400.00  Y= 28.38  Z=  0.00
752  X= 425.00  Y= 28.38  Z=  0.00
753  X= 450.00  Y= 28.38  Z=  0.00
754  X= 475.00  Y= 28.38  Z=  0.00
761  X= 500.00  Y= 28.38  Z=  0.00
;
; PIER 1
611  X=   0.00  Y= 25.00  Z=  0.00
;
```

```
; PIER 2
620  X= 100.00  Y= 28.38  Z= -11.26
520  X= 100.00  Y= 25.00  Z= -11.26
420  X= 100.00  Y= -5.00  Z= -11.26
320  X= 100.00  Y=-10.00  Z= -11.26
220  X= 100.00  Y=-13.00  Z= -11.26
;
622  X= 100.00  Y= 28.38  Z=  11.26
522  X= 100.00  Y= 25.00  Z=  11.26
422  X= 100.00  Y= -5.00  Z=  11.26
322  X= 100.00  Y=-10.00  Z=  11.26
222  X= 100.00  Y=-13.00  Z=  11.26
;
221  X= 100.00  Y=-13.00  Z=   0.00
;
; PIER 3
630  X= 200.00  Y= 28.38  Z= -11.26
530  X= 200.00  Y= 25.00  Z= -11.26
430  X= 200.00  Y=-20.00  Z= -11.26
330  X= 200.00  Y=-25.00  Z= -11.26
230  X= 200.00  Y=-29.00  Z= -11.26
;
632  X= 200.00  Y= 28.38  Z=  11.26
532  X= 200.00  Y= 25.00  Z=  11.26
432  X= 200.00  Y=-20.00  Z=  11.26
332  X= 200.00  Y=-25.00  Z=  11.26
232  X= 200.00  Y=-29.00  Z=  11.26
;
231  X= 200.00  Y=-29.00  Z=   0.00
;
; PIER 4
640  X= 300.00  Y= 28.38  Z= -11.26
540  X= 300.00  Y= 25.00  Z= -11.26
440  X= 300.00  Y=-25.00  Z= -11.26
340  X= 300.00  Y=-30.00  Z= -11.26
240  X= 300.00  Y=-36.00  Z= -11.26
;
642  X= 300.00  Y= 28.38  Z=  11.26
542  X= 300.00  Y= 25.00  Z=  11.26
442  X= 300.00  Y=-25.00  Z=  11.26
342  X= 300.00  Y=-30.00  Z=  11.26
242  X= 300.00  Y=-36.00  Z=  11.26
;
241  X= 300.00  Y=-36.00  Z=   0.00
;
; PIER 5
650  X= 400.00  Y= 28.38  Z= -11.26
550  X= 400.00  Y= 25.00  Z= -11.26
450  X= 400.00  Y=-20.00  Z= -11.26
350  X= 400.00  Y=-25.00  Z= -11.26
250  X= 400.00  Y=-29.00  Z= -11.26
;
652  X= 400.00  Y= 28.38  Z=  11.26
552  X= 400.00  Y= 25.00  Z=  11.26
452  X= 400.00  Y=-20.00  Z=  11.26
352  X= 400.00  Y=-25.00  Z=  11.26
252  X= 400.00  Y=-29.00  Z=  11.26
```

```
;
251  X= 400.00  Y=-29.00  Z=   0.00
;
; PIER 6
661  X= 500.00  Y= 25.00  Z=   0.00


;RESTRAINT
;  ADD=611,661,50 DOF=UY,UZ,RX    ;Abutments
;  ADD=221,251,10 DOF=ALL         ;Int Piers


SPRING

CSYS=0
  ADD=221        UX=3.33E5 UY=2.47E5 UZ=3.21E5 RX=7.59E8 RY=1.19E9 RZ=1.90E8
  ADD=231        UX=3.43E5 UY=2.73E5 UZ=3.31E5 RX=8.39E8 RY=1.23E9 RZ=2.10E8
  ADD=241        UX=4.59E5 UY=2.86E5 UZ=4.47E5 RX=8.80E8 RY=1.68E9 RZ=2.20E8
  ADD=251        UX=3.43E5 UY=2.73E5 UZ=3.31E5 RX=8.39E8 RY=1.23E9 RZ=2.10E8
  ADD=611,661,50 UX=0.0    UY=1.60E5 UZ=7.30E4 RX=2.99E7 RY=0.0    RZ=0.0
  ADD=711,761,50 U1=375.0


MASS
; ADD 15.3 KIPS AT EACH OF THE FIVE INTERMEDIATE DIAPHRAGMS
  ADD=713,753,10 UX=15.3/32.2 UY=15.3/32.2 UZ=15.3/32.2

; ADD 117.6 KIPS AT EACH OF THE TWO END DIAPHRAGMS
  ADD=711,761,50 UX=117.6/32.2 UY=117.6/32.2 UZ=117.6/32.2

; ADD 102.0 KIPS AT EACH OF THE FOUR INTERMEDIATE PIER CROSS BEAMS
  ADD=721,751,10 UX=102.0/32.2 UY=102.0/32.2 UZ=102.0/32.2


MATERIAL

; WEIGHT OF SUPERSTRUCTURE = (0.15K/FT^3)*(72.74FT^2) = 10.83 K/FT
; SUPERIMPOSED DEAD LOAD   = 2.19 K/FT
; TOTAL DEAD LOAD          = 10.83+2.19 = 13.02 K/FT
;                        ==> 13.02/72.18 = 0.180 K/FT^3
  NAME=SUPER TYPE=ISO M=0.180/32.2  W=0.180  IDES=C
    E=552000  U=0.18  A=6.0E-06
  NAME=SUB   TYPE=ISO M=0.150/32.2  W=0.150  IDES=C
    E=552000  U=0.18  A=6.0E-06
  NAME=RIGID TYPE=ISO M=0.0         W=0.0    IDES=C
    E=552000  U=0.18  A=6.0E-06
  NAME=3KSI  TYPE=ISO M=0.140/32.2  W=0.140  IDES=C
    E=431000  U=0.18  A=6.0E-06


FRAME SECTION

; ASSUME 0.4*Ig FOR COLUMN CRACKED SECTION PROPERTIES
;   FOR COLUMN  Jg=25.13FT^4 AND Ig=12.57FT^4
  NAME=SUPER TYPE=PRISM MAT=SUPER SH=G A=72.18  J=1177.0    I=401.0,9697.0
  NAME=XBEAM TYPE=PRISM MAT=SUB   SH=G A=27.00  J=10000.0   I=10000.0,10000.0
  NAME=LINK  TYPE=PRISM MAT=RIGID SH=G A=10.00  J=10000.0   I=10000.0,10000.0
```

```
    NAME=COL   TYPE=PRISM MAT=SUB   SH=G A=12.57  J=10.0      I=5.0,5.0
    NAME=FTG   TYPE=PRISM MAT=SUB   SH=G A=506.0  J=109634.0  I=89225.0,20409.0
    NAME=SEAL  TYPE=PRISM MAT=3KSI  SH=G A=196.0  J=6403.0    I=89225.0,20409.0

  FRAME
    CSYS=0
    711 J=711,712 SEC=SUPER
    712 J=712,713 SEC=SUPER
    713 J=713,714 SEC=SUPER
    714 J=714,721 SEC=SUPER
    721 J=721,722 SEC=SUPER
    722 J=722,723 SEC=SUPER
    723 J=723,724 SEC=SUPER
    724 J=724,731 SEC=SUPER
    731 J=731,732 SEC=SUPER
    732 J=732,733 SEC=SUPER
    733 J=733,734 SEC=SUPER
    734 J=734,741 SEC=SUPER
    741 J=741,742 SEC=SUPER
    742 J=742,743 SEC=SUPER
    743 J=743,744 SEC=SUPER
    744 J=744,751 SEC=SUPER
    751 J=751,752 SEC=SUPER
    752 J=752,753 SEC=SUPER
    753 J=753,754 SEC=SUPER
    754 J=754,761 SEC=SUPER

    620 J=620,721 SEC=XBEAM
    622 J=622,721 SEC=XBEAM
    630 J=630,731 SEC=XBEAM
    632 J=632,731 SEC=XBEAM
    640 J=640,741 SEC=XBEAM
    642 J=642,741 SEC=XBEAM
    650 J=650,751 SEC=XBEAM
    652 J=652,751 SEC=XBEAM

    611 J=611,711 SEC=LINK
    661 J=661,761 SEC=LINK

    220 J=220,320 SEC=SEAL
    320 J=320,420 SEC=FTG
    420 J=420,520 SEC=COL
    520 J=520,620 SEC=LINK

    222 J=222,322 SEC=SEAL
    322 J=322,422 SEC=FTG
    422 J=422,522 SEC=COL
    522 J=522,622 SEC=LINK

    230 J=230,330 SEC=SEAL
    330 J=330,430 SEC=FTG
    430 J=430,530 SEC=COL
    530 J=530,630 SEC=LINK

    232 J=232,332 SEC=SEAL
    332 J=332,432 SEC=FTG
    432 J=432,532 SEC=COL
```

```
   532 J=532,632 SEC=LINK

   240 J=240,340 SEC=SEAL
   340 J=340,440 SEC=FTG
   440 J=440,540 SEC=COL
   540 J=540,640 SEC=LINK

   242 J=242,342 SEC=SEAL
   342 J=342,442 SEC=FTG
   442 J=442,542 SEC=COL
   542 J=542,642 SEC=LINK

   250 J=250,350 SEC=SEAL
   350 J=350,450 SEC=FTG
   450 J=450,550 SEC=COL
   550 J=550,650 SEC=LINK

   252 J=252,352 SEC=SEAL
   352 J=352,452 SEC=FTG
   452 J=452,552 SEC=COL
   552 J=552,652 SEC=LINK

   120 J=221,220 SEC=LINK
   122 J=221,222 SEC=LINK

   130 J=231,230 SEC=LINK
   132 J=231,232 SEC=LINK

   140 J=241,240 SEC=LINK
   142 J=241,242 SEC=LINK

   150 J=251,250 SEC=LINK
   152 J=251,252 SEC=LINK


LOAD
  CSYS=0
  NAME=DL
    TYPE=GRAVITY ELEM=FRAME
       ADD=* UY=-1

  NAME=TL
    TYPE=TEMPERATURE ELEM=FRAME
       ADD=711,714,1,751,10   T=10


MODES

  TYPE=EIGEN  N=20 ; 5 SPANS AND 4 MODES PER SPAN


FUNCTION

NAME=2500YR NPL=1
0.000   0.423
0.187   1.058
0.933   1.058
```

1.030	0.958
1.100	0.897
1.20	0.822
1.30	0.759
1.40	0.705
1.50	0.658
1.60	0.617
1.70	0.580
1.80	0.548
1.90	0.519
2.00	0.493
2.10	0.470
2.20	0.448
2.30	0.429
2.40	0.411
2.50	0.395
2.60	0.379
2.70	0.365
2.80	0.352
2.90	0.340
3.00	0.329
3.10	0.318
3.20	0.308
3.30	0.299
3.40	0.290
3.50	0.282
3.60	0.274
3.70	0.267
3.80	0.260
3.90	0.253
4.00	0.247
4.10	0.241
4.20	0.235
4.30	0.229
4.40	0.224
4.50	0.219
4.60	0.214
4.70	0.210
4.80	0.206
4.90	0.201
5.00	0.197
5.10	0.193
5.20	0.190
5.30	0.186
5.40	0.183
5.50	0.179
5.60	0.176
5.70	0.173
5.80	0.170
5.90	0.167
6.00	0.164
6.10	0.162
6.20	0.159
6.30	0.157
6.40	0.154
6.50	0.152
6.60	0.149

```
6.70     0.147
6.80     0.145
6.90     0.143
7.00     0.141
7.10     0.139
7.20     0.137
7.30     0.135
7.40     0.133
7.50     0.132
7.60     0.130
7.70     0.128
7.80     0.126
7.90     0.125
8.00     0.123
8.10     0.122
8.20     0.120
8.30     0.119
8.40     0.117
8.50     0.116
8.60     0.115
8.70     0.113
8.80     0.112
8.90     0.111
9.00     0.110
10.00    0.099


SPEC

  CSYS=0
  NAME=EQLONG    MODC=CQC    DAMP=0.05
    ACC=U1 FUNC=2500YR  SF=32.2
  NAME=EQTRAN    MODC=CQC    DAMP=0.05
    ACC=U3 FUNC=2500YR  SF=32.2


COMBO

  NAME=EQ          TYPE=SRSS
    SPEC=EQLONG  SF=1
    SPEC=EQTRAN  SF=1


OUTPUT

  ELEM=JOINT  TYPE=DISP,REAC  LOAD=*  SPEC=*  COMB=*
  ELEM=FRAME  TYPE=FORCE      LOAD=*  SPEC=*  COMB=*
```

NCHRP 12-49 DESIGN EXAMPLE 8/ 100 YR EQ / NON-LIQUEFIED / FOUNDATION SPRINGS

```
; NCHRP 12-49
; WASHINGTON SITE, DESIGN EXAMPLE NO. 8
;
; STRUCTURE DESCRIPTION
; SUPERSTRUCTURE = FIVE 100-FT SPANS, CIP POST-TENSIONED BOX GIRDER
;                  WITH 9-INCH MIDSPAN DIAPHRAGMS
;                  AND FOUR 12-INCH WEBS
; SUBSTRUCTURE   = 2-COLUMN 48-IN DIAMETER PIERS, NO SKEW,
;                  5-FT WIDE CROSSBEAMS
; FOUNDATIONS    = END AND INT PIERS SUPPORTED BY PILE CAP WITH CIP
;                  CONCRETE PILES WITH 24-IN STEEL CASINGS
;                = END DIAPHRAGMS IN CONTACT W/SOIL FOR LONGITUDINAL
;                  RESISTANCE
;                = SEALS UNDER INT PIER PILE CAPS, ASSUME 3KSI & 140PCF
;                  3FT AT PIER 2, 4FT AT PIERS 3&5, 6FT AT PIER4
;
; MODEL DESCRIPTION
;                = INTERMEDIATE PIERS ON FOUNDATION SPRINGS
;                = ABUTMENTS ON FOUNDATION SPRINGS
;                = LONGITUDINAL SPRING AT SUPERSTRUCTURE ENDS
;
;
; File O:\1999\A99067\ENGR\WASHINGTON\SAP2000\2500YR N\WA2500N.SDB

SYSTEM

DOF=UX,UY,UZ,RX,RY,RZ  LENGTH=FT  FORCE=KIP  PAGE=SECTIONS

JOINT
; SUPERSTRUCTURE AT CG OF BOX GIRDER
711  X=   0.00  Y= 28.38  Z=  0.00
712  X=  25.00  Y= 28.38  Z=  0.00
713  X=  50.00  Y= 28.38  Z=  0.00
714  X=  75.00  Y= 28.38  Z=  0.00
721  X= 100.00  Y= 28.38  Z=  0.00
722  X= 125.00  Y= 28.38  Z=  0.00
723  X= 150.00  Y= 28.38  Z=  0.00
724  X= 175.00  Y= 28.38  Z=  0.00
731  X= 200.00  Y= 28.38  Z=  0.00
732  X= 225.00  Y= 28.38  Z=  0.00
733  X= 250.00  Y= 28.38  Z=  0.00
734  X= 275.00  Y= 28.38  Z=  0.00
741  X= 300.00  Y= 28.38  Z=  0.00
742  X= 325.00  Y= 28.38  Z=  0.00
743  X= 350.00  Y= 28.38  Z=  0.00
744  X= 375.00  Y= 28.38  Z=  0.00
751  X= 400.00  Y= 28.38  Z=  0.00
752  X= 425.00  Y= 28.38  Z=  0.00
753  X= 450.00  Y= 28.38  Z=  0.00
754  X= 475.00  Y= 28.38  Z=  0.00
761  X= 500.00  Y= 28.38  Z=  0.00
;
; PIER 1
611  X=   0.00  Y= 25.00  Z=  0.00
;
```

```
; PIER 2
620  X= 100.00  Y= 28.38  Z= -11.26
520  X= 100.00  Y= 25.00  Z= -11.26
420  X= 100.00  Y= -5.00  Z= -11.26
320  X= 100.00  Y=-10.00  Z= -11.26
220  X= 100.00  Y=-13.00  Z= -11.26
;
622  X= 100.00  Y= 28.38  Z=  11.26
522  X= 100.00  Y= 25.00  Z=  11.26
422  X= 100.00  Y= -5.00  Z=  11.26
322  X= 100.00  Y=-10.00  Z=  11.26
222  X= 100.00  Y=-13.00  Z=  11.26
;
221  X= 100.00  Y=-13.00  Z=   0.00
;
; PIER 3
630  X= 200.00  Y= 28.38  Z= -11.26
530  X= 200.00  Y= 25.00  Z= -11.26
430  X= 200.00  Y=-20.00  Z= -11.26
330  X= 200.00  Y=-25.00  Z= -11.26
230  X= 200.00  Y=-29.00  Z= -11.26
;
632  X= 200.00  Y= 28.38  Z=  11.26
532  X= 200.00  Y= 25.00  Z=  11.26
432  X= 200.00  Y=-20.00  Z=  11.26
332  X= 200.00  Y=-25.00  Z=  11.26
232  X= 200.00  Y=-29.00  Z=  11.26
;
231  X= 200.00  Y=-29.00  Z=   0.00
;
; PIER 4
640  X= 300.00  Y= 28.38  Z= -11.26
540  X= 300.00  Y= 25.00  Z= -11.26
440  X= 300.00  Y=-25.00  Z= -11.26
340  X= 300.00  Y=-30.00  Z= -11.26
240  X= 300.00  Y=-36.00  Z= -11.26
;
642  X= 300.00  Y= 28.38  Z=  11.26
542  X= 300.00  Y= 25.00  Z=  11.26
442  X= 300.00  Y=-25.00  Z=  11.26
342  X= 300.00  Y=-30.00  Z=  11.26
242  X= 300.00  Y=-36.00  Z=  11.26
;
241  X= 300.00  Y=-36.00  Z=   0.00
;
; PIER 5
650  X= 400.00  Y= 28.38  Z= -11.26
550  X= 400.00  Y= 25.00  Z= -11.26
450  X= 400.00  Y=-20.00  Z= -11.26
350  X= 400.00  Y=-25.00  Z= -11.26
250  X= 400.00  Y=-29.00  Z= -11.26
;
652  X= 400.00  Y= 28.38  Z=  11.26
552  X= 400.00  Y= 25.00  Z=  11.26
452  X= 400.00  Y=-20.00  Z=  11.26
352  X= 400.00  Y=-25.00  Z=  11.26
252  X= 400.00  Y=-29.00  Z=  11.26
```

```
;
251  X= 400.00  Y=-29.00  Z=   0.00
;
; PIER 6
661  X= 500.00  Y= 25.00  Z=   0.00


;RESTRAINT
;  ADD=611,661,50 DOF=UY,UZ,RX   ;Abutments
;  ADD=221,251,10 DOF=ALL        ;Int Piers


SPRING

CSYS=0
  ADD=221         UX=3.33E5 UY=2.47E5 UZ=3.21E5 RX=7.59E8 RY=1.19E9 RZ=1.90E8
  ADD=231         UX=3.43E5 UY=2.73E5 UZ=3.31E5 RX=8.39E8 RY=1.23E9 RZ=2.10E8
  ADD=241         UX=4.59E5 UY=2.86E5 UZ=4.47E5 RX=8.80E8 RY=1.68E9 RZ=2.20E8
  ADD=251         UX=3.43E5 UY=2.73E5 UZ=3.31E5 RX=8.39E8 RY=1.23E9 RZ=2.10E8
  ADD=611,661,50 UX=0.0    UY=1.60E5 UZ=7.30E4 RX=2.99E7 RY=0.0    RZ=0.0
  ADD=711,761,50 U1=1500.0


MASS
; ADD 15.3 KIPS AT EACH OF THE FIVE INTERMEDIATE DIAPHRAGMS
  ADD=713,753,10 UX=15.3/32.2 UY=15.3/32.2 UZ=15.3/32.2

; ADD 117.6 KIPS AT EACH OF THE TWO END DIAPHRAGMS
  ADD=711,761,50 UX=117.6/32.2 UY=117.6/32.2 UZ=117.6/32.2

; ADD 102.0 KIPS AT EACH OF THE FOUR INTERMEDIATE PIER CROSS BEAMS
  ADD=721,751,10 UX=102.0/32.2 UY=102.0/32.2 UZ=102.0/32.2


MATERIAL

; WEIGHT OF SUPERSTRUCTURE = (0.15K/FT^3)*(72.74FT^2) = 10.83 K/FT
; SUPERIMPOSED DEAD LOAD    = 2.19 K/FT
; TOTAL DEAD LOAD           = 10.83+2.19 = 13.02 K/FT
;                         ==> 13.02/72.18 = 0.180 K/FT^3
  NAME=SUPER TYPE=ISO M=0.180/32.2  W=0.180   IDES=C
    E=552000  U=0.18  A=6.0E-06
  NAME=SUB   TYPE=ISO M=0.150/32.2  W=0.150   IDES=C
    E=552000  U=0.18  A=6.0E-06
  NAME=RIGID TYPE=ISO M=0.0         W=0.0     IDES=C
    E=552000  U=0.18  A=6.0E-06
  NAME=3KSI  TYPE=ISO M=0.140/32.2  W=0.140   IDES=C
    E=431000  U=0.18  A=6.0E-06


FRAME SECTION

; ASSUME 0.4*Ig FOR COLUMN CRACKED SECTION PROPERTIES
;   FOR COLUMN  Jg=25.13FT^4 AND Ig=12.57FT^4
  NAME=SUPER TYPE=PRISM MAT=SUPER SH=G A=72.18  J=1177.0    I=401.0,9697.0
  NAME=XBEAM TYPE=PRISM MAT=SUB   SH=G A=27.00  J=10000.0   I=10000.0,10000.0
  NAME=LINK  TYPE=PRISM MAT=RIGID SH=G A=10.00  J=10000.0   I=10000.0,10000.0
```

```
  NAME=COL   TYPE=PRISM MAT=SUB   SH=G A=12.57  J=10.0      I=5.0,5.0
  NAME=FTG   TYPE=PRISM MAT=SUB   SH=G A=506.0  J=109634.0  I=89225.0,20409.0
  NAME=SEAL  TYPE=PRISM MAT=3KSI  SH=G A=196.0  J=6403.0    I=89225.0,20409.0

FRAME
  CSYS=0
  711 J=711,712 SEC=SUPER
  712 J=712,713 SEC=SUPER
  713 J=713,714 SEC=SUPER
  714 J=714,721 SEC=SUPER
  721 J=721,722 SEC=SUPER
  722 J=722,723 SEC=SUPER
  723 J=723,724 SEC=SUPER
  724 J=724,731 SEC=SUPER
  731 J=731,732 SEC=SUPER
  732 J=732,733 SEC=SUPER
  733 J=733,734 SEC=SUPER
  734 J=734,741 SEC=SUPER
  741 J=741,742 SEC=SUPER
  742 J=742,743 SEC=SUPER
  743 J=743,744 SEC=SUPER
  744 J=744,751 SEC=SUPER
  751 J=751,752 SEC=SUPER
  752 J=752,753 SEC=SUPER
  753 J=753,754 SEC=SUPER
  754 J=754,761 SEC=SUPER

  620 J=620,721 SEC=XBEAM
  622 J=622,721 SEC=XBEAM
  630 J=630,731 SEC=XBEAM
  632 J=632,731 SEC=XBEAM
  640 J=640,741 SEC=XBEAM
  642 J=642,741 SEC=XBEAM
  650 J=650,751 SEC=XBEAM
  652 J=652,751 SEC=XBEAM

  611 J=611,711 SEC=LINK
  661 J=661,761 SEC=LINK

  220 J=220,320 SEC=SEAL
  320 J=320,420 SEC=FTG
  420 J=420,520 SEC=COL
  520 J=520,620 SEC=LINK

  222 J=222,322 SEC=SEAL
  322 J=322,422 SEC=FTG
  422 J=422,522 SEC=COL
  522 J=522,622 SEC=LINK

  230 J=230,330 SEC=SEAL
  330 J=330,430 SEC=FTG
  430 J=430,530 SEC=COL
  530 J=530,630 SEC=LINK

  232 J=232,332 SEC=SEAL
  332 J=332,432 SEC=FTG
  432 J=432,532 SEC=COL
```

```
   532 J=532,632 SEC=LINK

   240 J=240,340 SEC=SEAL
   340 J=340,440 SEC=FTG
   440 J=440,540 SEC=COL
   540 J=540,640 SEC=LINK

   242 J=242,342 SEC=SEAL
   342 J=342,442 SEC=FTG
   442 J=442,542 SEC=COL
   542 J=542,642 SEC=LINK

   250 J=250,350 SEC=SEAL
   350 J=350,450 SEC=FTG
   450 J=450,550 SEC=COL
   550 J=550,650 SEC=LINK

   252 J=252,352 SEC=SEAL
   352 J=352,452 SEC=FTG
   452 J=452,552 SEC=COL
   552 J=552,652 SEC=LINK

   120 J=221,220 SEC=LINK
   122 J=221,222 SEC=LINK

   130 J=231,230 SEC=LINK
   132 J=231,232 SEC=LINK

   140 J=241,240 SEC=LINK
   142 J=241,242 SEC=LINK

   150 J=251,250 SEC=LINK
   152 J=251,252 SEC=LINK


LOAD
  CSYS=0
  NAME=DL
    TYPE=GRAVITY ELEM=FRAME
       ADD=* UY=-1

  NAME=TL
    TYPE=TEMPERATURE ELEM=FRAME
       ADD=711,714,1,751,10   T=10


MODES

  TYPE=EIGEN  N=20 ; 5 SPANS AND 4 MODES PER SPAN


FUNCTION

NAME=100YR NPL=1
0.00     0.257
0.088    0.642
0.442    0.642
```

0.490	0.579
0.50	0.567
0.60	0.473
0.70	0.405
0.80	0.354
0.90	0.315
1.00	0.284
1.10	0.258
1.20	0.236
1.30	0.218
1.40	0.203
1.50	0.189
1.60	0.177
1.70	0.167
1.80	0.158
1.90	0.149
2.00	0.142
2.10	0.135
2.20	0.129
2.30	0.123
2.40	0.118
2.50	0.113
2.60	0.109
2.70	0.105
2.80	0.101
2.90	0.098
3.00	0.095
3.10	0.091
3.20	0.089
3.30	0.086
3.40	0.083
3.50	0.081
3.60	0.079
3.70	0.077
3.80	0.075
3.90	0.073
4.00	0.071
4.10	0.069
4.20	0.068
4.30	0.066
4.40	0.064
4.50	0.063
4.60	0.062
4.70	0.060
4.80	0.059
4.90	0.058
5.00	0.057
5.10	0.056
5.20	0.055
5.30	0.053
5.40	0.053
5.50	0.052
5.60	0.051
5.70	0.050
5.80	0.049
5.90	0.048
6.00	0.047

```
6.10     0.046
6.20     0.046
6.30     0.045
6.40     0.044
6.50     0.044
6.60     0.043
6.70     0.042
6.80     0.042
6.90     0.041
7.00     0.041
7.10     0.040
7.20     0.039
7.30     0.039
7.40     0.038
7.50     0.038
7.60     0.037
7.70     0.037
7.80     0.036
7.90     0.036
8.00     0.035
8.10     0.035
8.20     0.035
8.30     0.034
8.40     0.034
10.00    0.028


SPEC

  CSYS=0
  NAME=EQLONG   MODC=CQC   DAMP=0.05
    ACC=U1 FUNC=100YR  SF=32.2
  NAME=EQTRAN   MODC=CQC   DAMP=0.05
    ACC=U3 FUNC=100YR  SF=32.2


COMBO

  NAME=EQ       TYPE=SRSS
    SPEC=EQLONG  SF=1
    SPEC=EQTRAN  SF=1


OUTPUT

  ELEM=JOINT  TYPE=DISP,REAC  LOAD=*  SPEC=*  COMB=*
  ELEM=FRAME  TYPE=FORCE      LOAD=*  SPEC=*  COMB=*
```

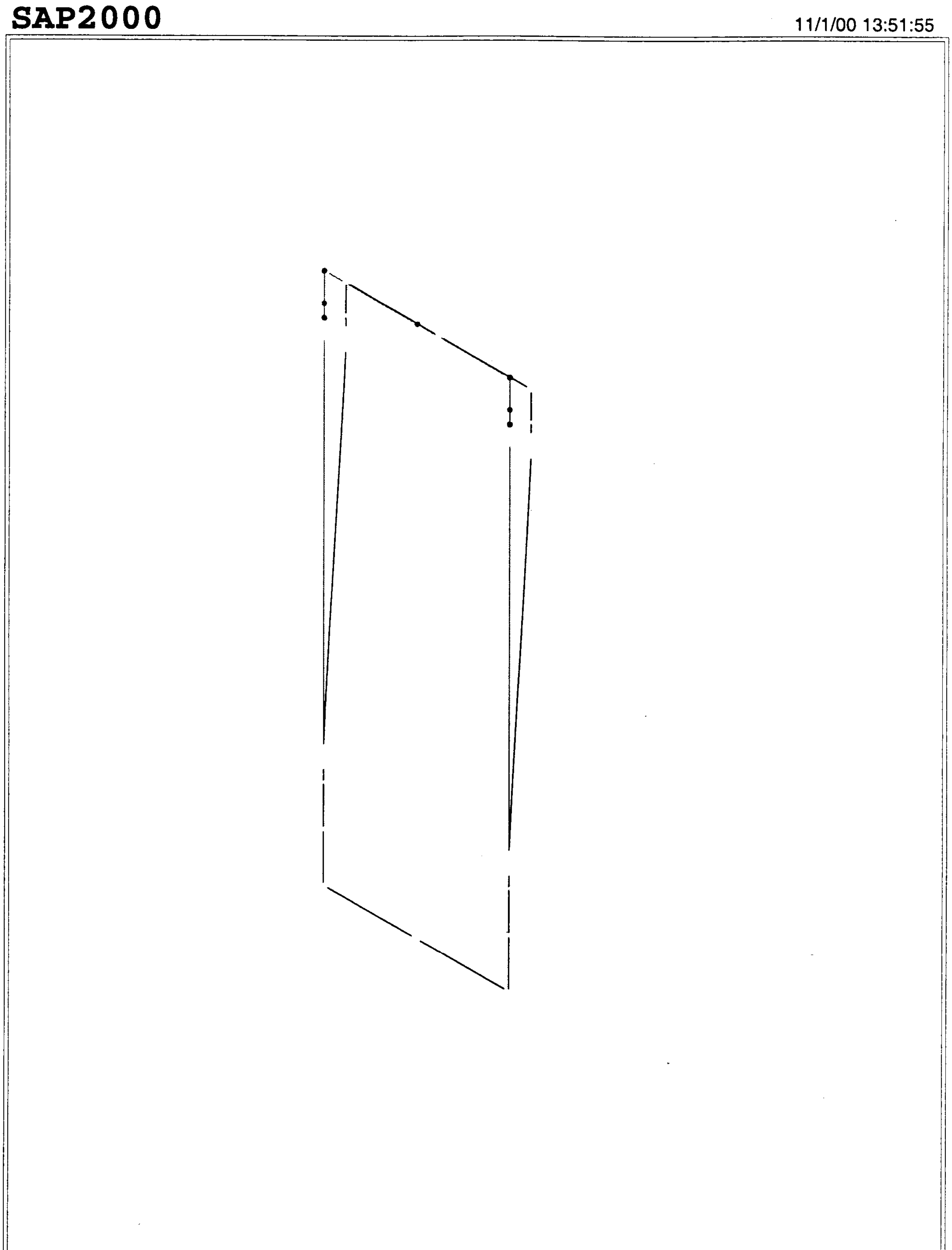
Appendix C
Pushover Data

NCHRP 12-49 WASHINGTON SITE/ BENT 3 PIER 4 PUSHOVER /NON-LIQUEFIED FOUNDATION

```
; File O:\1999\A99067\ENGR\WA\SAP2000\Pushover\P4push.$2k saved 11/1/00 13:56:10 in Kip-
ft

SYSTEM
  DOF=UX,UY,UZ,RX,RY,RZ  LENGTH=FT  FORCE=Kip  PAGE=SECTIONS

JOINT
  240   X=300   Y=-36     Z=-11.26
  241   X=300   Y=-36     Z=0
  242   X=300   Y=-36     Z=11.26
  340   X=300   Y=-30     Z=-11.26
  342   X=300   Y=-30     Z=11.26
  440   X=300   Y=-25     Z=-11.26
  442   X=300   Y=-25     Z=11.26
  540   X=300   Y=25      Z=-11.26
  542   X=300   Y=25      Z=11.26
  640   X=300   Y=28.38   Z=-11.26
  642   X=300   Y=28.38   Z=11.26
  741   X=300   Y=28.38   Z=0
  4401  X=300   Y=-23.46  Z=-11.26
  4409  X=300   Y=23.46   Z=-11.26
  4421  X=300   Y=-23.46  Z=11.26
  4429  X=300   Y=23.46   Z=11.26

PATTERN
  NAME=DEFAULT

SPRING
  ADD=241  U1=459000  U2=286000  U3=447000  R1=8.8E+08  R2=1.68E+09  R3=2.2E+08

MATERIAL
  NAME=SUB  IDES=C  M=4.658385E-03  W=.15
            T=0  E=552000  U=.18  A=.000006
  NAME=RIGID  IDES=C
            T=0  E=552000  U=.18  A=.000006
  NAME=3KSI  IDES=C  M=4.347826E-03  W=.14
              T=0  E=431000  U=.18  A=.000006
  NAME=STEEL  IDES=S  M=1.518708E-02  W=.489024
              T=0  E=4176000  U=.3  A=.0000065  FY=5184
  NAME=CONC  IDES=C  M=4.658087E-03  W=.1499904
              T=0  E=518400  U=.2  A=.0000055

FRAME SECTION
  NAME=XBEAM MAT=SUB   A=27    J=10000  I=10000,10000 AS=0,0 T=1,1
  NAME=LINK  MAT=RIGID A=10    J=10000  I=10000,10000 AS=0,0 T=1,1
  NAME=COL   MAT=SUB   A=12.57 J=10     I=5,5         AS=0,0 T=1,1
  NAME=FTG   MAT=SUB   A=506   J=109634 I=89225,20409 AS=0,0 T=1,1
  NAME=SEAL  MAT=3KSI  A=196   J=6403   I=89225,20409 AS=0,0 T=1,1

FRAME
  140  J=241,240    SEC=LINK  NSEG=2  ANG=0
  142  J=241,242    SEC=LINK  NSEG=2  ANG=0
  240  J=240,340    SEC=SEAL  NSEG=2  ANG=0
  242  J=242,342    SEC=SEAL  NSEG=2  ANG=0
  340  J=340,440    SEC=FTG   NSEG=2  ANG=0
  342  J=342,442    SEC=FTG   NSEG=2  ANG=0
  440  J=440,4401   SEC=COL   NSEG=2  ANG=0
  442  J=442,4421   SEC=COL   NSEG=2  ANG=0
  540  J=540,640    SEC=LINK  NSEG=2  ANG=0
  542  J=542,642    SEC=LINK  NSEG=2  ANG=0
  640  J=640,741    SEC=XBEAM NSEG=2  ANG=0
```

```
  642  J=642,741      SEC=XBEAM NSEG=2  ANG=0
  4401  J=4401,4409  SEC=COL    NSEG=2  ANG=0
  4409  J=4409,540   SEC=COL    NSEG=2  ANG=0
  4421  J=4421,4429  SEC=COL    NSEG=2  ANG=0
  4429  J=4429,542   SEC=COL    NSEG=2  ANG=0

LOAD
  NAME=DL  CSYS=0
    TYPE=FORCE
      ADD=741  UY=-1345.1
    TYPE=GRAVITY  ELEM=FRAME
      ADD=140  UY=-1
      ADD=142  UY=-1
      ADD=240  UY=-1
      ADD=242  UY=-1
      ADD=340  UY=-1
      ADD=342  UY=-1
      ADD=440  UY=-1
      ADD=442  UY=-1
      ADD=540  UY=-1
      ADD=542  UY=-1
      ADD=640  UY=-1
      ADD=642  UY=-1
      ADD=4401  UY=-1
      ADD=4409  UY=-1
      ADD=4421  UY=-1
      ADD=4429  UY=-1
  NAME=TRANS  CSYS=0
    TYPE=FORCE
      ADD=741  UZ=1

OUTPUT
; No Output Requested

END

; The following data is used for graphics, design and pushover analysis.
; If changes are made to the analysis data above, then the following data
; should be checked for consistency.

SAP2000 V7.40 SUPPLEMENTAL DATA
  GRID GLOBAL  X "1"  300
  GRID GLOBAL  Y "2"  -36
  GRID GLOBAL  Z "3"  -11.26
  GRID GLOBAL  X "4"  0
  GRID GLOBAL  Y "5"  0
  GRID GLOBAL  Z "6"  0
  GRID GLOBAL  Y "7"  28.38
  GRID GLOBAL  Z "8"  11.26

  MATERIAL STEEL FY 5184
  MATERIAL SUB   FYREBAR 8640 FYSHEAR 5760 FC 576 FCSHEAR 576
  MATERIAL RIGID FYREBAR 8640 FYSHEAR 5760 FC 576 FCSHEAR 576
  MATERIAL 3KSI  FYREBAR 8640 FYSHEAR 5760 FC 576 FCSHEAR 576
  MATERIAL CONC  FYREBAR 8640 FYSHEAR 5760 FC 576 FCSHEAR 576

  STATICLOAD DL TYPE DEAD
  STATICLOAD TRANS TYPE QUAKE

  PUSHCASE "PUSHDL"  CONTROL FORCE  MONITOREDJOINT 242  DOF U1
  PUSHCASE "PUSHDL"  PDELTA NO
  PUSHCASE "PUSHDL"  MINSTEPS 1  MAXNULLSTEPS 50  MAXTOTALSTEPS 200  MAXITER 10
  PUSHCASE "PUSHDL"  ITERTOL .0001  EVENTTOL .01
```

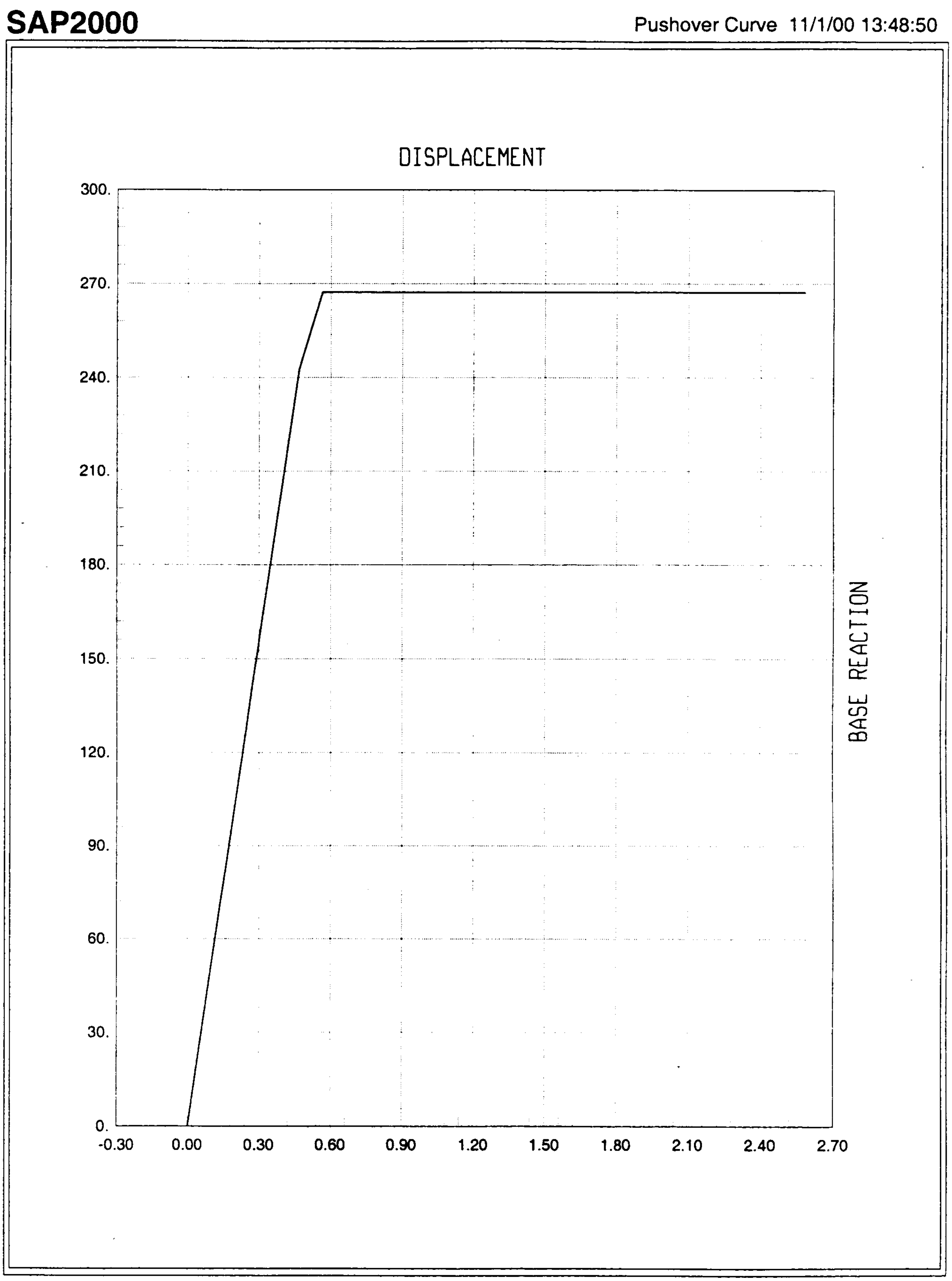
```
 PUSHCASE "PUSHDL"  LOADTYPE STATIC  LOAD DL  SCALEFACTOR 1

 PUSHCASE "PUSH2"  CONTROL CONJUGATEDISP  TARGETDISP 2.58  DOF U3  JOINT 741
 MONITOREDJOINT 741  DOF U3
 PUSHCASE "PUSH2"  STARTCASE "PUSHDL"  PDELTA NO
 PUSHCASE "PUSH2"  MINSTEPS 10  MAXNULLSTEPS 50  MAXTOTALSTEPS 200  MAXITER 10
 PUSHCASE "PUSH2"  ITERTOL .0001  EVENTTOL .01
 PUSHCASE "PUSH2"  LOADTYPE STATIC  LOAD TRANS  SCALEFACTOR 1

 HINGE "HINGE1"  TYPE PMM SYMMETRIC
 HINGE "HINGE1"  TYPE PMM  B 1 1  C 3 1  D 8 1  E 10 1
 HINGE "HINGE1"  TYPE PMM  ROTATIONSFP .01
 HINGE "HINGE1"  TYPE PMM  IO 2  LS 4  CP 6
 HINGE "HINGE1"  PCURVE PROPORTIONAL
 HINGE "HINGE1"  INTSURFACE USER  DSYMMETRY YES  NCURVES 5  NPOINTS 18
 HINGE "HINGE1"  INTPSCALE 1  INTMSCALE 1
 HINGE "HINGE1"  INTPOINT 1  -7567 0 0 0 0 0
 HINGE "HINGE1"  INTPOINT 2  -6891 1002 1002 1002 1002 1002
 HINGE "HINGE1"  INTPOINT 3  -5843 2348 2348 2348 2348 2348
 HINGE "HINGE1"  INTPOINT 4  -4764 3258 3258 3258 3258 3258
 HINGE "HINGE1"  INTPOINT 5  -3847 3729 3729 3729 3729 3729
 HINGE "HINGE1"  INTPOINT 6  -3353 3894 3894 3894 3894 3894
 HINGE "HINGE1"  INTPOINT 7  -2798 4016 4016 4016 4016 4016
 HINGE "HINGE1"  INTPOINT 8  -2425 3999 3999 3999 3999 3999
 HINGE "HINGE1"  INTPOINT 9  -2052 3929 3929 3929 3929 3929
 HINGE "HINGE1"  INTPOINT 10 -1800 3829 3829 3829 3829 3829
 HINGE "HINGE1"  INTPOINT 11 -1296 3564 3564 3564 3564 3564
 HINGE "HINGE1"  INTPOINT 12  -923 3289 3289 3289 3289 3289
 HINGE "HINGE1"  INTPOINT 13  -490 2914 2914 2914 2914 2914
 HINGE "HINGE1"  INTPOINT 14  -218 2614 2614 2614 2614 2614
 HINGE "HINGE1"  INTPOINT 15     4 2361 2361 2361 2361 2361
 HINGE "HINGE1"  INTPOINT 16   327 1952 1952 1952 1952 1952
 HINGE "HINGE1"  INTPOINT 17   942 1002 1002 1002 1002 1002
 HINGE "HINGE1"  INTPOINT 18  1536 0 0 0 0 0

 FRAMEHINGE 440   HINGE "HINGE1"  RDISTANCE 1
 FRAMEHINGE 442   HINGE "HINGE1"  RDISTANCE 1
 FRAMEHINGE 4409  HINGE "HINGE1"  RDISTANCE 0
 FRAMEHINGE 4429  HINGE "HINGE1"  RDISTANCE 0

END SUPPLEMENTAL DATA
```


SAP2000
11/1/00 13:51:55
SAP2000 v7.40 - File:P4push - Deformed Shape (PUSH2 - Step 11) - Kip-ft Units

P U S H O V E R   C U R V E

Pushover Case PUSH2

Step	Displacement	Base Force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	TOTAL
0	0.0000	0.0000	4	0	0	0	0	0	0	0	4
1	0.2580	134.1161	4	0	0	0	0	0	0	0	4
2	0.4669	242.7244	3	1	0	0	0	0	0	0	4
3	0.5649	267.1785	0	4	0	0	0	0	0	0	4
4	0.8229	267.1818	0	4	0	0	0	0	0	0	4
5	1.0809	267.1852	0	4	0	0	0	0	0	0	4
6	1.3389	267.1885	0	4	0	0	0	0	0	0	4
7	1.7845	267.1943	0	0	0	0	0	4	0	0	4
8	2.0425	267.1977	0	0	0	0	0	4	0	0	4
9	2.3005	267.2010	0	0	0	0	0	4	0	0	4
10	2.5585	267.2043	0	0	0	0	0	4	0	0	4
11	2.5800	267.2046	0	0	0	0	0	4	0	0	4



SAP2000 v7.40 - File:P4push - Kip-ft Units
Pushover Case PUSH2

FRAME ELEMENT FORCES

FRAME	LOAD	LOC	P	V2	V3	T	M2	M3
440	PUSHDL-000	0	0	0	0	0	0	0
440	PUSHDL-000	0.77	0	0	0	0	0	0
440	PUSHDL-000	1.54	0	0	0	0	0	0
440	PUSHDL-001	0	-812.428	8.77E-02	-1.06E-17	2.27E-20	-4.24E-16	3.498126
440	PUSHDL-001	0.77	-810.976	8.77E-02	-1.06E-17	2.27E-20	-4.15E-16	3.430584
440	PUSHDL-001	1.54	-809.524	8.77E-02	-1.06E-17	2.27E-20	-4.07E-16	3.363043
440	PUSH2-000	0	-812.428	8.77E-02	-1.06E-17	2.27E-20	-4.24E-16	3.498126
440	PUSH2-000	0.77	-810.976	8.77E-02	-1.06E-17	2.27E-20	-4.15E-16	3.430584
440	PUSH2-000	1.54	-809.524	8.77E-02	-1.06E-17	2.27E-20	-4.07E-16	3.363043
440	PUSH2-001	0	-643.987	67.146	-8.13E-15	2.27E-20	-2.04E-13	1686.408
440	PUSH2-001	0.77	-642.535	67.146	-8.13E-15	2.27E-20	-1.98E-13	1634.706
440	PUSH2-001	1.54	-641.083	67.146	-8.13E-15	2.27E-20	-1.92E-13	1583.004
440	PUSH2-002	0	-507.582	121.45	-1.47E-14	2.27E-20	-3.69E-13	3049.243
440	PUSH2-002	0.77	-506.13	121.45	-1.47E-14	2.27E-20	-3.58E-13	2955.727
440	PUSH2-002	1.54	-504.678	121.45	-1.47E-14	2.27E-20	-3.47E-13	2862.21
440	PUSH2-003	0	-478.822	121.57	-1.65E-14	-2.00E-14	-4.05E-13	3087.471
440	PUSH2-003	0.77	-477.37	121.57	-1.65E-14	-2.00E-14	-3.92E-13	2993.859
440	PUSH2-003	1.54	-475.919	121.57	-1.65E-14	-2.00E-14	-3.79E-13	2900.247
440	PUSH2-004	0	-478.818	121.58	-1.70E-14	-2.52E-14	-4.09E-13	3087.512
440	PUSH2-004	0.77	-477.366	121.58	-1.70E-14	-2.52E-14	-3.96E-13	2993.899
440	PUSH2-004	1.54	-475.914	121.58	-1.70E-14	-2.52E-14	-3.83E-13	2900.286
440	PUSH2-005	0	-478.813	121.58	-1.74E-14	-3.04E-14	-4.14E-13	3087.554
440	PUSH2-005	0.77	-477.362	121.58	-1.74E-14	-3.04E-14	-4.01E-13	2993.939
440	PUSH2-005	1.54	-475.91	121.58	-1.74E-14	-3.04E-14	-3.87E-13	2900.325
440	PUSH2-006	0	-478.809	121.58	-1.79E-14	-3.56E-14	-4.19E-13	3087.596
440	PUSH2-006	0.77	-477.357	121.58	-1.79E-14	-3.56E-14	-4.05E-13	2993.98
440	PUSH2-006	1.54	-475.905	121.58	-1.79E-14	-3.56E-14	-3.91E-13	2900.364
440	PUSH2-007	0	-478.801	121.58	-1.87E-14	-4.46E-14	-4.27E-13	3087.668
440	PUSH2-007	0.77	-477.35	121.58	-1.87E-14	-4.46E-14	-4.13E-13	2994.049
440	PUSH2-007	1.54	-475.898	121.58	-1.87E-14	-4.46E-14	-3.98E-13	2900.431
440	PUSH2-008	0	-478.797	121.58	-1.92E-14	-4.98E-14	-4.32E-13	3087.71
440	PUSH2-008	0.77	-477.345	121.58	-1.92E-14	-4.98E-14	-4.17E-13	2994.09
440	PUSH2-008	1.54	-475.893	121.58	-1.92E-14	-4.98E-14	-4.02E-13	2900.47
440	PUSH2-009	0	-478.793	121.59	-1.96E-14	-5.50E-14	-4.37E-13	3087.751
440	PUSH2-009	0.77	-477.341	121.59	-1.96E-14	-5.50E-14	-4.22E-13	2994.13
440	PUSH2-009	1.54	-475.889	121.59	-1.96E-14	-5.50E-14	-4.06E-13	2900.509
440	PUSH2-010	0	-478.788	121.59	-2.01E-14	-6.03E-14	-4.41E-13	3087.793
440	PUSH2-010	0.77	-477.336	121.59	-2.01E-14	-6.03E-14	-4.26E-13	2994.17
440	PUSH2-010	1.54	-475.885	121.59	-2.01E-14	-6.03E-14	-4.10E-13	2900.548
440	PUSH2-011	0	-478.788	121.59	-2.01E-14	-6.07E-14	-4.42E-13	3087.797
440	PUSH2-011	0.77	-477.336	121.59	-2.01E-14	-6.07E-14	-4.26E-13	2994.174
440	PUSH2-011	1.54	-475.884	121.59	-2.01E-14	-6.07E-14	-4.11E-13	2900.551
442	PUSHDL-000	0	0	0	0	0	0	0
442	PUSHDL-000	0.77	0	0	0	0	0	0
442	PUSHDL-000	1.54	0	0	0	0	0	0
442	PUSHDL-001	0	-812.428	-8.77E-02	1.06E-17	2.27E-20	4.24E-16	-3.498126
442	PUSHDL-001	0.77	-810.976	-8.77E-02	1.06E-17	2.27E-20	4.15E-16	-3.430584
442	PUSHDL-001	1.54	-809.524	-8.77E-02	1.06E-17	2.27E-20	4.07E-16	-3.363043
442	PUSH2-000	0	-812.428	-8.77E-02	1.06E-17	2.27E-20	4.24E-16	-3.498126
442	PUSH2-000	0.77	-810.976	-8.77E-02	1.06E-17	2.27E-20	4.15E-16	-3.430584
442	PUSH2-000	1.54	-809.524	-8.77E-02	1.06E-17	2.27E-20	4.07E-16	-3.363043
442	PUSH2-001	0	-980.869	66.97	-8.11E-15	2.27E-20	-2.03E-13	1679.412
442	PUSH2-001	0.77	-979.417	66.97	-8.11E-15	2.27E-20	-1.97E-13	1627.845
442	PUSH2-001	1.54	-977.966	66.97	-8.11E-15	2.27E-20	-1.91E-13	1576.278
442	PUSH2-002	0	-1117.27	121.27	-1.47E-14	2.27E-20	-3.68E-13	3042.247
442	PUSH2-002	0.77	-1115.82	121.27	-1.47E-14	2.27E-20	-3.57E-13	2948.866
442	PUSH2-002	1.54	-1114.37	121.27	-1.47E-14	2.27E-20	-3.46E-13	2855.484

FRAME ELEMENT FORCES

FRAME	LOAD	LOC	P	V2	V3	T	M2	M3
442	PUSH2-003	0	-1146.03	145.6	-1.59E-14	-2.00E-14	-4.13E-13	3661.713
442	PUSH2-003	0.77	-1144.58	145.6	-1.59E-14	-2.00E-14	-4.00E-13	3549.597
442	PUSH2-003	1.54	-1143.13	145.6	-1.59E-14	-2.00E-14	-3.88E-13	3437.482
442	PUSH2-004	0	-1146.04	145.61	-1.54E-14	-2.52E-14	-4.08E-13	3661.751
442	PUSH2-004	0.77	-1144.59	145.61	-1.54E-14	-2.52E-14	-3.96E-13	3549.634
442	PUSH2-004	1.54	-1143.14	145.61	-1.54E-14	-2.52E-14	-3.84E-13	3437.518
442	PUSH2-005	0	-1146.04	145.61	-1.49E-14	-3.04E-14	-4.03E-13	3661.789
442	PUSH2-005	0.77	-1144.59	145.61	-1.49E-14	-3.04E-14	-3.92E-13	3549.671
442	PUSH2-005	1.54	-1143.14	145.61	-1.49E-14	-3.04E-14	-3.80E-13	3437.553
442	PUSH2-006	0	-1146.05	145.61	-1.45E-14	-3.56E-14	-3.98E-13	3661.827
442	PUSH2-006	0.77	-1144.6	145.61	-1.45E-14	-3.56E-14	-3.87E-13	3549.708
442	PUSH2-006	1.54	-1143.14	145.61	-1.45E-14	-3.56E-14	-3.76E-13	3437.589
442	PUSH2-007	0	-1146.06	145.61	-1.37E-14	-4.46E-14	-3.90E-13	3661.893
442	PUSH2-007	0.77	-1144.6	145.61	-1.37E-14	-4.46E-14	-3.80E-13	3549.772
442	PUSH2-007	1.54	-1143.15	145.61	-1.37E-14	-4.46E-14	-3.69E-13	3437.651
442	PUSH2-008	0	-1146.06	145.61	-1.32E-14	-4.98E-14	-3.86E-13	3661.931
442	PUSH2-008	0.77	-1144.61	145.61	-1.32E-14	-4.98E-14	-3.75E-13	3549.808
442	PUSH2-008	1.54	-1143.16	145.61	-1.32E-14	-4.98E-14	-3.65E-13	3437.686
442	PUSH2-009	0	-1146.06	145.61	-1.27E-14	-5.50E-14	-3.81E-13	3661.969
442	PUSH2-009	0.77	-1144.61	145.61	-1.27E-14	-5.50E-14	-3.71E-13	3549.845
442	PUSH2-009	1.54	-1143.16	145.61	-1.27E-14	-5.50E-14	-3.61E-13	3437.722
442	PUSH2-010	0	-1146.07	145.62	-1.23E-14	-6.03E-14	-3.76E-13	3662.007
442	PUSH2-010	0.77	-1144.62	145.62	-1.23E-14	-6.03E-14	-3.67E-13	3549.882
442	PUSH2-010	1.54	-1143.16	145.62	-1.23E-14	-6.03E-14	-3.57E-13	3437.758
442	PUSH2-011	0	-1146.07	145.62	-1.22E-14	-6.07E-14	-3.76E-13	3662.01
442	PUSH2-011	0.77	-1144.62	145.62	-1.22E-14	-6.07E-14	-3.66E-13	3549.885
442	PUSH2-011	1.54	-1143.17	145.62	-1.22E-14	-6.07E-14	-3.57E-13	3437.76
4409	PUSHDL-000	0	0	0	0	0	0	0
4409	PUSHDL-000	0.77	0	0	0	0	0	0
4409	PUSHDL-000	1.54	0	0	0	0	0	0
4409	PUSHDL-001	0	-721.057	8.77E-02	-1.06E-17	2.27E-20	9.11E-17	-0.752609
4409	PUSHDL-001	0.77	-719.605	8.77E-02	-1.06E-17	2.27E-20	9.93E-17	-0.82015
4409	PUSHDL-001	1.54	-718.153	8.77E-02	-1.06E-17	2.27E-20	1.07E-16	-0.887692
4409	PUSH2-000	0	-721.057	8.77E-02	-1.06E-17	2.27E-20	9.11E-17	-0.752609
4409	PUSH2-000	0.77	-719.605	8.77E-02	-1.06E-17	2.27E-20	9.93E-17	-0.82015
4409	PUSH2-000	1.54	-718.153	8.77E-02	-1.06E-17	2.27E-20	1.07E-16	-0.887692
4409	PUSH2-001	0	-552.616	67.146	-8.13E-15	2.27E-20	1.90E-13	-1567.474
4409	PUSH2-001	0.77	-551.164	67.146	-8.13E-15	2.27E-20	1.96E-13	-1619.177
4409	PUSH2-001	1.54	-549.712	67.146	-8.13E-15	2.27E-20	2.02E-13	-1670.879
4409	PUSH2-002	0	-416.211	121.45	-1.47E-14	2.27E-20	3.43E-13	-2836.219
4409	PUSH2-002	0.77	-414.759	121.45	-1.47E-14	2.27E-20	3.55E-13	-2929.735
4409	PUSH2-002	1.54	-413.307	121.45	-1.47E-14	2.27E-20	3.66E-13	-3023.252
4409	PUSH2-003	0	-387.451	121.57	-1.65E-14	-2.00E-14	3.95E-13	-2803.997
4409	PUSH2-003	0.77	-385.999	121.57	-1.65E-14	-2.00E-14	4.08E-13	-2897.609
4409	PUSH2-003	1.54	-384.547	121.57	-1.65E-14	-2.00E-14	4.20E-13	-2991.221
4409	PUSH2-004	0	-387.446	121.58	-1.70E-14	-2.52E-14	4.13E-13	-2804.045
4409	PUSH2-004	0.77	-385.995	121.58	-1.70E-14	-2.52E-14	4.26E-13	-2897.658
4409	PUSH2-004	1.54	-384.543	121.58	-1.70E-14	-2.52E-14	4.39E-13	-2991.271
4409	PUSH2-005	0	-387.442	121.58	-1.74E-14	-3.04E-14	4.30E-13	-2804.092
4409	PUSH2-005	0.77	-385.99	121.58	-1.74E-14	-3.04E-14	4.44E-13	-2897.707
4409	PUSH2-005	1.54	-384.538	121.58	-1.74E-14	-3.04E-14	4.57E-13	-2991.322
4409	PUSH2-006	0	-387.438	121.58	-1.79E-14	-3.56E-14	4.48E-13	-2804.14
4409	PUSH2-006	0.77	-385.986	121.58	-1.79E-14	-3.56E-14	4.62E-13	-2897.756
4409	PUSH2-006	1.54	-384.534	121.58	-1.79E-14	-3.56E-14	4.75E-13	-2991.372
4409	PUSH2-007	0	-387.43	121.58	-1.87E-14	-4.46E-14	4.78E-13	-2804.222
4409	PUSH2-007	0.77	-385.978	121.58	-1.87E-14	-4.46E-14	4.93E-13	-2897.841
4409	PUSH2-007	1.54	-384.526	121.58	-1.87E-14	-4.46E-14	5.07E-13	-2991.459

FRAME ELEMENT FORCES

FRAME	LOAD	LOC	P	V2	V3	T	M2	M3
4409	PUSH2-008	0	-387.426	121.58	-1.92E-14	-4.98E-14	4.96E-13	-2804.27
4409	PUSH2-008	0.77	-385.974	121.58	-1.92E-14	-4.98E-14	5.11E-13	-2897.89
4409	PUSH2-008	1.54	-384.522	121.58	-1.92E-14	-4.98E-14	5.26E-13	-2991.51
4409	PUSH2-009	0	-387.421	121.59	-1.96E-14	-5.50E-14	5.14E-13	-2804.317
4409	PUSH2-009	0.77	-385.969	121.59	-1.96E-14	-5.50E-14	5.29E-13	-2897.938
4409	PUSH2-009	1.54	-384.518	121.59	-1.96E-14	-5.50E-14	5.44E-13	-2991.56
4409	PUSH2-010	0	-387.417	121.59	-2.01E-14	-6.03E-14	5.32E-13	-2804.365
4409	PUSH2-010	0.77	-385.965	121.59	-2.01E-14	-6.03E-14	5.47E-13	-2897.988
4409	PUSH2-010	1.54	-384.513	121.59	-2.01E-14	-6.03E-14	5.62E-13	-2991.61
4409	PUSH2-011	0	-387.417	121.59	-2.01E-14	-6.07E-14	5.33E-13	-2804.369
4409	PUSH2-011	0.77	-385.965	121.59	-2.01E-14	-6.07E-14	5.48E-13	-2897.992
4409	PUSH2-011	1.54	-384.513	121.59	-2.01E-14	-6.07E-14	5.64E-13	-2991.615
4429	PUSHDL-000	0	0	0	0	0	0	0
4429	PUSHDL-000	0.77	0	0	0	0	0	0
4429	PUSHDL-000	1.54	0	0	0	0	0	0
4429	PUSHDL-001	0	-721.057	-8.77E-02	1.06E-17	2.27E-20	-9.11E-17	0.752609
4429	PUSHDL-001	0.77	-719.605	-8.77E-02	1.06E-17	2.27E-20	-9.93E-17	0.82015
4429	PUSHDL-001	1.54	-718.153	-8.77E-02	1.06E-17	2.27E-20	-1.07E-16	0.887692
4429	PUSH2-000	0	-721.057	-8.77E-02	1.06E-17	2.27E-20	-9.11E-17	0.752609
4429	PUSH2-000	0.77	-719.605	-8.77E-02	1.06E-17	2.27E-20	-9.93E-17	0.82015
4429	PUSH2-000	1.54	-718.153	-8.77E-02	1.06E-17	2.27E-20	-1.07E-16	0.887692
4429	PUSH2-001	0	-889.498	66.97	-8.11E-15	2.27E-20	1.90E-13	-1565.969
4429	PUSH2-001	0.77	-888.046	66.97	-8.11E-15	2.27E-20	1.96E-13	-1617.536
4429	PUSH2-001	1.54	-886.594	66.97	-8.11E-15	2.27E-20	2.02E-13	-1669.104
4429	PUSH2-002	0	-1025.9	121.27	-1.47E-14	2.27E-20	3.43E-13	-2834.714
4429	PUSH2-002	0.77	-1024.45	121.27	-1.47E-14	2.27E-20	3.55E-13	-2928.095
4429	PUSH2-002	1.54	-1023	121.27	-1.47E-14	2.27E-20	3.66E-13	-3021.476
4429	PUSH2-003	0	-1054.66	145.6	-1.59E-14	-2.00E-14	3.56E-13	-3394.289
4429	PUSH2-003	0.77	-1053.21	145.6	-1.59E-14	-2.00E-14	3.68E-13	-3506.404
4429	PUSH2-003	1.54	-1051.76	145.6	-1.59E-14	-2.00E-14	3.80E-13	-3618.52
4429	PUSH2-004	0	-1054.67	145.61	-1.54E-14	-2.52E-14	3.38E-13	-3394.323
4429	PUSH2-004	0.77	-1053.22	145.61	-1.54E-14	-2.52E-14	3.50E-13	-3506.44
4429	PUSH2-004	1.54	-1051.76	145.61	-1.54E-14	-2.52E-14	3.62E-13	-3618.557
4429	PUSH2-005	0	-1054.67	145.61	-1.49E-14	-3.04E-14	3.20E-13	-3394.358
4429	PUSH2-005	0.77	-1053.22	145.61	-1.49E-14	-3.04E-14	3.32E-13	-3506.476
4429	PUSH2-005	1.54	-1051.77	145.61	-1.49E-14	-3.04E-14	3.43E-13	-3618.594
4429	PUSH2-006	0	-1054.68	145.61	-1.45E-14	-3.56E-14	3.03E-13	-3394.394
4429	PUSH2-006	0.77	-1053.22	145.61	-1.45E-14	-3.56E-14	3.14E-13	-3506.512
4429	PUSH2-006	1.54	-1051.77	145.61	-1.45E-14	-3.56E-14	3.25E-13	-3618.632
4429	PUSH2-007	0	-1054.68	145.61	-1.37E-14	-4.46E-14	2.72E-13	-3394.454
4429	PUSH2-007	0.77	-1053.23	145.61	-1.37E-14	-4.46E-14	2.83E-13	-3506.575
4429	PUSH2-007	1.54	-1051.78	145.61	-1.37E-14	-4.46E-14	2.93E-13	-3618.696
4429	PUSH2-008	0	-1054.69	145.61	-1.32E-14	-4.98E-14	2.55E-13	-3394.489
4429	PUSH2-008	0.77	-1053.24	145.61	-1.32E-14	-4.98E-14	2.65E-13	-3506.611
4429	PUSH2-008	1.54	-1051.78	145.61	-1.32E-14	-4.98E-14	2.75E-13	-3618.733
4429	PUSH2-009	0	-1054.69	145.61	-1.27E-14	-5.50E-14	2.37E-13	-3394.524
4429	PUSH2-009	0.77	-1053.24	145.61	-1.27E-14	-5.50E-14	2.47E-13	-3506.647
4429	PUSH2-009	1.54	-1051.79	145.61	-1.27E-14	-5.50E-14	2.57E-13	-3618.771
4429	PUSH2-010	0	-1054.7	145.62	-1.23E-14	-6.03E-14	2.19E-13	-3394.559
4429	PUSH2-010	0.77	-1053.25	145.62	-1.23E-14	-6.03E-14	2.29E-13	-3506.683
4429	PUSH2-010	1.54	-1051.79	145.62	-1.23E-14	-6.03E-14	2.38E-13	-3618.808
4429	PUSH2-011	0	-1054.7	145.62	-1.22E-14	-6.07E-14	2.18E-13	-3394.562
4429	PUSH2-011	0.77	-1053.25	145.62	-1.22E-14	-6.07E-14	2.27E-13	-3506.686
4429	PUSH2-011	1.54	-1051.79	145.62	-1.22E-14	-6.07E-14	2.37E-13	-3618.811

FRAME ELEMENT HINGE STATUS

FRAME	LOAD	HINGE ASSIGNED	HINGE GENERATED	LOC	P	V2	V3	T	M2	M3	U1P	U2P	U3P	R1P	R2P	R3P	STATUS
440	PUSHDL-000	HINGE1	HINGE1	1.54	0.000	0	0	0	0.000	0.000	0.00000	0	0	0	0.00000	0.00000	A-B
440	PUSHDL-001	HINGE1	HINGE1	1.54	-809.524	0	0	0	0.000	3.363	0.00000	0	0	0	0.00000	0.00000	A-B
440	PUSH2-000	HINGE1	HINGE1	1.54	-809.524	0	0	0	0.000	3.363	0.00000	0	0	0	0.00000	0.00000	A-B
440	PUSH2-001	HINGE1	HINGE1	1.54	-641.083	0	0	0	0.000	1583.004	0.00000	0	0	0	0.00000	0.00000	A-B
440	PUSH2-002	HINGE1	HINGE1	1.54	-504.678	0	0	0	0.000	2862.210	0.00000	0	0	0	0.00000	0.00000	A-B
440	PUSH2-003	HINGE1	HINGE1	1.54	-475.919	0	0	0	0.000	2900.247	0.00170	0	0	0	0.00173	0.00173	B-IO
440	PUSH2-004	HINGE1	HINGE1	1.54	-475.914	0	0	0	0.000	2900.286	0.00706	0	0	0	0.00722	0.00722	B-IO
440	PUSH2-005	HINGE1	HINGE1	1.54	-475.910	0	0	0	0.000	2900.325	0.01242	0	0	0	0.01271	0.01271	B-IO
440	PUSH2-006	HINGE1	HINGE1	1.54	-475.905	0	0	0	0.000	2900.364	0.01779	0	0	0	0.01819	0.01819	B-IO
440	PUSH2-007	HINGE1	HINGE1	1.54	-475.898	0	0	0	0.000	2900.431	0.02705	0	0	0	0.02767	0.02770	C-D
440	PUSH2-008	HINGE1	HINGE1	1.54	-475.893	0	0	0	0.000	2900.470	0.03241	0	0	0	0.03315	0.03319	C-D
440	PUSH2-009	HINGE1	HINGE1	1.54	-475.889	0	0	0	0.000	2900.509	0.03778	0	0	0	0.03864	0.03867	C-D
440	PUSH2-010	HINGE1	HINGE1	1.54	-475.885	0	0	0	0.000	2900.548	0.04314	0	0	0	0.04413	0.04416	C-D
440	PUSH2-011	HINGE1	HINGE1	1.54	-475.884	0	0	0	0.000	2900.551	0.04359	0	0	0	0.04458	0.04462	C-D
442	PUSHDL-000	HINGE1	HINGE1	1.54	0.000	0	0	0	0.000	0.000	0.00000	0	0	0	0.00000	0.00000	A-B
442	PUSHDL-001	HINGE1	HINGE1	1.54	-809.524	0	0	0	0.000	-3.363	0.00000	0	0	0	0.00000	0.00000	A-B
442	PUSH2-000	HINGE1	HINGE1	1.54	-809.524	0	0	0	0.000	-3.363	0.00000	0	0	0	0.00000	0.00000	A-B
442	PUSH2-001	HINGE1	HINGE1	1.54	-977.966	0	0	0	0.000	1576.278	0.00000	0	0	0	0.00000	0.00000	A-B
442	PUSH2-002	HINGE1	HINGE1	1.54	-1114.370	0	0	0	0.000	2855.484	0.00000	0	0	0	0.00000	0.00000	A-B
442	PUSH2-003	HINGE1	HINGE1	1.54	-1144.531	0	0	0	0.000	3466.155	0.00000	0	0	0	0.00000	0.00000	B-IO
442	PUSH2-004	HINGE1	HINGE1	1.54	-1144.535	0	0	0	0.000	3466.190	0.00439	0	0	0	0.00549	0.00549	B-IO
442	PUSH2-005	HINGE1	HINGE1	1.54	-1144.539	0	0	0	0.000	3466.226	0.00877	0	0	0	0.01097	0.01097	B-IO
442	PUSH2-006	HINGE1	HINGE1	1.54	-1144.544	0	0	0	0.000	3466.262	0.01316	0	0	0	0.01646	0.01646	B-IO
442	PUSH2-007	HINGE1	HINGE1	1.54	-1144.552	0	0	0	0.000	3466.323	0.02074	0	0	0	0.02593	0.02593	C-D
442	PUSH2-008	HINGE1	HINGE1	1.54	-1144.556	0	0	0	0.000	3466.359	0.02513	0	0	0	0.03142	0.03142	C-D
442	PUSH2-009	HINGE1	HINGE1	1.54	-1144.560	0	0	0	0.000	3466.395	0.02951	0	0	0	0.03691	0.03691	C-D
442	PUSH2-010	HINGE1	HINGE1	1.54	-1144.565	0	0	0	0.000	3466.430	0.03390	0	0	0	0.04239	0.04239	C-D
442	PUSH2-011	HINGE1	HINGE1	1.54	-1144.565	0	0	0	0.000	3466.433	0.03427	0	0	0	0.04285	0.04285	C-D
4409	PUSHDL-000	HINGE1	HINGE1	0	0.000	0	0	0	0.000	0.000	0.00000	0	0	0	0.00000	0.00000	A-B
4409	PUSHDL-001	HINGE1	HINGE1	0	-721.057	0	0	0	0.000	-0.753	0.00000	0	0	0	0.00000	0.00000	A-B
4409	PUSH2-000	HINGE1	HINGE1	0	-721.057	0	0	0	0.000	-0.753	0.00000	0	0	0	0.00000	0.00000	A-B
4409	PUSH2-001	HINGE1	HINGE1	0	-552.616	0	0	0	0.000	-1567.474	0.00000	0	0	0	0.00000	0.00000	A-B
4409	PUSH2-002	HINGE1	HINGE1	0	-416.211	0	0	0	0.000	-2836.219	0.00000	0	0	0	0.00000	0.00000	B-IO
4409	PUSH2-003	HINGE1	HINGE1	0	-387.451	0	0	0	0.000	-2803.997	0.00243	0	0	0	0.00217	0.00217	B-IO
4409	PUSH2-004	HINGE1	HINGE1	0	-387.446	0	0	0	0.000	-2804.045	0.00845	0	0	0	0.00753	0.00753	B-IO
4409	PUSH2-005	HINGE1	HINGE1	0	-387.442	0	0	0	0.000	-2804.092	0.01446	0	0	0	0.01290	0.01290	B-IO
4409	PUSH2-006	HINGE1	HINGE1	0	-387.438	0	0	0	0.000	-2804.140	0.02048	0	0	0	0.01827	0.01827	B-IO
4409	PUSH2-007	HINGE1	HINGE1	0	-387.430	0	0	0	0.000	-2804.222	0.03087	0	0	0	0.02753	0.02753	C-D

FRAME ELEMENT HINGE STATUS

FRAME	LOAD	HINGE ASSIGNED	HINGE GENERATED	LOC	P	V2	V3	T	M2	M3	U1P	U2P	U3P	R1P	R2P	R3P	STATUS
4409	PUSH2-008	HINGE1	HINGE1	0	-387.426	0	0	0	0.000	-2804.270	0.03688	0	0	0	0.03290	0.03290	C-D
4409	PUSH2-009	HINGE1	HINGE1	0	-387.421	0	0	0	0.000	-2804.317	0.04290	0	0	0	0.03827	0.03827	C-D
4409	PUSH2-010	HINGE1	HINGE1	0	-387.417	0	0	0	0.000	-2804.365	0.04892	0	0	0	0.04363	0.04363	C-D
4409	PUSH2-011	HINGE1	HINGE1	0	-387.417	0	0	0	0.000	-2804.369	0.04942	0	0	0	0.04408	0.04408	C-D
4429	PUSHDL-000	HINGE1	HINGE1	0	0.000	0	0	0	0.000	0.000	0.00000	0	0	0	0.00000	0.00000	A-B
4429	PUSHDL-001	HINGE1	HINGE1	0	-721.057	0	0	0	0.000	0.753	0.00000	0	0	0	0.00000	0.00000	A-B
4429	PUSH2-000	HINGE1	HINGE1	0	-721.057	0	0	0	0.000	0.753	0.00000	0	0	0	0.00000	0.00000	A-B
4429	PUSH2-001	HINGE1	HINGE1	0	-889.498	0	0	0	0.000	-1565.969	0.00000	0	0	0	0.00000	0.00000	A-B
4429	PUSH2-002	HINGE1	HINGE1	0	-1025.903	0	0	0	0.000	-2834.714	0.00000	0	0	0	0.00000	0.00000	A-B
4429	PUSH2-003	HINGE1	HINGE1	0	-1054.662	0	0	0	0.000	-3394.289	0.00000	0	0	0	0.00000	0.00000	B-IO
4429	PUSH2-004	HINGE1	HINGE1	0	-1054.667	0	0	0	0.000	-3394.323	0.00429	0	0	0	0.00537	0.00537	B-IO
4429	PUSH2-005	HINGE1	HINGE1	0	-1054.671	0	0	0	0.000	-3394.358	0.00858	0	0	0	0.01073	0.01073	B-IO
4429	PUSH2-006	HINGE1	HINGE1	0	-1054.676	0	0	0	0.000	-3394.394	0.01287	0	0	0	0.01610	0.01610	B-IO
4429	PUSH2-007	HINGE1	HINGE1	0	-1054.683	0	0	0	0.000	-3394.454	0.02028	0	0	0	0.02537	0.02537	C-D
4429	PUSH2-008	HINGE1	HINGE1	0	-1054.688	0	0	0	0.000	-3394.489	0.02458	0	0	0	0.03073	0.03073	C-D
4429	PUSH2-009	HINGE1	HINGE1	0	-1054.692	0	0	0	0.000	-3394.524	0.02887	0	0	0	0.03610	0.03610	C-D
4429	PUSH2-010	HINGE1	HINGE1	0	-1054.697	0	0	0	0.000	-3394.559	0.03316	0	0	0	0.04146	0.04146	C-D
4429	PUSH2-011	HINGE1	HINGE1	0	-1054.697	0	0	0	0.000	-3394.562	0.03352	0	0	0	0.04191	0.04191	C-D

J O I N T   D I S P L A C E M E N T S

JOINT	LOAD	U1	U2	U3	R1	R2	R3
240	DL	0.0000	-9.603E-03	0.0000	-1.557E-05	0.0000	0.0000
240	TRANS	0.0000	1.139E-06	3.257E-06	0.0000	0.0000	0.0000
241	DL	0.0000	-9.486E-03	0.0000	0.0000	0.0000	0.0000
241	TRANS	0.0000	0.0000	2.237E-06	0.0000	0.0000	0.0000
242	DL	0.0000	-9.603E-03	0.0000	1.557E-05	0.0000	0.0000
242	TRANS	0.0000	-1.139E-06	3.257E-06	0.0000	0.0000	0.0000
340	DL	0.0000	-9.694E-03	-9.324E-05	-1.557E-05	0.0000	0.0000
340	TRANS	0.0000	1.229E-06	4.011E-06	0.0000	0.0000	0.0000
342	DL	0.0000	-9.694E-03	9.324E-05	1.557E-05	0.0000	0.0000
342	TRANS	0.0000	-1.229E-06	4.011E-06	0.0000	0.0000	0.0000
440	DL	0.0000	-9.712E-03	-1.711E-04	-1.557E-05	0.0000	0.0000
440	TRANS	0.0000	1.251E-06	4.650E-06	0.0000	0.0000	0.0000
442	DL	0.0000	-9.712E-03	1.711E-04	1.557E-05	0.0000	0.0000
442	TRANS	0.0000	-1.251E-06	4.650E-06	0.0000	0.0000	0.0000
540	DL	0.0000	-0.0152	-2.736E-05	8.076E-06	0.0000	0.0000
540	TRANS	0.0000	1.030E-05	1.920E-03	1.001E-06	0.0000	0.0000
542	DL	0.0000	-0.0152	2.736E-05	-8.076E-06	0.0000	0.0000
542	TRANS	0.0000	-1.030E-05	1.920E-03	1.001E-06	0.0000	0.0000
640	DL	0.0000	-0.0157	0.0000	8.075E-06	0.0000	0.0000
640	TRANS	0.0000	1.107E-05	1.923E-03	0.0000	0.0000	0.0000
642	DL	0.0000	-0.0157	0.0000	-8.075E-06	0.0000	0.0000
642	TRANS	0.0000	-1.107E-05	1.923E-03	0.0000	0.0000	0.0000
741	DL	0.0000	-0.0157	0.0000	0.0000	0.0000	0.0000
741	TRANS	0.0000	0.0000	1.924E-03	0.0000	0.0000	0.0000
4401	DL	0.0000	-9.892E-03	-1.936E-04	-1.366E-05	0.0000	0.0000
4401	TRANS	0.0000	1.530E-06	1.013E-05	6.915E-06	0.0000	0.0000
4409	DL	0.0000	-0.0151	-4.016E-05	8.534E-06	0.0000	0.0000
4409	TRANS	0.0000	1.002E-05	1.913E-03	7.734E-06	0.0000	0.0000
4421	DL	0.0000	-9.892E-03	1.936E-04	1.366E-05	0.0000	0.0000
4421	TRANS	0.0000	-1.530E-06	1.013E-05	6.915E-06	0.0000	0.0000
4429	DL	0.0000	-0.0151	4.016E-05	-8.534E-06	0.0000	0.0000
4429	TRANS	0.0000	-1.002E-05	1.913E-03	7.734E-06	0.0000	0.0000

F R A M E   E L E M E N T   F O R C E S

FRAME	LOAD	LOC	P	V2	V3	T	M2	M3
140	DL							
		0.00	-8.772E-02	0.00	1356.57	0.00	15270.49	0.00
		5.63	-8.772E-02	0.00	1356.57	0.00	7633.01	0.00
		11.26	-8.772E-02	0.00	1356.57	0.00	-4.46	0.00
140	TRANS							
		0.00	-5.000E-01	0.00	-1.26	0.00	-32.19	0.00
		5.63	-5.000E-01	0.00	-1.26	0.00	-25.12	0.00
		11.26	-5.000E-01	0.00	-1.26	0.00	-18.05	0.00
142	DL							
		0.00	-8.772E-02	0.00	-1356.57	0.00	-15270.49	0.00
		5.63	-8.772E-02	0.00	-1356.57	0.00	-7633.01	0.00
		11.26	-8.772E-02	0.00	-1356.57	0.00	4.46	0.00
142	TRANS							

DESIGN EXAMPLE NO. 2LRFD

PURPOSE OF DESIGN EXAMPLE

This is the ninth in a series of seismic design examples originally developed for the FHWA. The original seven examples were developed to illustrate the use of the AASHTO Division I-A Specification for seismic design. The eighth and ninth examples illustrate the use of the *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges*, MCEER/ATC 49 (2003) for seismic design, which is a comprehensive revision of the AASHTO seismic design provisions. Each example emphasizes different features that must be considered in the seismic analysis and design process. The matrix below is a summary of the features of the nine examples. The ninth example, Design Example No. 2LRFD, is intended to illustrate the use of the *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges*, MCEER/ATC 49 (2003) on the bridge in the original Design Example No. 2.

DESIGN EXAMPLE NO.	DESIGN EXAMPLE DESCRIPTION	SEISMIC CATEGORY	PLAN GEOMETRY	SUPER-STRUCTURE TYPE	PIER TYPE	ABUTMENT TYPE	FOUNDATION TYPE	CONNECTIONS AND JOINTS
1	Two-Span Continuous	SPC - C	Tangent Square	CIP Concrete Box	Three-Column Integral Bent	Seat Stub Base	Spread Footings	Monolithic Joint at Pier Expansion Bearing at Abutment
2	Three-Span Continuous	SPC - B	Tangent Skewed	Steel Girder	Wall Type Pier	Tall Seat	Spread Footings	Elastomeric Bearing Pads (Piers and Abutments)
3	Single-Span	SPC - C	Tangent Square	AASHTO Precast Concrete Girders	(N/A)	Tall Seat (Closed-In)	Spread Footings	Elastomeric Bearing Pads
4	Three-Span Continuous	SPC - C	Tangent Skewed	CIP Concrete	Two-Column Integral Bent	Seat	Spread Footings	Monolithic at Col. Tops Pinned Column at Base Expansion Bearings at Abutments
5	Nine-Span Viaduct with Four-Span and Five-Span Continuous Structs.	SPC - B	Curved Square	Steel Girder	Single-Column (Variable Heights)	Seat	Steel H-Piles	Conventional Steel Pins and PTFE Sliding Bearings
6	Three-Span Continuous	SPC - C	Sharply-Curved Square	CIP Concrete Box	Single Column	Monolithic	Drilled Shaft at Piers, Steel Piles at Abutments	Monolithic Concrete Joints
7	12-Span Viaduct with (3) Four-Span Structures	SPC - B	Tangent Square	AASHTO Precast Concrete Girders	Pile Bents (Battered and Plumb)	Seat	Concrete Piles and Steel Piles	Pinned and Expansion Bearings

SECTION I INTRODUCTION

DESIGN EXAMPLE NO.	DESIGN EXAMPLE DESCRIPTION							
8	Five-Span Continuous	SDAP E	Tangent Square	CIP Concrete Box Girder	Two-Column Integral Bent	Stub Abutment with Over-hanging Diaphragm	CIP Concrete Piles with Steel Casings	Monolithic at Interior Piers Expansion Bearings at Abutments
2LRFD	Three-Span Continuous	SDAP A2 and C	Tangent Skewed	Steel Girder	Four-Column Bent and Wall Type Pier	Tall Seat	Spread Footings	Conventional and Elastomeric Bearing Pads (Piers and Abutments)

REFERENCE AASHTO SPECIFICATIONS

Example Nos. 1 through 7 conform to the following specifications.

AASHTO Division I (herein referred to as "Division I")

Standard Specifications for Highway Bridges, American Association of State Highway and Transportation Officials, Inc., 15th Edition, as amended by the Interim Specifications-Bridges-1993 through 1995.

AASHTO Division I-A (herein referred to as "Division I-A" or the "Specification")

Standard Specifications for Highway Bridges, Division I-A, Seismic Design, American Association of State Highway and Transportation Officials, Inc., 15th Edition, as amended by the Interim Specifications-Bridges-1995.

Example Nos. 8 and 2LRFD conform to the following.

***Recommended LRFD Guidelines for the Seismic Design of Highway Bridges*, MCEER/ATC 49 (2003) (herein referred to as the Guide Specification)**

Additionally, these examples cross reference the original NCHRP Specification that is the source document of the Guide Specification.

NCHRP 12-49 *Comprehensive Specification for the Seismic Design of Bridges*, Revised LRFD Design Specifications, Third Draft, March 2001.

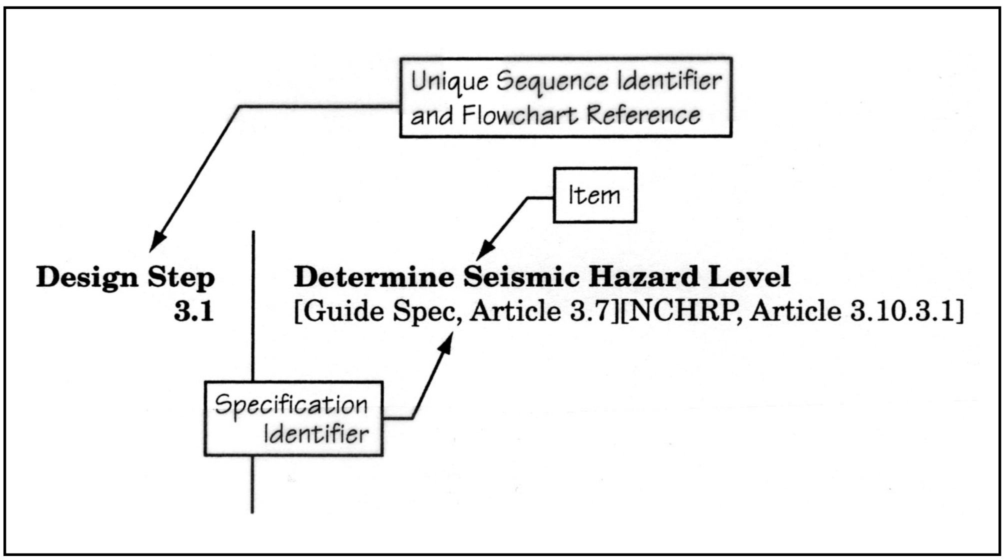
SECTION I INTRODUCTION

FLOWCHARTS AND DESIGN STEPS

This ninth example follows the outline given in detailed flowcharts presented in Section II, Flowcharts. The flowchart generally follows the one currently used in the proposed seismic Guide Specification.

The purpose of Design Steps is to present the information covered by the example in a logical and sequential manner that allows for easy referencing within the example itself. Each Design Step has a unique number in the left margin of the calculation document. The title is located to the right of the Design Step number. Where appropriate, a reference to both the Guide Specification and the NCHRP Specification follows the title.

An example is shown below.

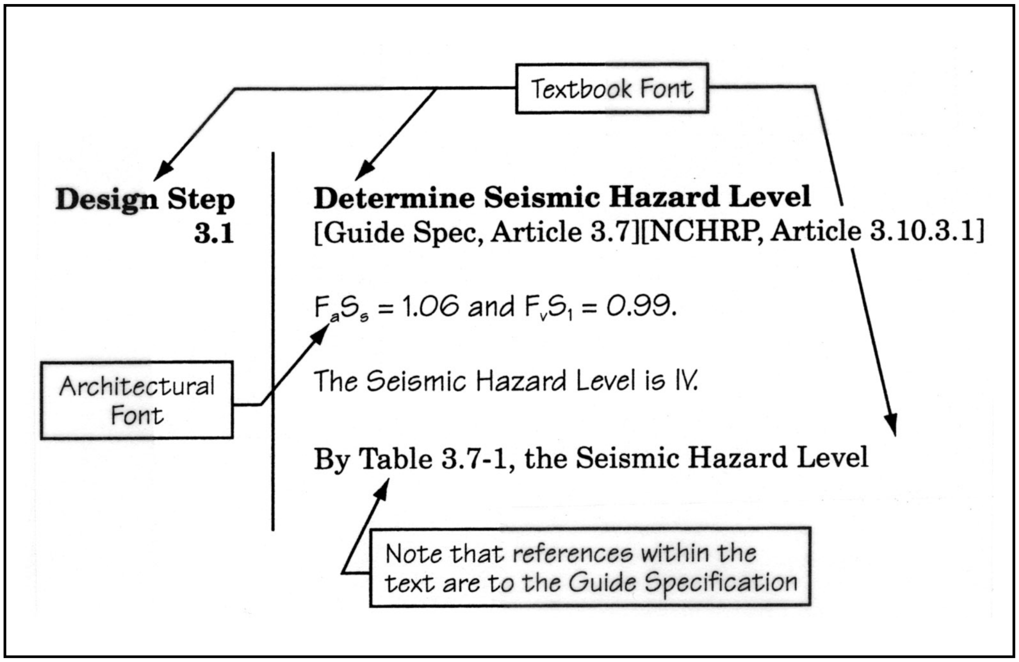


SECTION I INTRODUCTION

USE OF DIFFERENT TYPE FONTS

In the example, two primary type fonts have been used. One font, similar to the type used for textbooks, is used for all section headings and for commentary. The other, an architectural font that appears hand printed, is used for all primary calculations. The material in the architectural font is the essential calculation material and essential results.

An example of the use of the fonts is shown below.



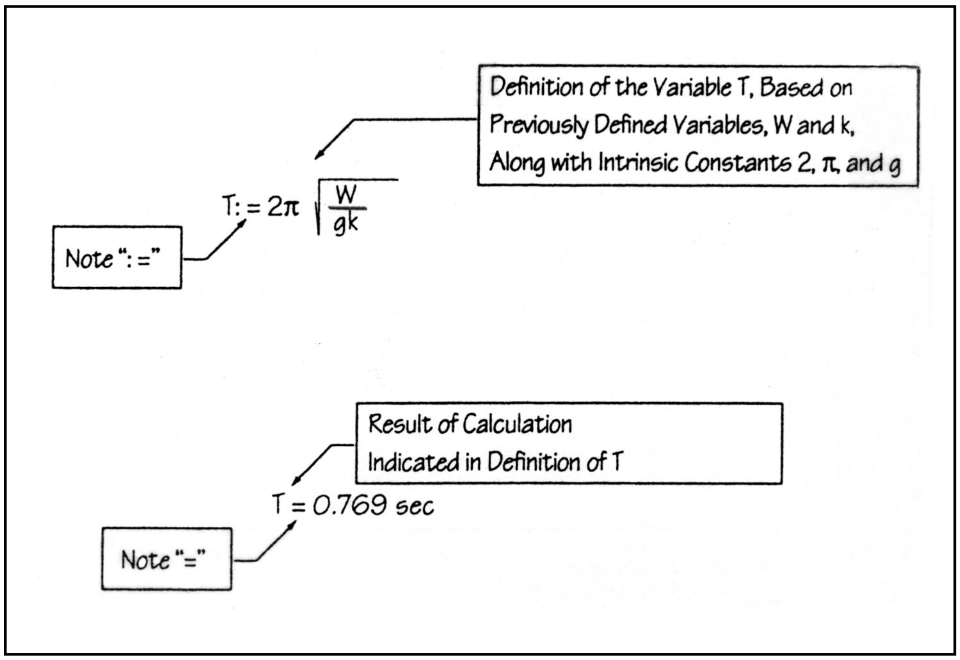
SECTION I INTRODUCTION

USE OF MATHCAD®

To provide consistent results and quality control, all calculations have been performed using the program Mathcad®.

The variables used in equations calculated by the program are defined before the equation, and the **definition** of either a variable or an equation is distinguished by a ':=' symbol. The **echo** of a variable or the result of a calculation is distinguished by a '=' symbol, i.e., no colon is used.

An example is shown below.



Note that Mathcad® carries the full precision of the variables throughout the calculations, even though the listed result of a calculation is rounded off. Thus, hand-calculated checks made using intermediate rounded results may not yield the same result as the number being checked.

Also, Mathcad® does not allow the superscript " ´ " to be used in a variable name. Therefore, the specified compressive strength of concrete is defined as $f`_{c}$ in this example (not $f´_{c}$).

SECTION I INTRODUCTION

BASIC BRIDGE DATA

The bridge is to be built in the northeast United States along the Merrimack River. Two sites along the river are investigated in the subsequent sections.

Two earthquake loadings will be considered in the design, one for a rare event, called the maximum considered earthquake (MCE), and one for a frequent or expected event. The rare event has a 3 percent chance of exceedence in 75 years, and the frequent event has a 50 percent chance of exceedence in 75 years. Seventy-five years is the nominal "design life" of a bridge as defined by the LRFD Specifications.

The configuration of the bridge is a three-span steel plate girder superstructure with a composite deck. The substructure and bearing elements vary in subsequent sections for purposes of illustrating different aspects of design. The bridge is located on a rock site and all footings are founded on rock. The rock is a hard, fresh, and sound quartz biotite schist at all locations over the site. Figure 1 (a to f) provides details of the bridge configuration and Appendix A contains the geotechnical information for the site.

The alignment of the roadway over the bridge is straight and there is no vertical curve. The bridge has a 25-degree skew at all four substructure elements.

The bridge spans a river, and the two intermediate piers are located within the normal flow of the river. Due to the presence of the piers in the river, flow issues and ice loading have required that the intermediate piers have thick cross sections.

REQUIRED

Design the bridge for seismic loading using the *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges*, MCEER/ATC 49 (2003).

FEATURES

ISSUES EMPHASIZED FOR THIS EXAMPLE

Proposed LRFD Seismic Guide Specification, including

- Basic Application of the Provisions
- SDAP C Capacity Spectrum Analysis with Conventional Bearings
- SDAP C Capacity Spectrum Analysis with Seismic Isolation Systems
- Consideration of Elastomeric Bearings
- SDAP A2 Provisions

SECTION I INTRODUCTION

ROADMAP

Design Example No. 2LRFD is separated into sections to address several different applications of SDAP A2 and C provisions.

Section III illustrates the SDAP C Capacity Spectrum Method on a bridge with conventional, mechanical bearings. The original bridge, with wall piers and elastomeric bearings requires the use of Isolation Provisions. The Isolation Provisions are a special aspect and a more straightforward initial application of the method is desirable. Because conventional bearings that do not permit transverse movement cannot be used with a wall pier (disallowed for the Capacity Spectrum approach, Guide Specification, Article 4.4.2), the wall piers were replaced with multicolumn bents for the purposes of the example.

Section IV illustrates the SDAP C Capacity Spectrum Method with Seismic Isolation, applying it directly to the original bridge with wall piers and elastomeric bearings. The difference in application of the method with and without isolation is then apparent by comparing Sections III and IV.

Section V illustrates the SDAP A2 provisions for the bridge of Section III, with conventional bearings.

Section VI illustrates the SDAP A2 provisions for the bridge of Section IV, with elastomeric bearings.

SECTION I INTRODUCTION

BRIDGE DATA
(continued)

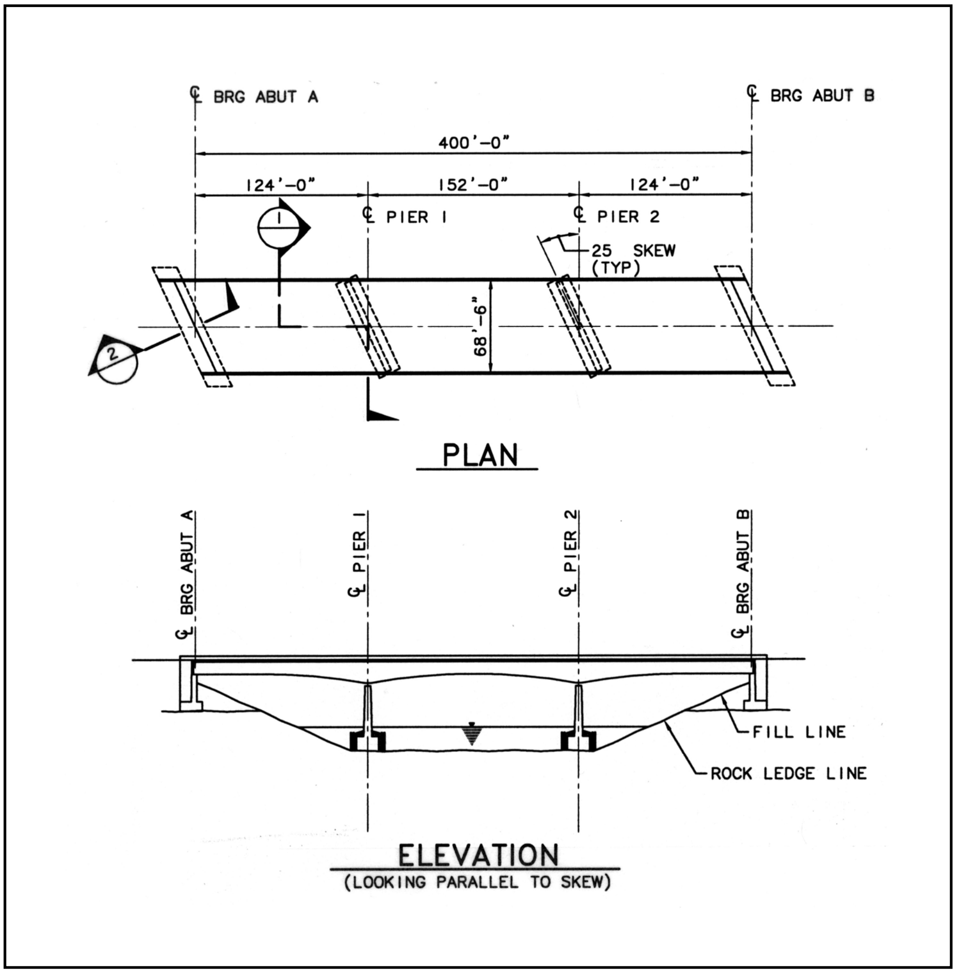


Figure 1a — Bridge No. 2LRFD - Plan and Elevation

SECTION I INTRODUCTION

BRIDGE DATA
(continued)

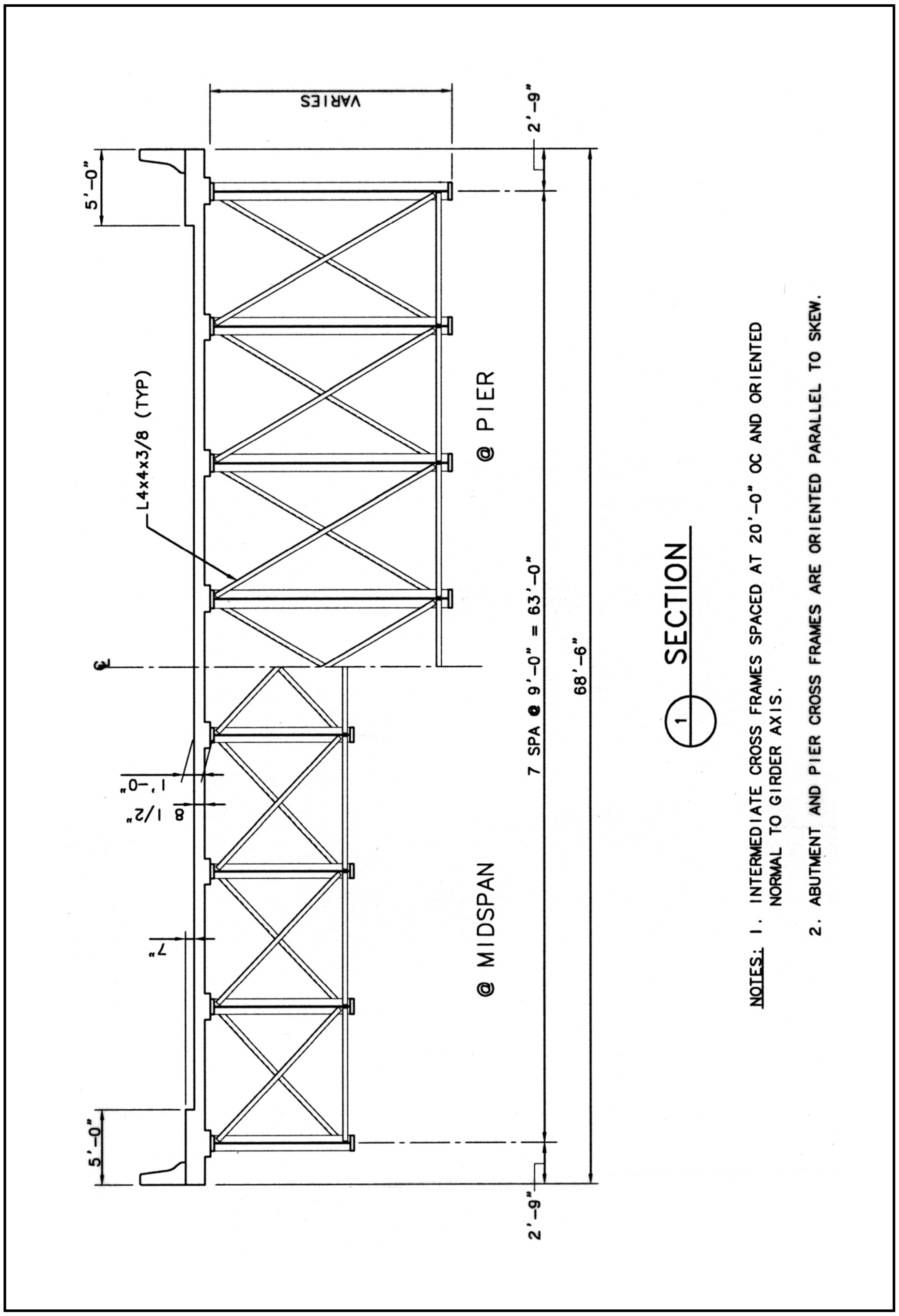


Figure 1b — Bridge No. 2LRFD - Typical Cross Section

SECTION I INTRODUCTION

BRIDGE DATA
(continued)

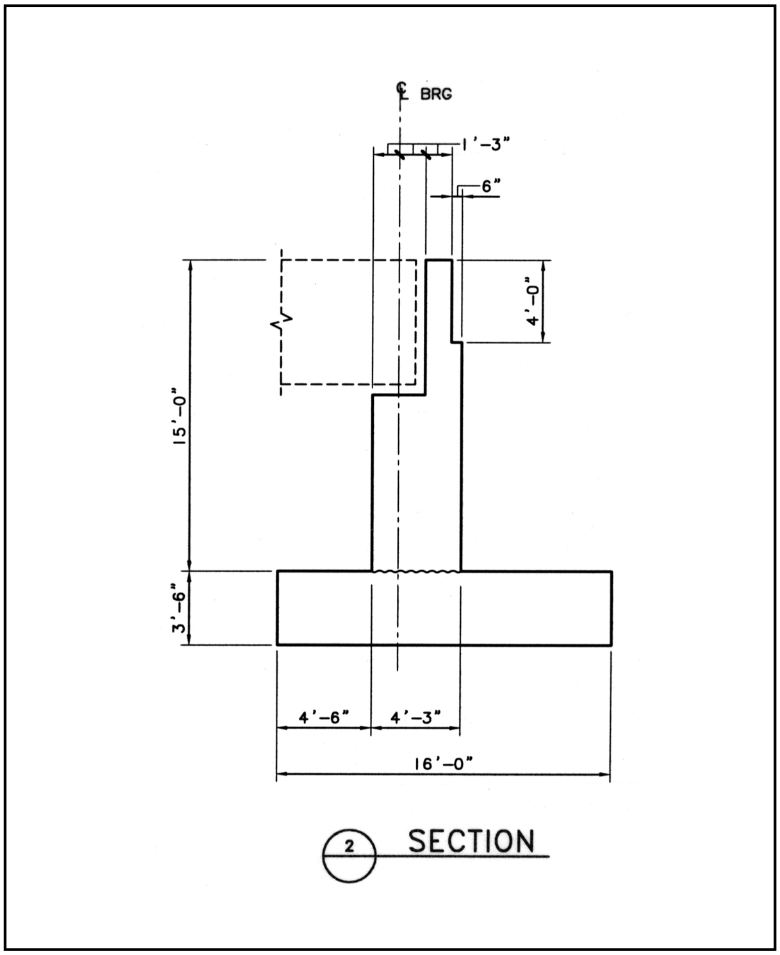


Figure 1c — Bridge No. 2LRFD - Seat-Type Abutment

SECTION I INTRODUCTION

BRIDGE DATA
(continued)

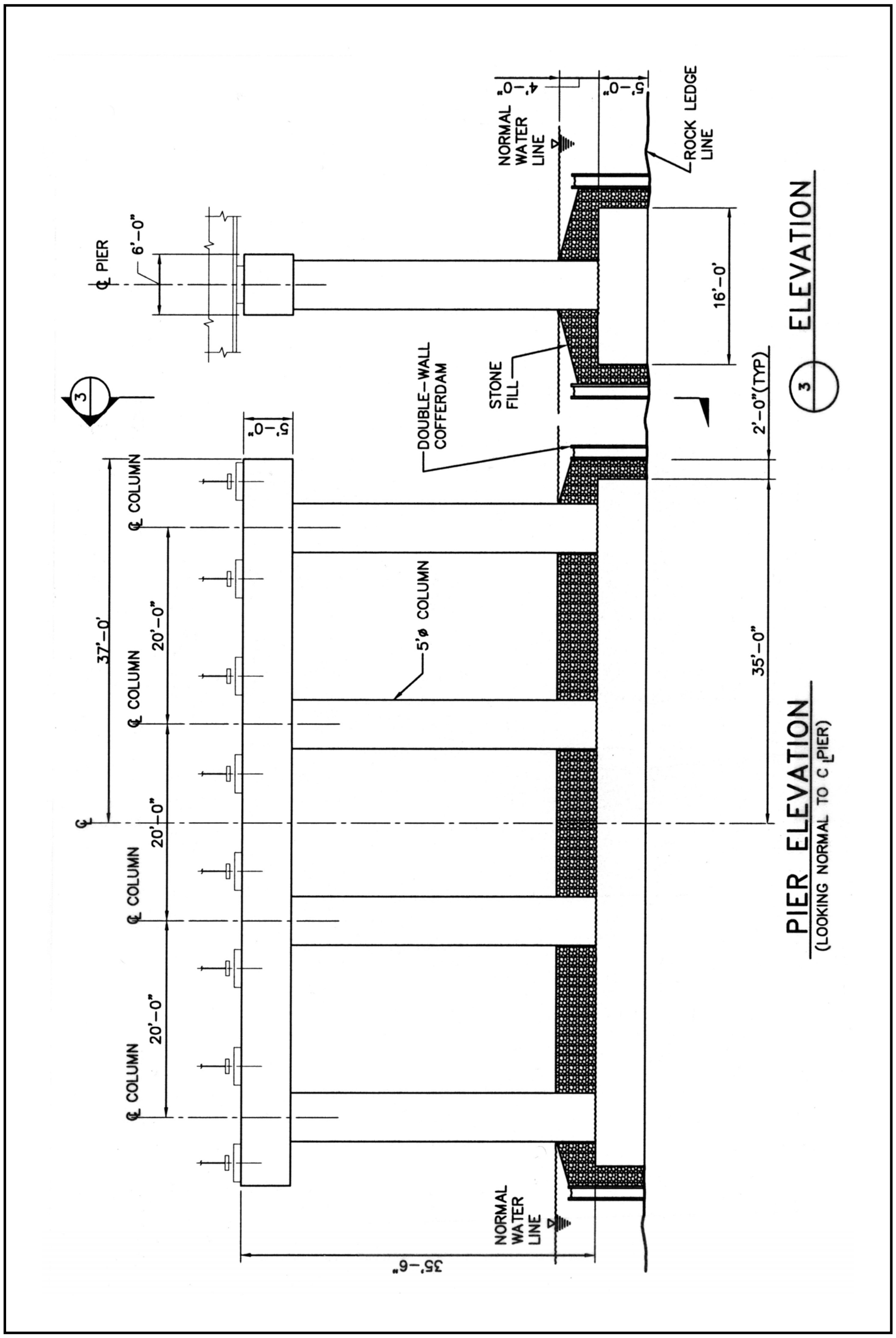


Figure 1d — Bridge No. 2LRFD – Four-Column Bent Elevation (for Section III)

SECTION I INTRODUCTION

BRIDGE DATA
(continued)

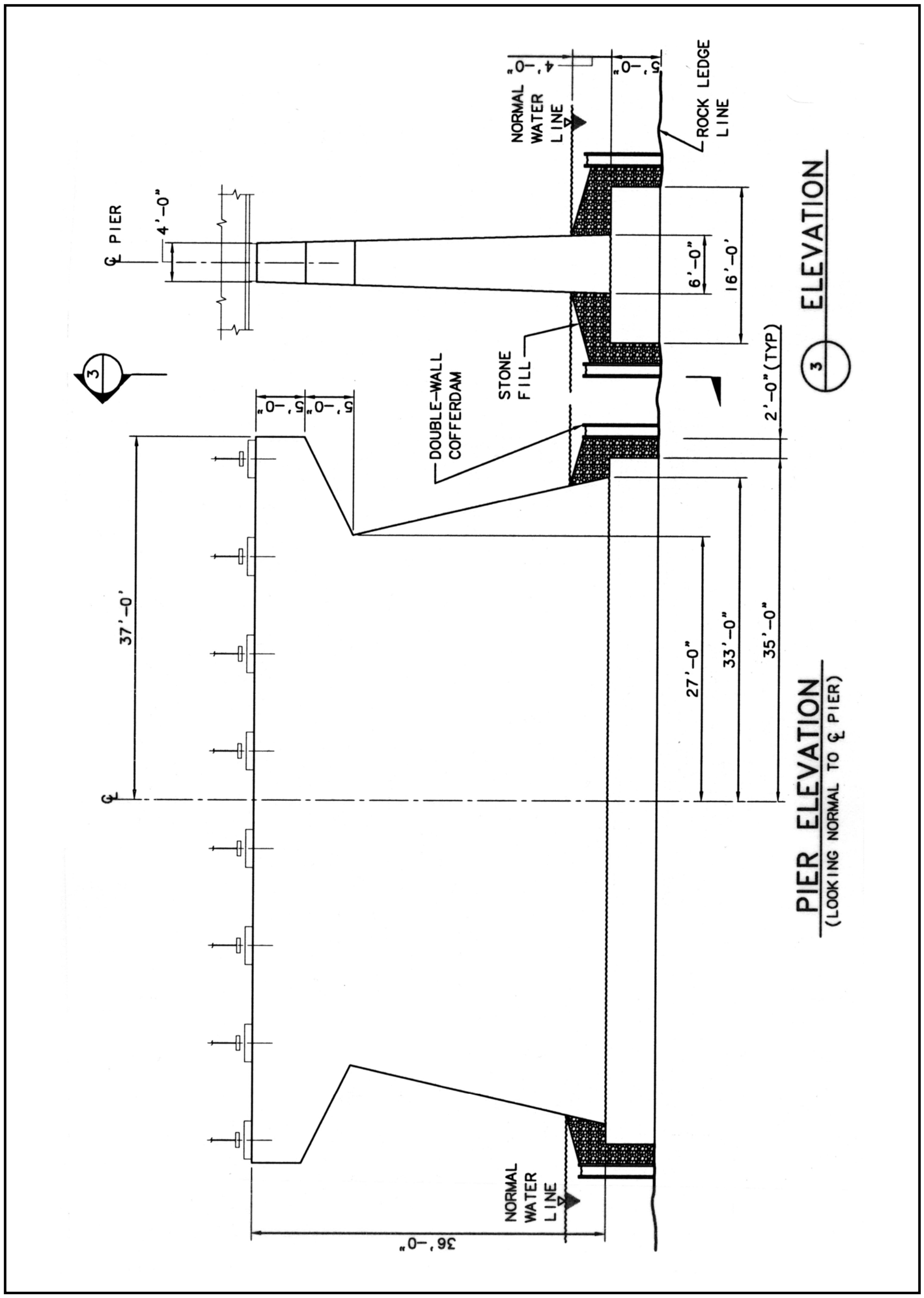


Figure 1e — Bridge No. 2LRFD – Wall Pier Elevation (for Section IV)

SECTION I INTRODUCTION

BRIDGE DATA
(continued)

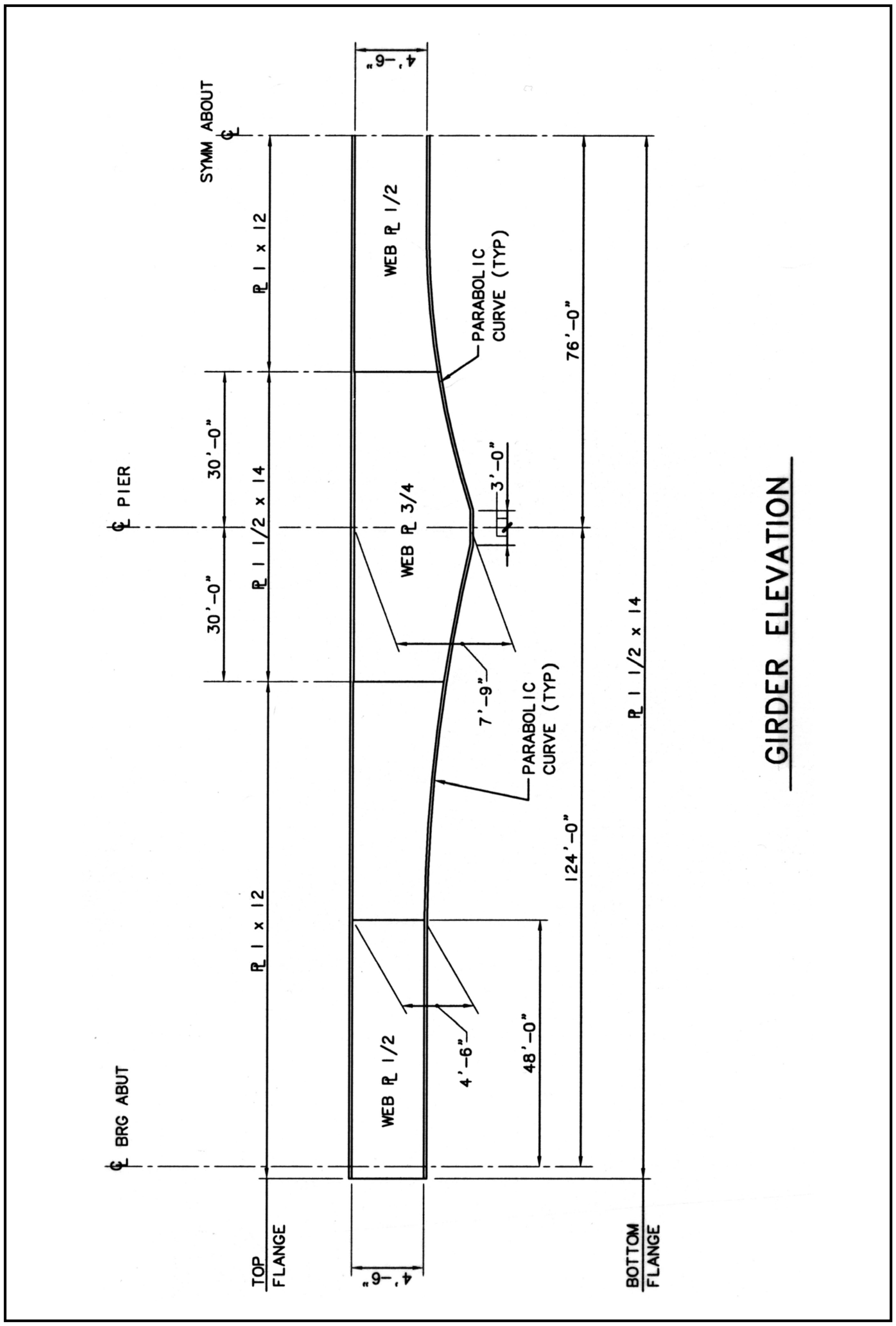


Figure 1f — Bridge No. 2LRFD – Plate Girder Detail

SECTION II FLOWCHARTS

Start

Design Step 1.0	<u>Preliminary Design</u> - Seismic Design Approach - Earthquake Resisting Systems
Design Step 2.0	<u>Basic Requirements</u> - Applicability - Seismic Performance Objectives - Spectral Accelerations - Site Class & Coefficients - Vertical Acceleration Effects - Liquefaction and Collateral Seismic Hazard Considerations
Design Step 3.0	<u>Determine Seismic Design and Analysis Procedure</u> - Seismic Hazard Level - Seismic Design and Analysis Procedure - Seismic Detailing Requirements - Response Modification Factors
Design Step 4.0	<u>Determine Elastic Seismic Forces and Displacements</u> - Not Required for SDAP A1, A2 or B - Capacity Spectrum Method (SDAP C) - Elastic Response Spectrum Method (SDAP D & E) - Modeling Requirements / Structure and Foundations
Design Step 5.0	<u>Determine Design Forces</u> - Directional Combination of Forces - Modified Seismic Design Forces (R Factor) - Load Combinations
Design Step 6.0	<u>Design Primary Earthquake Resisting Elements</u> (e.g. Elements Intended to Dissipate Energy)
Design Step 7.0	<u>Design Displacements and Checks</u> - Seat Widths - P-Δ Checks - Displacement Capacity Verification (SDAP E)

SECTION II FLOWCHARTS

Design Step 8.0	<u>Design Structural Components</u> - Seismic Detailing Requirements SDRs - Transverse Steel in Columns and Walls - Connections, Shear Keys, Joint Designs, Restrainers, Bearings - Superstructure Checks / Design Requirements - Capbeams, Diaphragms
Design Step 9.0	<u>Design Foundations</u> - Seismic Detailing Requirements SDRs - Footings, Piles, Shafts - Connections, Joints
Design Step 10.0	<u>Design Abutments</u> - Seismic Detailing Requirements SDRs - Shear Keys, Connections - Footings, Piles, Shafts
Design Step 11.0	<u>Consideration of Liquefaction-Induced Flow or Spread</u> - Evaluation of Foundation Displacement Demands / Capacities - Ground Improvement / Structural Improvement
Design Step 12.0	<u>Seismic Design Complete ?</u> - Revise As Necessary to Meet Criteria

SECTION III SDAP C CONVENTIONAL BEARING EXAMPLE

Design Step 1, Preliminary Design

DESIGN STEP 1 PRELIMINARY DESIGN

The bridge is located on the north Merrimack River, north of Concord, New Hampshire. The preliminary design of the bridge has been completed.

Conventional, mechanical bearings that restrain both longitudinal and transverse movements and which allow rotation about axes perpendicular to each girder are used at both bents, and sliding bearings are used at the abutments. The intermediate substructures are four-column bents with cap beams that support the bearings and superstructure. Seismic behavior will be as shown in Figure 2. Thermal loads induced by conventional bearings may pose a problem in design of the substructure and foundation. Thermal loads are neglected in this design example. It is assumed in this example that nonseismic considerations lead to the column diameter of 5 feet and reinforcement ratio of 1 percent.

Design Step 1.1 Seismic Design Objectives

[Guide Spec, Article 3.3] [NCHRP, Article 2.5.6]

Section 3.3 of the LRFD Guide Specification requires that a "clearly identifiable earthquake resisting system (ERS)" be selected to achieve the appropriate performance objectives defined in Table 3.2-1.

In this example, the ERS includes conventional inelastic action (plastic hinging) in the columns.

Design Step 1.2 Earthquake Resisting Systems

[Guide Spec, Article 3.3.1] [NCHRP, Article 2.5.6.1]

Section 3.3.1 of the LRFD Guide Specification introduces the concept of ERS and earthquake resisting elements (ERE). This concept is new and it organizes commonly occurring systems and elements into three categories: 1) Permissible, 2) Permissible with Owner's Approval, and 3) Not Recommended for New Bridges.

In this example, the bridge system is classified as "Permissible."

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 1, Preliminary Design

DESIGN STEP 1
(continued)

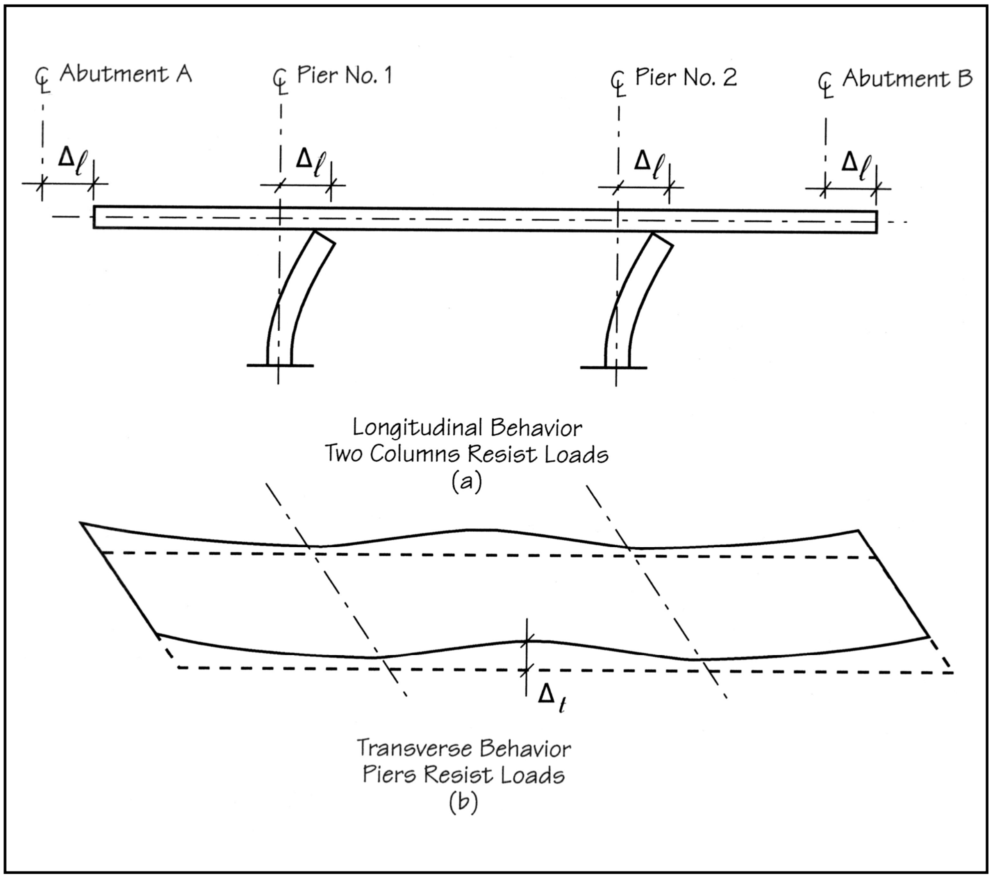


Figure 2 — Seismic Behavior with Conventional Bearings

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 2, Basic Requirements

DESIGN STEP 2 BASIC REQUIREMENTS

Design Step 2.1 Applicability of Specification

[Guide Spec, Article 3.1] [NCHRP, Article 3.10.1.1]

The bridge has three spans that total 400 feet. The end spans are 124 feet, the center span is 152 feet, and the bridge superstructure is steel plate girders with a composite concrete deck. Because no span is longer than 500 feet, and the construction is conventional, the Specification applies.

Design Step 2.2 Seismic Performance Objectives

[Guide Spec, Article 3.2] [NCHRP, Article 3.10.1.2]

For this example, the selected performance level is "Life Safety," the minimum required for all bridges. This is the case for both the MCE and the Frequent earthquake.

Table 3.2-1 defines the performance levels for service and damage the bridge is to be designed for. In this case, the choice of Life Safety as the performance level implies that for the Frequent earthquake minimal damage is expected and the structure is expected to fully open to normal traffic following an inspection of the bridge. The Life Safety choice also implies that in the MCE earthquake significant damage is expected, and the bridge will likely not be available to full traffic following an earthquake. The bridge may, in fact, be damaged to the point where it needs to be replaced following the MCE event. Displacement limits are established by the provisions to guide the designer in assessing geometrically what is implied by the specified service levels. Per the LRFD Guide Specification, displacements should be checked "to satisfy geometric, structural, and foundation constraints on performance" as outlined in Table C3.2-1 of the Specification.

Design Step 2.3 Spectral Acceleration Parameters

[Guide Spec, Article 3.4.1] [NCHRP, Article 3.10.2.1]

The site is on the north Merrimack River, north of Concord, New Hampshire. Using national ground motion maps, the MCE short-period (0.2 second) acceleration, S_S, is 0.46g and the 1.0-second acceleration, S_1, is 0.12g.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 2, Basic Requirements

The spectral accelerations for the Frequent earthquake were determined by the geotechnical engineer, and likewise are based on national ground motion maps. The short-period (0.2 second) acceleration, S_S, is 0.15g and the 1.0-second acceleration, S_1, as 0.04g.

Design Step 2.4 **Site Class**
[Guide Spec, Article 3.4.2.1] [NCHRP, Article 3.10.2.2.1]

The site class is B because the founding soil is rock. In this case, the shear wave velocity is taken as greater than 2,500 feet per second.

Design Step 2.5 **Site Coefficients**
[Guide Spec, Article 3.4.3.2] [NCHRP, Article 3.10.2.2.3]

<u>Maximum Considered Earthquake (3% in 75 years)</u>

The site coefficient for the short-period range, F_a, is 1.0 for site Class B.
The site coefficient for the long-period range, F_v, is 1.0 for site Class B.

<u>Frequent Earthquake (50% in 75 years)</u>

The site coefficient for the short-period range, F_a, is 1.0 for site Class B.
The site coefficient for the long-period range, F_v, is 1.0 site Class B.

Design Step 2.6 **Design Earthquake Response Spectra**
[Guide Spec, Article 3.4.1] [NCHRP, Article 3.10.2.1]

Not computed. See Design Step 3.

Design Step 2.7 **Vertical Acceleration Effects**
[Guide Spec, Article 3.4.5] [NCHRP, Article 3.10.2.6]

The bridge site is in the eastern part of the United States where vertical acceleration effects are not required to be considered in the design.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 3, Determine Seismic Design and Analysis Procedure

DESIGN STEP 3 DETERMINE SEISMIC DESIGN AND ANALYSIS PROCEDURE

Design Step 3.1 Determine Seismic Hazard Level

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3.1]

$F_aS_s = 0.46$ and $F_vS_1 = 0.12$.

The Seismic Hazard Level is III.

By Table 3.7-1, the Seismic Hazard Level is III because F_aS_s exceeds 0.35. Based on F_vS_1, the Seismic Hazard Level would only be I. The controlling value is taken to be the more restrictive of the two values.

Design Step 3.2 Determine Seismic Design and Analysis Procedure (SDAP)

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3.1]

SDAP C will be used.

Table 3.7-2 of the Specification gives the requirements for determining what Seismic Design and Analysis Procedure (SDAP) should be used. The table suggests either B, C, D, or E can be used for the Life-Safety performance level in Seismic Hazard Level III. For this example, use SDAP C.

In general, the capacity spectrum design approach may begin with a structure designed for nonseismic load cases. Then the structure is checked for seismic adequacy, and elements that are not adequate are revised to meet the requirements. Adequacy is checked for those elements that are part of the primary ERS, such as columns. If their lateral resistance is sufficient, then the remainder of the structure is designed using the capacity protection procedures of Section 4.8. If the lateral resistance is not adequate, then it is increased until it is acceptable, then capacity protection is used for the remainder of the design.

Design Step 3.3 Determine Seismic Detailing Requirements (SDR)

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3.1]

SDR 3 is applicable for SDAP C.

Because the structure is classified for Life-Safety Performance and Seismic Hazard Level III, Table 3.7-2 requires SDR 3. The detailing provisions for various components of the structure will be discussed in more detail in subsequent design steps in this design example.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

DESIGN STEP 4 DETERMINE ELASTIC SEISMIC FORCES AND DISPLACEMENTS

SDAP C Capacity Spectrum Design Method
[Guide Spec, Article 4.4] [NCHRP, Article 3.10.3.4]

This method combines a demand and capacity analysis, including the effect of inelastic behavior of ductile earthquake resisting elements. It applies only to single degree of freedom systems and is restricted to bridges that meet the regularity requirements of Guide Specification 4.4.2.

The basic relationship of the capacity spectrum method is

$$C_s \cdot \Delta = \left(\frac{F_v \cdot S_1}{2 \cdot \pi \cdot B_L} \right)^2 \cdot g$$

Check the regularity requirements of Guide Specification 4.4.2

a) Bridge has three spans, less than six maximum.
b) Bridge has three spans, which is equal to minimum number required.
c) Sliding bearings at abutments do not resist significant seismic forces in either direction.
d) Maximum span length is 152 feet, less than 200-foot maximum.
e) Ratio of span lengths is 1.22, less than maximum of 1.5.
f) No pier walls.
g) The maximum skew angle is 25 degrees, less than the maximum of 30 degrees. Piers are parallel to each other.
h) Bridge is not horizontally curved.
g) Bent stiffnesses are equal.
h) Bent strengths are equal.
i) There is no potential for liquefaction.

Note that the requirement that the abutments resist no significant lateral forces means that the superstructure must carry the inertial forces to the two piers with diaphragm action. This means the superstructure must be designed to effectively cantilever laterally to the abutments.

The capacity spectrum method does not require the consideration of earthquake loading in two directions, simultaneously. Thus the SRSS or 100%-40% directional combination rules are not used. For this reason, the checks of the substructure against the basic relationship of the capacity

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

DESIGN STEP 4
(continued)

spectrum method will be made independently in each direction. Thus the remaining design steps of the example, as outlined in the flowcharts, will be executed first for the longitudinal direction then repeated for the transverse direction.

In the case of the skew, checks will actually be in the weak direction of the bent and then in the strong direction of the bent. The pinned bearings are assumed to be rotationally released on an axis along the skewed bent. This is true even though the bearings are oriented such that rotation is perpendicular to the girder line. The least resistance to rotation is about an axis along the skewed bent. This behavior was discussed in the original Design Example 2. The superstructure design will require that the weak and strong direction results be resolved into longitudinal and transverse forces.

The SDAP C process is described in Sections 4.4.1 and 5.4.1 of the Guide Specification. As applied here as a design check, the process applied is as follows: First, the vertical and 'lateral' weights tributary to each pier are calculated. For the most part, the check will be executed on a pier-by-pier basis. These weights will be used for the checks of both the frequent and MCE earthquakes. The check for the frequent earthquake requires that the yield displacement be calculated for use in the basic capacity spectrum relationship. Because the structure should remain essentially elastic in the frequent earthquake, the yield displacement is used directly in the basic relationship. The check for the MCE earthquake requires that a minimum value of $C_S\Delta$ be supplied. Thus the design check can be accomplished by first calculating the structure C_S, then calculating a required minimum displacement capacity to be supplied. Then check the actual capacity against this value for the worst pier.

Design Step 4.1

Frequent Earthquake (50% in 75 years)
Longitudinal/Weak Direction

$F_v := 1.0$

$S_1 := 0.04$

Design Step 4.1.1

Effective Weights

Calculate the weight of the superstructure and distribute to the bent columns.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.1.1 (continued)

The total superstructure weight is made up of the following:

$w_{misc} := 3.69 \cdot \frac{kip}{ft}$ Weight of overlay, deck forms, barriers, and crossframes/stiffeners

$w_{deck} := 8.16 \cdot \frac{kip}{ft}$ Weight of deck and sidewalks

$w_{girders} := 2.0 \cdot \frac{kip}{ft}$ Assumed "average" weight of girders Actual is 3.04 kip/ft at the piers and 1.63 kip/ft at minimum depth.

$L := 400 \cdot ft$ Length of bridge

$$W_{super} := \left(w_{misc} + w_{deck} + w_{girders}\right) \cdot L$$

$$W_{super} = 5540\,kip$$

Estimate the vertical load, Ppier:

$$P_{pier} := W_{super} \cdot \frac{\frac{124 \cdot 5 \cdot ft}{8} + \frac{152 \cdot ft}{2}}{400 \cdot ft}$$

$$P_{pier} = 2126\,kip$$

Estimate the dead load of column, Pcol:

$D_{col} := 5 \cdot ft$

$H_{clr} := 30.5 \cdot ft$

$$P_{col} := \frac{\pi}{4} \cdot D_{col}^{\ 2} \cdot 0.15 \cdot \frac{kip}{ft^3} \cdot H_{clr} \qquad P_{col} = 90\,kip$$

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Estimate the weight of the cap beam, Pcap_bm:

$$P_{cap\_bm} := (6 \cdot ft) \cdot (5 \cdot ft) \cdot (74 \cdot ft) \cdot 0.15 \cdot \frac{kip}{ft^3} \qquad P_{cap\_bm} = 333\,kip$$

Total dead load at bottom of column:

$$P_{col\_dl} := \frac{P_{pier}}{4} + \frac{P_{cap\_bm}}{4} + P_{col} \qquad P_{col\_dl} = 705\,kip$$

Design Step 4.1.2

Yield Displacement

Compute the yield displacement, Δy for each column and each pier, which is the same for the two piers. Columns are 5-foot diameter with 1 percent reinforcement.

$M_n := 4500 \cdot kip \cdot ft$ Nominal moment at top of column. See Figure 3

Moment at bottom of column is slightly higher. This difference is neglected in this example.

$H := 36 \cdot ft$ Height to pin bearing from foundation

This distance will be used to calculate the displacement capacity and strengths in the weak direction of the pier.

$E_c := 3605ksi$

$$I_{cr} := \frac{\pi}{64} \cdot \frac{D_{col}^{\;4}}{2}$$ Cracked section taken as one-half gross section.

$$\Delta_y := \frac{M_n H^2}{3 \cdot E_c \cdot I_{cr}} \qquad \Delta_y = 0.24\,ft$$

$1.3 \cdot \Delta_y = 0.32\,ft$

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.1.2 (continued)

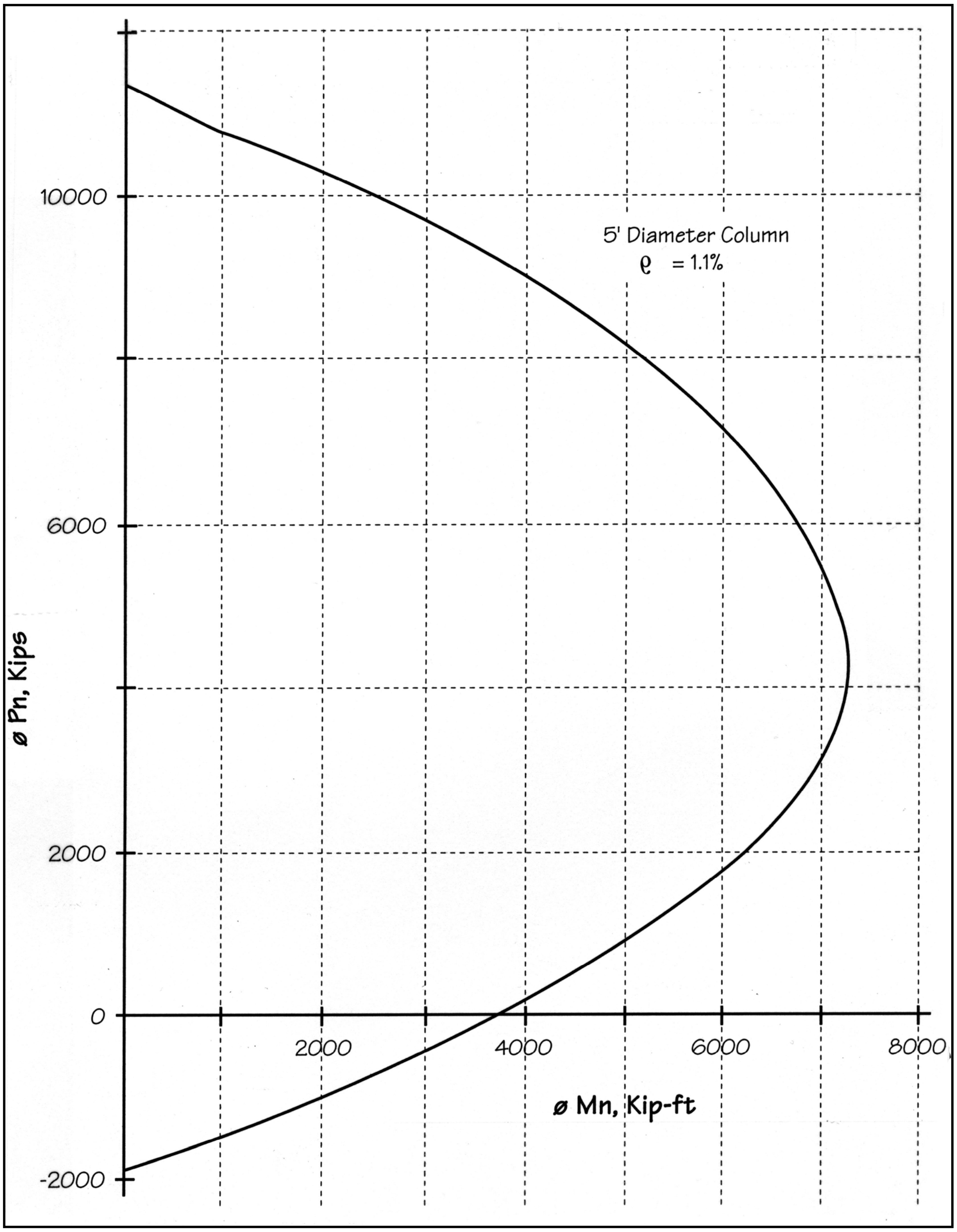


Figure 3 — Column Interaction Capacity Curve

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.1.3

Required Lateral Strength
[Guide Spec, Article 5.4.1] [NCHRP, Article 4.8.5.1]

Compute the required lateral strength of each participating column.

$$V_{up} := \frac{M_n}{H} \qquad V_{up} = 125\ \text{kip}$$

Sum the strengths to give the strength of the bridge.

$n := 8$ number of columns

$$\Sigma V_{up} := n \cdot V_{up} \qquad \Sigma V_{up} = 1000\ \text{kip}$$

Compute the seismic coefficient, Cs; this coefficient is based on the entire bridge strength and weight.

$$C_s := \frac{\Sigma V_{up}}{W_{super}} \qquad C_s = 0.18$$

Design Step 4.1.4

Basic Relationship
[Guide Spec, Article 5.4.1] [NCHRP, Article 4.8.5.1]

$B_L := 1.0$ 5% nominal damping

$C_s \cdot 1.3\Delta_y = 0.06\ \text{ft}$ Supplied $C_s \cdot \Delta$

$$\left(\frac{F_v \cdot S_1}{2 \cdot \pi \cdot \frac{\text{rad}}{\text{sec}} \cdot B_L}\right)^2 \cdot g = 0.0013\ \text{ft}$$ Required $C_s \cdot \Delta$

$$C_s \cdot 1.3\Delta_y > \left(\frac{F_v \cdot S_1}{2 \cdot \pi \cdot B_L}\right)^2 \cdot g$$ Therefore, bent columns are adequate for Frequent EQ

Note that Equation 5.4.1-1 typically controls over Equation 5.4.1-2.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.2

MCE EQ (3% in 75 years)
Longitudinal/Weak Direction

Use C_S as calculated above.

$$F_v := 1.0$$

$$S_1 := 0.12$$

Design Step 4.2.1

Maximum Displacement

Determine the maximum displacement capacity that must be supplied.

$$B_L := 1.6$$

The performance level for this bridge is "Life Safety," which by Table 5.4.1-1 of the Guide Specification, sets B_L at 1.6. This implies some energy dissipation in the bridge system, and this dissipation is accompanied by some damage. If a B_L of 1.0 were used instead, as was the case for the Frequent earthquake, then operational performance would be achieved. In this example, if a B_L of 1.0 were used to design the piers, or alternatively if the design provided can resist B_L = 1.0 forces, and if the bearings at the abutments can accommodate the displacements corresponding to B_L = 1.0, then operational performance will be achieved, even in the MCE.

$$\Delta_{max} := \frac{1}{C_s}\left(\frac{F_v \cdot S_1}{2\cdot\pi\cdot\frac{rad}{sec}\cdot B_L}\right)^2 \cdot g \qquad \Delta_{max} = 0.03\,ft \ \text{ required}$$

Design Step 4.2.2

Deformation Capacity
[Guide Spec, Article C5.4.1] [NCHRP, Article C4.8.5.1]

Check that the maximum required displacement capacity is less than the supplied deformation capacity for the pier. Per the commentary, only θp, plastic rotational capacity, is used in calculation. The yield capacity is not considered. This step is essentially the same as for SDAP E, except that the yield component of displacement is conservatively omitted.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

$\theta_p := .035$

$\theta_p \cdot H = 1.26\ \text{ft}$

$\Delta_{max} < \theta_p \cdot H_{col}$ OK

Design Step 4.2.3

P-Delta Requirement
[Guide Spec, Article 7.3.4] [NCHRP, Article 3.10.3.10.4]

This step is required only for the MCE (or rare) earthquake.

$C_s \cdot H \cdot 0.25 = 1.62\ \text{ft}$

$\Delta_{max} < C_s \cdot H \cdot 0.25$ OK

Bridge is adequate for MCE EQ.

Design Step 4.3

Frequent Earthquake (50% in 75 years)
Transverse/Strong Direction

$F_v := 1.0$

$S_1 := 0.04$

Design Step 4.3.1

Effective Weights

Same as longitudinal direction.

Design Step 4.3.2

Yield Displacement

Compute the yield displacement, Δy for each column.

$M_n := 4500 \cdot \text{kip} \cdot \text{ft}$

$\Delta_y := \dfrac{M_n H_{clr}^{\ 2}}{6 \cdot E_c \cdot I_{cr}}$ $\Delta_y = 0.09\ \text{ft}$

$1.3 \cdot \Delta_y = 0.11\ \text{ft}$

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.3.3

Required Lateral Strength
[Guide Spec, Article 5.4.1] [NCHRP, Article 4.8.5.1]

Compute the required lateral strength of each participating column. For the transverse direction, the method of Section 4.8.1.2 is used.

First iteration on bent with P_{col_dl} on each column:

$$V_{up} := \frac{2M_n}{H_{clr}} \qquad V_{up} = 295\,\text{kip}$$ per column

$$V_{up1} := V_{up} \qquad M_{n1} := M_n$$

$$V_{up2} := V_{up} \qquad M_{n2} := M_n$$

$$V_{up3} := V_{up} \qquad M_{n3} := M_n$$

$$V_{up4} := V_{up} \qquad M_{n4} := M_n$$

Calculate increments in column axial loads. Refer to Figure 1d. It is assumed that the outer columns carry ΔP and that the inner columns carry ΔP/3. By summing moments, we can solve for ΔP:

$$H_{cg} := 41\cdot\text{ft}$$ Height to cg of steel superstructure

$$\Delta P := \frac{\left(V_{up1} + V_{up2} + V_{up3} + V_{up4}\right)\cdot H_{cg} - \left(M_{n1} + M_{n2} + M_{n3} + M_{n4}\right)}{66.67\cdot\text{ft}}$$

$$\Delta P = 455.88\,\text{kip}$$ increment on outer columns

$$P_1 := P_{col\_dl} + \Delta P \qquad P_1 = 1160.45\,\text{kip}$$ outer column

$$P_2 := P_{col\_dl} + \frac{\Delta P}{3} \qquad P_2 = 856.53\,\text{kip}$$ inner column

$$P_3 := P_{col\_dl} - \frac{\Delta P}{3} \qquad P_3 = 552.61\,\text{kip}$$ inner column

$$P_4 := P_{col\_dl} - \Delta P \qquad P_4 = 248.69\,\text{kip}$$ outer column

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.3.3 (continued)

From the column interaction diagram:

$M_{n1} := 4800 \cdot kip \cdot ft \qquad V_{up1} := \dfrac{2 \cdot M_{n1}}{H_{clr}} \qquad V_{up1} = 314.75\, kip$

$M_{n2} := 4650 \cdot kip \cdot ft \qquad V_{up2} := \dfrac{2 \cdot M_{n2}}{H_{clr}} \qquad V_{up2} = 304.92\, kip$

$M_{n3} := 4300 \cdot kip \cdot ft \qquad V_{up3} := \dfrac{2 \cdot M_{n3}}{H_{clr}} \qquad V_{up3} = 281.97\, kip$

$M_{n4} := 4200 \cdot kip \cdot ft \qquad V_{up4} := \dfrac{2 \cdot M_{n4}}{H_{clr}} \qquad V_{up4} = 275.41\, kip$

Calculate increments in column axial loads:

$$\Delta P := \frac{\left(V_{up1} + V_{up2} + V_{up3} + V_{up4}\right) \cdot H_{cg} - \left(M_{n1} + M_{n2} + M_{n3} + M_{n4}\right)}{66.67 \cdot ft}$$

$\Delta P = 454.61\, kip$ increment on outer columns

$P_1 := P_{col\_dl} + \Delta P \qquad P_1 = 1159.19\, kip$

$P_2 := P_{col\_dl} + \dfrac{\Delta P}{3} \qquad P_2 = 856.11\, kip$

$P_3 := P_{col\_dl} - \dfrac{\Delta P}{3} \qquad P_3 = 553.04\, kip$

$P_4 := P_{col\_dl} - \Delta P \qquad P_4 = 249.96\, kip$

$V_{bent} := V_{up1} + V_{up2} + V_{up3} + V_{up4} \qquad V_{bent} = 1177.05\, kip$

$$\frac{V_{bent}}{4 \cdot V_{up}} = 1.00$$

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Two iterations is sufficient to converge.

Sum the strengths to give the strength of the bridge.

$$C_s := \frac{2 \cdot V_{bent}}{W_{super}} \qquad C_s = 0.42$$

Design Step 4.3.4

Basic Relationship
[Guide Spec, Article 5.4.1] [NCHRP, Article 4.8.5.1]

$$B_L := 1.0$$

$$C_s \cdot 1.3 \Delta_y = 0.05\,ft$$

$$\left(\frac{F_v \cdot S_1}{2 \cdot \pi \cdot \frac{rad}{sec} \cdot B_L} \right)^2 \cdot g = 0.0013\,ft$$

$$C_s \cdot 1.3 \Delta_y > \left(\frac{F_v \cdot S_1}{2 \cdot \pi} \right)^2 \cdot g$$

Therefore, bridge is adequate for Frequent EQ

Design Step 4.4

MCE EQ (3% in 75 years)
Transverse/Strong Direction

$$F_v := 1.0$$

$$S_1 := 0.12$$

Design Step 4.4.1

Maximum Displacement

Determine the maximum displacement capacity that must be supplied.

$$B_L := 1.6$$

$$\Delta_{max} := \frac{1}{C_s} \left(\frac{F_v \cdot S_1}{2 \cdot \pi \cdot \frac{rad}{sec} \cdot B_L} \right)^2 \cdot g \qquad \Delta_{max} = 0.01\,ft$$

SECTION III SDAP C CONVENTIONAL BEARING EXAMPLE

Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.4.2

Deformation Capacity
[Guide Spec, Article C5.4.1] [NCHRP, Article C4.8.5.1]

Check that the maximum required displacement capacity is less than the supplied deformation capacity for the pier.

$$\theta_p := .035$$

$$\theta_p \cdot H_{clr} = 1.07\ ft$$

$$\Delta_{max} < \theta_p \cdot H_{clr} \qquad OK$$

Design Step 4.4.3

P-Delta Requirement
[Guide Spec, Article 7.3.4] [NCHRP, Article 3.10.3.10.4]

$$C_s \cdot H_{clr} \cdot 0.25 = 3.24\ ft$$

$$\Delta_{max} < C_s \cdot H_{clr} \cdot 0.25 \qquad OK$$

Bridge is adequate for MCE EQ.

At this point, the SDAP C capacity spectrum method has incorporated Design Steps 5, 6, and 7 as outlined in the flowcharts, with the exception of the seat width requirement.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 7, Design Displacements and Checks

DESIGN STEP 7 DESIGN DISPLACEMENTS AND CHECKS

Design Step 7.1 Minimum Seat Width Requirement

[Guide Spec 7.3.2] [NCHRP, Article 3.10.3.10]

These checks are made for the MCE earthquake only.

$L := 400 \cdot ft$ $L = 121.9\,m$ $L := 121.9$ distance btwn joints

$H := 36 \cdot ft$ $H = 10.97\,m$ $H := 10.97$ tallest pier btwn joints

$B := 68.5 \cdot ft$ $B = 20.88\,m$ $B := 20.88$ width of superstructure

$F_v := 1.0$ from Design Step 2.5

$S_1 := 0.12$ from Design Step 2.3

$\alpha := 25 \cdot deg$ skew angle

$$N := \left[.10 + .0017 \cdot L + .007 \cdot H + .05\sqrt{H} \cdot \sqrt{1 + \left(\frac{2 \cdot B}{L}\right)^2} \right] \cdot \frac{\left(1 + 1.25 \cdot F_v \cdot S_1\right)}{\cos(\alpha)}$$

$N = 0.71$ meters

$N = 2.33\,ft$ minimum seat width

Seat width of 2.5 feet is provided at abutment per Figure 1c; thus the provided seat width is adequate.

In this example, the columns are seen to be much larger than needed for seismic considerations. If nonseismic considerations allow, the size or reinforcement of the column could be reduced (ρ_{min} = 0.8%) until nonseismic or $C_S\Delta$ limits are reached.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 8, Design Structural Components

DESIGN STEP 8 DESIGN STRUCTURAL COMPONENTS

[Guide Spec 7.2.2] [NCHRP, Article 3.10.3.8]

In SDAP C, capacity protection is required. Use Capacity Design Procedures to determine column shear and confinement reinforcement and connection forces.

Design Step 8.1 Transverse Steel in Columns

For transverse reinforcement in the column, the implicit approach of Guide Specification 7.8.2.3 was used. The calculation is not included in this example. If the explicit approach were used, an overstrength factor of 1.5 would be required.

Design Step 8.2 Calculate the Connection Force at each Bearing

For the weak direction:

$OS := 1.5$ Overstrength factor; see Guide Spec 4.8.1

$V_{up} = 125\ \text{kip}$ Plastic shear strength per column - weak direction

There are four columns per bent and eight bearings per bent. Therefore, the required connection design shear force for weak direction bending is

$$V_{conn} := OS\,\frac{4V_{up}}{8} \qquad V_{conn} = 93.75\ \text{kip}$$

SECTION III SDAP C CONVENTIONAL BEARING EXAMPLE
Design Step 8, Design Structural Components

Design Step 8.2 (continued)

For the strong direction:

Compute the plastic hinging forces for the bent. For the transverse direction, the method of Guide Specification 4.8.1.2 is used.

First iteration on bent with P_{col_dl} on each column:

Recall $M_n = 4500\,kip\cdot ft$

$OS := 1.5$

$M_{po} := OS\cdot M_n \qquad M_{po} = 6750\,kip\cdot ft$

$V_{po} := \dfrac{2M_{po}}{H_{clr}} \qquad V_{po} = 443\,kip$ per column

$M_{p1} := M_{po} \qquad V_{po1} := V_{po}$

$M_{p2} := M_{po} \qquad V_{po2} := V_{po}$

$M_{p3} := M_{po} \qquad V_{po3} := V_{po}$

$M_{p4} := M_{po} \qquad V_{po4} := V_{po}$

Calculate increments in column axial loads:

$$\Delta P := \frac{\left(V_{po1} + V_{po2} + V_{po3} + V_{po4}\right)\cdot H_{cg} - \left(M_{p1} + M_{p2} + M_{p3} + M_{p4}\right)}{66.67\cdot ft}$$

$\Delta P = 684\,kip$ increment on outer columns

$P_{col\_dl} = 705\,kip$

$P_{p1} := P_{col\_dl} + \Delta P \qquad P_{p1} = 1388\,kip$

$P_{p2} := P_{col\_dl} + \dfrac{\Delta P}{3} \qquad P_{p2} = 933\,kip$

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 8, Design Structural Components

Design Step 8.2 (continued)

$$P_{p3} := P_{col\_dl} - \frac{\Delta P}{3} \qquad P_{p3} = 477\,\text{kip}$$

$$P_{p4} := P_{col\_dl} - \Delta P \qquad P_{p4} = 21\,\text{kip}$$

From the column interaction diagram:

$$M_{p1} := OS \cdot 4900 \cdot \text{kip} \cdot \text{ft}$$

$$V_{po1} := \frac{2 \cdot M_{p1}}{H_{clr}} \qquad V_{po1} = 482\,\text{kip}$$

$$M_{p2} := OS \cdot 4700 \cdot \text{kip} \cdot \text{ft}$$

$$V_{po2} := \frac{2 \cdot M_{p2}}{H_{clr}} \qquad V_{po2} = 462\,\text{kip}$$

$$M_{p3} := OS \cdot 4200 \cdot \text{kip} \cdot \text{ft}$$

$$V_{po3} := \frac{2 \cdot M_{p3}}{H_{clr}} \qquad V_{po3} = 413\,\text{kip}$$

$$M_{p4} := OS \cdot 4000 \cdot \text{kip} \cdot \text{ft}$$

$$V_{po4} := \frac{2 \cdot M_{p4}}{H_{clr}} \qquad V_{po4} = 393\,\text{kip}$$

Calculate increments in column axial loads:

$$\Delta P := \frac{\left(V_{po1} + V_{po2} + V_{po3} + V_{po4}\right) \cdot H_{cg} - \left(M_{p1} + M_{p2} + M_{p3} + M_{p4}\right)}{66.67 \cdot \text{ft}}$$

$$\Delta P = 676\,\text{kip}$$ increment on outer columns

$$P_{p1} := P_{col\_dl} + \Delta P \qquad P_{p1} = 1381\,\text{kip}$$

$$P_{p2} := P_{col\_dl} + \frac{\Delta P}{2} \qquad P_{p2} = 1043\,\text{kip}$$

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 8, Design Structural Components

Design Step 8.2 (continued)

$P_{p3} := P_{col\_dl} - \frac{\Delta P}{2}$ $P_{p3} = 366\,kip$

$P_{p4} := P_{col\_dl} - \Delta P$ $P_{p4} = 28\,kip$

$V_{bent} := V_{po1} + V_{po2} + V_{po3} + V_{po4}$ $V_{bent} = 1751\,kip$

$\frac{V_{bent}}{4 \cdot V_{po}} = 0.99$ Two iterations is sufficient to converge.

There are eight bearings per bent. Therefore

$V_{conn} := \frac{V_{bent}}{8}$ $V_{conn} = 219\,kip$ Strong direction

The maximum connection force is 219 kip from the strong direction. The design of the anchor bolts, plates, etc is not included in this example.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 9, Design Foundations

DESIGN STEP 9 DESIGN FOUNDATIONS

[Guide Spec 7.2.2] [NCHRP, Article 3.10.3.8]

Use Capacity Design Procedures to determine foundation design forces.

Design Step 9.1 Calculate the Foundation Forces for Overturning, Sliding, and Soil Bearing Capacity in the Weak Direction

$OS := 1.0$ See Guide Spec 4.4.1, Step 6

For SDR 3, the design for geotechnical aspects only requires the nominal moment of the columns to be used, rather than the full overstrength.

This means that at the MCE limited overload may occur in the foundation (see Guide Specification Commentary 4.3.3).

$P_{col\_dl} = 705\,kip$ at bottom of column

$M_{OT} := OS \cdot M_n \cdot 4$ $M_{OT} = 18000\,ft\,kip$

$H := 36 \cdot ft$

$V_{OT} := \dfrac{M_{OT}}{H}$ $V_{OT} = 500\,kip$

$D_f := 5 \cdot ft$

$M_v := V_{OT} \cdot D_f$ $M_v = 2500\,ft\,kip$

The dead load forces must be augmented to account for foundation weight, buoyancy, and overburden effects. The shear forces and moments, however, do not require adjustment.

Based on the foundation configuration shown in Figure 4, calculate the additional axial force acting at the base of the foundation due to the stone fill overburden. Recall that the length of the footing is 70 feet, and assume that stone fill with a saturated unit weight of 0.130 kip per cubic foot is used.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 9, Design Foundations

Design Step 9.1
(continued)

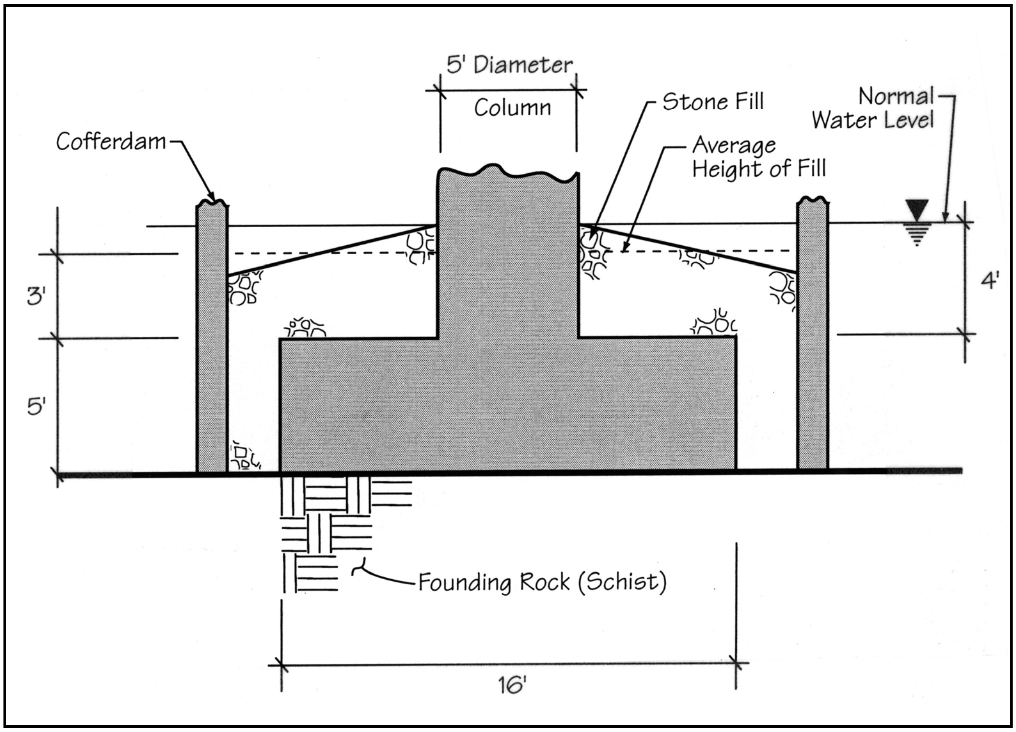


Figure 4 — Configuration of Bent Foundation

Overburden weight:

$$V_{sf} := 3 \cdot ft \left[(16 \cdot ft) \cdot (70 \cdot ft) - \frac{\pi}{4} \cdot D_{col}^{\ 2} \right]$$

$$P_{sf} := V_{sf} \cdot \left(.130 \cdot \frac{kip}{ft^3} \right)$$

$$P_{sf} = 429\ kip$$

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 9, Design Foundations

Design Step 9.1 (continued)

Calculate the uplift force due to buoyancy assuming the water level corresponds to the normal level, 4 feet above the top of the footing. Per the Commentary of the Guide Specification 3.5, mean discharge levels may be used for the Extreme Event I load combination.

Buoyancy force:

$V_{ftg} := (16 \cdot ft) \cdot (5 \cdot ft) \cdot (70 \cdot ft)$ Volume of footing

$V_{sf} = 3301.10\, ft^3$ Volume of stone fill

$V_{cols} := 4 \cdot \frac{\pi}{4} \cdot D_{col}{}^2 \cdot (3 \cdot ft)$ Volume of columns

$$P_b := \left(V_{ftg} + V_{sf} + V_{cols}\right) \cdot 0.0624 \cdot \frac{kip}{ft^3}$$

$$P_b = 570\, kip$$

Foundation weight:

$P_{ftg} := V_{ftg} \cdot .15 \cdot \frac{kip}{ft^3}$ $P_{ftg} = 840\, kip$

Axial Force:

Adjusted axial force acting at base of foundation

$$P := 4P_{col\_dl} + P_{ftg} + P_{sf} - P_b$$

$$P = 3517\, kip$$

Design Moment and Shear Forces.

Calculate the design moment to be used for the overturning check.

$M_{weak} := M_{OT} + M_v$ Weak direction driving moment

$$M_{weak} = 20500\, ft\, kip$$

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 9, Design Foundations

Calculate the design shear forces to be used in the sliding check.

$$V_{weak} := \frac{M_{OT}}{H}$$

$$V_{weak} = 500\,kip$$ Shear in weak direction.

Design Step 9.2 Check Foundation for Overturning in the Weak Direction

Per Section 7.4.2.1, footing lift off shall not exceed 50 percent at the peak displacement. This can be taken to mean under the action of the overstrength forces. In the case of SDR 3, which applies for this example, the overstrength is 1.0 for geotechnical effects.

To ensure that there is no more than one-half uplift on the footing, the eccentricity e must be less than $\frac{L_f}{3}$.

The preliminary length of the footing in the weak direction is:

$$L_f := 16 \cdot ft$$

The overturning induced eccentricity must be less than or equal to:

$$\frac{L_f}{3} = 5.33\,ft$$

The eccentricity of the axial load caused by the overturning moment can be calculated by:

$$e := \frac{M_{weak}}{P} \qquad e = 5.83\,ft$$

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 9, Design Foundations

Design Step 9.2 (continued)

The length of footing in the longitudinal direction must be increased to meet the one-half uplift requirement.

$$L_f := 3 \cdot e$$

$$L_f = 17.48\ \text{ft}$$

$$\frac{L_f}{3} = 5.83\ \text{ft}$$

Therefore, in order to meet the capacity design requirements, the foundation length in the longitudinal direction must be increased from 16 feet to 17.5 feet.

For there to be any uplift, the eccentricity must be greater than $\frac{L_f}{6}$.

$$\frac{L_f}{6} = 2.91\ \text{ft}$$

Design Step 9.3

Check the Soil Bearing Capacity in the Weak Direction

The contact stress can be calculated using the following method because the eccentricity is greater than one-sixth of the footing length. The equation can be derived assuming a triangular stress distribution.

$B_f := 70 \cdot \text{ft}$ Width of footing

$$q := \frac{2 \cdot P}{3 \cdot B_f \cdot \left(\frac{L_f}{2} - e\right)}$$ Maximum contact stress at edge of footing

$$q = 11.5\ \text{ksf}$$

By inspection, q is much less than the ultimate bearing capacity of 50 ksf. Thus the footing width is adequate to ensure that the foundation will not "yield" prior to the column hinging.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 9, Design Foundations

Design Step 9.4

Check Foundation for Sliding in the Weak Direction

The check of sliding is made by comparing the ultimate sliding resistance with the driving force. For this footing founded on a competent rock, the coefficient of friction may be taken as 0.8.

$V_r := 0.8 \cdot P$

$V_r = 2814 \text{ kip}$

The driving force is:

$V_{OT} = 500 \text{ kip}$

Because the resistance is larger than the driving force, the footing is adequate for sliding.

Design Step 9.5

Calculate the Foundation Forces for Design of Footing Reinforcement in the Weak Direction

The use of a 1.0 overstrength factor in SDR 3 only applies for geotechnical effects. For the design of structural elements, the normal overstrength factors, 1.5 for concrete columns, apply.

$OS := 1.5$

$P_{col\_dl} = 705 \text{ kip}$

$M_{po} := OS \cdot M_n \cdot 4$ $\qquad M_{po} = 27000 \text{ ft kip}$

$V_{po} := \dfrac{M_{po}}{H}$ $\qquad V_{po} = 750 \text{ kip}$

$D_f := 5 \cdot ft$

$M_v := V_{po} \cdot D_f$ $\qquad M_v = 3750 \text{ ft kip}$

Recall adjusted axial force acting at base of foundation

$P = 3517 \text{ kip}$

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 9, Design Foundations

Design Step 9.5 (continued)

Design Moment and Shear Forces.

$M_{weak} := M_{po} + M_v$ Weak direction driving moment

$M_{weak} = 30750\ \text{ft kip}$

$V_{weak} := \dfrac{M_{po}}{H}$

$V_{weak} = 750\ \text{kip}$ Shear in weak direction.

Recall the length of the footing in the longitudinal direction is:

$L_f = 17.48\ \text{ft}$

The eccentricity of the axial load caused by the overturning moment can be calculated by:

$e := \dfrac{M_{weak}}{P}$ $e = 8.74\ \text{ft}$

Note that the eccentricity is at $\dfrac{L_f}{2}$, the bounds of the footing. The maximum overturning moment that can be developed in the footing is that corresponding to a soil pressure diagram that is a block at the ultimate soil pressure magnitude, extending a distance "a" from the toe of the footing. This moment is the maximum that will develop just as rocking occurs.

$B_f := 70 \cdot \text{ft}$ Width of footing

$q_{ult} := 50 \cdot \text{ksf}$

$a := \dfrac{P}{q_{ult} \cdot B_f}$ $a = 1.00\ \text{ft}$ length of ultimate soil pressure block

$e_{max} := \dfrac{L_f - a}{2}$ $e_{max} = 8.24\ \text{ft}$

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 9, Design Foundations

Design Step 9.5 (continued)

Because $e > e_{max}$, the length of footing in the longitudinal direction will be increased to accommodate a sufficient length of the ultimate soil pressure block for equilibrium.

$L_{f\_min} := 2 \cdot e + a$ $\qquad L_{f\_min} = 18.49\,ft$

Therefore say,

$L_f := 18.5 \cdot ft$

$e_{max} := \dfrac{L_f - a}{2}$ $\qquad e_{max} = 8.75\,ft$

Now e < e_{max}, and the footing is reaching the moment at which rocking has fully developed. Note that if the footing began to rock before the attainment of the overstrength moment, then the rocking moment would define the design moment of the footing.

The final length of footing in the weak direction is 18.5 feet.

Using the ultimate soil pressure block at the toe of the footing, the designer can now design the footing for flexure and shear.

The ultimate shear is

$V_u := P$ $\qquad V_u = 3517\,kip$

The ultimate moment for bottom reinforcement is

$M_u := P \cdot \left(\dfrac{L_f - D_{col} - a}{2}\right)$ $\qquad M_u = 21974\,kip \cdot ft$

For top reinforcement, the weight of soil above the footing during uplift must be included.

This completes the capacity spectrum checks and design requirements for the weak direction.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 9, Design Foundations

Design Step 9.6 **Calculate the Foundation Forces for Overturning, Sliding, and Soil Capacity in the Strong Direction**

$OS := 1.0$ See Guide Spec 4.4.1, Step 6

Recall from Step 4.4.2

$\Sigma P := P_1 + P_2 + P_3 + P_4$ $\Sigma P = 2818 \text{ kip}$

$\Sigma V_{up} := V_{up1} + V_{up2} + V_{up3} + V_{up4}$ $\Sigma V_{up} = 1177 \text{ kip}$

$\Sigma M_n := M_{n1} + M_{n2} + M_{n3} + M_{n4}$ $\Sigma M_n = 17950 \text{ kip} \cdot \text{ft}$

$D_f := 5 \cdot \text{ft}$

$M_v := \Sigma V_{up} \cdot D_f$ $M_v = 5885 \text{ ft kip}$ Moment due to column shears.

$M_{\Delta P} := 30 \cdot \text{ft} \cdot P_1 + 10 \cdot \text{ft} \cdot P_2 - 10 \cdot \text{ft} \cdot P_3 - 30 \cdot \text{ft} \cdot P_4$

$M_{\Delta P} = 30308 \text{ kip} \cdot \text{ft}$

$P = 3517 \text{ kip}$

Design Moment and Shear Forces.

Calculate the design moment to be used for the overturning check.

$M_{strong} := \Sigma M_n + M_v + M_{\Delta P}$ Strong direction driving moment

$M_{strong} = 54143 \text{ kip} \cdot \text{ft}$

Calculate the design shear forces to be used in the sliding check.

$V_{strong} := \Sigma V_{up}$

$V_{strong} = 1177 \text{ kip}$ Shear in strong direction.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 9, Design Foundations

Design Step 9.7

Check Foundation for Overturning in the Strong Direction

$$L_f := 70 \cdot ft$$

The overturning induced eccentricity must be less than or equal to:

$$\frac{L_f}{3} = 23.33\,ft$$

The eccentricity of the axial load caused by the overturning moment can be calculated by:

$$e := \frac{M_{strong}}{P} \qquad e = 15.39\,ft \qquad OK$$

For there to be any uplift, the eccentricity must be greater than $\frac{L_f}{6}$.

$$\frac{L_f}{6} = 11.67\,ft$$

Design Step 9.8

Check the Soil Bearing Capacity in the Strong Direction

The contact stress can be calculated using the following method because the eccentricity is greater than one-sixth of the footing length. The equation can be derived assuming a triangular stress distribution.

$$B_f := 18.5 \cdot ft \qquad \text{Width of footing}$$

$$q := \frac{2 \cdot P}{3 \cdot B_f \cdot \left(\frac{L_f}{2} - e\right)} \qquad \text{Maximum contact stress at edge of footing}$$

$$q = 6.5\,ksf$$

By inspection q is much less than the ultimate bearing capacity of 50 ksf. Thus the footing width is adequate.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 9, Design Foundations

Design Step 9.9

Check Foundation for Sliding in the Strong Direction

The check of sliding is made by comparing the ultimate sliding resistance with the driving force. For this footing founded on a competent rock, the coefficient of friction may be taken as 0.8.

$V_r := 0.8 \cdot P$

$V_r = 2814\ \text{kip}$

The driving force is:

$V_{strong} = 1177\ \text{kip}$

Because the resistance is larger than the driving force, the footing is adequate for sliding.

Design Step 9.10

Calculate the Foundation Forces for Design of Footing Reinforcement in the Strong Direction

$OS := 1.5$

$\Sigma P_p := P_{p1} + P_{p2} + P_{p3} + P_{p4}$ $\Sigma P_p = 2818\ \text{kip}$

$\Sigma V_{po} := V_{po1} + V_{po2} + V_{po3} + V_{po4}$ $\Sigma V_{po} = 1751\ \text{kip}$

$\Sigma M_p := M_{p1} + M_{p2} + M_{p3} + M_{p4}$ $\Sigma M_p = 26700\ \text{kip} \cdot \text{ft}$

$D_f := 5 \cdot \text{ft}$

$M_v := \Sigma V_{po} \cdot D_f$ $M_v = 8754\ \text{ft kip}$ Moment due to column shears.

$M_{\Delta P} := 30 \cdot \text{ft} \cdot P_{p1} + 10 \cdot \text{ft} \cdot P_{p2} - 10 \cdot \text{ft} \cdot P_{p3} - 30 \cdot \text{ft} \cdot P_{p4}$

$M_{\Delta P} = 47335\ \text{kip} \cdot \text{ft}$

$P = 3517\ \text{kip}$

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 9, Design Foundations

Design Step 9.10 (continued)

Design Moment and Shear Forces.

$$M_{strong} := \Sigma M_p + M_v + M_{\Delta P}$$ Strong direction driving moment

$$M_{strong} = 82790 \text{ ft kip}$$

$$V_{strong} := \Sigma V_{po}$$

$$V_{strong} = 1751 \text{ kip}$$ Shear in strong direction.

Recall the length of the footing in the strong direction is:

$$L_f = 70.00 \text{ ft} \qquad \frac{L_f}{3} = 23.33 \text{ ft}$$

The eccentricity of the axial load caused by the overturning moment can be calculated by:

$$e := \frac{M_{strong}}{P} \qquad e = 23.54 \text{ ft}$$

Footing is slightly over 50 percent uplifted.

$$B_f = 18.50 \text{ ft}$$ Width of footing

$$q := \frac{2 \cdot P}{3 \cdot B_f \cdot \left(\frac{L_f}{2} - e\right)}$$ Maximum contact stress at edge of footing

$$q = 11.06 \text{ ksf}$$

By inspection, q is less than the ultimate bearing capacity of 50 ksf. Thus the footing length is adequate. The final length of footing in the strong direction is 70 feet.

Using the soil pressure diagram at the toe of the footing, the designer can now design the footing for flexure and shear. Note that in strong direction, with a combined footing and one-half uplift, the footing must be designed to distribute the gravity loads and lateral shears on the columns over the zone of uplift. This is not illustrated in this example; however, feasibility of such a design may drive the final configuration of the foundations.

The final footing size is 18.5 by 70 feet.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 10, Design Abutments

DESIGN STEP 10 DESIGN ABUTMENTS

Abutments are not assumed to take significant seismic load. This is a requirement of the SDAP C analysis. In an actual design, the abutments would, of course, be designed for the gravity and soil loads, including seismic effects.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 11, Consider Liquefaction

DESIGN STEP 11 CONSIDER LIQUEFACTION

There is no liquefaction potential.

SECTION III **SDAP C CONVENTIONAL BEARING EXAMPLE**
Design Step 12, Seismic Design Complete

DESIGN STEP 12 SEISMIC DESIGN COMPLETE?

The final footing size was 18.5 by 70 feet. The maximum bearing connection design force was 219 kips. This force is based on an overstrength factor of 1.5 and capacity protection on the strong direction. A reduction in the footing width of 18.5 feet or in the bearing connection force could be pursued by going to SDAP D with an elastic analysis. This was not done here.

SECTION IV SDAP C WITH ELASTOMERIC BEARING EXAMPLE

Design Step 1, Preliminary Design

This section illustrates the design of the same bridge and location as that of Section III, with the exception that wall piers are used at the two intermediate pier locations and elastomeric bearings are used to support the superstructure at the abutments and intermediate pier locations.

DESIGN STEP 1 PRELIMINARY DESIGN

As before, the bridge is located on the north Merrimack River, north of Concord, New Hampshire. The preliminary design of the bridge has been completed.

The form of the intermediate bents was established to accommodate ice loadings; and, therefore, the pier size is not controlled by seismic loading. The seat abutments are provided to accommodate thermal movements. They provide the ability for the bridge to move in the longitudinal direction.

Elastomeric bearings are used at each wall pier and at the abutments. The relatively low stiffness of the bearings will cause much of the earthquake-induced lateral movement to occur in the bearings. Consequently, the superstructure will tend to move essentially as a rigid body under seismic loading in both directions, and the forces transmitted to the substructure will be substantially smaller than those required to fully restrain the superstructure. Because elastomeric bearings are much more flexible than the wall piers, especially in the strong direction of the pier, little (if any) inelastic response is expected in the piers. The seismic behavior for this system is shown in Figure 5. The bearing details are shown in Figures 6 and 7.

The bearings are designed for the expected thermal movements and for the service loads. They are not intended to be true isolation bearings, which provide extra damping; they provide only the typical 5 percent damping. In the event that the bearings are overstrained under seismic loading, transverse girder stops will be provided as a failsafe mechanism. Longitudinally, the abutment back walls provide failsafe restraint at one end to prevent the end spans from dropping off the abutments at the other end.

SECTION IV SDAP C WITH ELASTOMERIC BEARING EXAMPLE

Design Step 1, Preliminary Design

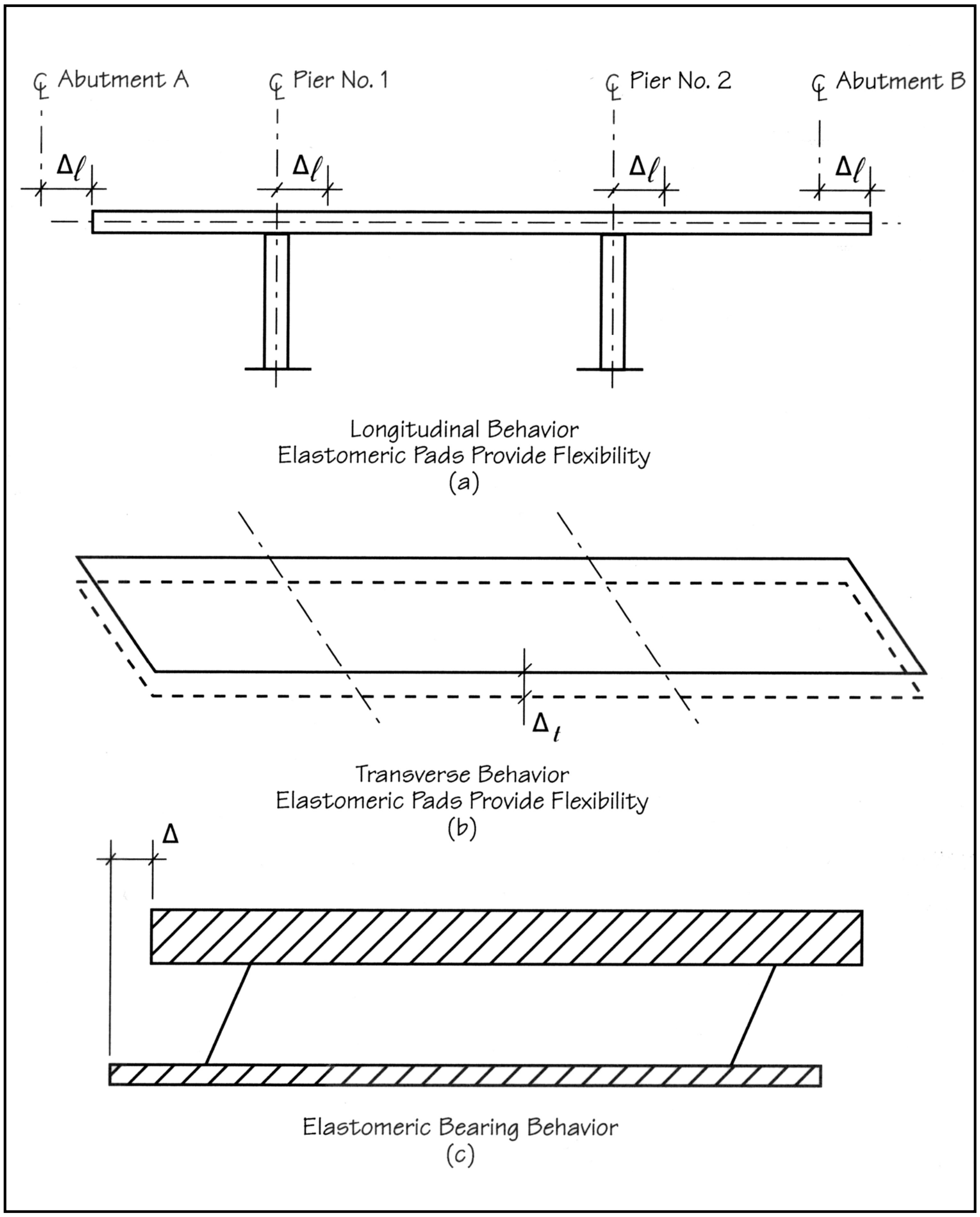


Figure 5 — Seismic Behavior with Elastomeric Bearings

SECTION IV SDAP C WITH ELASTOMERIC BEARING EXAMPLE

Design Step 1, Preliminary Design

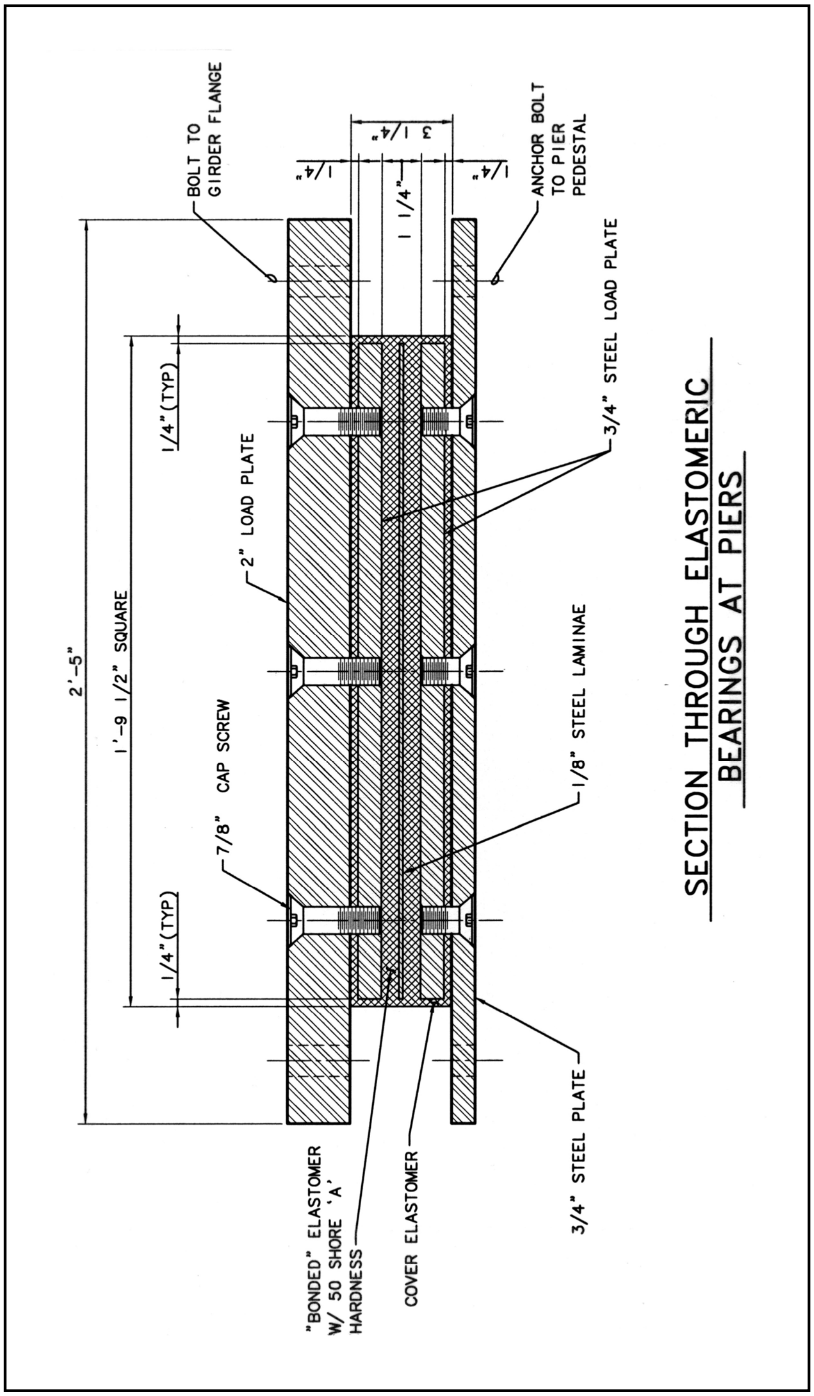


Figure 6 — Elastomeric Bearing at Pier

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**

Design Step 1, Preliminary Design

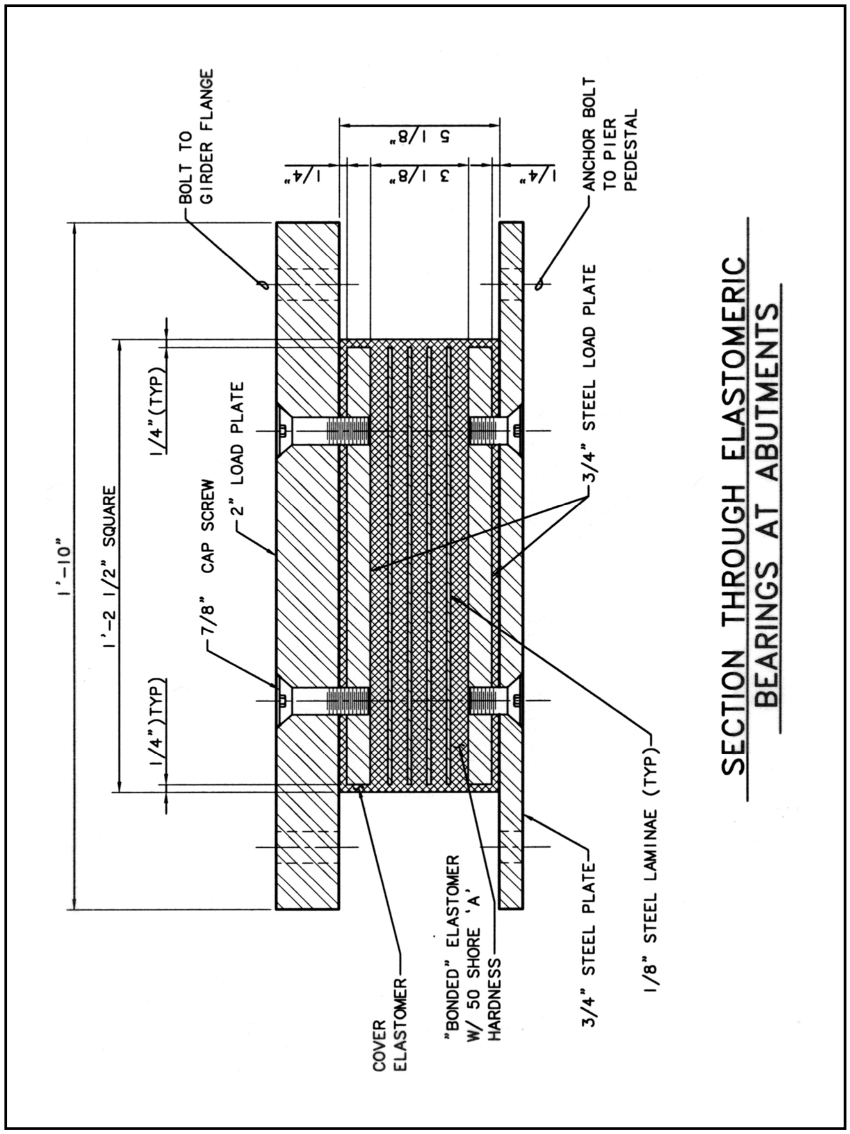


Figure 7 — Elastomeric Bearing at Abutment

SECTION IV SDAP C WITH ELASTOMERIC BEARING EXAMPLE

Design Step 1, Preliminary Design

Design Step 1.1 Seismic Design Objectives

[Guide Spec, Article 3.3] [NCHRP, Article 2.5.6]

Section 2.5 of the LRFD Guide Specification requires that a "clearly identifiable earthquake resisting system (ERS)" be selected to achieve the appropriate performance objectives defined in Table 3.2-1.

In this example, the ERS includes the behavior of the elastomeric bearings. Although they are not intended to dissipate energy, they are classified as isolation bearings for the seismic design. This will invoke the provisions of Chapter 15 for Isolation Design, including the requirements for testing of the bearings.

Design Step 1.2 Earthquake Resisting Systems

[Guide Spec, Article 3.3.1] [NCHRP, Article 2.5.6.1]

Section 3.3.1 of the LRFD Guide Specification introduces the concept of ERS and earthquake resisting elements (ERE). This concept is new and it organizes commonly occurring systems and elements into three categories: 1) Permissible, 2) Permissible with Owner's Approval, and 3) Not Recommended for New Bridges.

In this example, the bridge system is classified as "Permissible," because the bearings will be designed to accommodate the full seismic displacement.

SECTION IV **SDAP C ELASTOMERIC BEARING EXAMPLE**
Design Step 2, Basic Requirements

DESIGN STEP 2 BASIC REQUIREMENTS

Design Step 2.1 Applicability of Specification

[Guide Spec, Article 3.1] [NCHRP, Article 3.10.1.1]

The bridge has three spans that total 400 feet. The end spans are 124 feet, the center span is 152 feet, and the bridge superstructure is steel plate girders with a composite concrete deck. Because no span is longer than 500 feet, and the construction is conventional, the Specification applies.

Design Step 2.2 Seismic Performance Objectives

[Guide Spec, Article 3.2] [NCHRP, Article 3.10.1.2]

For this example, the selected performance level is "Life Safety," the minimum required for all bridges. This is the case for both the MCE and the Frequent earthquake.

Table 3.2-1 defines the performance levels for service and damage the bridge is to be designed for. In this case, the choice of Life Safety as the performance level implies that for the Frequent earthquake minimal damage is expected and the structure is expected to fully open to normal traffic following an inspection of the bridge. The Life Safety choice also implies that in the MCE earthquake significant damage is expected, and the bridge will likely not be available to full traffic following an earthquake. The bridge may, in fact, be damaged to the point where it needs to be replaced following the MCE event. Displacement limits are established by the provisions to guide the designer in assessing geometrically what is implied by the specified service levels. Per the LRFD Guide Specification, displacements should be checked "to satisfy geometric, structural, and foundation constraints on performance" as outlined in Table C3.2-1 of the Guide Specification commentary.

Design Step 2.3 Spectral Acceleration Parameters

[Guide Spec, Article 3.4.1] [NCHRP, Article 3.10.2.1]

The site is on the north Merrimack River, north of Concord, New Hampshire. Using national ground motion maps, the MCE short-period (0.2 second) acceleration, S_S, is 0.46g and the 1.0-second acceleration, S_1, is 0.12g.

The spectral accelerations for the Frequent earthquake were determined by the geotechnical engineer, and likewise are based on national ground motion maps. The short-period (0.2 second) acceleration, S_S, is 0.15g and the 1.0-second acceleration, S_1, as 0.04g.

SECTION IV SDAP C ELASTOMERIC BEARING EXAMPLE
Design Step 2, Basic Requirements

Design Step 2.4

Site Class
[Guide Spec, Article 3.4.2.1] [NCHRP, Article 3.10.2.2.1]

The site class is B because the founding soil is rock. In this case, the shear wave velocity is taken as greater than 2,500 feet per second.

Design Step 2.5

Site Coefficients
[Guide Spec, Article 3.4.2.3] [NCHRP, Article 3.10.2.2.3]

Maximum Considered Earthquake (3% in 75 years)

The site coefficient for the short-period range, F_a, is 1.0 for site Class B.
The site coefficient for the long-period range, F_v, is 1.0 for site Class B.

Frequent Earthquake (50% in 75 years)

The site coefficient for the short-period range, F_a, is 1.0 for site Class B.
The site coefficient for the long-period range, F_v, is 1.0 site Class B.

Design Step 2.6

Design Earthquake Response Spectra
[Guide Spec, Article 3.4.1] [NCHRP, Article 3.10.2.1]

Not computed. See Design Step 3.

Design Step 2.7

Vertical Acceleration Effects
[Guide Spec, Article 3.4.5] [NCHRP, Article 3.10.2.6]

The bridge site is in the eastern part of the United States where vertical acceleration effects are not required to be considered in the design.

SECTION IV **SDAP C ELASTOMERIC BEARING EXAMPLE**
Design Step 3, Determine Seismic Design and Analysis Procedure

DESIGN STEP 3 — DETERMINE SEISMIC DESIGN AND ANALYSIS PROCEDURE

Design Step 3.1 — Determine Seismic Hazard Level

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3.1]

$F_aS_s = 0.46$ and $F_vS_1 = 0.12$.

The Seismic Hazard Level is III.

By Table 3.7-1, the Seismic Hazard Level is III because F_aS_s exceeds 0.35. Based on F_vS_1, the Seismic Hazard Level would only be I. The controlling value is taken to be the more restrictive of the two values.

Design Step 3.2 — Determine Seismic Design and Analysis Procedure (SDAP)

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3.1]

SDAP C will be used.

Table 3.7-2 of the Specification gives the requirements for determining what Seismic Design and Analysis Procedure (SDAP) should be used. The table suggests either B, C, D, or E can be used for the Life-Safety performance level in Seismic Hazard Level III. For this example, use SDAP C.

Design Step 3.3 — Determine Seismic Detailing Requirements (SDR)

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3.1]

SDR 3 is applicable for SDAP C.

Because the structure is classified for Life-Safety Performance and Seismic Hazard Level III, Table 3.7-2 requires SDR 3. The detailing provisions for various components of the structure will be discussed in more detail in subsequent design steps in this design example.

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

DESIGN STEP 4 DETERMINE ELASTIC SEISMIC FORCES AND DISPLACEMENTS

SDAP C Capacity Spectrum Design Method with Elastomeric Bearings
[Guide Spec 4.4, 5.4.1, and 15.4.1]
[NCHRP, Articles 3.10.3.4, 4.8.5.2, 15.4.1]

The basic relationship of the capacity spectrum method is

$$C_s \cdot \Delta = \left(\frac{F_v \cdot S_1}{2 \cdot \pi \cdot B_L}\right)^2 \cdot g$$

With elastomeric bearings there is no clear or capped value of C_s as there would be with a yielding system, so the following relationship is used

$$C_s = \frac{V}{W} = \frac{K\Delta}{W}$$

B_L is the damping coefficient. For bridges with isolation, B is substituted for B_L.

Combining equations

$$\Delta = \sqrt{\left(\frac{F_v \cdot S_1}{2 \cdot \pi \cdot B}\right)^2 \cdot g \cdot \frac{W}{K}}$$

This equation for Δ is essentially the same as Eqn 15.4.1-3b, which applies to isolation bearings. The only difference is round-off approximation for $g / (2\pi)^2$.

Check the eligibility requirements of Guide Specification 5.4.1.1.

a) Bridge must meet the regularity requirements for the Uniform Load Method of Guide Specification Table 5.4.2.1-1.

Bridge is not curved.

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

DESIGN STEP 4
(continued)

For three spans, ratio of span lengths span to span is 1.22, which is less than the maximum of 2.0.

Pier wall stiffnesses are equal.

b) Effective vibration period is less than or equal to 3 seconds.

c) Effective damping is less than or equal to 30 percent of critical.

Note that the requirement that the abutments resist no significant lateral forces is no longer present. Therefore, the abutments may provide resistance; however, such resistance should be supplied with isolation bearings at the abutments if isolation bearings are also used at the intermediate piers. This preserves the SDOF response that the capacity spectrum method is based on.

The capacity spectrum method does not require the consideration of earthquake loading in two directions, simultaneously. Thus the SRSS or 100%-40% directional combination rules are not used. For this reason the checks of the substructure against the basic relationship of the capacity spectrum method will be made independently in each direction. In the case of the skew, checks will actually be in the weak direction of the pier and then in the strong direction of the pier that is checked. A superstructure design will require that the weak and strong direction results be resolved into longitudinal and transverse forces.

For elastomeric bearings, 5% nominal damping is assumed. They are not used for seismic isolation and no energy dissipation is assumed. Therefore, B = 1.0 by Table 5.4.1.1-1 of the Guide Specification. The bearing sizes have been set to accommodate service loads and thermal effects.

The implication of the use of B = 1.0 is that operational performance may be achieved, because no energy dissipation (or damage) is presumed in the design. Such performance, of course, is only achieved if all the bridge elements are designed to resist the forces and displacements calculated from B = 1.0.

The sequence of application of the method differs from the conventional case. For non-isolated bridges, the designer sums the lateral strength of the columns to obtain C_s and then determines if the displacement capacity of the columns is adequate to satisfy the required $C_s \Delta$. If not, the columns are strengthened.

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

DESIGN STEP 4 (continued)

In an isolation design, the designer uses the stiffness characteristics of the isolation bearings to determine the design displacement. The lateral force that the substructure must resist is then calculated. K_{eff} the sum of the effective linear stiffnesses of all bearings and substructures supporting the superstructure. The stiffnesses will be oriented normal and parallel to the skew of the substructure. By using the stiffness of the structure in the expression for C_s the designer is effectively solving for the correct response.

Because an effective stiffness is used in the calculations of displacements and forces, SDAP C as applied to isolation systems is virtually identical to the analysis of the SDAP D uniform load method. In essence, an elastic analysis is being performed in the Capacity Spectrum Method when the isolation-specific equations are used.

An important difference between SDAP C and D is that in SDAP D the directional combinations are used.

K_{eff} & W, weight of superstructure, will be calculated in Design Steps 4.1 and 4.2 and Δ required is solved for in Design Step 4.3.

Design Step 4.1 **Determine the System Stiffness, K**

In this example, longitudinal and transverse refer to weak and strong directions on the piers.

Design Step 4.1.1 Horizontal Translational Stiffness of Pier Bearings, K_{pier_brg}

The stiffness of an individual bearing pad can be calculated by determining the shear force required to produce a unit deflection on the pad. See Figure 8.

The translational stiffness will be the same in both principal directions since the pads are square; and therefore the stiffness will be the same in all directions.

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.1.1 (continued)

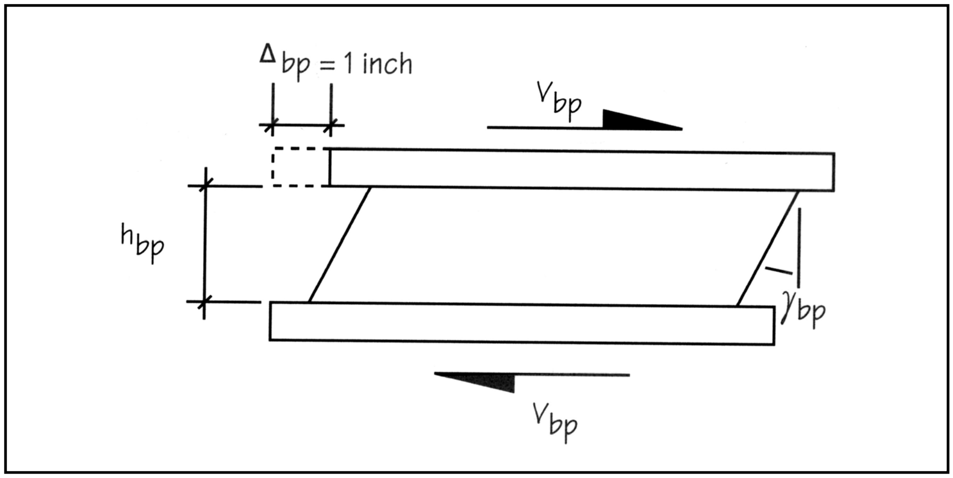


Figure 8 — Translational Deflection of Bearing Pad

Assume:

$\Delta_{bp} := 1.0 \cdot in$ Unit deflection of bearing pad

Given:

$G_b := 115 \cdot psi$ Shear modulus of elastomer

$A_{bp} := (21 \cdot in)^2$ Area of each pier bearing pad

$T_{bp} := 1.125 \cdot in$ Height of elastomer in pier bearing pads

Calculate the shear strain for a unit deflection:

$\gamma_{bp} := \dfrac{\Delta_{bp}}{T_{bp}}$ Shear strain in pad; note that the steel reinforcing plate thickness is not included.

Calculate the shear stress:

$v_{bp} := G_b \cdot \gamma_{bp}$ Shear stress in bearing pad

Calculate the shear force:

$V_{bp} := v_{bp} \cdot A_{bp}$ Shear force acting across bearing pad

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

$$k_{trans} := \frac{V_{bp}}{\Delta_{bp}}$$ Translational stiffness of pier bearing pads

$$k_{trans} = 541 \frac{kip}{ft}$$

Then the total translational stiffness for all eight bearing pads is given by:

$$K_{pier\_brg} := 8 \cdot k_{trans}$$

$$K_{pier\_brg} = 4328 \frac{kip}{ft}$$

Design Step 4.1.2

Horizontal Translational Stiffness of Abutment Bearings, K_{abut_brg}

Assume:

$\Delta_{bp} := 1.0 \cdot in$ Unit deflection of bearing pad

Given:

$G := 115 \cdot psi$ Shear modulus of elastomer

$A_{bp} := (14 \cdot in)^2$ Area of each pier bearing pad

$T_{bp} := 2.625 \cdot in$ Height of elastomer in pier bearing pads

Calculate the shear strain for a unit deflection:

$$\gamma_{bp} := \frac{\Delta_{bp}}{T_{bp}}$$ Shear strain in pad; note that the steel reinforcing plate thickness is not included.

Calculate the shear stress:

$v_{bp} := G \cdot \gamma_{bp}$ Shear stress in bearing pad

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.1.2 (continued)

Calculate the shear force:

$V_{bp} := v_{bp} \cdot A_{bp}$ Shear force acting across bearing pad

Calculate the translational stiffness:

$k_{trans} := \frac{V_{bp}}{\Delta_{bp}}$ Translational stiffness of pier bearing pads

$$k_{trans} = 103 \frac{kip}{ft}$$

Then the total translational stiffness for all eight bearing pads is given by:

$K_{abut\_brg} := 8 \cdot k_{trans}$

$$K_{abut\_brg} = 824 \frac{kip}{ft}$$

The stiffness of the bearings is the same in both principal directions. Thus the stiffness is the same in all directions in a horizontal plane.

Design Step 4.1.3

Pier Stiffness – Weak Axis Bending, $K_{pier\_wall\_weak}$

The pier stiffness is calculated in the weak direction by approximating the wall as a cantilever of uniform thickness and width.

The assumed thickness is 5.5 feet and the assumed width is 60 feet. The following logic was used to obtain these values: The thickness varies between 4 and 6 feet and the curvature of the cantilever will probably be the highest in the lower half, so weight the lower half properties to a greater extent; use 5.5 feet. The width of the lower wall varies between 54 and 66 feet; 60 feet is the average of these.

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

$$I_{wall} := \frac{60 \cdot ft \cdot (5.5 \cdot ft)^3}{12} \qquad I_{wall} = 832\, ft^4$$

$$E := 519000 \cdot \frac{kip}{ft^2}$$

$$h_{wall} := 36 \cdot ft$$

$$K_{pier\_wall\_weak} := \frac{3 \cdot E \cdot I_{wall}}{h_{wall}^3} \qquad K_{pier\_wall\_weak} = 27761 \frac{kip}{ft}$$

Design Step 4.1.4

Pier Stiffness – Strong Axis Bending, $K_{pier\_wall\_strong}$

The pier stiffness is so large in the strong direction that $K_{pier\_wall\_strong}$ is taken as infinite,

say $K_{pier\_wall\_strong} := 1000000 \cdot \frac{kip}{ft}$

Design Step 4.1.5

Abutment Stiffness, $K_{abutment}$

The abutment stiffness is so large in both directions that $K_{abutment}$ is taken as infinite,

say $K_{abutment} := 1000000 \cdot \frac{kip}{ft}$

Design Step 4.1.6

Longitudinal System Stiffness, K_{total_Long}

Recall that longitudinal is synonymous with pier weak direction for this calculation.

Recall the pier bearing stiffness

$$K_{pier\_brg} = 4328 \frac{kip}{ft}$$

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.1.6 (continued)

Combine the pier stiffness and pier bearing stiffness in series

$$K_{pier\_Long} := \frac{1}{\left(\frac{1}{K_{pier\_brg}} + \frac{1}{K_{pier\_wall\_weak}}\right)}$$

$$K_{pier\_Long} = 3744 \frac{kip}{ft}$$

Recall the abutment bearing stiffness

$$K_{abut\_brg} := 824 \cdot \frac{kip}{ft}$$

Combine the abutment stiffness and abutment bearing stiffness in series

$$K_{abut\_Long} := \frac{1}{\left(\frac{1}{K_{abut\_brg}} + \frac{1}{K_{abutment}}\right)}$$

$$K_{abut\_Long} = 823 \frac{kip}{ft}$$

Total longitudinal stiffness for two piers and two abutments

$$K_{total\_Long} := 2 \cdot K_{pier\_Long} + 2 \cdot K_{abut\_Long}$$

$$K_{total\_Long} = 9135 \frac{kip}{ft}$$

Effective System Stiffness in weak direction of pier wall

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.1.7

Transverse System Stiffness, K_{total_Trans}.

Recall that transverse is synonymous with pier strong direction in this calculation.

Recall

$$K_{pier\_brg} = 4328 \frac{kip}{ft}$$

$$K_{abut\_brg} = 824 \frac{kip}{ft}$$

by inspection

$$K_{pier\_Trans} := K_{pier\_brg}$$

$$K_{abut\_Trans} := K_{abut\_brg}$$

$$K_{total\_Trans} := 2 \cdot K_{pier\_Trans} + 2 \cdot K_{abut\_Trans}$$

$$K_{total\_Trans} = 10303 \frac{kip}{ft}$$

Effective System Stiffness in strong direction of pier wall

Design Step 4.2

Total Superstructure Weight

Recall from Section III that

$$W_{super} := 5540 \cdot kips$$

Design Step 4.3

Calculate Design Displacements of the Superstructure

$$F_v := 1.0$$

$$S_1 := 0.12$$

MCE controls the design.

$$g = 32.17 \frac{ft}{sec^2}$$

SECTION IV SDAP C WITH ELASTOMERIC BEARING EXAMPLE

Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.3 (continued)

$$\Delta_{design\_Long} := \sqrt{\left(\frac{F_v \cdot S_1 \cdot sec}{2 \cdot \pi}\right)^2 \cdot g \cdot \frac{W_{super}}{K_{total\_Long}}}$$

$$\Delta_{design\_Long} = 1.01 \text{ in}$$

Δ_{design_Long} is the sum of $\Delta_{pier\_brg\_Long}$ and Δ_{pier_Long} at the pier

$$\Delta_{pier\_brg\_Long} := \frac{K_{pier\_Long} \cdot \Delta_{design\_Long}}{K_{pier\_brg}}$$

$$\Delta_{pier\_brg\_Long} = 0.88 \text{ in}$$

$$\Delta_{pier\_Long} := \frac{K_{pier\_Long} \cdot \Delta_{design\_Long}}{K_{pier\_wall\_weak}}$$

$$\Delta_{pier\_Long} = 0.14 \text{ in}$$

At the piers, the bearings deflect 0.88 inches and the pier wall deflects 0.14 inches in the weak direction, for a total of 1.02 inches.

$$\Delta_{design\_Trans} := \sqrt{\left(\frac{F_v \cdot S_1 \cdot sec}{2 \cdot \pi}\right)^2 \cdot g \cdot \frac{W_{super}}{K_{total\_Trans}}}$$

$$\Delta_{design\_Trans} = 0.95 \text{ in}$$

$$\Delta_{pier\_brg\_Trans} := \Delta_{design\_Trans}$$

$$\Delta_{pier\_brg\_Trans} = 0.95 \text{ in}$$

At the piers, the bearings deflect 0.95 inches in the strong direction. The pier wall does not significantly deflect.

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 4, Determine Elastic Seismic Forces and Displacements

Design Step 4.4

Calculate Lateral Force Coefficient

$$C_{s\_reqd\_Long} := \frac{K_{total\_Long} \cdot \Delta_{design\_Long}}{W_{super}} \qquad C_{s\_reqd\_Long} = 0.14$$

$$C_{s\_reqd\_Trans} := \frac{K_{total\_Trans} \cdot \Delta_{design\_Trans}}{W_{super}} \qquad C_{s\_reqd\_Trans} = 0.15$$

At this point, Design Step 4, the SDAP C Capacity Spectrum Method, has incorporated Design Steps 5 and 6 as outlined in the flowcharts.

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 7, Design Displacements and Checks

DESIGN STEP 7 DESIGN DISPLACEMENTS AND CHECKS

[Guide Spec 15.11.2] [NCHRP, Article 15.11.2]

Design Step 7.1 Displacement Capacity of the Elastomeric Bearings at Piers

The Seismic Isolation provisions of Guide Specification Chapter 15 provide procedures for assessing the maximum permissible earthquake induced displacements that elastomeric bearings can carry.

Elastomeric bearings shall satisfy

$$\gamma_c < 2.5$$

and

$$\gamma_c + \gamma_{s\_eq} + \gamma_r < 5.5$$

where γ_c is the shear strain due to compression by vertical loads

γ_{s_eq} is the shear strain due to earthquake imposed lateral loads

γ_r is the shear strain due to rotation

Shear strain due to compression by vertical loads at piers

$T_{bp} := 1.125 \cdot in$ Height of elastomeric pad

$L_{bp} := 21 \cdot in$ Length of bearing pad

$W_{bp} := 21 \cdot in$ Width of bearing pad

Estimate the vertical load on the bearing:

$$P_{pier} := W_{super} \cdot \frac{\frac{124 \cdot ft \cdot 5}{8} + \frac{152 \cdot ft}{2}}{400 \cdot ft}$$

$$P_{pier} = 2126\ kip$$

$$P_{brg} := \frac{P_{pier}}{8} \qquad P_{brg} = 266\ kip$$

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 7, Design Displacements and Checks

Design Step 7.1 (continued)

Calculate the shape factor S

$$S := \frac{L_{bp} \cdot W_{bp}}{T_{bp} \cdot \left(L_{bp} + W_{bp}\right)} \qquad S = 9.3$$

Estimate the Area

$$A_r := L_{bp} \cdot W_{bp}$$

$k_{bar} := 50$ hardness constant for bearings

$$G := 115 \cdot psi$$

$$\gamma_c := \frac{3 \cdot S \cdot P_{brg}}{2 \cdot A_r \cdot G \cdot \left(1 + 2 \cdot k_{bar} \cdot S^2\right)}$$

$$\gamma_c = 0.008$$

This formulation is different than what was used in the original design example No. 2 where γ_c was found to be 0.025.

Shear strain due to rotation will be neglected.

Therefore, maximum shear strain due to earthquake imposed lateral displacement at piers is

$$\gamma_{s\_eq} := 5.5 - \gamma_c \qquad \gamma_{s\_eq} = 5.49$$

Maximum Pier Bearing Displacement due to earthquake is

$$\Delta_{s\_eq} := \gamma_{s\_eq} \cdot T_{bp} \qquad \Delta_{s\_eq} = 6.18\,in$$

maximum allowable bearing lateral displacement

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 7, Design Displacements and Checks

Design Step 7.2 **Compare Design Displacement to Displacement Capacity of Bearing**

$\Delta_{pier\_brg\_Long} = 0.88\,in$ less than $\Delta_{s\_eq} = 6.18\,in$ OK

$\Delta_{pier\_brg\_Trans} = 0.95\,in$ less than $\Delta_{s\_eq} = 6.18\,in$ OK

Design Step 7.3 **Check P-Δ in Longitudinal/Weak Direction**

$h_{wall} = 36.00\,ft$

$C_{s\_reqd\_Long} \cdot h_{wall} \cdot 0.25 = 15.02\,in$

$\Delta_{design\_Long} = 1.01\,in$ < 15 in OK

Design Step 7.4 **Displacement Capacity of the Elastomeric Bearings at Abutment and Check**

Maximum Abutment Bearing Displacement due to earthquake is

$T_{bp} := 2.625 \cdot in$

$L_{bp} := 14 \cdot in$

$W_{bp} := 14 \cdot in$

$$P_{abut} := W_{super} \cdot \frac{\frac{3 \cdot 124 \cdot ft}{8}}{400 \cdot ft} \qquad P_{abut} = 644\,kip$$

$$P_{brg} := \frac{P_{abut}}{8} \qquad P_{brg} = 81\,kip$$

$$S := \frac{L_{bp} \cdot W_{bp}}{T_{bp} \cdot \left(L_{bp} + W_{bp}\right)} \qquad S = 2.67$$

$A_r := L_{bp} \cdot W_{bp}$

$k_{bar} := 50$ hardness constant for bearings

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 7, Design Displacements and Checks

Design Step 7.4 (continued)

$G := 115 \cdot psi$

$$\gamma_c := \frac{3 \cdot S \cdot P_{brg}}{2 \cdot A_r \cdot G \cdot \left(1 + 2 \cdot k_{bar} \cdot S^2\right)}$$

$\gamma_c = 0.020$

Therefore, maximum shear strain due to earthquake imposed lateral displacement at abutment is

$\gamma_{s\_eq} := 5.5 - \gamma_c$ $\gamma_{s\_eq} = 5.48$

Maximum Abutment Bearing Displacement due to earthquake is

$\Delta_{s\_eq} := \gamma_{s\_eq} \cdot T_{bp}$ $\Delta_{s\_eq} = 14.38\,in$ maximum allowable bearing lateral displacement

Compare Design Displacement to Displacement Capacity of Bearing

$\Delta_{design\_Long} = 1.01\,in$ less than $\Delta_{s\_eq} = 14.38\,in$ OK

$\Delta_{design\_Trans} = 0.95\,in$ less than $\Delta_{s\_eq} = 14.38\,in$ OK

Design Step 7.5 **Minimum Seat Width Requirement**
[Guide Spec 7.3.2] [NCHRP, Article 3.10.3.10]

The check of minimum seat width at the abutments is identical to that illustrated in Design Step 7.1 of Section III.

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 8, Design Structural Components

DESIGN STEP 8 — DESIGN STRUCTURAL COMPONENTS

Design Step 8.1 — Calculate Lateral Force that Pier must Resist

$$V_{u\_Long} := C_{s\_reqd\_Long} \cdot W_{super} \cdot \frac{K_{pier\_Long}}{K_{total\_Long}} \qquad V_{u\_Long} = 316\,\text{kip}$$

$$V_{u\_Trans} := C_{s\_reqd\_Trans} \cdot W_{super} \cdot \frac{K_{pier\_Trans}}{K_{total\_Trans}} \qquad V_{u\_Trans} = 344\,\text{kip}$$

These are elastic seismic forces.

Design Step 8.2 — Check Pier Lateral Capacity

[Guide Spec 7.8.3] [NCHRP, Article 5.10.11.4.2]

The pier wall has been detailed with ρ_v and ρ_h equal to 0.0025. See Figure 9 for wall reinforcement and details. The nominal moment capacities for the wall are given below.

$W_{pier\_wall} := 1695 \cdot \text{kip}$ calculated from volume of concrete

$P_{dl} := W_{pier\_wall} + P_{pier}$ $\qquad P_{dl} = 3821\,\text{kip}$

$M_{n\_Long} := 33400 \cdot \text{kip} \cdot \text{ft}$ See Figure 10 for PCACOL diagram.

$M_{n\_Trans} := 394000 \cdot \text{kip} \cdot \text{ft}$ See Figure 11 for PCACOL diagram.

$$\phi := 0.90 - \frac{1.5 \cdot 3812 \cdot \text{kip}}{4.0 \cdot \text{ksi} \cdot 66 \cdot \text{ft} \cdot 6 \cdot \text{ft}} \qquad \phi = 0.87$$

This value of ϕ is based on the transition described in Section 5.5.4.2.1 of the LRFD Bridge Design Specification.

$H_{pier} := 36 \cdot \text{ft}$

SECTION IV SDAP C WITH ELASTOMERIC BEARING EXAMPLE
Design Step 8, Design Structural Components

Design Step 8.2
(continued)

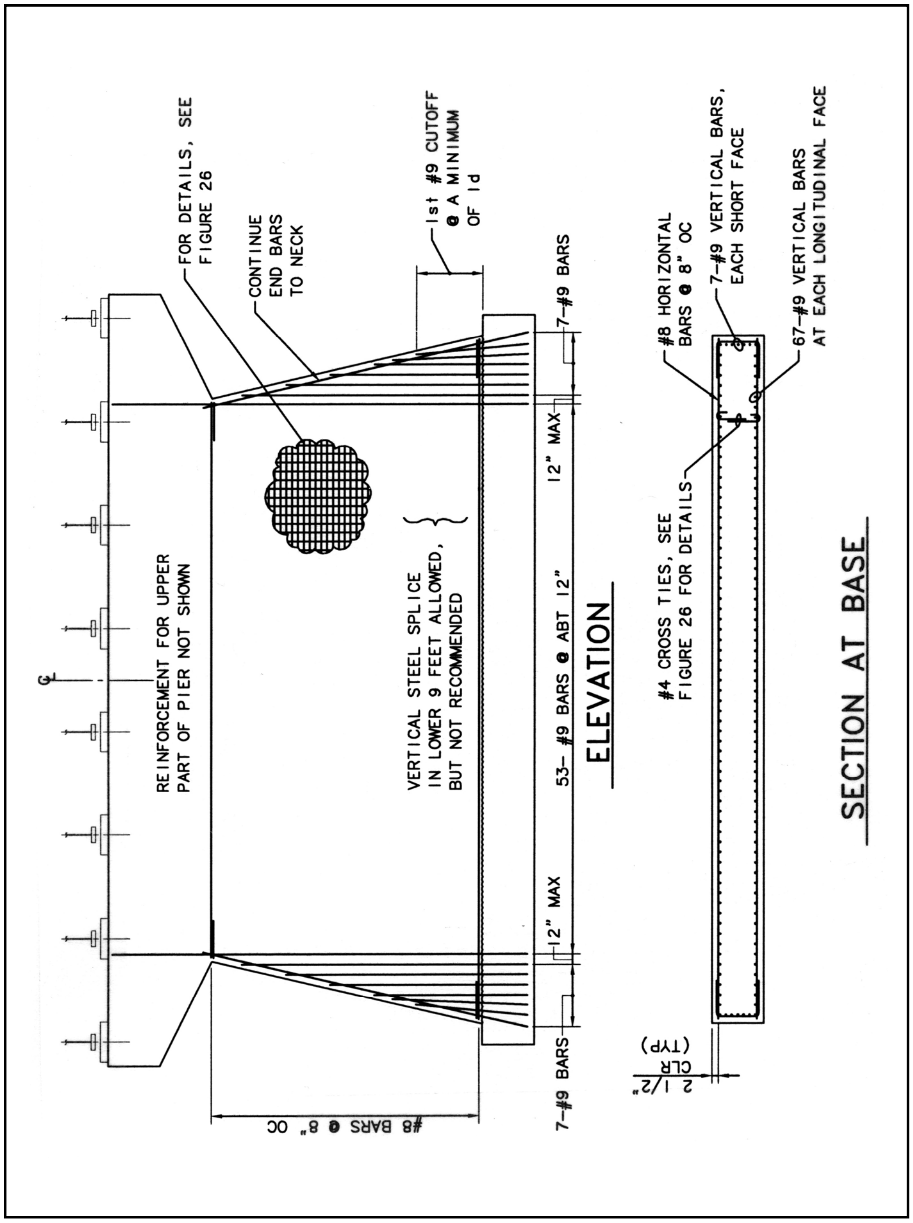


Figure 9 — Reinforcement in Lower Part of Pier Wall

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 8, Design Structural Components

Design Step 8.2
(continued)

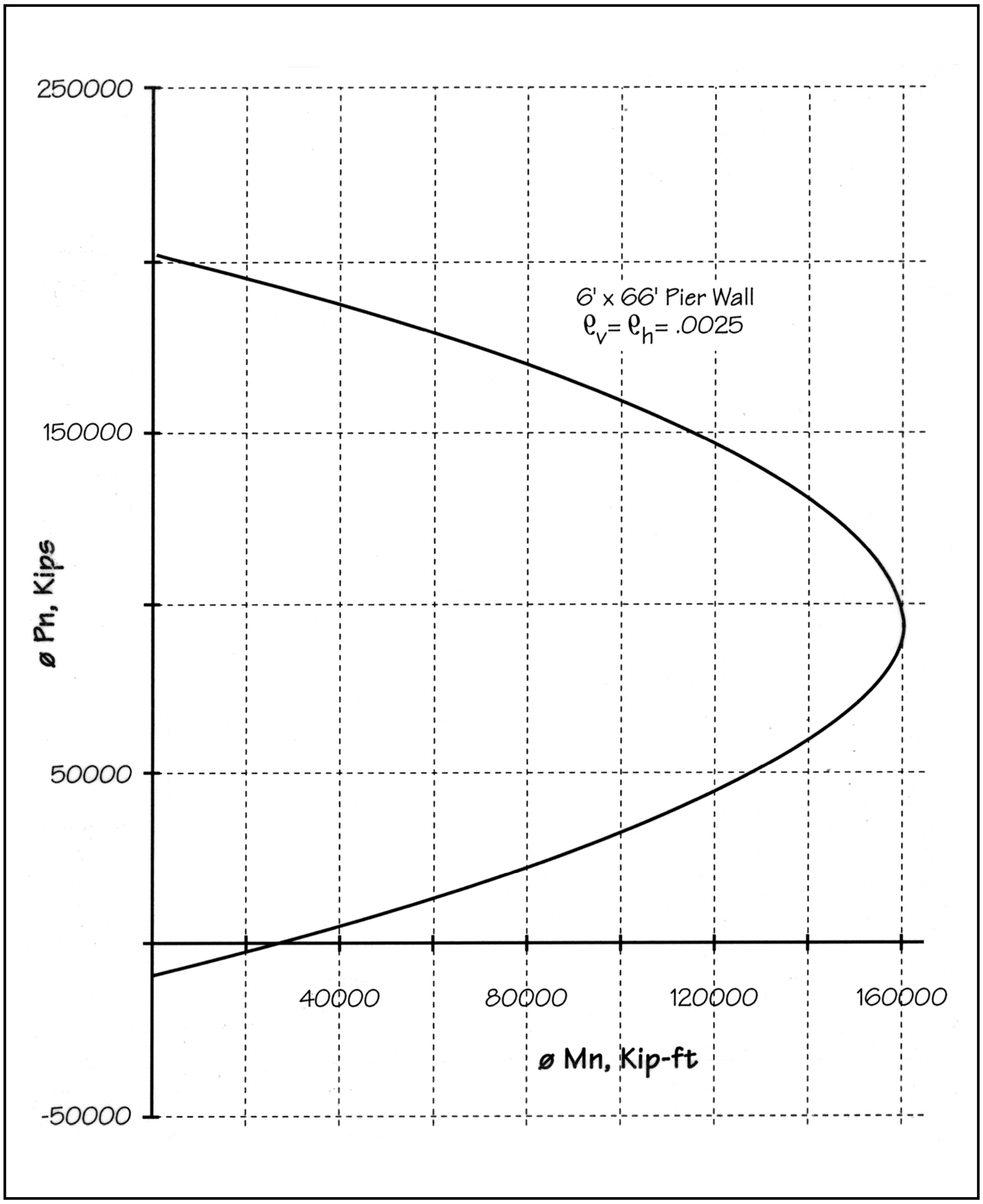


Figure 10 — Pier Interaction Capacity Curve Weak Direction

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 8, Design Structural Components

Design Step 8.2
(continued)

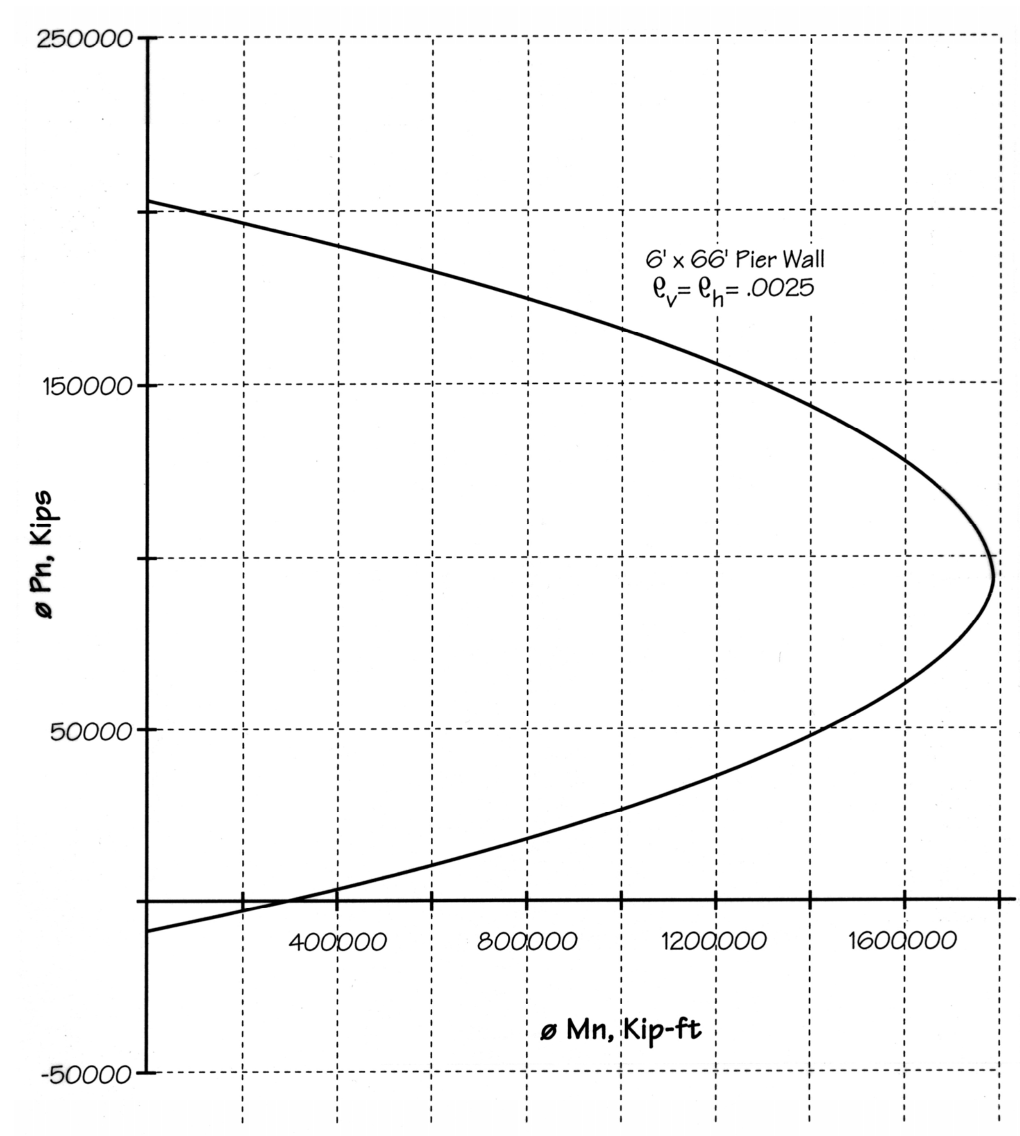


Figure 11 — Pier Interaction Capacity Curve Strong Direction

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 8, Design Structural Components

Design Step 8.2 (continued)

Calculate elastic moments and check against capacities.

$$M_{u\_Long} := \frac{V_{u\_Long} \cdot H_{pier}}{\phi} \qquad M_{u\_Long} = 12997 \text{ ft kip} \qquad \text{OK}$$

$$M_{u\_Trans} := \frac{V_{u\_Trans} \cdot H_{pier}}{\phi} \qquad M_{u\_Trans} = 14145 \text{ ft kip} \qquad \text{OK}$$

$b := 4 \cdot ft$ minimum width of wall

$d := 54 \cdot ft$ minimum length of wall

Note that the following equations from the provisions have been converted from metric into U.S. customary units.

$V_r = 3 \ \text{sqrt}\,(f'_c)\ b\ d$

$V_{r1} := 0.190 \cdot ksi \cdot b \cdot d \qquad V_{r1} = 5910 \text{ kip}$

$V_r = 0.9(0.7 \ \text{sqrt}\,(f'_c) + \rho_h \cdot f_y)\ b\ d$

$V_{r2} := 0.175 \cdot ksi \cdot b \cdot d \qquad V_{r2} = 5443 \text{ kip}$

$V_u < V_r$ OK

Minimum wall ties per Guide Specification Section 7.8.1 will be used. There are no seismic provisions for wall ties, except when the wall is considered a column. This wall is not so considered.

Design Step 8.3

Calculate Connection Force at Each Bearing
[Guide Spec 15.8] [NCHRP, Article 15.8]

The seismic design force for the connection between the superstructure and the substructure is given by

$F_a := K_{eff} \cdot \Delta_t$

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 8, Design Structural Components

Design Step 8.3 (continued)

Calculate design force

$K_{eff\_L} := K_{total\_Long}$ $K_{eff\_L} = 9135 \dfrac{kip}{ft}$

$K_{eff\_T} := K_{total\_Trans}$ $K_{eff\_T} = 10303 \dfrac{kip}{ft}$

$\Delta_{t\_L} := \Delta_{design\_Long}$ $\Delta_{t\_L} = 1.01 \text{ in}$

$\Delta_{t\_T} := \Delta_{design\_Trans}$ $\Delta_{t\_T} = 0.95 \text{ in}$

$F_{a\_L} := K_{eff\_L} \cdot \Delta_{t\_L}$ $F_{a\_L} = 771 \text{ kip}$

$F_{a\_T} := K_{eff\_T} \cdot \Delta_{t\_T}$ $F_{a\_T} = 818 \text{ kip}$

$F_{conn\_L} := \dfrac{F_{a\_L}}{8}$ $F_{conn\_L} = 96 \text{ kip}$

$F_{conn\_T} := \dfrac{F_{a\_T}}{8}$ $F_{conn\_T} = 102 \text{ kip}$

The connection force to be designed for will be 102 kips.

Note that a structural load path must be provided through the cross frames (diaphragms) for the bearing forces calculated above.

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 9, Design Foundations

DESIGN STEP 9 DESIGN FOUNDATIONS

[Guide Spec 15.8] [NCHRP, Article 15.8]

If the elastic foundation forces are less than the forces resulting from plastic hinging, they may be used for the foundation design with an R equal to 1.0. From Design Step 8.2, this is seen to be the case.

Design Step 9.1 Calculate the Foundation Forces for Overturning, Sliding, and Soil Bearing Capacity in the Weak Direction

$P_{dl} = 3821\,\text{kip}$ at top of footing

$M_{OT} := M_{u\_Long}$ $\qquad M_{OT} = 12635\,\text{ft kip}$

$V_{OT} := \dfrac{M_{OT}}{H_{pier}}$ $\qquad V_{OT} = 351\,\text{kip}$

$D_f := 5\cdot\text{ft}$

$M_v := V_{OT}\cdot D_f$ $\qquad M_v = 1755\,\text{ft kip}$

The dead load forces must be augmented to account for footing weight, buoyancy, and overburden effects. The shear forces and moments, however, do not require adjustment.

Based on the foundation configuration shown in Figure 12, calculate the additional axial force acting at the base of the foundation due to the stone fill overburden. Recall that the length of the footing is 70 feet, and assume that stone fill with a saturated unit weight of 0.130 kip per cubic foot is used.

Overburden weight:

$$P_{sf} := (16\cdot\text{ft} - 6\cdot\text{ft})\cdot(3\cdot\text{ft})\cdot(70\cdot\text{ft})\cdot\left(.130\cdot\frac{\text{kip}}{\text{ft}^3}\right)$$

$$P_{sf} = 273\,\text{kip}$$

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 9, Design Foundations

Design Step 9.1 (continued)

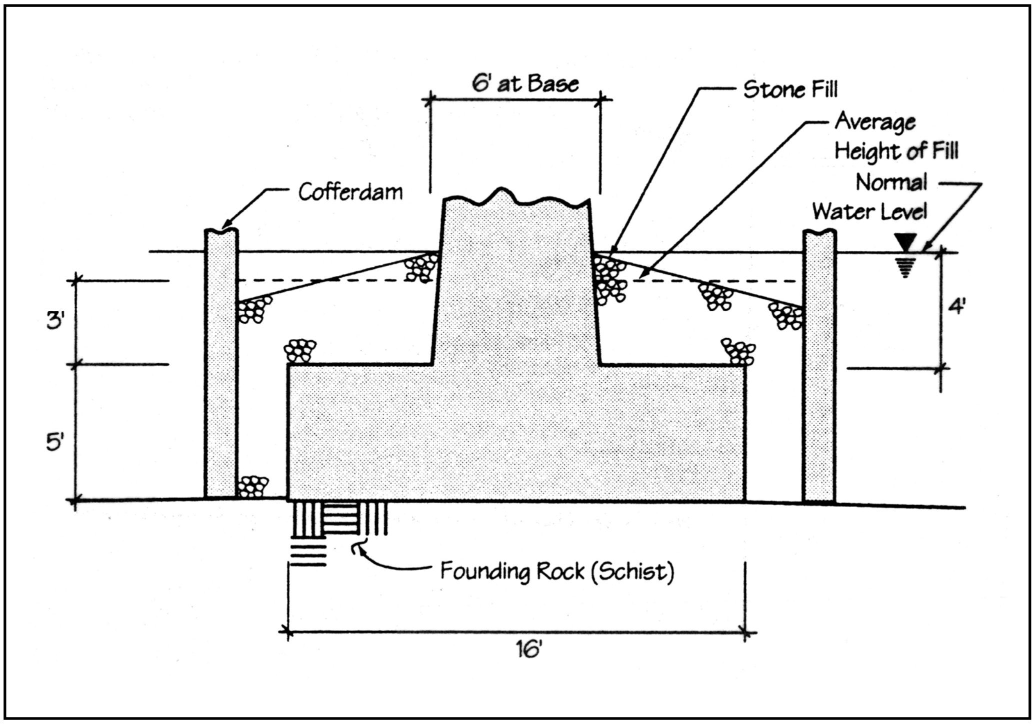


Figure 12 — Reinforcement in Lower Part of Pier Wall

Calculate the uplift force due to buoyancy assuming the water level corresponds to the normal level, 4 feet above the top of the footing. Per the commentary of Guide Specification 3.5, mean discharge levels may be used for the Extreme Event 1 load combination.

Buoyancy force:

$V_{ftg} := (16 \cdot ft) \cdot (5 \cdot ft) \cdot (70 \cdot ft)$ Volume of footing

$V_{sf} := (16 \cdot ft - 6 \cdot ft) \cdot (3 \cdot ft) \cdot (70 \cdot ft)$ Volume of stone fill

$V_{stem} := (6 \cdot ft) \cdot (4 \cdot ft) \cdot (66 \cdot ft)$ Volume of wall stem

$$P_b := \left(V_{ftg} + V_{sf} + V_{stem}\right) \cdot 0.0624 \cdot \frac{kip}{ft^3}$$

$$P_b = 579\ kip$$

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 9, Design Foundations

Design Step 9.1 (continued)

Weight of footing:

$$W_{ftg} := V_{ftg} \cdot .15 \cdot \frac{kip}{ft^3} \qquad W_{ftg} = 840\,kip$$

Axial Force:

Adjusted axial force acting at base of foundation

$$P := P_{dl} + W_{ftg} + P_{sf} - P_b$$

$$P = 4355\,kip$$

Design Moment and Shear Forces.

Calculate the design moment to be used for the overturning check.

$$M_{weak} := M_{OT} + M_v$$ Weak direction driving moment

$$M_{weak} = 14389\,ft\,kip$$

Calculate the design shear forces to be used in the sliding check.

$$V_{weak} := \frac{M_{OT}}{H_{pier}}$$

$$V_{weak} = 351\,kip$$ Shear in weak direction.

Design Step 9.2

Check Foundation for Overturning in the Weak Direction

Per Guide Specification 7.4.2.1, footing lift off shall not exceed 50 percent at the peak displacement.

To ensure that there is no more than one-half uplift on the footing, the eccentricity e must be less than $\frac{L_f}{3}$.

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 9, Design Foundations

Design Step 9.2 (continued)

The preliminary length of the footing in the longitudinal direction is:

$$L_f := 16 \cdot ft$$

The overturning induced eccentricity must be less than or equal to:

$$\frac{L_f}{3} = 5.33\,ft$$

The eccentricity of the axial load caused by the overturning moment can be calculated by:

$$e := \frac{M_{weak}}{P} \qquad e = 3.3\,ft$$

There is no uplift.

Design Step 9.3

Check the Soil Bearing Capacity in the Weak Direction

$B_f := 70 \cdot ft$ Width of footing

$$q := \left(\frac{1}{B_f \cdot L_f} + \frac{6 \cdot e}{B_f \cdot L_f^{\,2}}\right) \cdot P$$ Maximum contact stress at edge of footing

$q = 8.7\,ksf$ OK

By inspection, q is much less than the ultimate bearing capacity of 50 ksf. Thus the footing width is adequate.

SECTION IV SDAP C WITH ELASTOMERIC BEARING EXAMPLE
Design Step 9, Design Foundations

Design Step 9.4

Check Foundation for Sliding in the Weak Direction

The check of sliding is made by comparing the ultimate sliding resistance with the driving force.

For this footing founded on a competent rock, the coefficient of friction may be taken as 0.8.

$$V_r := 0.8 \cdot P$$

$$V_r = 3484\,\text{kip}$$

The driving force is:

$$V_{OT} = 351\,\text{kip}$$

Because the resistance is larger than the driving force, the footing is adequate for sliding.

Design Step 9.5

Calculate the Foundation Forces for Design of Footing Reinforcement in the Weak Direction

An overstrength factor of 1.5 has been added for this calculation although it is not required by the provisions:

$$OS := 1.5$$

$$P_{dl} = 3820.97\,\text{kip}$$

$$M_{po} := OS \cdot M_{u\_Long} \qquad M_{po} = 18952\,\text{ft kip}$$

$$V_{po} := \frac{M_{po}}{H_{pier}} \qquad V_{po} = 526\,\text{kip}$$

$$D_f := 5 \cdot \text{ft}$$

$$M_v := V_{po} \cdot D_f \qquad M_v = 2632\,\text{ft kip}$$

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 9, Design Foundations

Design Step 9.5 (continued)

Recall adjusted axial force acting at base of foundation

$P = 4355\,\text{kip}$

Design Moment and Shear Forces.

$M_{weak} := M_{po} + M_v$ Weak direction driving moment

$M_{weak} = 21584\,\text{ft kip}$

$$V_{weak} := \frac{M_{po}}{H_{pier}}$$

$V_{weak} = 526\,\text{kip}$ Shear in weak direction.

Recall the length of the footing in the longitudinal direction is:

$L_f = 16\,\text{ft}$

The eccentricity of the axial load caused by the overturning moment can be calculated by:

$$e := \frac{M_{weak}}{P} \qquad e = 4.96\,\text{ft}$$

Check the soil bearing capacity.

The contact stress can be calculated using the following method because the eccentricity is greater than one-sixth of the footing length. The equation can be derived assuming a triangular stress distribution.

$B_f := 70\cdot\text{ft}$ Width of footing

$$q := \frac{2\cdot P}{3\cdot B_f\cdot\left(\frac{L_f}{2} - e\right)}$$ Maximum contact stress at edge of footing

$q = 13.6\,\text{ksf}$

By inspection, q is much less than the ultimate bearing capacity of 50 ksf. Thus the footing width is adequate.

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 9, Design Foundations

Using the soil diagram, the designer can now design the footing for flexure and shear.

For top reinforcement, the weight of soil above the footing during uplift must be included.

The final footing length for the longitudinal direction will be 16 feet as in the preliminary nonseismic design.

Design Step 9.6 Calculate the Foundation Forces for Overturning, Sliding, and Soil Bearing Capacity in the Strong Direction

$P_{dl} = 3821\ \text{kip}$ at top of footing

$M_{OT} := M_{u\_Trans}$ $M_{OT} = 13751\ \text{ft kip}$

$V_{OT} := \dfrac{M_{OT}}{H_{pier}}$ $V_{OT} = 382\ \text{kip}$

$D_f := 5 \cdot \text{ft}$

$M_v := V_{OT} \cdot D_f$ $M_v = 1910\ \text{ft kip}$

Recall adjusted axial force acting at base of foundation

$P = 4355\ \text{kip}$

Design Moment and Shear Forces.

Calculate the design moment to be used for the overturning check.

$M_{strong} := M_{OT} + M_v$ Strong direction driving moment

$M_{strong} = 15661\ \text{ft kip}$

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 9, Design Foundations

Calculate the design shear forces to be used in the sliding check.

$$V_{strong} := \frac{M_{OT}}{H_{pier}}$$

$V_{strong} = 382\,\text{kip}$ Shear in weak direction.

Design Step 9.7 **Check Foundation for Overturning in the Strong Direction**

$$L_f := 70 \cdot ft$$

The eccentricity of the axial load caused by the overturning moment can be calculated by:

$$e := \frac{M_{strong}}{P} \qquad e = 3.6\,ft$$

$\frac{L_f}{6} = 11.67\,ft$ therefore no uplift

Design Step 9.8 **Check the Soil Bearing Capacity in the Strong Direction**

$B_f := 16 \cdot ft$ Width of footing

$$q := \left(\frac{1}{B_f \cdot L_f} + \frac{6 \cdot e}{B_f \cdot L_f^{\,2}}\right) \cdot P$$

Maximum contact stress at edge of footing

$q = 5.1\,\text{ksf}$ OK

Design of footing reinforcement is not shown.

The final footing length for the strong direction will be 70 feet as in the preliminary design.

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 10, Design Abutments

DESIGN STEP 10 DESIGN ABUTMENTS

The design of the abutments is not included in this design example.

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 11, Consider Liquefaction

DESIGN STEP 11 CONSIDER LIQUEFACTION

There is no liquefaction potential.

SECTION IV **SDAP C WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 12, Seismic Design Complete

DESIGN STEP 12 SEISMIC DESIGN COMPLETE?

The bearing connection design force is 102 kips, which is reasonable, and the foundation size remains at 16 feet by 70 feet. The design is satisfactory.

The designer is reminded that an appropriate structural load path is required between each bearing and its supporting structural element, and from each supporting structural element to the source of the inertial seismic load.

SECTION V **SDAP A2 CONVENTIONAL BEARING EXAMPLE**
Design Step 1, Preliminary Design

DESIGN STEP 1 PRELIMINARY DESIGN

The bridge is essentially the same as the one used in Section III. The intermediate substructure is multicolumn construction. However, the site for this section now has lower spectral acceleration levels to illustrate the application of SDAP A2.

SECTION V **SDAP A2 CONVENTIONAL BEARING EXAMPLE**
Design Step 2, Basic Requirements

DESIGN STEP 2 BASIC REQUIREMENTS

Basic requirements are the same as in Section III except for Design Step 2.3.

Design Step 2.3 Spectral Acceleration Parameters

[Guide Spec, Article 3.4.1] [NCHRP, Article 3.10.2.1]

The site is on the southern Merrimack River, in northeast Massachusetts. Using national ground motion maps, the MCE short-period (0.2 second) acceleration, S_S, is 0.34g and the 1.0-second acceleration, S_1, is 0.094g.

The spectral accelerations for the Frequent earthquake were determined by the geotechnical engineer, and likewise are based on national ground motion maps. The short-period (0.2 second) acceleration, S_S, is 0.12g and the 1.0-second acceleration, S_1, as 0.03g.

SECTION V **SDAP A2 CONVENTIONAL BEARING EXAMPLE**
Design Step 3, Determine Seismic Design and Analysis Procedure

DESIGN STEP 3 DETERMINE SEISMIC DESIGN AND ANALYSIS PROCEDURE

Design Step 3.1 Determine Seismic Hazard Level

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3.1]

$F_aS_s = 0.34$ and $F_vS_1 = 0.094$.

The Seismic Hazard Level is II.

By Table 3.7-1, the Seismic Hazard Level is II because F_aS_s is less than 0.35. Based on F_vS_1, the Seismic Hazard Level would only be I. The controlling value is taken to be the more restrictive of the two values.

Design Step 3.2 Determine Seismic Design and Analysis Procedure (SDAP)

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3.1]

We will use SDAP A2.

Table 3.7-2 of the Specification gives the requirements for determining what Seismic Design and Analysis Procedure (SDAP) should be used. A2 is to be used for the Life-Safety performance level in Seismic Hazard Level II.

Design Step 3.3 Determine Seismic Detailing Requirements (SDR)

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3.1]

SDR 2 is applicable for SDAP A2.

Because the structure is classified for Life-Safety Performance and Seismic Hazard Level II, Table 3.7-2 requires SDR 2. The detailing provisions for various components of the structure will be discussed in more detail in subsequent design steps in this design example.

NOTE: Design Steps 4 through 6 are not used for SDAP A2. A2 only requires the seat width requirements and connection force requirements to be met; detailed force and displacement calculations are not required.

SECTION V **SDAP A2 CONVENTIONAL BEARING EXAMPLE**
Design Step 7, Design Displacements and Checks

DESIGN STEP 7 DESIGN DISPLACEMENTS AND CHECKS

[Guide Spec 6.3] [NCHRP, Article 3.10.3.10]

Minimum Seat Width Requirement

$L := 400 \cdot ft$ $L = 121.9\ m$ $L := 121.9$ distance btwn joints

$H := 36 \cdot ft$ $H = 10.97\ m$ $H := 10.97$ tallest pier btwn joints

$B := 68.5 \cdot ft$ $B = 20.88\ m$ $B := 20.88$ width of superstructure

$F_v := 1.0$ from Design Step 2.5

$S_1 := 0.094$ from Design Step 2.3

$\alpha := 25 \cdot deg$ skew angle

$$N := \left[0.10 + 0.0017 \cdot L + 0.007 \cdot H + 0.05\sqrt{H} \cdot \sqrt{1 + \left(\frac{2 \cdot B}{L}\right)^2}\right] \cdot \frac{\left(1 + 1.25 \cdot F_v \cdot S_1\right)}{\cos(\alpha)}$$

$N = 0.69$ meters

$N = 2.26\ ft$ minimum seat width

Seat width of 2.5 feet is provided at abutment per Figure 1c, thus the provided seat width is adequate.

SECTION V **SDAP A2 CONVENTIONAL BEARING EXAMPLE**
Design Step 8, Design Structural Components

DESIGN STEP 8 DESIGN STRUCTURAL COMPONENTS

[Guide Spec 6.8.2] [NCHRP, Article 3.10.3.2]

Design Step 8.1 Bearing Connection Design

[Guide Spec 6.2] [NCHRP, Article 3.10.3.2]

Each pinned bearing and its connection to the substructure must be designed to resist a connection force of 0.25 times the vertical reaction at the bearing to provide some resistance against loads induced during a large earthquake.

Recall the total superstructure weight

$$W_{super} := 5540 \cdot kip$$

Estimate the vertical load, P:

$$P_{pier} := W_{super} \cdot \frac{\frac{5 \cdot 124 \cdot ft}{8} + \frac{152 \cdot ft}{2}}{400 \cdot ft} \qquad P_{pier} = 2126\,kip$$

$$F_{pier} := 0.25 \cdot P_{pier} \qquad F_{pier} = 531\,kip$$

Note that more refined estimates of the vertical loads typically are available during design. Also, the consideration of live load per Article 3.5 may potentially increase the vertical load at the bearings. In this case, live load effects are not included.

Calculate the connection force at each bearing

$$V_{conn} := \frac{F_{pier}}{8} \qquad V_{conn} = 66\,kip$$

8 bearings per bent

In Section III with SDAP C, Vconn equals 94 kips in the weak direction and 219 kips in the strong direction, for comparison.

All details that fasten the bearing to the sole and masonry plates (including the anchor bolts) must resist the 66 kip connection force as a minimum. This is a simple but effective strategy to minimize risk of collapse due to girder unseating.

SECTION V **SDAP A2 CONVENTIONAL BEARING EXAMPLE**
Design Step 8, Design Structural Components

In low seismic zones, it is not necessary to design the substructures or their foundations for these forces because it is expected that the substructure will have adequate inherent ductility to survive without collapse. Only the bearing and its connections are designed for the specified force.

Note that although the full seismic load path is not formally designed in SDAP A2, it is prudent that the designer provide a reasonable load path for such forces.

Design Step 8.2 **Shear and Transverse Reinforcement**
[Guide Spec 6.8.2.1] [NCHRP, Article 5.10.11.4.1c]

Columns will be designed using the implicit method as required by Article 6.8.2.1. This is a proscriptive means of capacity protection for column shear resistance only. Because shear failures in columns can lead to collapse, it is rational to design the columns to avoid shear critical behavior. Other types of failure, for instance confinement of plastic hinge zones, do not represent potential life safety hazards. Thus, only shear strength is critical to prevent collapse.

NOTE: Design Steps 9 through 11: Not used.

SECTION V **SDAP A2 CONVENTIONAL BEARING EXAMPLE**
Design Step 12, Seismic Design Complete

DESIGN STEP 12 SEISMIC DESIGN COMPLETE?

The bearing connection design force was 66 kips with $C_{s\_effective} = 0.25$.

SDAP C produced 219 kips, using OS = 1.5, and capacity protection in the strong direction.

Thus, SDAP A2 is a simple and economical way to design for seismic where allowed.

SECTION VI SDAP A2 WITH ELASTOMERIC BEARING EXAMPLE
Design Step 1, Preliminary Design

DESIGN STEP 1 PRELIMINARY DESIGN

The bridge is essentially the same as the one used in Section IV. The intermediate substructure is wall pier construction. However, the site for this section now has lower spectral acceleration levels to illustrate the application of SDAP A2.

SECTION VI **SDAP A2 WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 2, Basic Requirements

DESIGN STEP 2 BASIC REQUIREMENTS

Basic requirements are the same as in Section IV except for Design Step 2.3.

Design Step 2.3 Spectral Acceleration Parameters

The site is on the southern Merrimack River, in northeast Massachusetts. Using national ground motion maps, the MCE short-period (0.2 second) acceleration, S_S, is 0.34g and the 1.0-second acceleration, S_1, is 0.094g.

The spectral accelerations for the Frequent earthquake were determined by the geotechnical engineer, and likewise are based on national ground motion maps. The short-period (0.2 second) acceleration, S_S, is 0.12g and the 1.0-second acceleration, S_1, as 0.03g.

SECTION VI **SDAP A2 WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 3, Determine Seismic Design and Analysis Procedure

DESIGN STEP 3 DETERMINE SEISMIC DESIGN AND ANALYSIS PROCEDURE

Design Step 3.1 Determine Seismic Hazard Level

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3.1]

$F_aS_s = 0.34$ and $F_vS_1 = 0.094$.

The Seismic Hazard Level is II.

By Table 3.7-1, the Seismic Hazard Level is II because F_aS_s is less than 0.35. Based on F_vS_1, the Seismic Hazard Level would only be I. The controlling value is taken to be the more restrictive of the two values.

Design Step 3.2 Determine Seismic Design and Analysis Procedure (SDAP)

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3.1]

We will use SDAP A2.

Table 3.7-2 of the Specification gives the requirements for determining what Seismic Design and Analysis Procedure (SDAP) should be used. A2 is to be used for the Life-Safety performance level in Seismic Hazard Level II.

Design Step 3.3 Determine Seismic Detailing Requirements (SDR)

[Guide Spec, Article 3.7] [NCHRP, Article 3.10.3.1]

SDR 2 is applicable for SDAP A2.

Because the structure is classified for Life-Safety Performance and Seismic Hazard Level II, Table 3.7-2 requires SDR 2. The detailing provisions for various components of the structure will be discussed in more detail in subsequent design steps in this design example.

NOTE: Design Steps 4 through 6 are not used for SDAP A2. A2 only requires the seat width requirements and connection force requirements to be met; detailed force and displacement calculations are not required.

SECTION VI **SDAP A2 WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 7, Design Displacement and Checks

DESIGN STEP 7 DESIGN DISPLACEMENTS AND CHECKS

[Guide Spec 6.3] [NCHRP, Article 3.10.3.10]

Minimum Seat Width Requirement

$L := 400 \cdot ft$ $L = 121.9\,m$ $L := 121.9$ distance btwn joints

$H := 36 \cdot ft$ $H = 10.97\,m$ $H := 10.97$ tallest pier btwn joints

$B := 68.5 \cdot ft$ $B = 20.88\,m$ $B := 20.88$ width of superstructure

$F_v := 1.0$ from Design Step 2.5

$S_1 := 0.094$ from Design Step 2.3

$\alpha := 25 \cdot deg$ skew angle

$$N := \left[0.10 + 0.0017 \cdot L + 0.007 \cdot H + 0.05\sqrt{H} \cdot \sqrt{1 + \left(\frac{2 \cdot B}{L}\right)^2} \right] \cdot \frac{\left(1 + 1.25 \cdot F_v \cdot S_1\right)}{\cos(\alpha)}$$

$N = 0.69$ meters

$N = 2.26\,ft$ minimum seat width

Seat width of 2.5 feet is provided at abutment per Figure 1c, thus the provided seat width is adequate.

SECTION VI **SDAP A2 WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 8, Design Structural Components

DESIGN STEP 8 **DESIGN STRUCTURAL COMPONENTS**
[Guide Spec 15.7] [NCHRP, Article 15.7]

Design Step 8.1 **Bearing Connection Design**

Per the isolation section, each elastomeric bearing and its connection to the substructure must be designed to resist a connection force of

$$F_a := K_{eff} \cdot \Delta$$

where the design displacement, Δ, is based upon

$F_v \cdot S_1$ which must be greater than or equal to 0.25

In order to proceed, the effective stiffness of the bridge must be determined. Furthermore, the deflection of the bridge, Δ, must be determined. This is clearly more work than with the mechanical bearing of Section V.

$$F_v := 1.0$$

$S_1 := 0.25$ minimum value allowed

Recall from the calculations of Section IV

$$K_{total\_Long} := 9135 \cdot \frac{kip}{ft}$$

$$K_{pier\_Long} := 3744 \cdot \frac{kip}{ft}$$

$$W_{super} := 5540 \cdot kip$$

$$\Delta := \left(\sqrt{\frac{W_{super} \cdot g}{K_{total\_Long}}}\right) \cdot \left(\frac{F_v \cdot S_1}{2 \cdot \pi \cdot \frac{rad}{sec}}\right) \qquad \Delta = 2.11\ in$$

SECTION VI **SDAP A2 WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 8, Design Structural Components

Design Step 8.1 (continued)

therefore,

$F_a := K_{total\_Long} \cdot \Delta$ $F_a = 1606\,\text{kip}$

$C_{s\_effective} := \dfrac{F_a}{W_{super}}$ $C_{s\_effective} = 0.29$

This is a much higher C_s than was obtained in SDAP C where $F_vS_1 = 0.12$ and $C_s = 0.14$.

$F_{pier} := \dfrac{K_{pier\_Long} \cdot F_a}{K_{total\_Long}}$ $F_{pier} = 658\,\text{kip}$

$V_{conn} := \dfrac{F_{pier}}{8}$ $V_{conn} = 82\,\text{kip}$ 8 bearings per pier

In SDAP C, the connection force was 102 kip.

All details that fasten the bearing to the sole and masonry plates (including the anchor bolts) must resist this 82 kip connection force. This is a simple but effective strategy to minimize risk of collapse due to girder unseating.

In low seismic zones, it is not necessary to design the substructures or their foundations for these forces because it is expected that the substructure will have adequate inherent ductility to survive without collapse.

SECTION VI **SDAP A2 WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 8, Design Structural Components

Design Step 8.2

Shear and Transverse Reinforcement

Per the requirements of 6.8.2.1, the wall could be designed in its weak direction as a column for shear and transverse reinforcement, using the implicit method. This is a proscriptive means of capacity protection for column shear resistance.

Alternately, shear reinforcement was designed per 7.8.3 to provide for limited ductility in the wall piers. The walls were checked to ensure that the plastic hinging demand shear is less than the shear capacity provided in the wall.

$M_{n\_Long} := 33400 \cdot kip \cdot ft$ See Figure 10 (Section IV, Design Step 8), for PCACOL diagram.

$H_{pier} := 36 \cdot ft$

$V_{u\_Long} := \frac{M_{n\_Long}}{H_{pier}}$ $V_{u\_Long} = 928\ kip$

$b := 4 \cdot ft$ minimum width of wall

$d := 54 \cdot ft$ minimum length of wall

Note that the following equations from the provisions have been converted from metric into U.S. customary units.

$V_r = 3\ \ sqrt\ (f_c')\ \ b\ d$

$V_{r1} := 0.190 \cdot ksi \cdot b \cdot d$ $V_{r1} = 5910\ kip$

$V_r = (0.7\ \ sqrt\ (f_c') + \rho_h \cdot f_y)\ \ b\ d$

$V_{r2} := 0.194 \cdot ksi \cdot b \cdot d$ $V_{r2} = 6034\ kip$

$V_{u\_Long} < V_r$ OK

NOTE: Design Steps 9 through 11: Not used.

SECTION VI **SDAP A2 WITH ELASTOMERIC BEARING EXAMPLE**
Design Step 12, Seismic Design Complete?

DESIGN STEP 12 SEISMIC DESIGN COMPLETE?

SDAP A2 is a reasonably simple and economical way to design for seismic where allowed.

Note that although the full seismic load path is not formally designed in SDAP A2, it is prudent that the designer provide a reasonable load path for such forces.

SECTION VII CLOSING STATEMENT

SECTION VII CLOSING STATEMENT

The multicolumn bent substructure example was a straightforward application of SDAP A2 and C. Because SDAP B requires capacity protection as in SDAP C, it was not included in the example. The pier wall substructure example illustrated the impacts of the Isolation Provisions, where nonseismic issues may control the design of the substructure.

For the pier walls, bearings that permit movement at least in the strong direction of the pier wall, are required by the SDAP C provisions (Guide Specification 4.4.2). The elastomeric bearings allow such movement and the Isolation Provisions of Chapter 15 are used. Use of elastomeric bearings reduces the seismic forces transmitted to the substructure by allowing displacements at the bearings. The reduced elastic forces, which are calculated within the isolation procedures, are allowed to be used to design the connections and foundations. If capacity protection were required, the required connections and foundations would have been unreasonably large in the strong direction. Because the elastic forces are based on the MCE earthquake, which is based on a 2,500-year return period, not the 500-year return period of Division I-A, this approach is reasonable for pier walls.

SECTION VIII REFERENCES

SECTION VIII REFERENCES

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Frankel, A.D. and E.V. Leyendecker (2000), *Uniform Hazard Response Spectra and Seismic Hazard Curves for the United States*, CD-ROM Published by U.S. Geological Survey National Seismic Hazard Mapping Project, March.

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MCEER/ATC 49 (2003), *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges, Part I: Specifications and Part 2: Commentary and Appendices*, Multidisciplinary Center for Earthquake Engineering Research, and Applied Technology Council, Redwood City, CA.

Appendix A
Geotechnical Data

APPENDIX A GEOTECHNICAL DATA

APPENDIX A GEOTECHNICAL DATA, DESIGN EXAMPLE 2

SUBSURFACE CONDITIONS

Subsurface conditions were derived from four borings drilled along the bridge alignment. As shown on Figure A1, the site is underlain by hard, fresh, and sound quartz biotite schist. The water table, which is controlled by the river, is above the ground surface at the interior piers and approximately 30 feet below the ground surface at the abutments.

ROCK PROPERTIES

Rock properties for the subsurface materials encountered in the explorations are shown on Figure A1. These properties were estimated from a series of laboratory test results. Additionally, the measured shear wave velocity is greater than 2,500 feet per second.

SITE CLASS

Site Class B — Rock at the ground surface and the average shear wave velocity in the upper 100 feet exceeds 2,500 feet per second.

SITE ACCELERATION

See design sections of example.

FOUNDATION DESIGN PARAMETERS

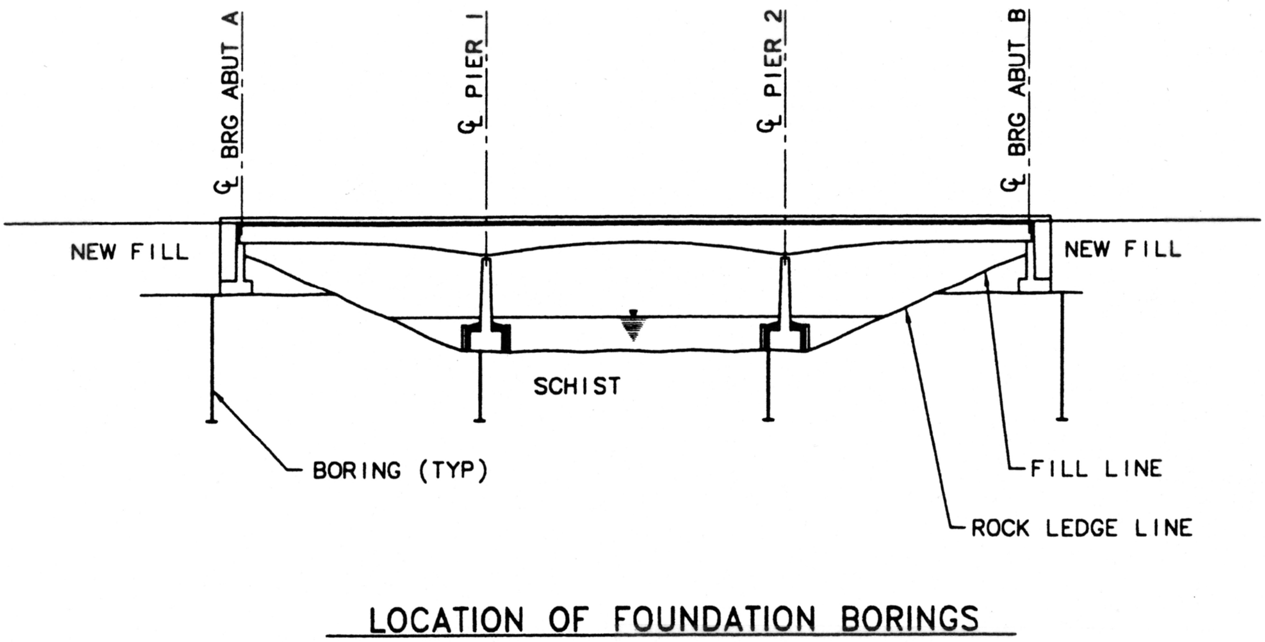
For spread footings on rock, the rock is estimated to have an ultimate bearing capacity of at least 50 ksf based on local experience. The ultimate coefficient of friction between the rock and cast-in-place concrete footings is 0.8.

OTHER ISSUES

Liquefaction will not occur because of the presence of rock.

Assuming the new fill is placed and compacted in accordance with typical Department of Transportation or local jurisdiction requirements, the abutment slopes should be stable during earthquake shaking.

APPENDIX A GEOTECHNICAL DATA



SUBSURFACE PROPERTIES

Type	Depth (ft)	Description	RGD (%)	γ (pcf)	qu (psi)
Rock	See above	Hard, fresh sound quartz biotite schist	90	165	8,000

Where:

RGD rate quality designation (percent)
γ total unit weight (pounds per cubic foot)
qu unconfined compressive strength (pounds per square inch)

Figure A1 — Subsurface Conditions

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