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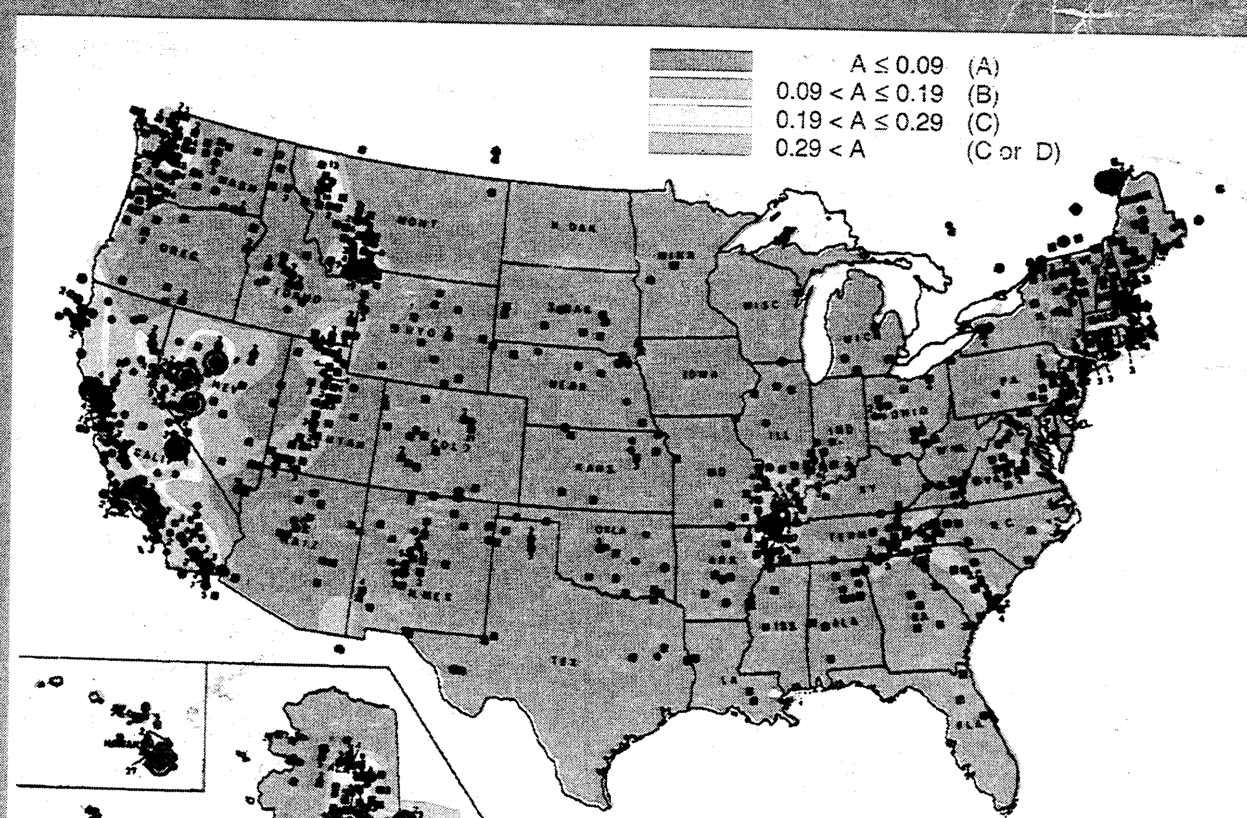
U.S. Department
of Transportation
Federal Highway
Administration

October 1996

Seismic Design of Bridges
Design Example No. 2
Three-Span Continuous Steel Girder Bridge



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16. Abstract

This document describes one of seven seismic design examples that illustrate "how" to apply AASHTO's seismic analysis and design requirements on actual different bridge types across the United States. Each provides a complete set of "designer's notes" covering the seismic analysis, design, and details for that particular bridge including flow charts, references to applicable AASHTO requirements, and thorough commentary that explains each step. In addition, each example highlights separate issues (skew effects, wall piers, elastomeric bearings, pile foundations, etc.).

The **first example** is a 242' reinforced concrete box girder two span overcrossing with spread footing foundations, SPC-C & A = 0.28g. The **second example** is a 400' 3-span skewed steel plate girder bridge over a river in New England with spread footing foundations, SPC-B & A = 0.15g. The **third example** is a skewed 70' single span prestressed concrete girder bridge with tall-closed seat-type abutments on spread footings, SPC-C & A = 0.36g. The **fourth example** is a 320' reinforced concrete box girder 3-span skewed bridge in the western United States with spread footing foundations, SPC-C & A = 0.30g. The **fifth example** is a 1488' steel plate girder bridge in the inland Pacific Northwest with pile foundations, SPC-B & A = 0.15g. It has nine spans and consists of two units: a four-span tangent (Unit 1) and a five-span with a 1300-foot radius curve (Unit 2). The **sixth example** is a 290' sharply curved (104 degrees) 3-span concrete box girder bridge in the Northwestern United States with pile abutment foundations and drilled shaft pier foundations, SPC-C & A = 0.20g. The **seventh example** is a 717' 10-span prestressed girder bridge with open pile bents, SPC-B & A = 0.10g. The superstructure consists of three continuous span units arranged in a 3-4-3 span series.

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Seismic Design Course

Design Example No. 2

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PLEASE NOTE

Data, specifications, suggested practices, and drawings presented herein are based on the best available information, are delineated in accord with recognized professional engineering principles and practices, and are provided for general information only. None of the procedures suggested or discussed should be used without first securing competent advice regarding their suitability for any given application.

This document was prepared with the help and advice of FHWA, State, academic, and private engineers. The intent of this document is to aid practicing engineers in the application of the AASHTO seismic design specification. BERGER/ABAM and the United States Government assume no liability for its contents or use thereof.

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Section I
Introduction

PURPOSE OF DESIGN EXAMPLE

This is the second in a series of seismic design examples developed for the FHWA. A different bridge configuration is used in each example. The bridges are in either Seismic Performance Category B or C sites. 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 first seven 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

REFERENCE AASHTO SPECIFICATIONS

The examples 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")

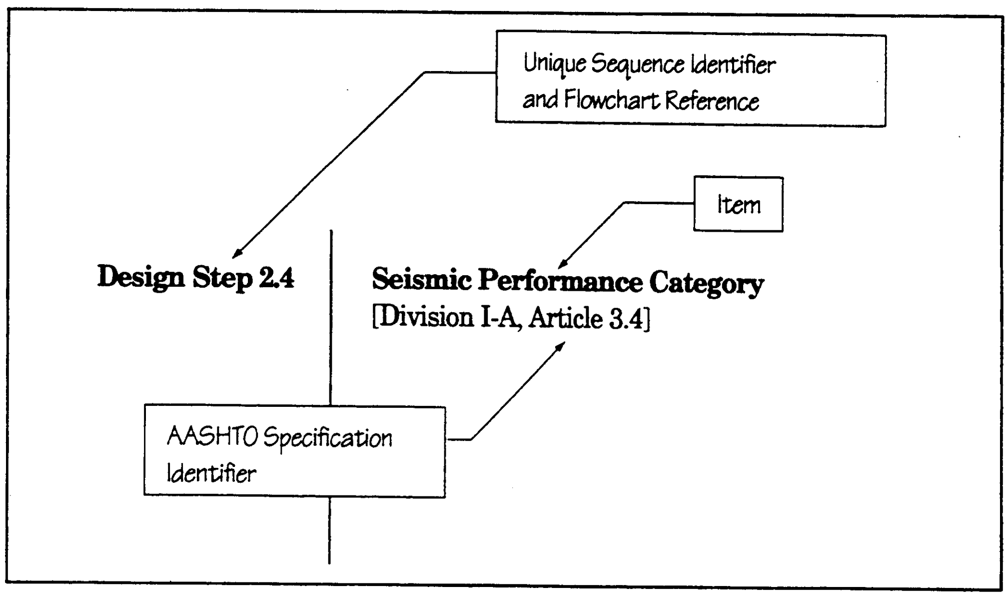
Standard Specifications for Highway Bridges, Division I-A, Seismic Design, American Association of State Highway and Transportation Officials, Inc., 15th Edition, as amended by the Interim Specifications-Bridges-1995.

FLOWCHARTS AND DESIGN STEPS

This second 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 AASHTO Division I-A, and several subcharts that detail the operations that occur for each Design Step.

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 either Division I or Division I-A of the AASHTO Specification follows the title.

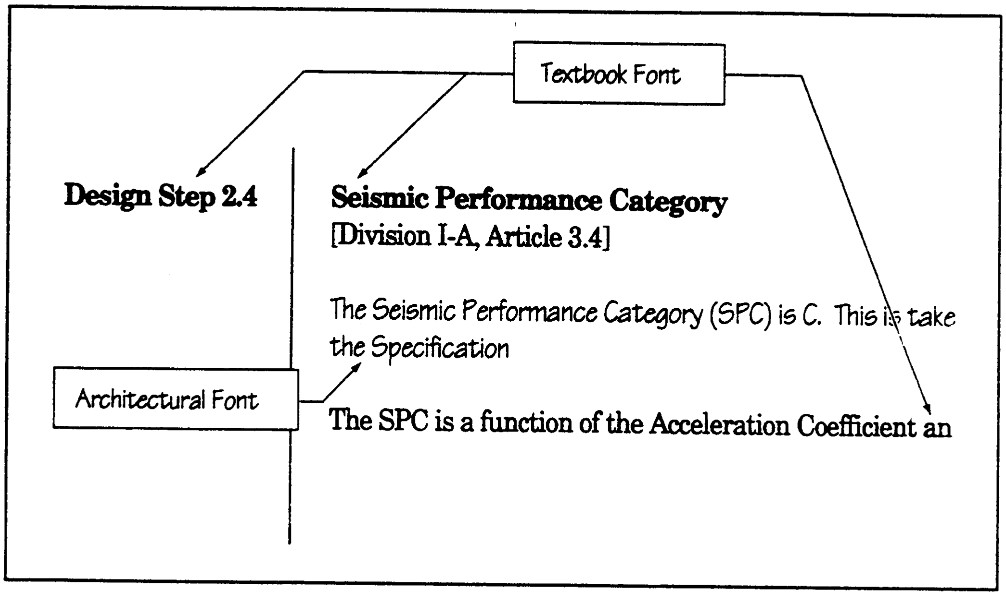
An example is shown below.



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.

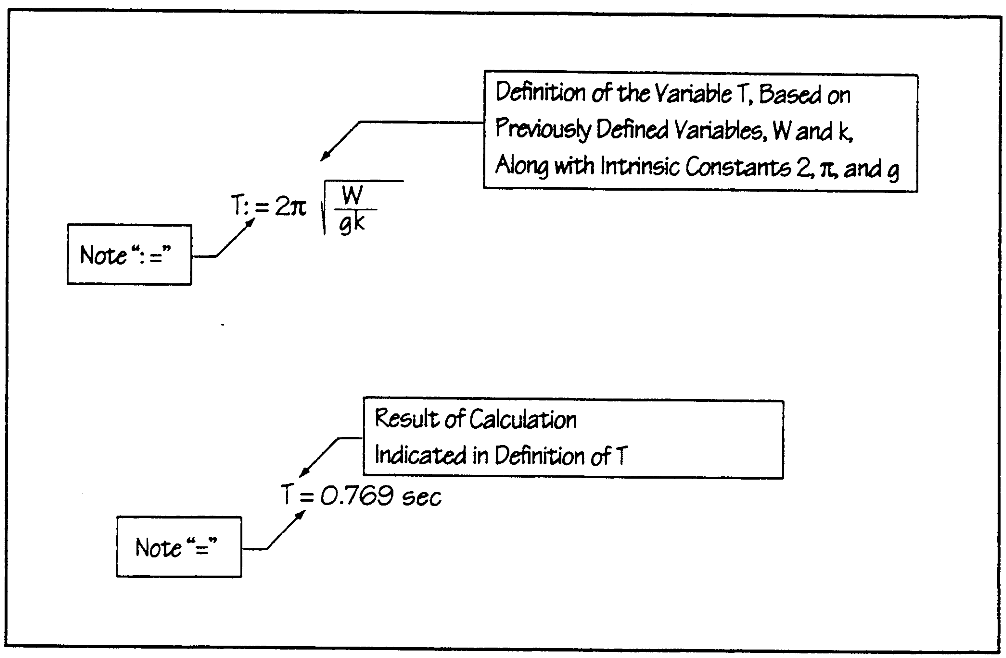


USE OF MATHCAD®

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.

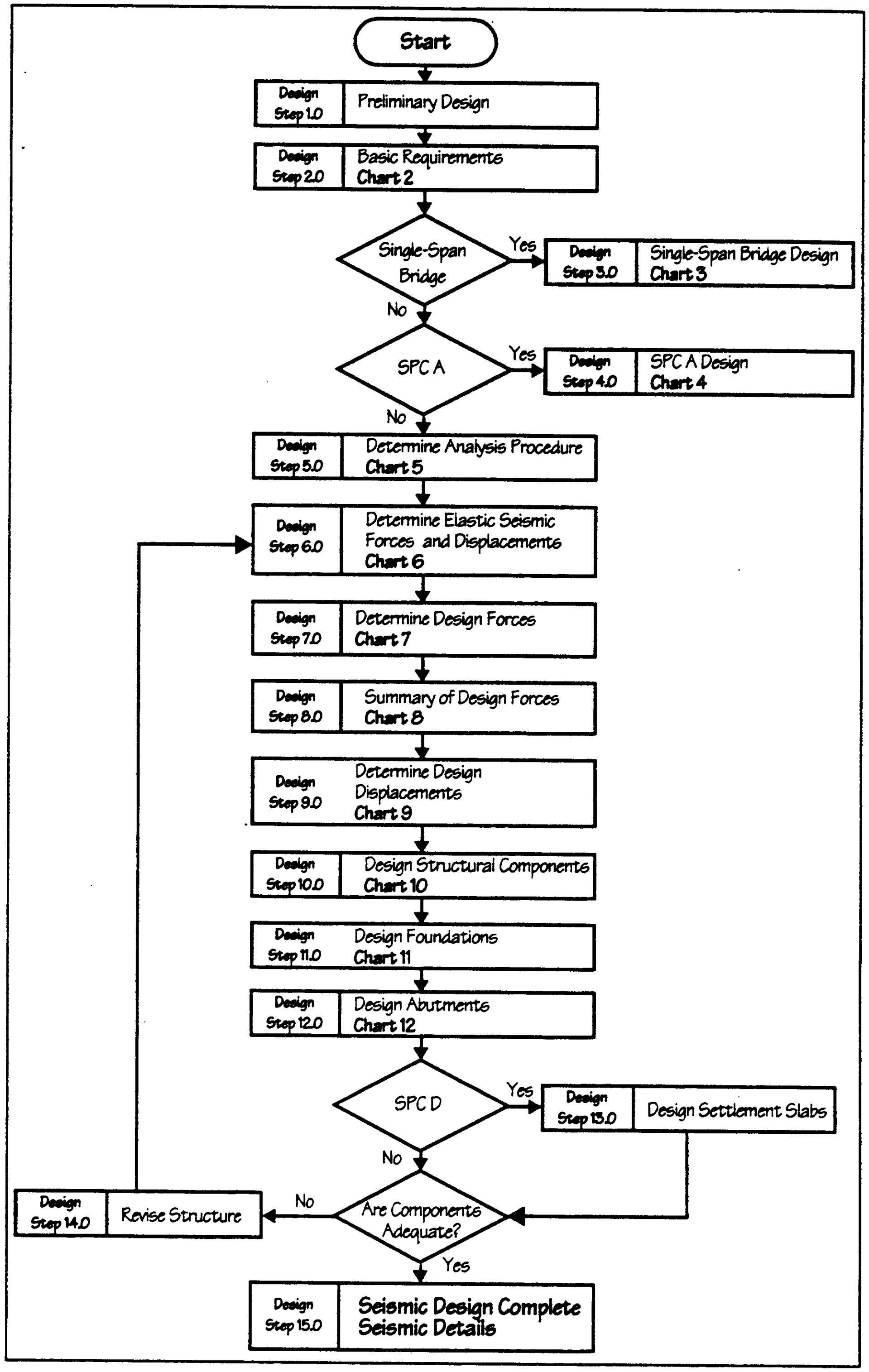


Note that Mathcad® carries the full precision of the variables throughout the calculations, even though the listed result of a calculation is rounded off. Thus, hand-calculated checks made using intermediate rounded results may not yield the same results as the number being checked.

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

FLOWCHARTS



Main Flowchart — Seismic Design AASHTO Division I-A

FLOWCHARTS
(continued)

Key to Detailed Flowcharts

FLOWCHARTS
(continued)

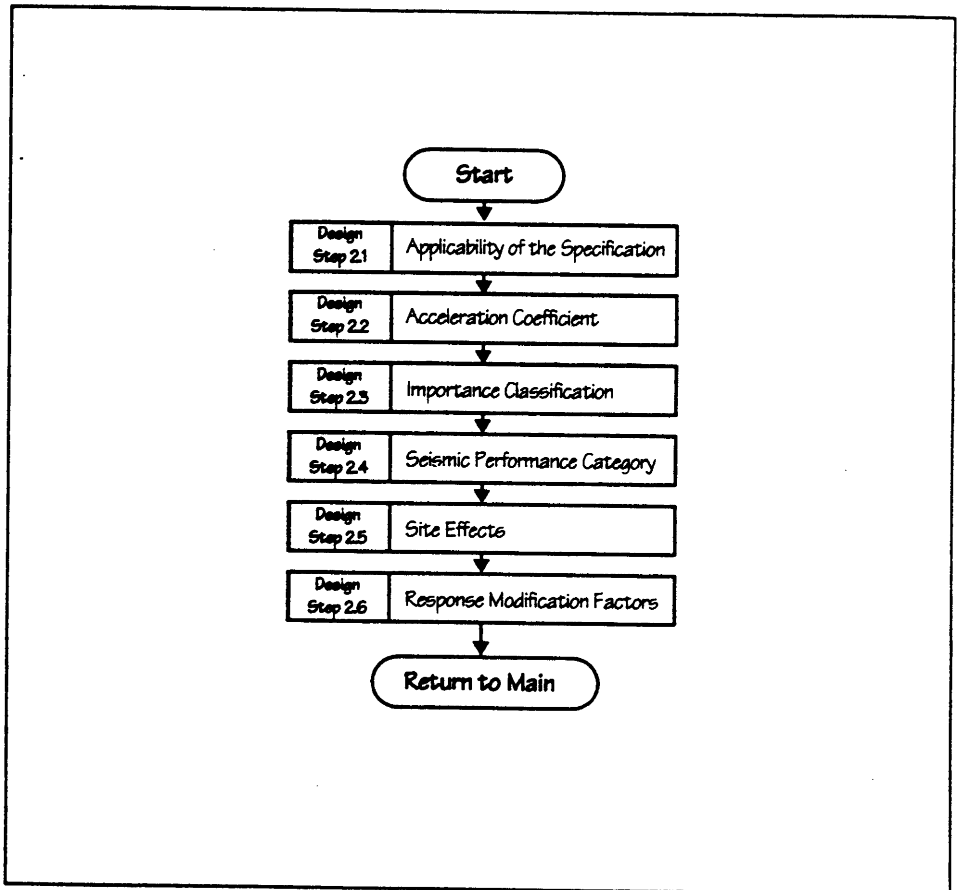


Chart 2 — Basic Requirements

FLOWCHARTS
(continued)

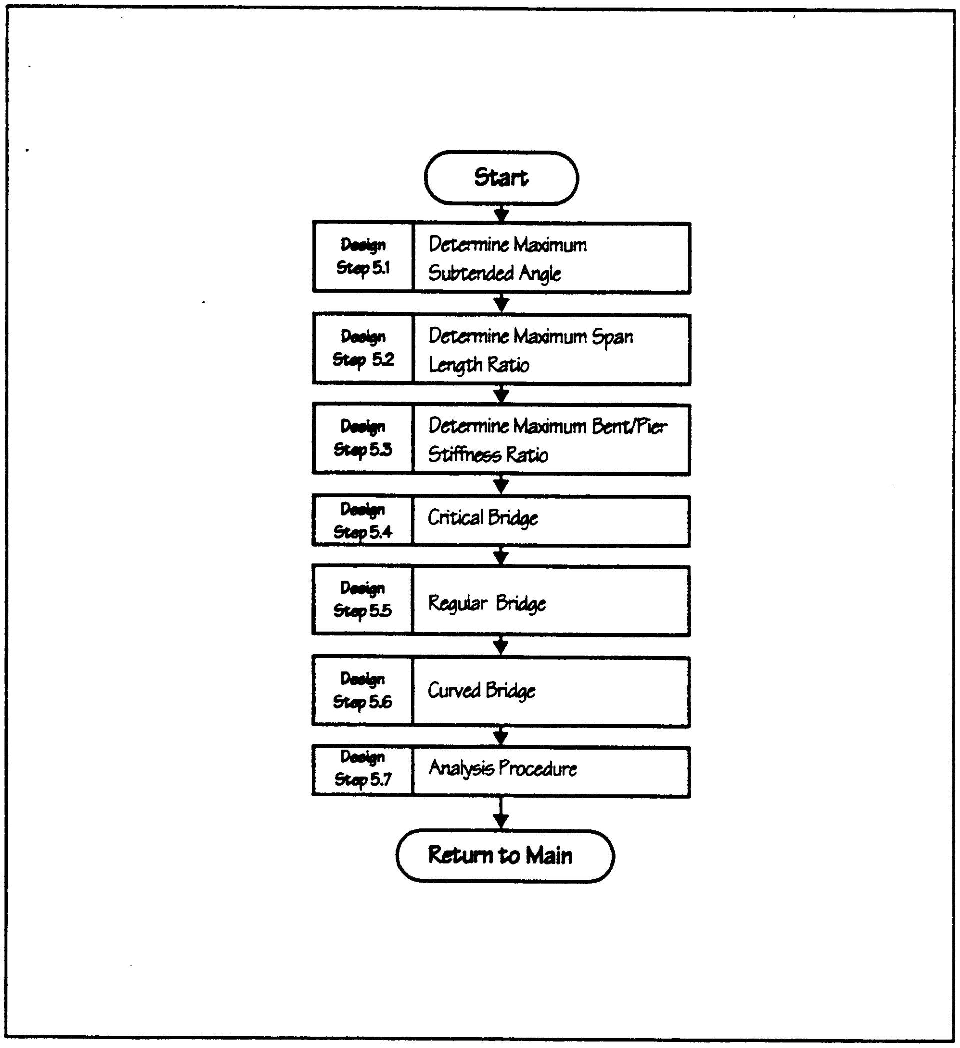


Chart 5 — Determine Analysis Procedure

FLOWCHARTS
(continued)

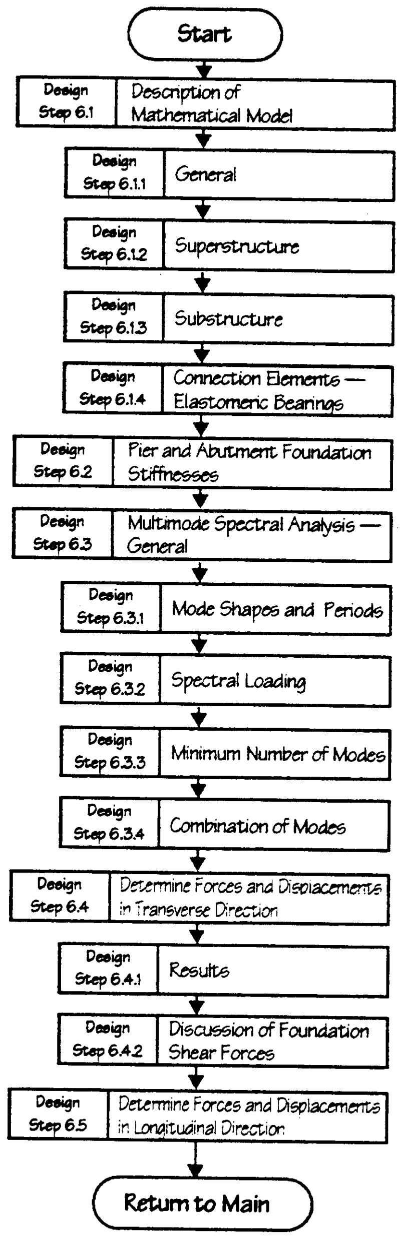


Chart 6 — Determine Elastic Seismic Forces and Displacements

FLOWCHARTS
(continued)

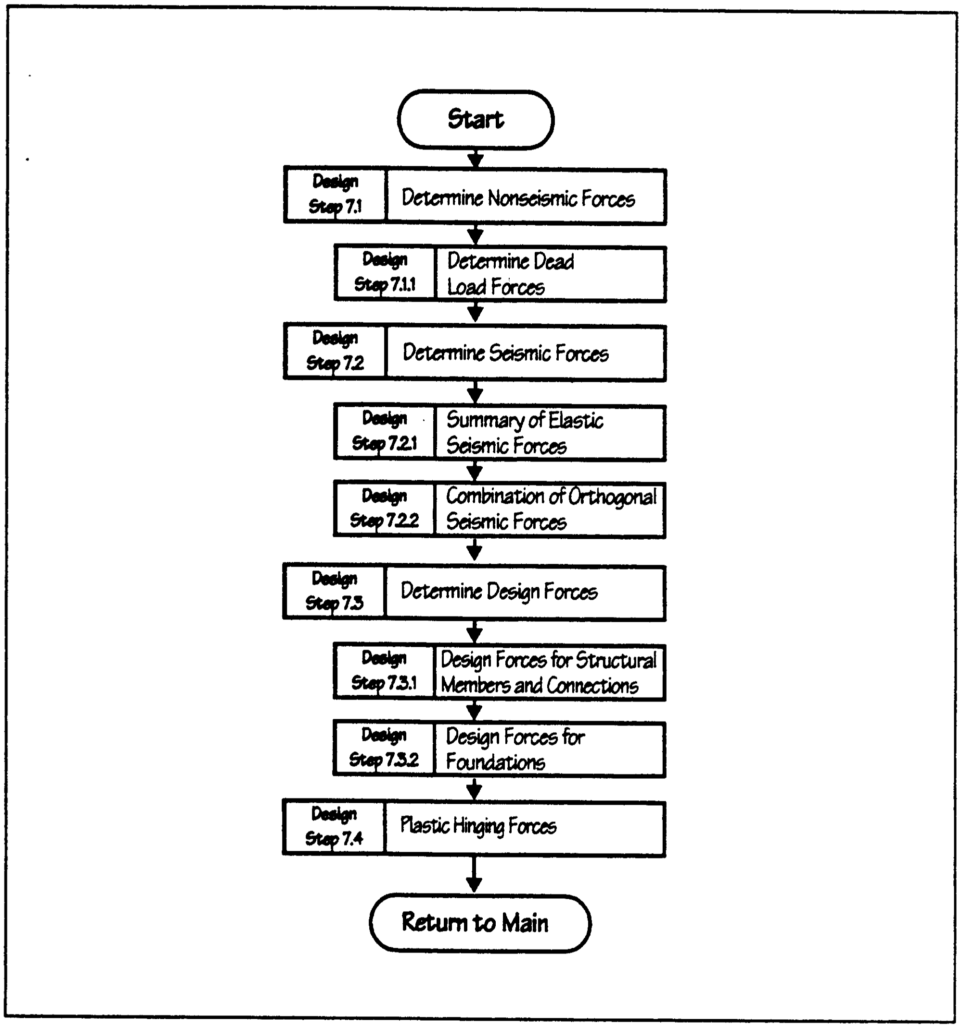


Chart 7 — Determine Design Forces (SPC B)

FLOWCHARTS
(continued)

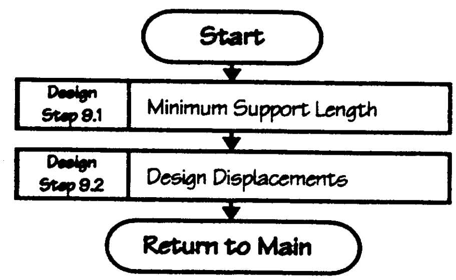


Chart 9 — Determine Design Displacements

FLOWCHARTS
(continued)

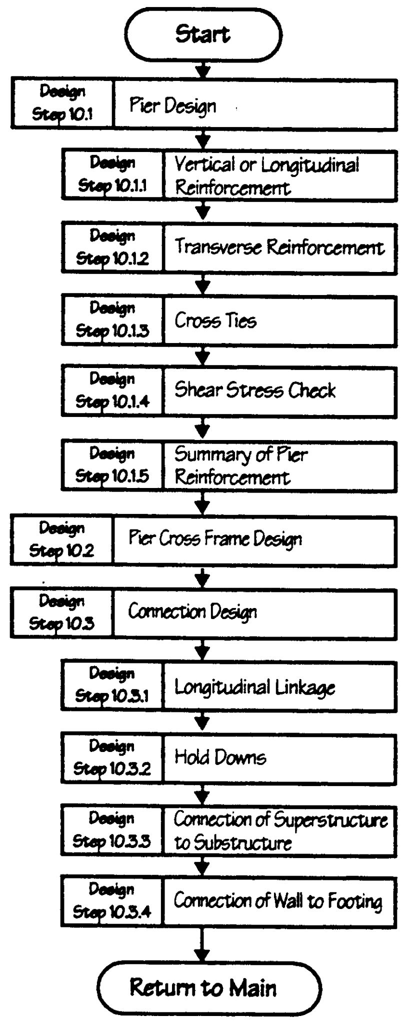


Chart 10 — Design Structural Components

FLOWCHARTS
(continued)

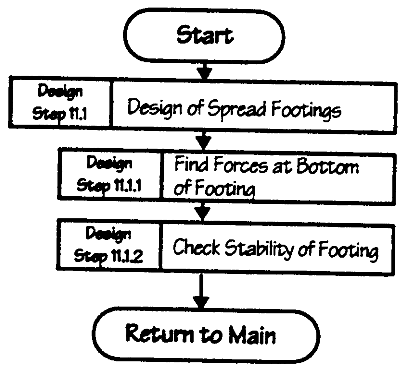


Chart 11 — Design Foundations

Section III
Analysis and Design

SECTION III — ANALYSIS AND DESIGN

DATA

The bridge is to be built in the Northeast United States in a seismic zone with an acceleration coefficient of 0.15g.

The configuration of the bridge is a three-span steel plate girder superstructure with a composite deck. The substructure elements are seat-type abutments and wall piers. 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 g) 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 be wall piers with a thick cross section.

REQUIRED

Design the bridge for seismic loading using the *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.

FEATURES

EMPHASIZED ISSUES IN THIS EXAMPLE

- Wall Pier Design
- Consideration of Elastomeric Bearings
- SPC B Design
- Skew Effects on Girder Systems
- Consideration of Varying Cross Sections

BRIDGE DATA
(continued)

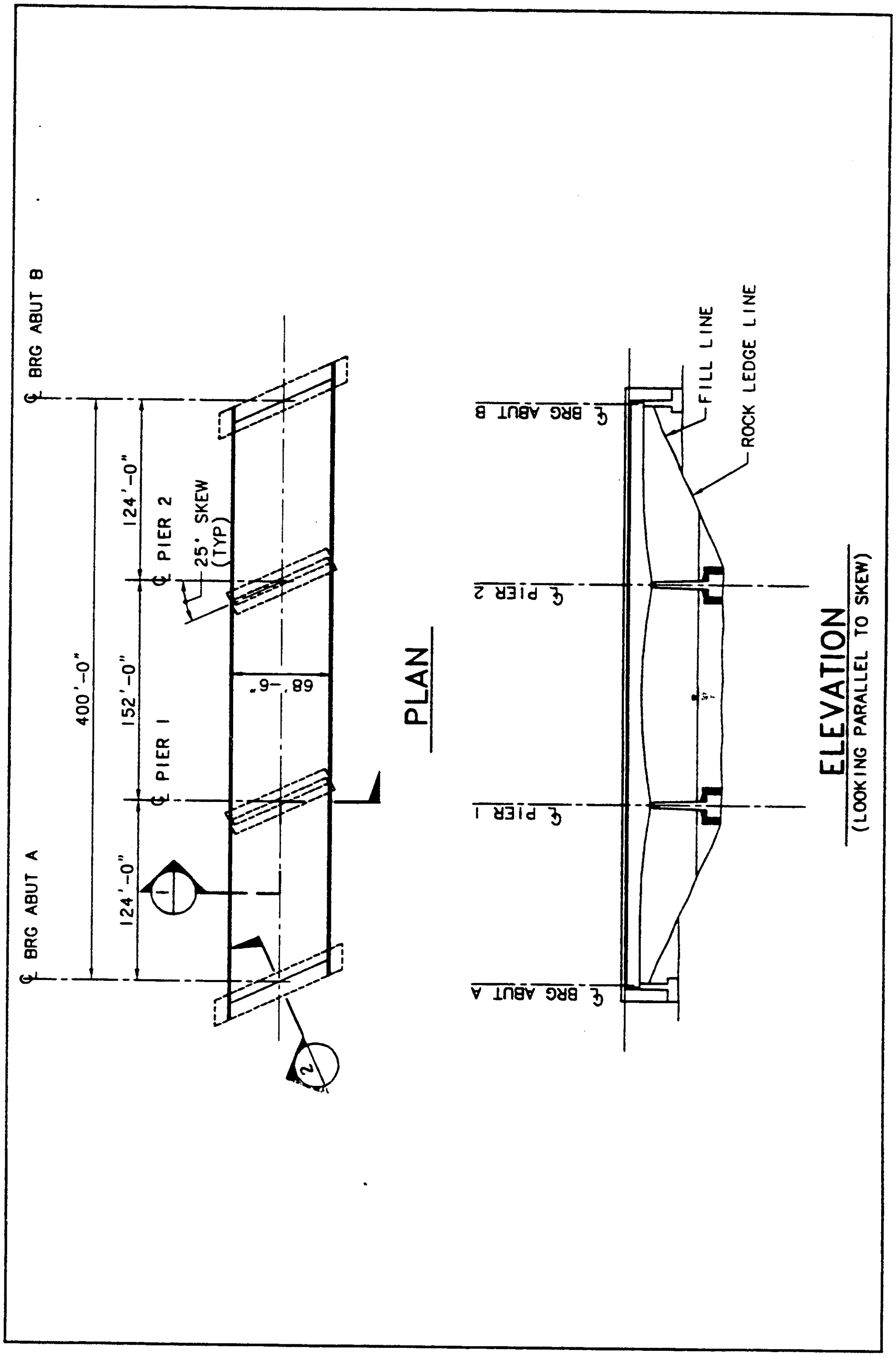


Figure 1a — Bridge No. 2 - Plan and Elevation

BRIDGE DATA
(continued)

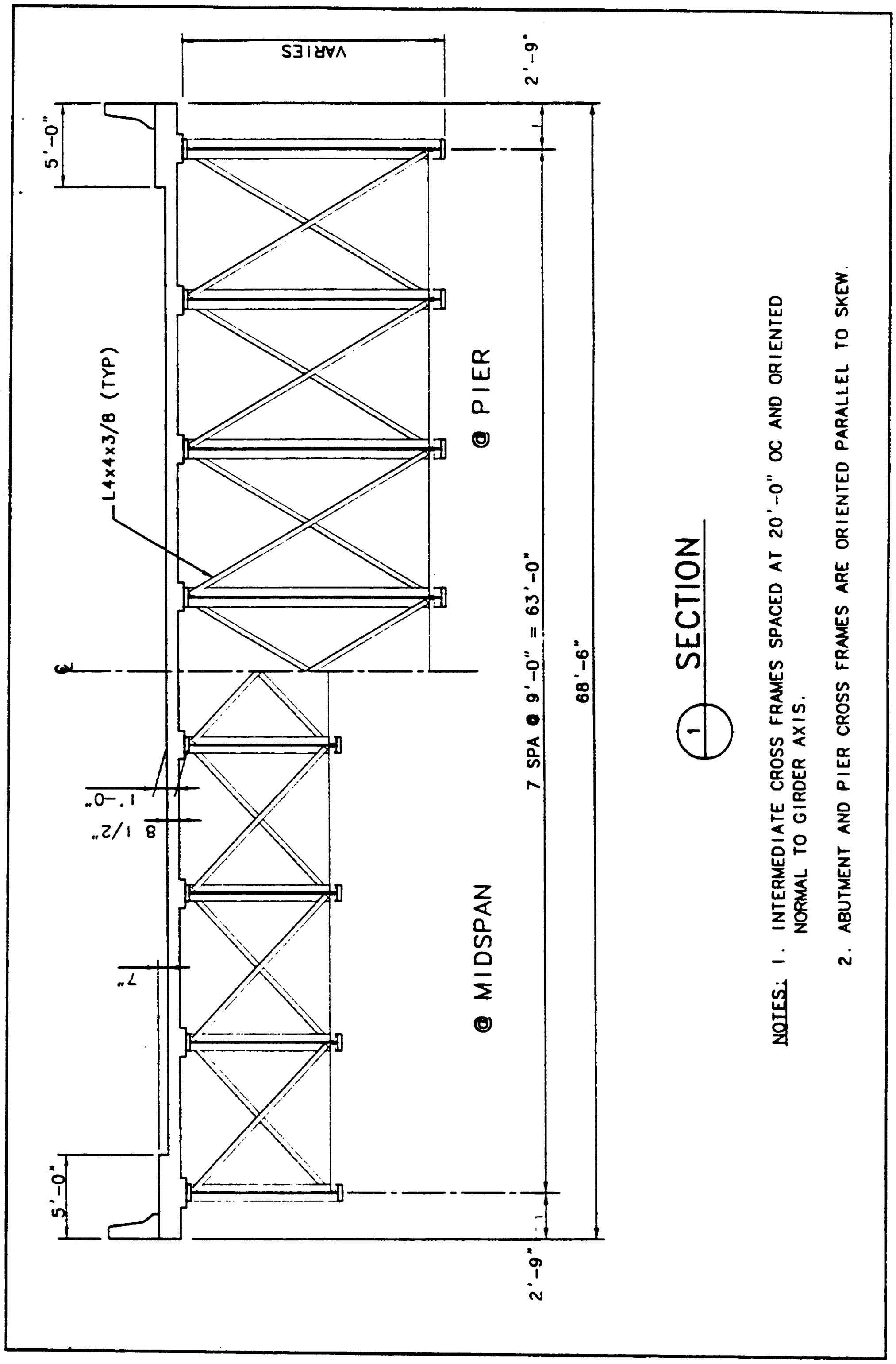


Figure 1b — Bridge No. 2 - Typical Cross Section

BRIDGE DATA
(continued)

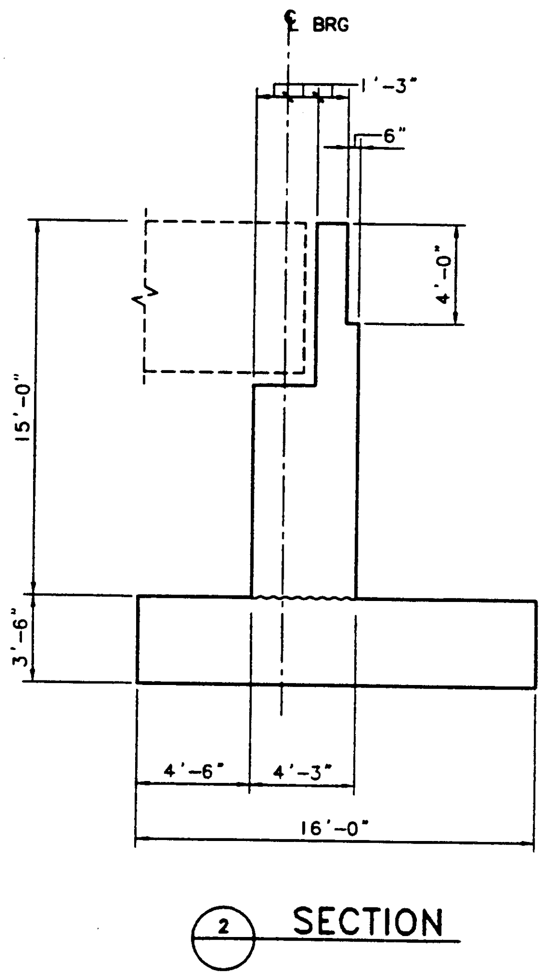


Figure 1c — Bridge No. 2 - Seat-Type Abutment

BRIDGE DATA
(continued)

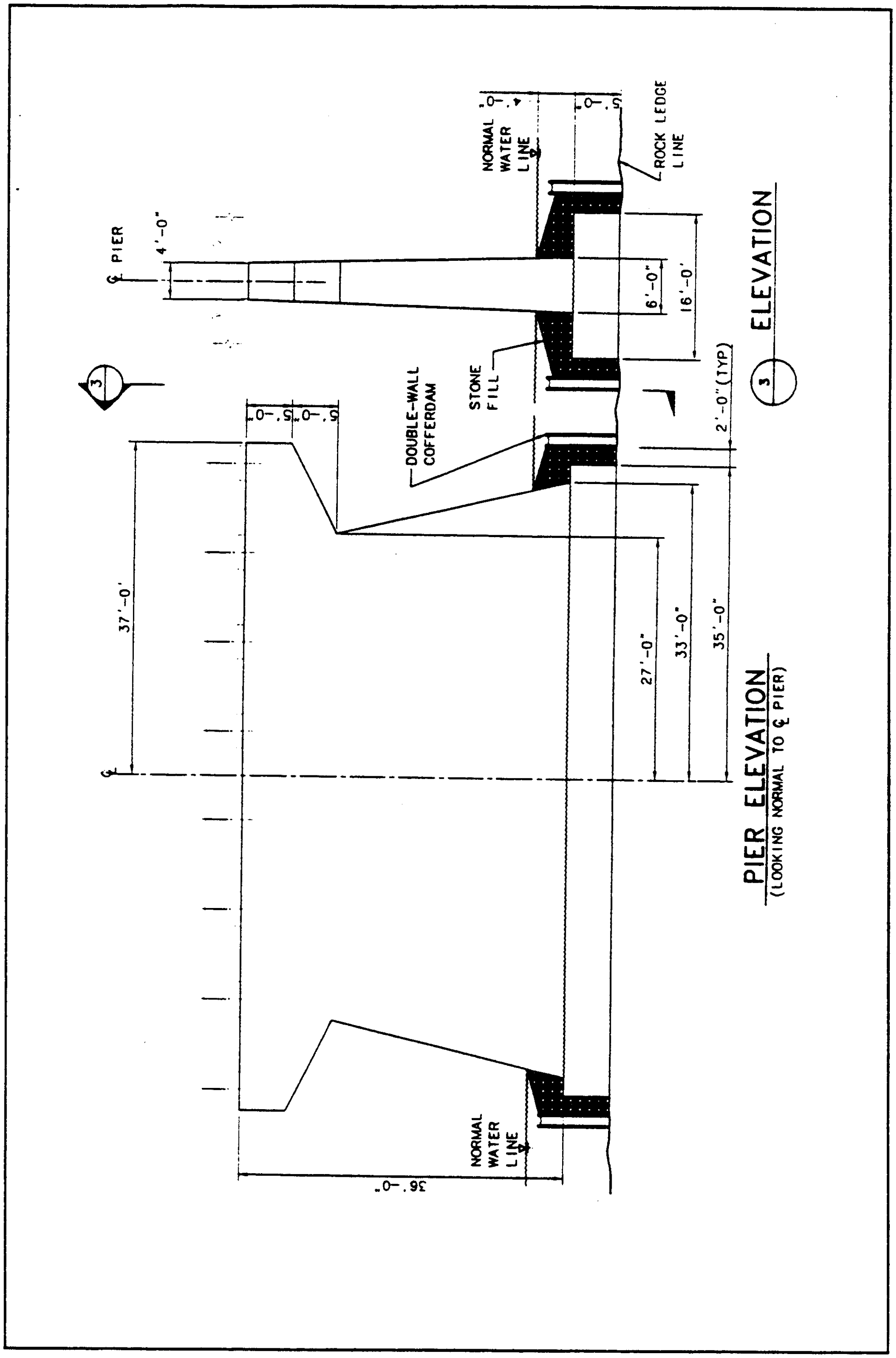


Figure 1d — Bridge No. 2 - Pier Elevation

BRIDGE DATA
(continued)

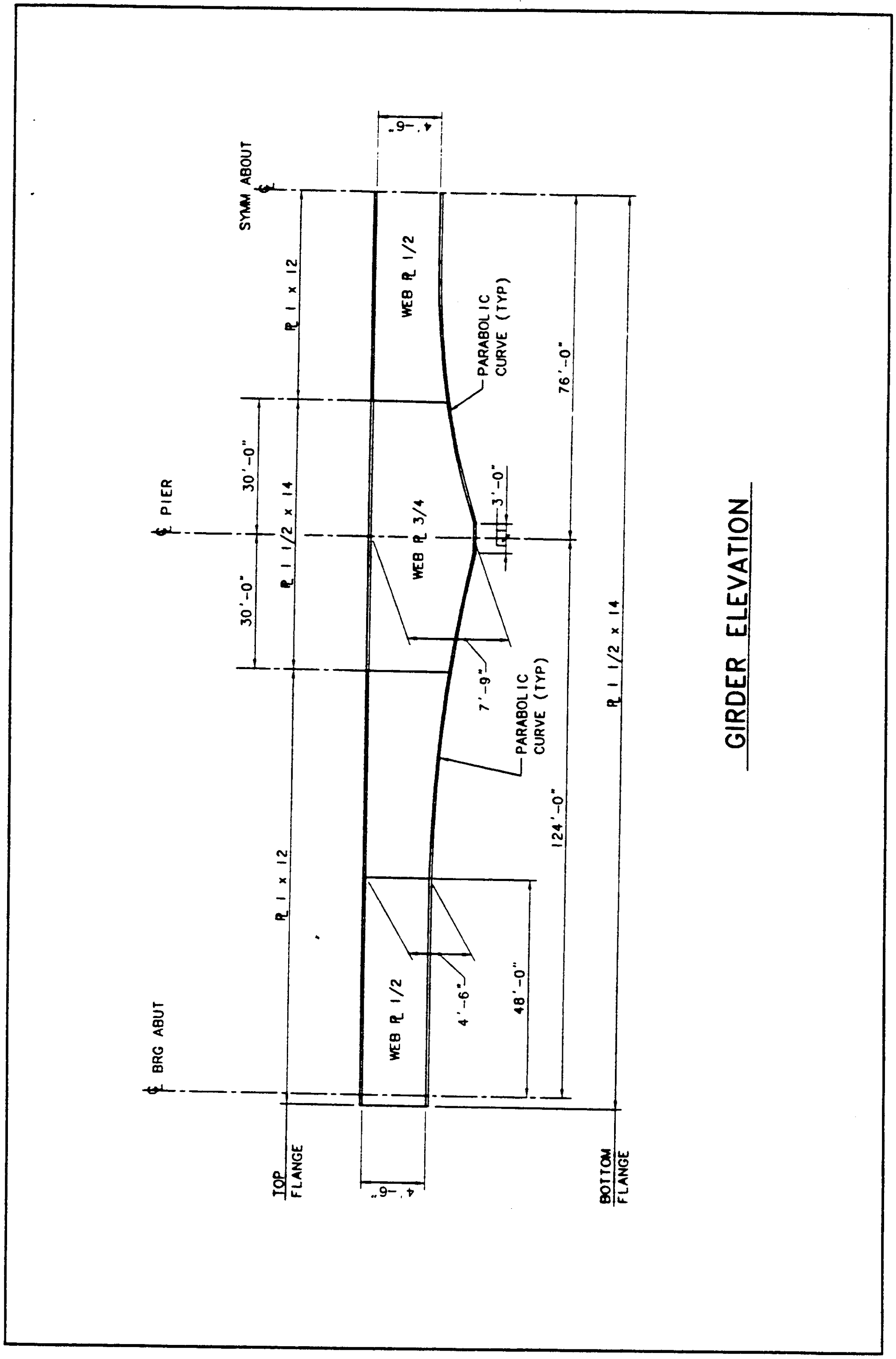


Figure 1e — Bridge No. 2 - Plate Girder Details

BRIDGE DATA
(continued)

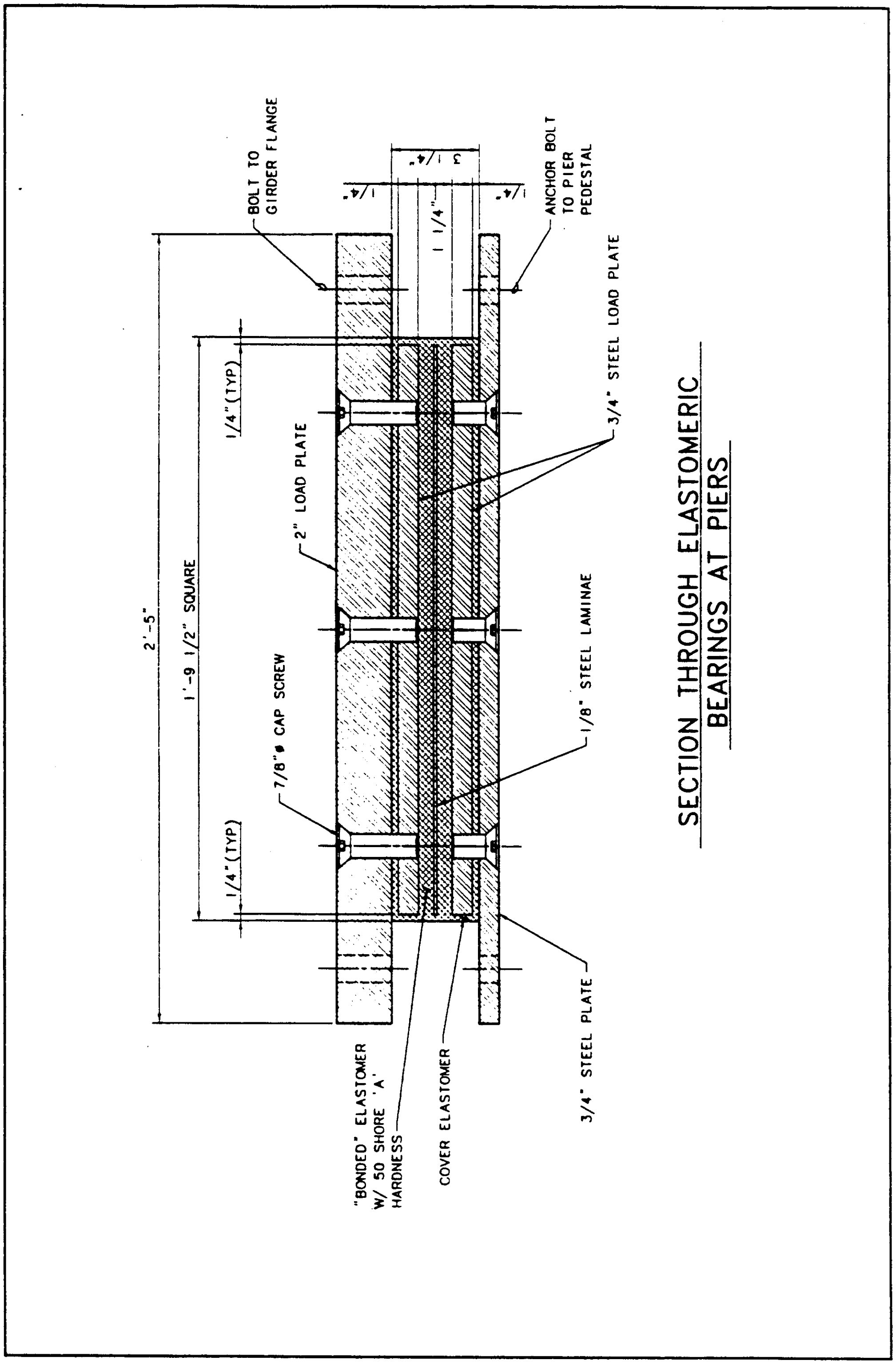


Figure 1f — Bridge No. 2 - Elastomeric Bearings at Piers

BRIDGE DATA
(continued)

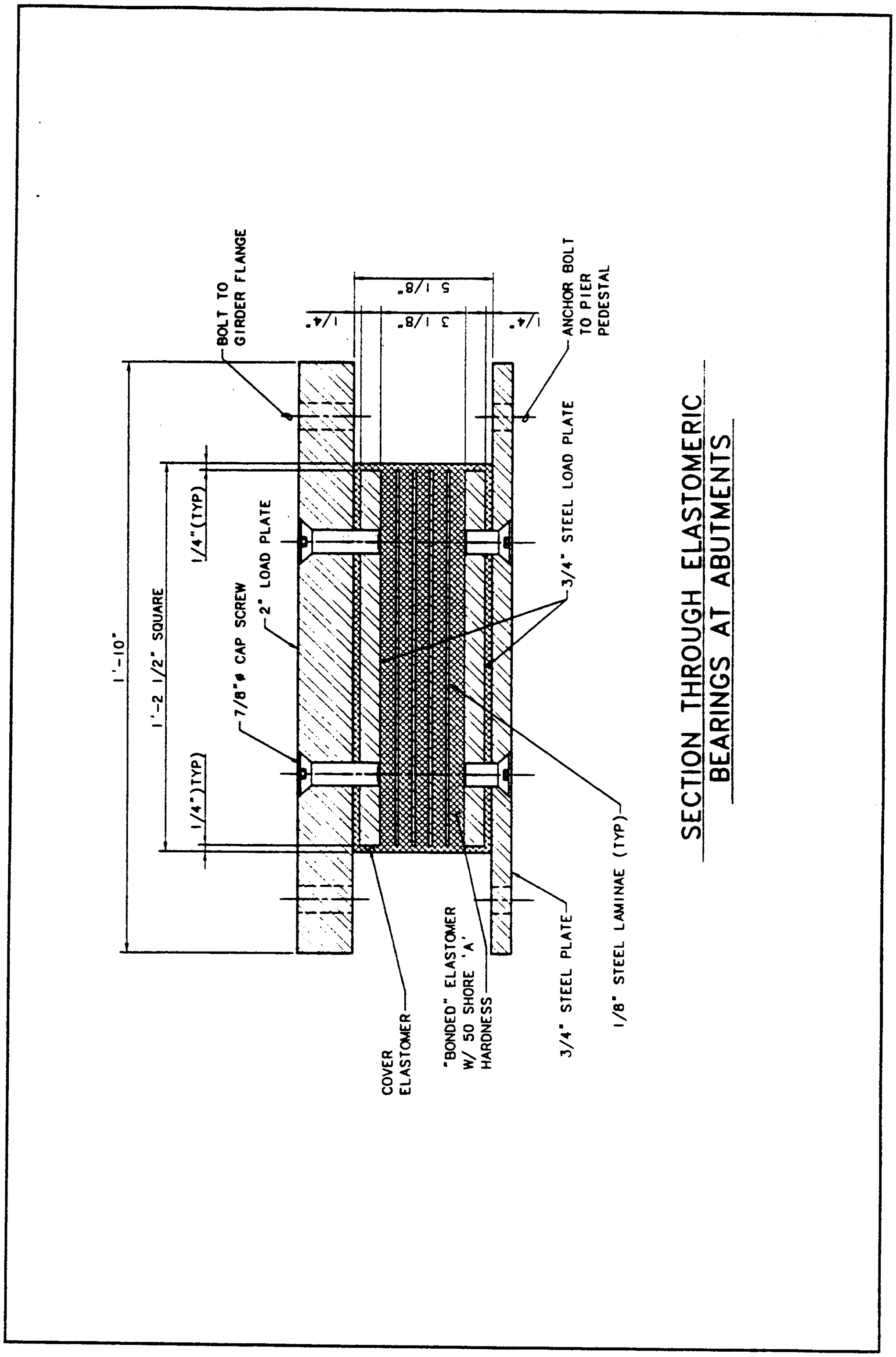


Figure 1g — Bridge No. 2 - Elastomeric Bearings at Abutments

SOLUTION

DESIGN STEP 1 — PRELIMINARY DESIGN

The preliminary seismic design of the bridge has been completed. The results are shown in this section.

The form of the intermediate wall piers 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 and may be used to provide restraint in the transverse direction if the design concept requires such restraint. Several types of seismic behavior have been considered in the preliminary design phase. The behavior and the consequent design forces are strongly dependent on the manner in which the superstructure is connected to the piers and abutments.

If conventional pin bearings are used at one wall pier and sliding bearings, which provide movement only in the longitudinal direction, are used elsewhere, then seismic behavior will be as shown in Figure 2. Such an arrangement allows thermal movements to occur essentially unrestrained. However, the wall pier with the pin bearings must resist relatively large overturning forces for seismic loading. These may not pose a problem in the design of the wall itself, but they may complicate the footing design.

If 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 as a rigid body under seismic loading, and the forces transmitted to the substructure will be substantially smaller than those required to fully restrain the superstructure. The seismic behavior for a system with elastomeric pads is illustrated in Figure 3.

For this example, the system with the elastomeric bearings has been chosen for the seismic design. The bridge superstructure is allowed to move in both the longitudinal and the transverse directions.

DESIGN STEP 1
(continued)

The form of the bearings is conventional in that the bearings are designed for the expected thermal movements and not as base isolation bearings. In the event that the bearings are overstrained under seismic loading, transverse girder stops will be provided as a fail-safe mechanism. Longitudinally, the abutment back walls provide fail-safe restraint to prevent the end spans from dropping off the abutments.

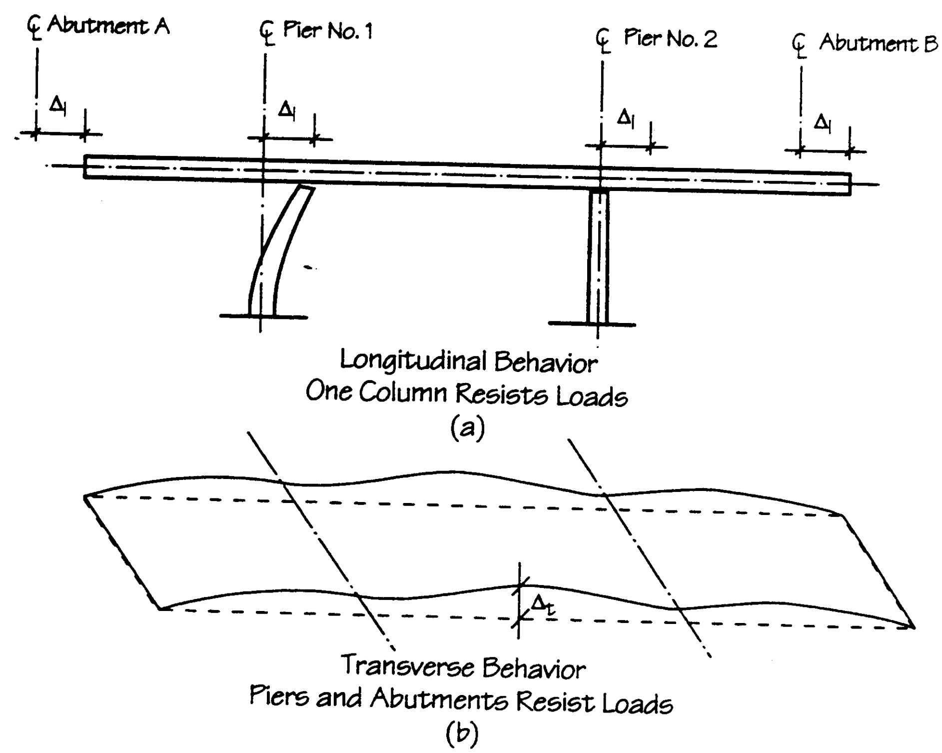


Figure 2 — Seismic Behavior with Conventional Bearings

DESIGN STEP 1
(continued)

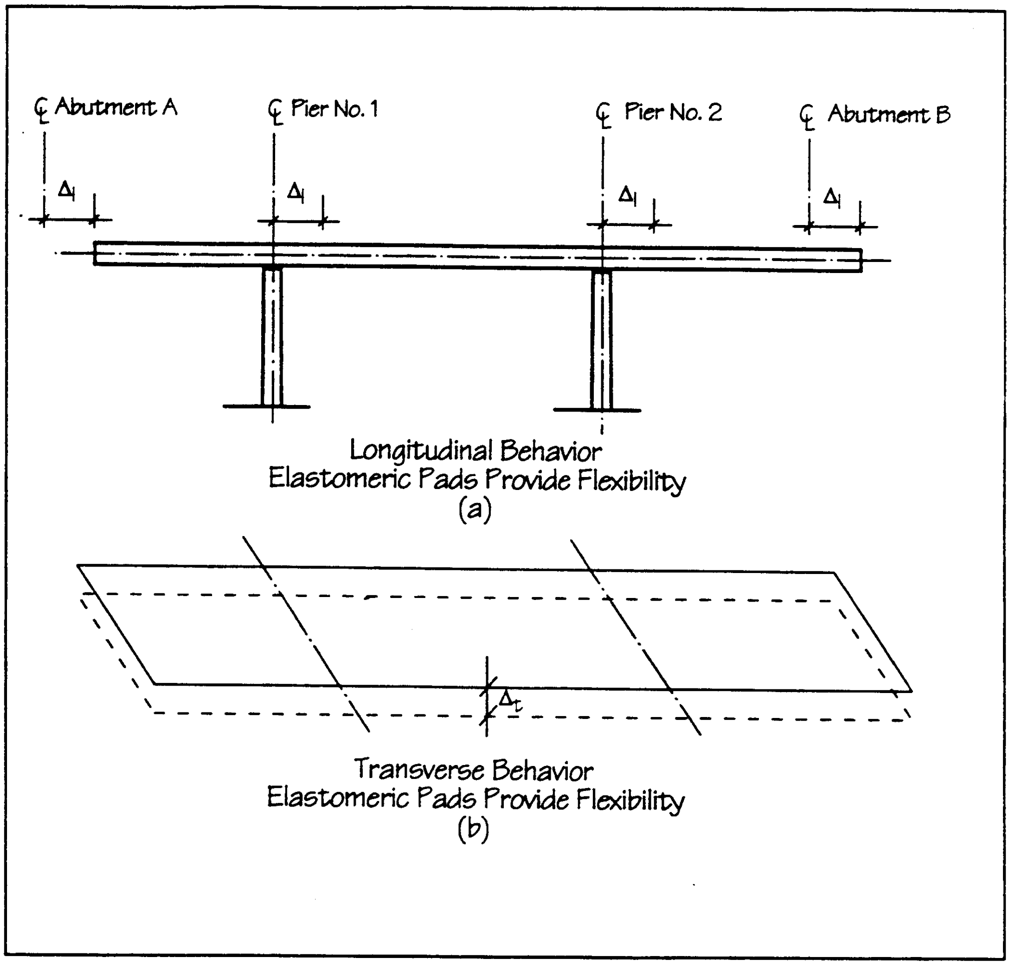


Figure 3 — Seismic Behavior with Elastomeric Bearings

DESIGN STEP 2 BASIC REQUIREMENTS

Design Step 2.1 Applicability of Specification
[Division I-A, Article 3.1]

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 Acceleration Coefficient
[Division I-A, Article 3.2]

The bridge is sited in an area where the Acceleration Coefficient (A) is 0.15.

A = 0.15

Design Step 2.3 Importance Classification
[Division I-A, Article 3.3]

The Importance Classification (IC) of this bridge is taken to be II. The bridge is assumed not to be essential for use following an earthquake.

IC = II

Design Step 2.4 Seismic Performance Category
[Division I-A, Article 3.4]

The Seismic Performance Category (SPC) is B . This is taken from Table 1 of the Specification.

SPC = B

Design Step 2.5 Site Effects
[Division I-A, Article 3.5]

The site conditions affect the design through a coefficient based on the soil profile. In this case, SOIL PROFILE TYPE I corresponds to rock as the founding material.

The Site Coefficient (S) for this type soil is 1.0 per Table 2 of the Specification.

S = 1.0

Design Step 2.6

Response Modification Factors

[Division I-A, Article 3.7]

Because this bridge is classified as SPC B, appropriate Response Modification Factors (R Factors) must be selected for use later in establishing appropriate design force levels.

In this case, Table 3 of the Specification gives the following R Factors.

R = 2 For the substructure wall piers in their strong direction

R = 3 For the weak direction of the wall piers if they are designed as columns in the weak direction. If they are considered wall piers in the weak direction, then R is 2.

R = 1 For the superstructure connections to the wall piers (bearings and shear keys)

Additionally, the cross frame elements that transfer forces from the deck to piers are designed for R = 1.

R = 0.8 For the superstructure connections to the abutments

These factors will be used to ensure that inelastic effects are restricted to elements that 1) can be designed to provide reliable, ductile response, 2) can be inspected after an earthquake to assess damage, and 3) can be repaired relatively easily.

The foundations do not fit this constraint and thus will be designed to resist the probable forces that can be delivered by the piers without incurring any damage. For SPC C and D bridges, this is accomplished by designing the foundations to withstand the plastic hinging forces likely to be developed in the piers. Presently, a relaxed version of this requirement is used for SPC B; an R factor equal to the pier's R Factor divided by 2. However, at the time of writing this requirement is under consideration by AASHTO, since there are instances where the foundation may be weaker than the column or pier. This situation has been discussed by Gajer and Wagh (1994 and 1995).

Issues related to the use of the reduced R Factor for the foundations will be discussed in more detail in the foundation design steps.

DESIGN STEP 3 SINGLE-SPAN BRIDGE DESIGN

Not applicable.

DESIGN STEP 4 SEISMIC PERFORMANCE CATEGORY A DESIGN

Not applicable.

DESIGN STEP 5 — DETERMINE ANALYSIS PROCEDURE

Design Step 5.1 — Determine Maximum Subtended Angle
[Division I-A, Article 4.2]

The bridge is not curved in the horizontal plane.

Design Step 5.2 — Determine Maximum Span Length Ratio
[Division I-A, Article 4.2]

The maximum span length ratio is 1.23 = 152 ft/124 ft.

Design Step 5.3 — Determine Maximum Bent/Pier Stiffness Ratio
[Division I-A, Article 4.2]

The piers are identical; thus the ratio of their stiffnesses is 1.

Design Step 5.4 — Critical Bridge
[Division I-A, Article 4.2.3]

Assume that the bridge is not critical.

Design Step 5.5 — Regular Bridge
[Division I-A, Article 4.2]

Table 5 of the Specification gives the requirements for determining whether a bridge is regular. The requirements are based on limiting values of the parameters determined in the steps above.

The bridge is regular since there is no curve, the span length ratio is less than 2, and maximum pier stiffness ratio is less than 4.

Design Step 5.6 — Curved Bridge
[Division I-A, Article 4.2.2]

Not applicable; no curvature.

Design Step 5.7

Analysis Procedure

[Division I-A, Article 4.2]

Since this bridge is not a single-span bridge or a SPC A bridge, the analysis requirements of Article 4 must be satisfied. Table 4 of the Specification is used to select the minimum analysis requirements.

From Table 4 of the Specification, either the Uniform Load Method (Procedure 1) or the Single-Mode Spectral Method (Procedure 2), may be used to analyze this structure since it has less than six spans.

These are the minimum methods that can be used; the Multimode Spectral Method (Procedure 3) or the Time-History Method (Procedure 4) could also be used in lieu of Procedures 1 and 2.

For this example, Procedure 3 is used for the analysis for two reasons: 1) the multimode method is easy to apply using most analysis programs, and 2) the application of the method to a bridge with relatively flexible bearings connecting the superstructure and substructure is to be discussed.

Procedure 1 or 2 could be used to estimate the superstructure forces. If these methods were used, the substructure would be considered rigid, and the rigid body inertial earthquake forces — the pier mass times the ground acceleration — would be added to the superstructure forces.

DESIGN STEP 6

DETERMINE ELASTIC SEISMIC FORCES AND DISPLACEMENTS

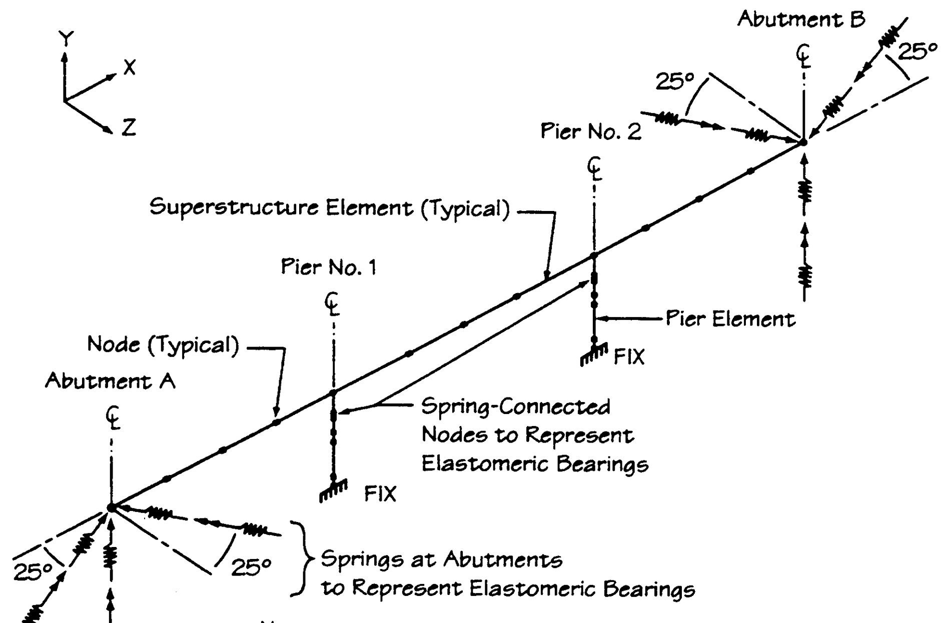
Design Step 6.1

Description of Mathematical Model

Design Step 6.1.1

General
[Division I-A, Article 4.5.2]

The structural analysis program SAP90 Version 6.0 Beta (CSI, 1994) was used for the analyses. The model used is shown in Figure 4 and includes a single line of frame elements for the superstructure and a single vertical line of elements for the piers (columns). The elastomeric bearing pads have been included as elastic springs located between the superstructure and the substructure elements. A copy of the SAP input file for the analyses is provided in Appendix B.



Notes:
1. For Span Lengths, See Figure 1.
2. For Pier Element Details, See Figure 5.
3. For Elastomeric Bearing Spring Details, See Figure 7.
4. All Angles Shown Are Measured in Horizontal Plane.
5. Superstructure Nodes Are Located Vertically at the Superstructure Centroid.

Figure 4 — SAP90 Model

Design Step 6.1.2

Superstructure

a) Geometry

The superstructure has been modeled using four elements per span, and the longitudinal axes of the elements are located along the centroid of the superstructure. Since the girders are haunched, the centrodial depth varies along the length of the structure. This variation of depth is reflected in the model. The computer program will lump the distributed mass of the elements at the nodes, and enough nodes per span should be used to properly estimate the actual dynamic response. In this case, four elements per span or three nodes per span should be sufficient.

The centroid of the superstructure at the piers is located approximately 6.4 feet above the bottom flange of the plate girder. The connection of the superstructure to the bearings and substructure is made in the SAP90 model with a rigid link element that extends from centroid to the bearings. This element is the uppermost pier element shown in Figure 5.

As shown in Figure 4, the superstructure has been collapsed into a single line of 3-D frame elements. This is a reasonable approach for most bridges that have regular geometry. The model is used solely for the determination of seismic forces, so the fact that such a "stick" model does not give the correct forces for other loadings, for instance dead loads, is not a concern. Many designers use such an approach for the seismic model, and further discussion of setting up the seismic model is given by FHWA (1987) and Caltrans (1989).

b) Properties

Areas and Moments of Inertia. The properties of the elements have been calculated at the quarter points of each span. These properties are listed in Table 1.

The properties reported are reported as equivalent concrete properties, since the superstructure is a composite of steel and concrete. 4000 psi concrete has been assumed. The areas are based on the gross area of concrete and steel. The moment of inertia about the horizontal axis I_{horiz} is based on full composite gross sections in both positive and negative gravity moment regions. The moment of inertia about the vertical axis I_{vert} also assumes gross sections comprised of the deck, sidewalks, and all of the girders.

Design Step
6.1.2
(continued)

Table 1
Properties of Superstructure

Location	Area	Effective Density	Moment of Inertia		
			Abt. Vert. Axis	Abt. Horiz. Axis	
	A (ft^2)	g [1] (k/ft^3)	I vert [2] (ft^4)	y bar [3] (ft)	I horiz (ft^4)
Abutment	81.0	0.166	36207	1.377	296
End Span 1/4 Pt	81.0	0.166	36207	1.377	296
1/2 Pt	81.3	0.166	36353	1.407	311
3/4 Pt	84.3	0.162	37607	1.698	473
Pier	104.0	0.143	45988	2.477	996
Center Span 1/4 Pt	83.4	0.163	37206	1.603	417
1/2 Pt	81.0	0.166	36207	1.377	296

Notes:

1. Includes weight of barriers, overlay, forms, stiffeners, and cross frames.
2. I vert based on full composite action of deck and girders.
3. 'y bar' is measured from the top of the 9-inch deck.

As shown in Table 1, the density of concrete has been increased to include the following additional dead loads: traffic barriers, wearing surface overlays, cross frames and stiffeners, and stay-in-place steel forms with concrete. These items are considered uniformly distributed along the length of the bridge. The weight of these additional items totals 3.69 kips per lineal foot.

SAP90 can model members that have smoothly varying cross sections along their lengths. A linear variation of properties (not dimensions) has been used to approximate the effect of the haunched girders.

For this example with the superstructure supported on elastomeric bearings, it is appropriate to use full composite action between the deck and the girders, and to assume that the concrete deck is not cracked. This argument is based on the bearings acting to prevent significant inertial

Design Step 6.1.2 (continued)

forces from developing in the superstructure. In cases where the inertial forces that are developed in the deck are large enough to cause cracking, reduction of the deck stiffness should be considered. This would include determining if the induced deck forces are sensitive to the stiffness assigned to the deck. A starting point for reduced section properties might be to halve the contribution of the deck to the stiffness.

Full composite action between the deck and the girders is assumed by many designers, although some slippage; and, therefore, noncomposite action may develop in some bridges. There is no established practice on how to handle this problem; however, some designers count only the deck and the top flange or top flange and some fraction of the girder web in the calculation of the transverse bending stiffness. The problem may be addressed by assessing the sensitivity of response to the assumed stiffness. Quick assessments of this may be made by arbitrarily reducing the stiffness and looking at the results. A lack of sensitivity means that the precise estimates of the properties are not warranted.

The presence of the skew is accounted for in the orientation of the substructure and bearing elements.

<u>Torsional Properties.</u> The torsional constant of the superstructure was calculated using only the deck. The contribution to torsional resistance offered by warping of the sections has been neglected. The calculation of the torsional constant J is given below.

$b_d := 68.5 \cdot ft$ Width of deck

$h_d := 8.5 \cdot in$ Thickness of deck

$J := \dfrac{b_d \cdot h_d^{\,3}}{3}$ $J = 8.11 \cdot ft^4$

The cross section is open — there is no horizontal truss at the bottom flanges — and thus the approach for open sections discussed by Heins (1975) is used. In the calculation, only the deck is considered. If the individual girders are included, the result changes by only about 1 percent. This can be demonstrated using the approach outlined by Heins and Kuo (1972) for determining the torsional properties of composite sections.

Design Step 6.1.2 (continued)

The approach, whereby warping effects are ignored, is reasonable for the seismic model of this straight bridge. The designer might wish to include warping effects in some instances, for example curved bridges with heavy cross framing. Due to the difficulties of properly including warping in most computer models, a simple bounding using different J values may be easier to perform and will give an indication of sensitivity to torsional stiffness.

Design Step 6.1.3

Substructure

The single line of elements representing each pier has been divided into elements with nodes at each change in cross section, as shown in Figure 5. The piers and abutments are skewed 25 degrees; thus the properties of these substructure elements are rotated in the model to properly account for the skew, as shown in Figure 6. The rotation of the elements is handled with the member local axis control in SAP90. As with the superstructure, SAP90's non-prismatic feature is used to model the continual varying cross section of the piers. For this model full, uncracked moments of inertia are used for the pier.

The use of the uncracked moments of inertia are justified here based on the fact that little, if any, inelastic action is expected in the pier due to its size and isolation effect of the elastomeric bearings. Typically, the use of the uncracked properties will produce higher forces for seismic loading, although the displacements will be smaller than if cracked properties are used.

Because the main part of the pier is relatively long (26 feet), a short column element, which is only 0.2 foot long, has been included near the base of the pier. The short column element allows a more refined estimate of the pier shear to be output.

Design Step
6.1.3
(continued)

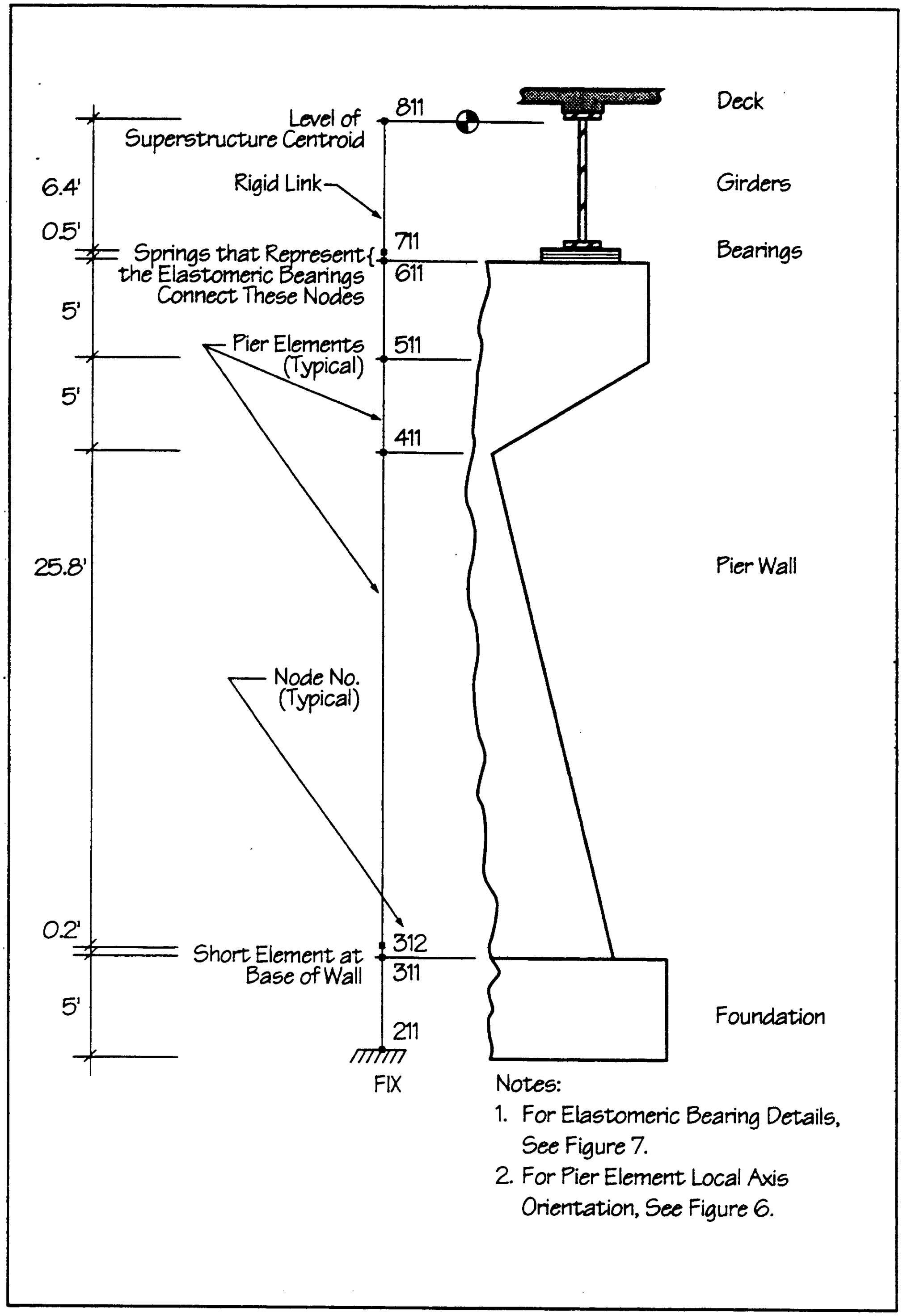


Figure 5 — Details of Column Elements

Design Step
6.1.3
(continued)

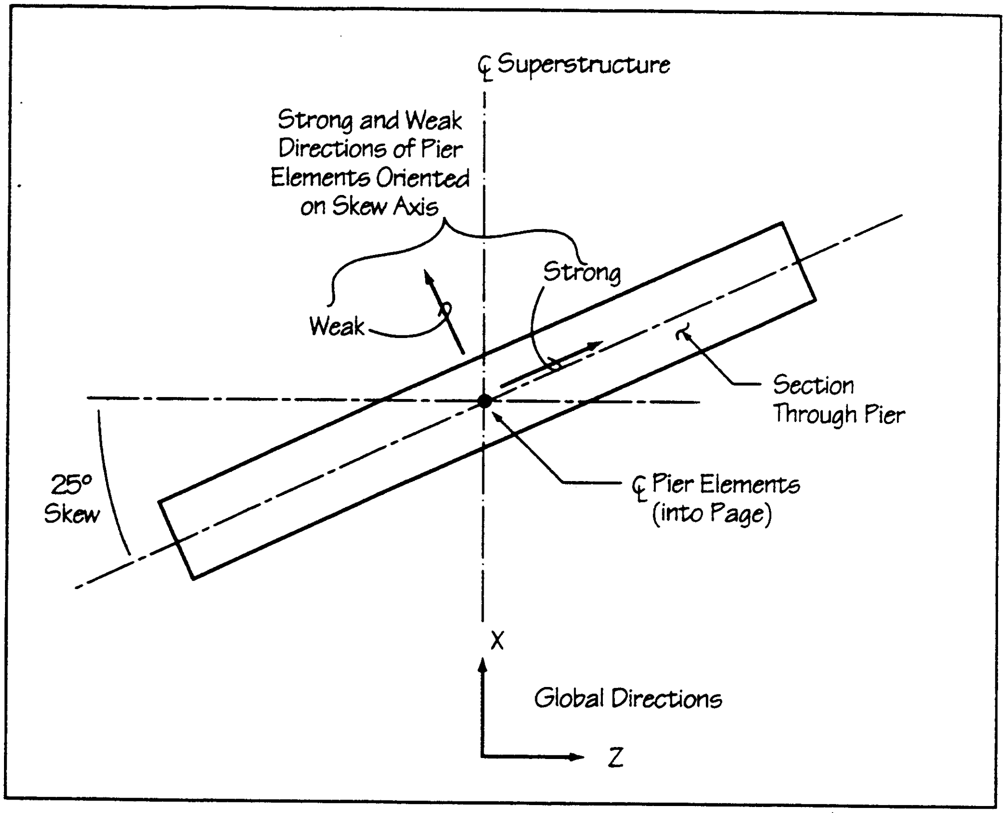


Figure 6 — Plan of Pier Showing Rotation of Pier Elements

The short element is needed because the inertial effects in the pier are modeled with masses lumped at the nodes. For the 26-foot-long part of the pier, roughly the lower half of the mass is lumped at the base of the pier at the top of the footing. Thus shear values that are output for the pier element just above the foundation do not include the inertial force of the lower part of the pier. The short element circumvents this problem, since nearly all the lower mass of the pier is lumped at the node just above this element. Thus, the shear in the short element includes the inertial effect of the lower part of the pier. An alternative to the short element is to space several nodes along the height of the pier. The problem of obtaining the proper shear force in the pier is more acute for the type of wall pier in this problem, where much of the wall mass is located near the base.

The stiffnesses of the pier foundations and the abutment foundations are as described in Design Step 6.2.

Design Step 6.1.4

Connection Elements — Elastomeric Bearings

a) Summary

The elastomeric bearing pads at the piers and at the abutments have been included in the model as linear springs. The superstructure is not restrained in either the longitudinal or transverse directions; thus springs are provided in all three translational directions. The orientation of the springs is shown in Figure 7, and the corresponding spring stiffnesses are summarized in Table 2. Rotational springs have been provided around the vertical axis and about an axis normal to the strong directions of the piers and abutments. Rotational releases have been provided around the axes parallel to the pier and abutment strong directions. Note that the spring stiffnesses are given in a local coordinate system that coincides with the strong and weak directions of the piers and abutments. These stiffnesses have been generated and input into SAP90 in this local system.

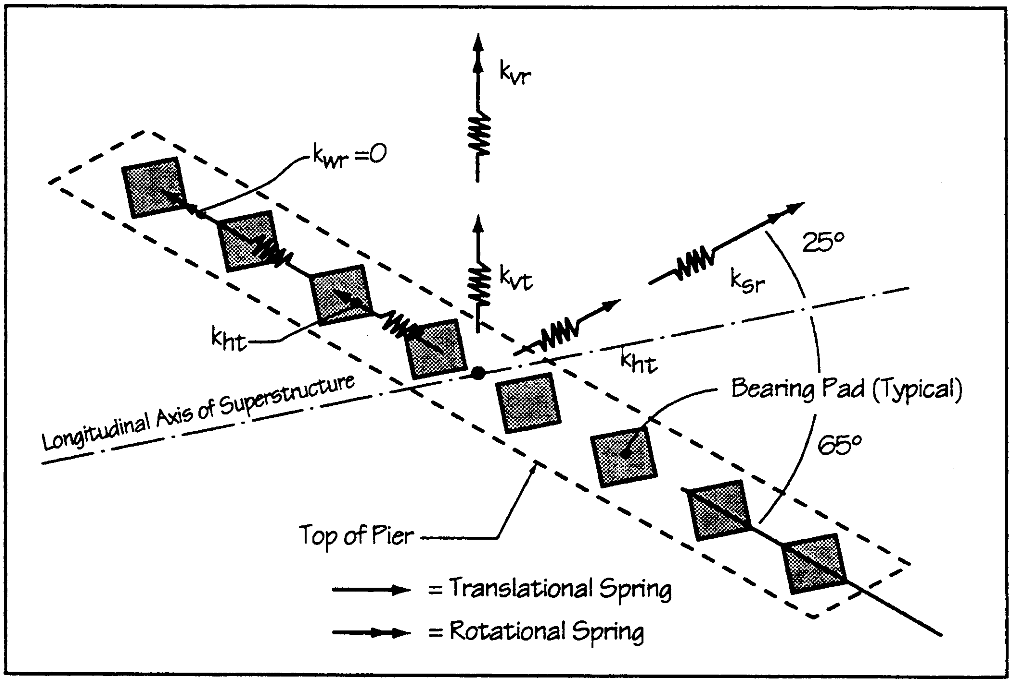


Figure 7 — Orientation of Bearing Springs

Design Step
6.1.4
(continued)

Table 2
Elastomeric Bearing Spring Constants

	Piers	Abutments
Plan Dimensions (Based on Bonded Area)	21 Inches Square	14 Inches Square
Elastomer Height	1.125 in. total (2 layers)	2.625 in. total (5 layers)
k ht (kip/ft) Horizontal Translation	4328	824
k vt (kip/ft) Vertical Translation	813,000	148,000
k vr (kip-ft/rad) Rotation About Vertical	1,840,000	350,000
k sr (kip-ft/rad) Rotation About Strong Axis	346,000,000	62,900,000
k wr (kip-ft/rad) Rotation About Weak Axis	0	0

The bearings have been designed to accommodate thermal movements. The elastomer is Durometer 50 Shore 'A' hardness, but is being specified in terms of its shear modulus Gb, which is 115 psi. The properties are established using the provisions given in Chapter 14 of AASHTO Division I.

Design Step 6.1.4 (continued)

The actual properties of the elastomer should be coordinated with bearing manufacturers. Additionally, the configuration and tolerances of the elastomer, reinforcing plates, and load plates should be coordinated with the manufacturer. Such communication with the suppliers helps insure that the final bearing configuration can be economically constructed and will behave as designed.

As shown in Table 2, the plan dimensions of the bonded area of the pier bearings are 21 inches on each side. The total thickness of the elastomer is 1.125 inches with one internal reinforcement plate and two equal thickness layers of elastomer. Likewise, the plan dimensions of the abutment bearings are 14 inches on each side. The total thickness of the elastomer is 2.625 inches.

It is recommended that the calculated stiffnesses should be based on the bonded area, which is the contact area between the elastomer and the reinforcing plates (1/8-inch steel laminae shown in Figure 1). This practice neglects the elastomer used as a protective cover around the perimeter of the bearing assembly. Because the cover elastomer is not reinforced, may be subject to some environmental deterioration due to exposure and may even be a different material. Depending on the construction of the bearing, it is conservative to neglect the cover.

b) Horizontal Translational Stiffness of Pier Bearings

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.

Note that the bearings pads are oriented square to the girders and not to the pier as shown in Figure 7. However, 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. The equivalence of translational stiffness in all directions also means that the rotational stiffness about the vertical axis can be calculated without regard for the individual bearing orientation. This calculation will be discussed in Section (c) following this section.

Assume:

$\Delta_{bp} := 1.0 \cdot in$ Unit deflection of bearing pad

Design Step 6.1.4 (continued)

Given:

$G_b := 115 \cdot psi$ — Shear modulus of elastomer

$A_{bp} := (21 \cdot in)^2$ — Area of each pier bearing pad

$h_{bp} := 1.125 \cdot in$ — Height of elastomer in pier bearing pads

Calculate the shear strain for a unit deflection.

$$\gamma_{bp} := \frac{\Delta_{bp}}{h_{bp}}$$

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

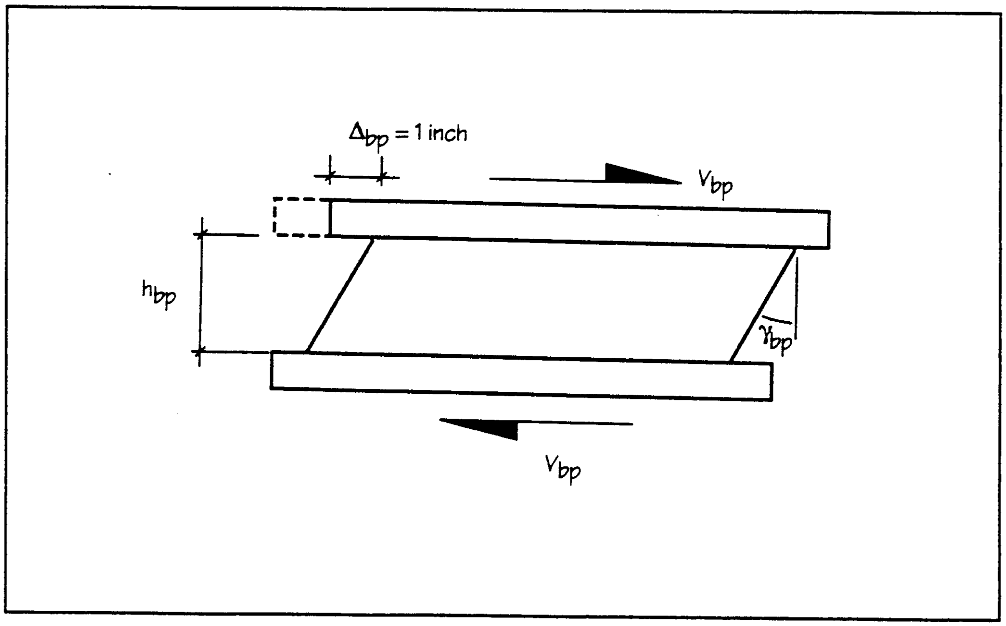


Figure 8 — Translational Deflection of Bearing Pad

Calculate the shear force.

$V_{bp} := v_{bp} \cdot A_{bp}$ — Shear force acting across bearing pad

Design Step 6.1.4 (continued)

Calculate the translational stiffness.

$$k_{trans} := \frac{V_{bp}}{\Delta_{bp}}$$ Translational stiffness of pier bearing pads

$$k_{trans} = 541 \cdot \frac{kip}{ft}$$

Then the total translational stiffness for all eight bearing pads is given by

$$K_{ht} := 8 \cdot k_{trans}$$

$$K_{ht} = 4328 \cdot \frac{kip}{ft}$$

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. This extends to the total translational stiffness at each pier as well.

c) Rotational Stiffness of Pier Bearings

The rotational stiffness of the bearings about the vertical axis (or torsion on the pier) is found by adding the individual bearing contributions when a unit rotation is applied to the entire group. See Figure 9. The bearings are assumed to be connected with a rigid link that transmits forces from the individual bearings to the point where the moment, which produces the unit rotation, is applied:

Design Step
6.1.4
(continued)

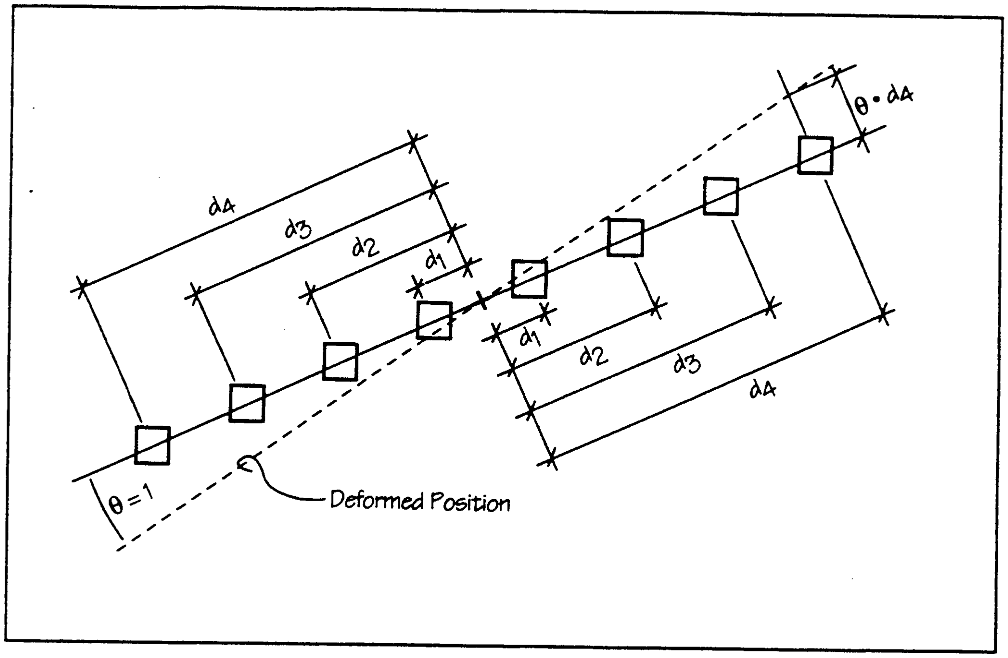


Figure 9 — Rotational Deflection of Bearing Pads

Assume:

$\theta := 1 \cdot rad$ Unit rotation

Given:

$d_1 := 4.5 \cdot ft$ $d_2 := 13.5 \cdot ft$

$d_3 := 22.5 \cdot ft$ $d_4 := 31.5 \cdot ft$

Distances from the center of the superstructure to the individual bearings

$n_{brg} := 4$ Number of bearings per side of centerline

Calculate the horizontal force acting at each pad.

$V_1 := k_{trans} \cdot \theta \cdot d_1$ $V_2 := k_{trans} \cdot \theta \cdot d_2$

$V_3 := k_{trans} \cdot \theta \cdot d_3$ $V_4 := k_{trans} \cdot \theta \cdot d_4$

Design Step 6.1.4 (continued)

Calculate the moment about the centerline produced by the force at each bearing.

$M_1 := V_1 \cdot d_1 \qquad M_2 := V_2 \cdot d_2$

$M_3 := V_3 \cdot d_3 \qquad M_4 := V_4 \cdot d_4$

Calculate the spring constant for rotation by summing these moments and dividing by the rotation.

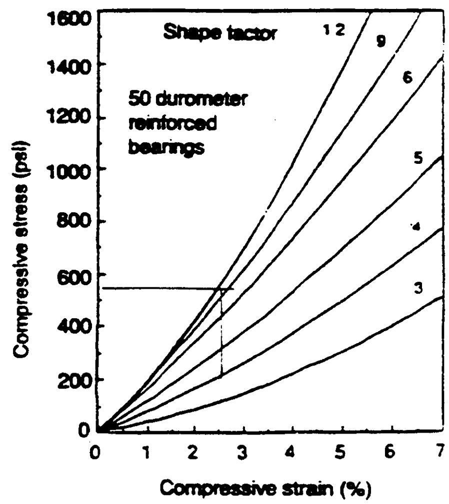
$$k_{vr} := \frac{2 \cdot \sum_{i=1}^{n_{brg}} M_i}{\theta} \qquad k_{vr} = 1.84 \cdot 10^6 \cdot \frac{kip \cdot ft}{rad}$$

d) Vertical Translational Stiffness of Pier Bearings

The vertical stiffness of the bearings and the rotational stiffness about an axis perpendicular to the pier strong axis can be found using the method outlined in Chapter 14 of AASHTO, Division I. These stiffnesses are relatively large and therefore provide nearly fixed conditions. An example calculation of the vertical stiffness for the pier bearings is shown below.

The stiffness is the standard axial stiffness 'AE/L,' but Young's modulus E is an equivalent linear stiffness based on Figure 14.4.1.2A of AASHTO Division I. This figure is reproduced here as Figure 10. Required for the calculation of E is the shape factor SF and the compressive stress on the bearing.

Design Step
6.1.4
(continued)



Taken from
AASHTO Division I
Section 14
Figure 14.4.1.2A

Figure 10 — Compressive Stress versus Strain for 50 Durometer Steel-Reinforced Bearings

Recall:

$h_{layer} := \frac{h_{bp}}{2}$ Height of elastomer layers

$h_{layer} = 0.563 \cdot in$

$L_{bp} := 21 \cdot in$ Length of bearing pad

$W_{bp} := 21 \cdot in$ Width of bearing pad

The weight of the superstructure is required to estimate the compressive stress.

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 cross frames/stiffeners

Design Step 6.1.4 (continued)

$w_{deck} := 8.16 \cdot \frac{kip}{ft}$ 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

$$W_{super} := \left(w_{misc} + w_{deck} + w_{girders}\right) \cdot L$$

$$W_{super} = 5540 \cdot kip$$

Estimate the vertical load at the piers.

$$W_{pier} := W_{super} \cdot \frac{\frac{124 \cdot ft}{2} + \frac{152 \cdot ft}{2}}{400 \cdot ft}$$

$$W_{pier} = 1911 \cdot kip$$

Calculate the compressive stress.

$$\sigma_{bp} := \frac{W_{pier}}{L_{bp} \cdot W_{bp}} \cdot \left(\frac{1}{8}\right) \qquad \sigma_{bp} = 542 \cdot psi$$

Calculate the shape factor SF as defined in Chapter 14 of Division I.

$$SF := \frac{L_{bp} \cdot W_{bp}}{2 \cdot h_{layer} \cdot \left(L_{bp} + W_{bp}\right)} \qquad SF = 9.3$$

From Figure 10, the compressive strain is

$$\varepsilon_c := 0.025$$

Design Step 6.1.4 (continued)

The equivalent Young's modulus is then

$$E := \frac{\sigma_{bp}}{\varepsilon_c} \qquad E = 21.67 \cdot ksi$$

The vertical stiffness can then be calculated as

$$k_{vert} := \frac{A_{bp} \cdot E}{h_{bp}} \qquad k_{vert} = 8495 \cdot \frac{kip}{in}$$

The total vertical stiffness for all eight pads is then

$$k_{vt} := 8 \cdot k_{vert} \qquad k_{vt} = 8.15 \cdot 10^{5} \cdot \frac{kip}{ft}$$

e) Rotation Releases

Rotation has been released about an axis parallel to the strong direction of the pier ($k_{wr} = 0$). Stiffness (or restraint) in this direction is considered negligible. The release is shown as a double-headed arrow in Figure 11(a).

The rotational release should be provided as shown and not perpendicular to the girder, since the minimum restraint occurs about the weak axis of the pier. This holds not only for elastomeric bearings, but also for conventional steel pin bearings that are oriented perpendicular to the girders. The reason is illustrated in Figures 11(b) and 11(c).

Figure 11(b) depicts an end view of the pier and its bearings looking along the skew. In this figure it is evident that the lever arm available for the bearing contact stresses to provide rotational restraint is relatively small and is limited by the plan dimensions of the bearings. In contrast, Figure 11(c) depicts the pier looking from the side of the bridge. Due to the skew a relatively large lever arm is available for contact stresses to develop rotational restraint. As a result, the vertical forces present in the outer bearings can develop a significant restraining moment about an axis transverse to the bridge.

Design Step
6.1.4
(continued)

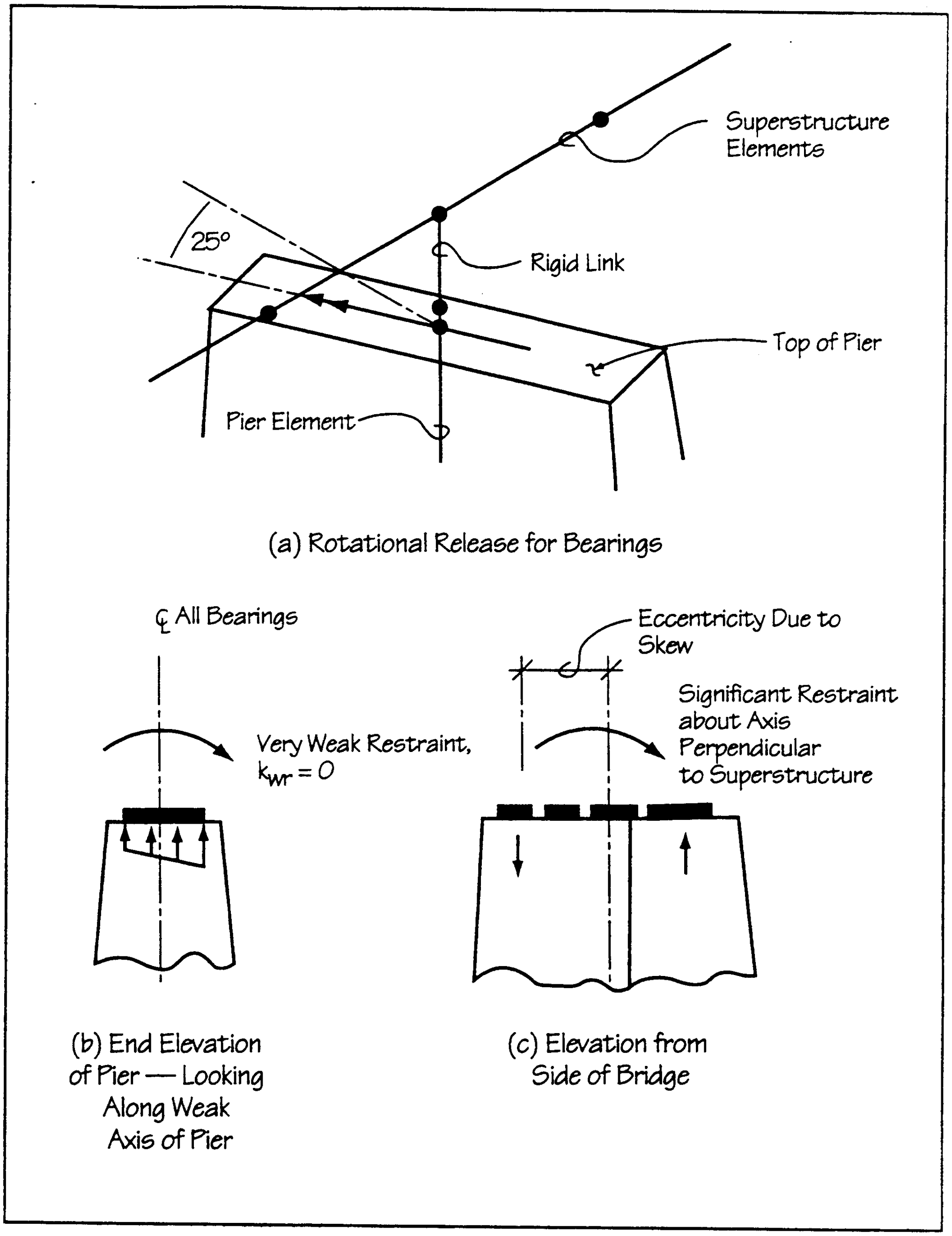


Figure 11 — Rotational Release for the Bearings

The springs used to represent the elastomeric bearings in SAP90 are used to resolve the restraint issues by using k_{wr} equal to zero and the relatively high stiffness value of k_{sr}. The orientation of k_{wr} and k_{sr} are shown in Figure 7.

Design Step 6.1.4 (continued)

The recommended release directions apply for the relatively large magnitudes of applied forces and displacements developed during large earthquakes. The releases also are based on composite action that will be effective for earthquake loading. The behavior of the bearings under dead and live load may be somewhat different than that described. In the case of this bridge with its elastomeric bearings, rotation is free to develop in any direction. For the case of conventional pin bearings that are oriented perpendicular to the girders, the rotation under dead and live load may be more nearly aligned with the rollers. However, for the larger magnitude earthquake movements, rotation will occur about the strong axis along the top of the pier.

The releases used in this example are simplifications of the actual movements and are required because a spine model (single line of elements) is used for the seismic analysis. If a more elaborate model such as a grid model had been used, then the actual support conditions could be more directly modeled. For instance, all the support points would be modeled and the rotational stiffness of the individual elastomeric bearings could be used. If conventional pins were used for the bearing elements, rotation would be released parallel to the pins. However, a grid model is not necessary for determining the seismic forces; thus the simplified modeling and simplified releases are used.

Design Step 6.2

Pier and Abutment Foundation Stiffnesses

Because the bridge is founded on rock, no attempt has been made to include foundation stiffness. The pier foundations have been considered fixed in all directions at the base of the footing. The abutments have also been considered fixed in all directions. Neglecting the abutment stiffness is reasonable because the elastomeric bearings at the abutments are much more flexible than the abutments, and the gap between the superstructure and the abutment is assumed not to close. This should be verified after the spectral analysis results are obtained, and the model adjusted if necessary.

Design Step 6.3

Multimode Spectral Analysis - General

Design Step 6.3.1

Mode Shapes and Periods
[Division I-A, Article 4.5.3]

The structure has been modeled using four elements per span and elements at each pier, as shown in Figures 4 and 5. Twenty-five vibration modes have been determined for use in the multimodal spectral analysis, which involves the superposition of individual modal responses to estimate the overall structural seismic response.

The SAP90 program (or any other dynamic spectral analysis program) will lump the tributary mass of each element to the adjacent nodes. Massless spring elements provide the flexibility introduced by the elastomeric bearing pads. SAP90 will determine 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 number of foundation restraints for the system. Enough modes need to be included in the modal superposition to ensure that the response of each significant structural element, particularly those with large mass, is captured.

The bearing pads' stiffness is significantly lower than that of either the superstructure or the piers. Thus the vibration periods associated with movement of the superstructure on the bearing pads are expected to be significantly longer than those corresponding to movement of the substructure elements. This results in a so-called "separation" of vibration periods.

Recall from the Preliminary Design Step that one reason for using the bearing pads was to reduce the response of the superstructure mass, thereby reducing the earthquake forces transmitted to the substructure. Such a reduction is accomplished by lengthening the fundamental vibration period, which reduces the induced inertial forces. This fact may be deduced from the response spectrum for seismic loading, which is discussed in the next section.

A potential difficulty with the separation of the vibration periods for movements of the superstructure and substructure is that more modes are required in the spectral analysis to properly estimate the structure forces. This is the result of the modes being developed in descending order of period magnitude. Consequently, less important modes such as vertical

Design Step
6.3.1
(continued)

modes of the superstructure occur between the more important lateral modes.

To reduce the number of modes required, the Ritz vector technique has been used. This feature is provided in SAP90, but it may not be provided in all spectral analysis programs. If it is not available, then more modes may be required in the analysis to obtain the same level of accuracy. A discussion of the technique has been given by Clough and Penzien (1993). The reason that Ritz vectors are more efficient than the exact natural vibration modes is that the location and direction of the inertial loading is considered in the Ritz vector determination. This important piece of information is not included in the determination of the natural vibration modes. By using Ritz vectors, vibration modes that are related to the deformations caused by seismic loading are determined. Thus for the same number of modes, the Ritz vector technique provides more meaningful information.

The natural periods of vibration for the bridge are shown in Table 3 for the first 25 Ritz vectors. The vibration shapes for the first three modes and Mode 10 are given in Figures 12, 13, 14, and 15. Modes 1 through 3 represent the vibration of the superstructure on the bearing pads; the first mode (Figure 12) is a rotational mode and the second and third are translational modes along the weak and strong axis of the piers, respectively (Figure 13 and 14). The 10th mode (Figure 15) is the first mode with significant translation of the piers. As seen in the figure, both of the pier tops are vibrating in the weak direction of the walls.

If the exact undamped natural mode shapes (the usual eigenvectors) had been used, 52 modes would have been required to obtain the same level of accuracy.

Design Step
6.3.1
(continued)

Table 3
Modal Periods and Frequencies for the First 25 Ritz Vectors

```
PROGRAM SAP90, VERSION BETA6.00                         FILE:p2ritzc2.OUT
 FHWA BRIDGE NO 2 PRELIMINARY DESIGN CALCS

 E I G E N V A L U E S   A N D   F R E Q U E N C I E S
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MODE	PERIOD (TIME)	FREQUENCY (CYC/TIME)	FREQUENCY (RAD/TIME)	EIGENVALUE (RAD/TIME)2	
1	0.874945	1.142929	7.181236	51.570154	*
2	0.868166	1.151853	7.237309	52.378637	**
3	0.813949	1.228578	7.719381	59.588839	***
4	0.569808	1.754976	11.026839	121.591174	
5	0.379822	2.632815	16.542467	273.653223	
6	0.372320	2.685858	16.875746	284.790800	
7	0.277193	3.607589	22.667150	513.799711	
8	0.141679	7.058230	44.348168	1966.760	
9	0.134212	7.450887	46.815301	2191.672	
10	0.133808	7.473370	46.956571	2204.920	****
11	0.108783	9.192608	57.758863	3336.086	
12	0.098867	10.114549	63.551587	4038.804	
13	0.074340	13.451767	84.519945	7143.621	
14	0.073492	13.606968	85.495102	7309.413	
15	0.054965	18.193527	114.313304	13067.532	
16	0.051327	19.483009	122.415356	14985.519	
17	0.038901	25.706297	161.517430	26087.880	
18	0.036907	27.094801	170.241654	28982.221	
19	0.027985	35.733611	224.520899	50409.634	
20	0.023442	42.658978	268.034262	71842.365	
21	0.020193	49.523089	311.162745	96822.254	
22	0.019280	51.866426	325.886364	106201.923	
23	0.006217	160.844682	1010.617	1.0213E+06	
24	0.006198	161.346816	1013.772	1.0277E+06	
25	0.005657	176.766678	1110.658	1.2336E+06	

* Rotation of Superstructure

** Translation in Pier Weak Direction

*** Translation in Pier Strong Direction

**** Pier Vibration in Weak Direction

Design Step 6.3.1 (continued)

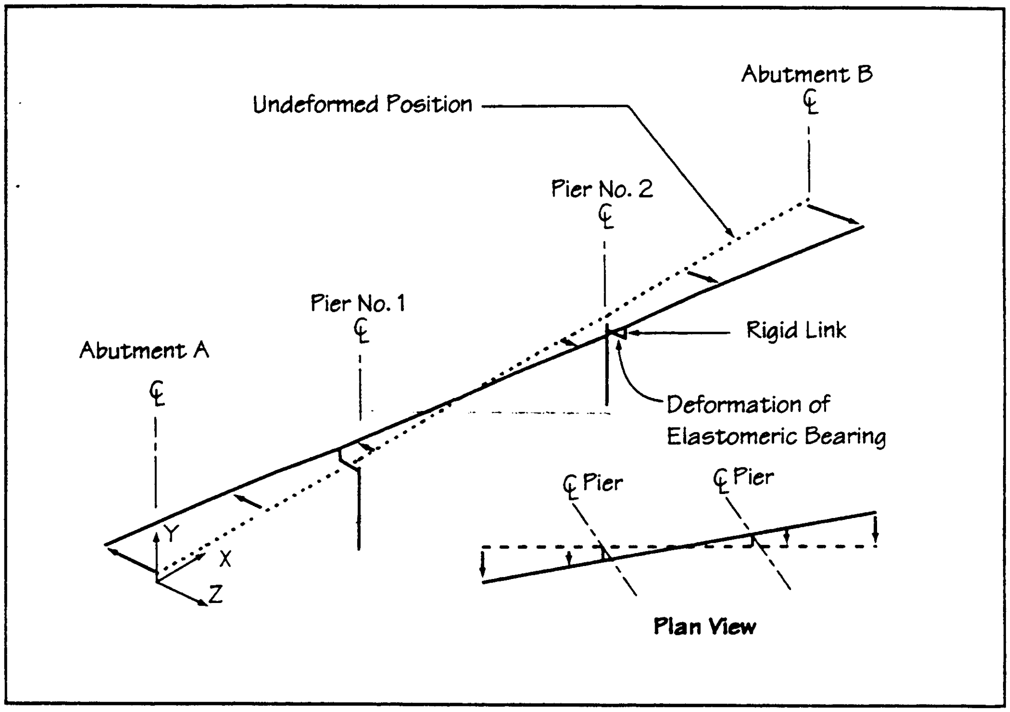

Figure 12 — Vibration Shape for Mode 1

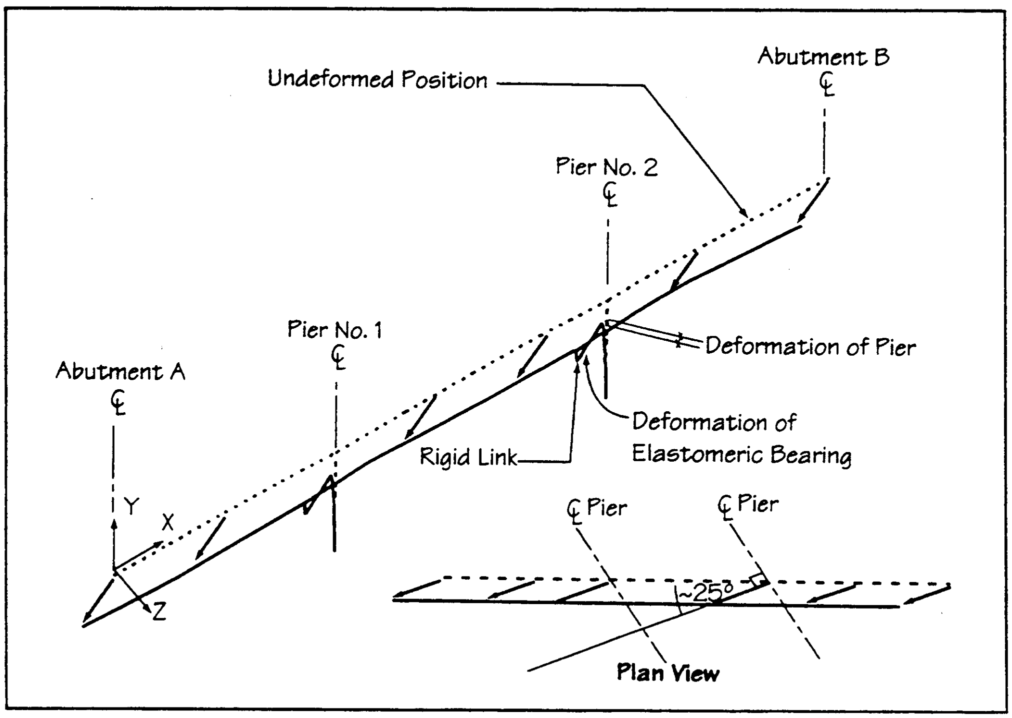

Figure 13 — Vibration Shape for Mode 2

Design Step
6.3.1
(continued)

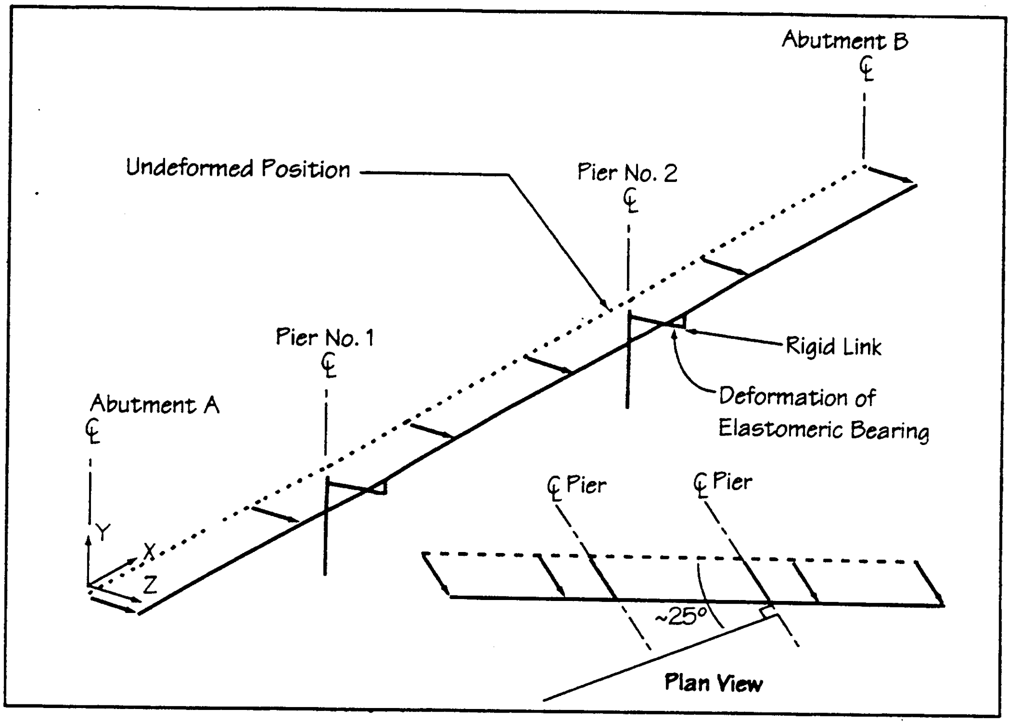

Figure 14 — Vibration Shape for Mode 3

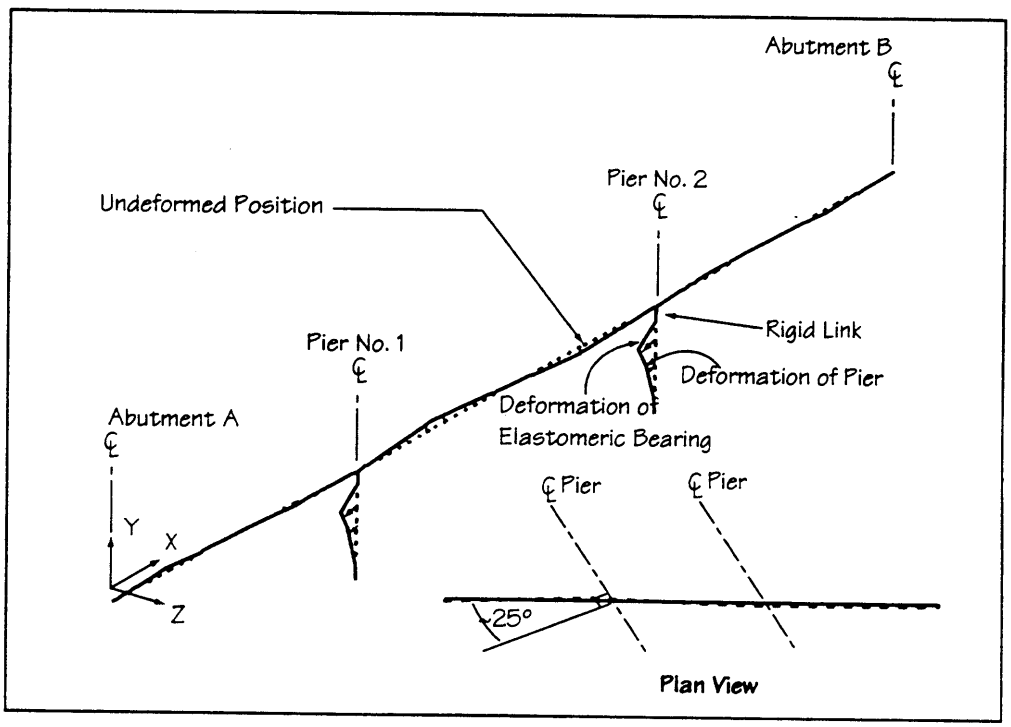

Figure 15 — Vibration Shape for Mode 10

Design Step
6.3.1
(continued)

Hand Check ✓ Check No. 1, Fundamental Period of Vibration — Translation in Pier Weak Direction

This check applies in the weak direction of the pier since the bearing stiffness is the same in all directions.

The mass of the superstructure is the primary mass moving in this case.

Recall the weight of the superstructure.

$$W_{super} = 5540 \cdot kip$$

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.

$$I_{wall} := \frac{60 \cdot ft \cdot (5.5 \cdot ft)^3}{12} \qquad I_{wall} = 832 \cdot ft^4$$

$$E := 519000 \cdot \frac{kip}{ft^2}$$

$$h_{wall} := 36 \cdot ft$$

$$K_{wall} := \frac{3 \cdot E \cdot I_{wall}}{h_{wall}^{\;3}} \qquad K_{wall} = 27761 \cdot \frac{kip}{ft}$$

Design Step 6.3.1 (continued)

Recall the pier bearing stiffness.

$$K_{ht} = 4.328 \cdot 10^{3} \cdot \frac{kip}{ft}$$

Combine the pier stiffness and pier bearing stiffness in series.

$$K_{pier} := \frac{1}{\left(\frac{1}{K_{ht}} + \frac{1}{K_{wall}}\right)} \qquad K_{pier} = 3744 \cdot \frac{kip}{ft}$$

Abutment bearing stiffness.

$$K_{aht} := 824 \cdot \frac{kip}{ft}$$

Total stiffness for two piers and two abutments.

$$K_{total} := 2 \cdot K_{aht} + 2 \cdot K_{pier} \qquad K_{total} = 9136 \cdot \frac{kip}{ft}$$

Calculate period.

$$T := 2 \cdot \pi \cdot \sqrt{\frac{W_{super}}{K_{total} \cdot g}} \qquad T = 0.863 \cdot sec$$

Compare this period with the period obtained from SAP90 for vibration in the weak direction of the pier.

$$T_{SAP} := 0.868 \cdot sec$$

$$\frac{T}{T_{SAP}} = 0.994$$ Quite close, say okay

Design Step 6.3.1 (continued)

Hand Check ✓ Check No. 2, Fundamental Period of Vibration — Translation in Pier Strong Direction

The only difference in this check and the previous check is that the pier stiffness is assumed to be infinite in the strong direction. Since the stiffness of the pads is the same in all directions, many of the values from the previous check also apply here.

Recall the weight of the superstructure.

$$W_{super} = 5540 \cdot kip$$

Recall the pier bearing stiffness.

$$K_{ht} = 4328 \cdot \frac{kip}{ft}$$

Recall the abutment bearing stiffness.

$$K_{aht} = 824 \cdot \frac{kip}{ft}$$

Total stiffness of the two pier and abutment bearings.

$$K_{total} := 2 \cdot K_{ht} + 2 \cdot K_{aht} \qquad K_{total} = 10303 \cdot \frac{kip}{ft}$$

The period of the structure vibrating in the strong direction of the pier.

$$T := 2 \cdot \pi \cdot \sqrt{\frac{W_{super}}{K_{total} \cdot g}} \qquad T = 0.812 \cdot sec$$

Compare this period with that obtained from SAP90.

$$T_{SAP} := 0.814 \cdot sec$$

Design Step 6.3.1 (continued)

$$\frac{T}{T_{SAP}} = 0.998$$ Quite close, say okay

Both this check and the previous check provided periods essentially equal to the SAP90 value. This should be the case because nearly all the deflection occurs in the bearing pads.

Hand Check ✓ Check No. 3, Vibration Period for Rotation of Superstructure

Because the superstructure of this bridge is free to move in any direction, a rotational mode of vibration exists. For this structure, this particular mode has the longest period; thus it is Mode 1. The shape of this mode is shown in Figure 12. The period of this mode is easily verified, and this check is shown below.

The period can be found using the rotational stiffness of the superstructure resting on the elastomeric bearings and the mass moment of inertia of the superstructure. Neglect the flexibility of the superstructure.

Calculate the rotational stiffness as shown in Figure 16.

Assume:

$$\theta := 1.0 \cdot \text{rad}$$

Recall the pier and abutment translational stiffnesses.

$$k_{ht} := 4328 \cdot \frac{\text{kip}}{\text{ft}} \qquad k_{aht} := 824 \cdot \frac{\text{kip}}{\text{ft}}$$

Calculate the lateral forces at the piers and abutments.

$$P_{ht} := k_{ht} \cdot (76 \cdot \text{ft}) \cdot \theta \qquad P_{aht} := k_{aht} \cdot (200 \cdot \text{ft}) \cdot \theta$$

Calculate the moment that the pier and abutment forces cause about the center of rotation.

$$M_{cr} := 2 \cdot \left(P_{ht} \cdot 76 \cdot \text{ft} + P_{aht} \cdot 200 \cdot \text{ft}\right)$$

Design Step
6.3.1
(continued)

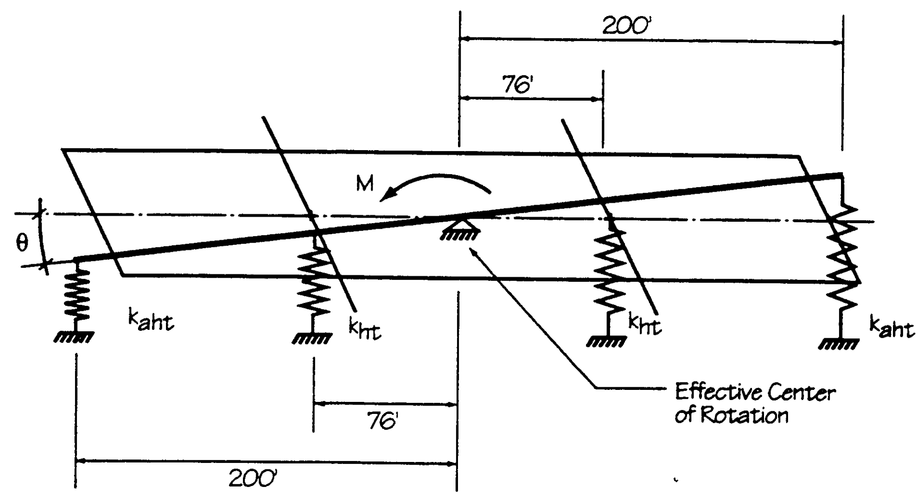

Figure 16 — Rotational Stiffness Calculation

Calculate the rotational stiffness.

$$k_{rot} := \frac{M_{cr}}{\theta} \qquad k_{rot} = 1.159 \cdot 10^{8} \cdot \frac{kip \cdot ft}{rad}$$

Calculate the mass moment of inertia for the superstructure. Assume that the superstructure is a bar with its mass (weight) distributed uniformly along its length. The mass moment of inertia can be found in most dynamics texts, for example Clough and Penzien (1993).

Recall:

$$W_{super} = 5540 \cdot kip \qquad L = 400 \cdot ft$$

$$I_o := \frac{W_{super}}{g} \cdot \frac{L^2}{12} \qquad I_o = 2.296 \cdot 10^{6} \cdot ft \cdot kip \cdot sec^2$$

Design Step 6.3.1 (continued)

The rotational period of vibration is then

$$T := 2\cdot\pi\cdot\sqrt{\frac{I_o}{k_{rot}}} \qquad T = 0.884\cdot sec$$

Compare this period with that from SAP90.

$$T_{SAP} := 0.875\cdot sec$$

$$\frac{T}{T_{SAP}} = 1.011 \qquad \text{Very close}$$

Design Step 6.3.2

Spectral Loading
[Division I-A, Article 3.6.2]

The input response spectra for this bridge is shown in Figure 17. The curve shown in the figure is given by the equation for C_{sm} shown below.

$$C_{sm}\left(T_m\right) := \frac{1.2\cdot A\cdot S}{T_m^{\frac{2}{3}}} < 2.5\cdot A \qquad \text{Division I-A Eqn (3-2)}$$

Where:
A is the acceleration coefficient
S is the site coefficient
T_m is the period

A design response spectrum must be input to provide loading for the models. This spectrum is specified in Section 3.6.2 of Division I-A, and it applies in both the transverse and longitudinal directions. Equation (3-2) of Division I-A and the equation's corresponding upper limit of two and a half times A effectively define the spectrum as a function of period T. This can be seen in Figure 17 for this bridge. Most programs will require period-spectrum data pairs to be input. Thus, the user must calculate the C_{sm} values that will define a smooth function within the analysis software.

Design Step 6.3.2 (continued)

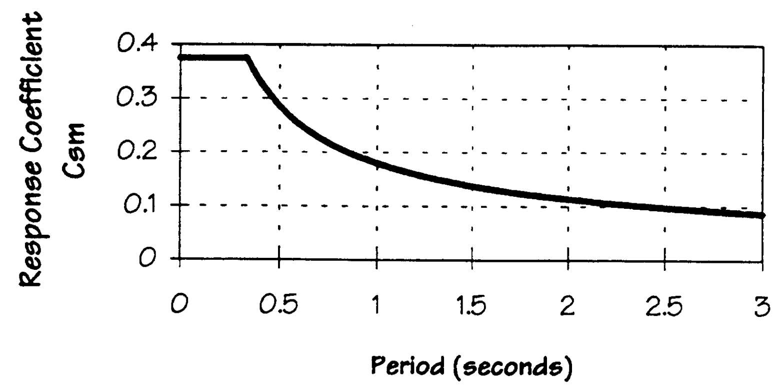

Figure 17 — Relationship Between Elastic Seismic Response Coefficient and Period

The range used must cover the entire range of expected periods for the structure.

Figure 17 and Equation 3-2 in AASHTO Division I-A are based on 5 percent damping. The 5 percent damping value has evolved into the standard that is used for the spectral loading equations and curves reported in Division I-A. The commentary gives some advice for obtaining spectral loading values for damping values not equal to 5 percent. The 5 percent damping value may be assumed to apply to the elastomeric bearings used, since no additional damping devices, such as lead cores, are included in the bearings. The damping values for the elastomeric bearings are primarily a material property, and therefore more accurate values should be obtained from the elastomer manufacturer. If a target damping value is important for the design of a particular structure, then the damping required of the elastomer should be coordinated with the supplier.

Design Step 6.3.3

Minimum Number of Modes
[Division I-A, Article 4.5.4]

Twenty-five modes have been included to provide an accurate estimate of the response and internal forces.

Design Step 6.3.3 (continued)

As mentioned above, most dynamic response can be adequately characterized without using all the vibration modes. The minimum number of modes required is specified in Division I-A, Article 4.5.4 as three times the number of spans, which in this case is nine modes. Due to the separation of modes described in Section 6.1.2, nine modes is not sufficient to estimate the response.

Table 4 lists the participating mass of the 25 Ritz vectors used as mode shapes.

As can be seen in the table, all 25 modes are necessary to obtain more than 90 percent mass participation in the two horizontal directions. It is apparent that several modes above mode 9 have significant participating mass, and in fact, if only nine modes were considered, the mass in the "global x" direction would be just 60 percent and in the "global z" direction only 57 percent. The low mass participation with only nine modes is due to the combination of heavy piers, in which roughly half the total mass of the bridge is located, and the high stiffness of the piers. Since the mass of the piers is associated with the higher modes, the error that would be introduced if the mass were neglected would primarily affect only the shears in the lower part of the walls and in the foundations.

Design Step
6.3.3
(continued)

Table 4
Participating Mass

PROGRAM SAP90, VERSION BETA6.00 FILE:p2ritzc2.OUT
FHWA BRIDGE NO 2 PRELIMINARY DESIGN CALCS

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

MODE	PERIOD	INDIVIDUAL MODE (PERCENT) UX	UY	UZ	CUMULATIVE SUM (PERCENT) UX	UY	UZ
1	0.874945	0.0000	0.0002	0.0000	0.0000	0.0002	0.0000
2	0.868166	48.7517	0.0000	11.2914	48.7517	0.0002	11.2914
3	0.813949	10.7023	0.0000	45.9236	59.4540	0.0002	57.2150
4	0.569808	0.0000	0.2103	0.0000	59.4540	0.2105	57.2150
5	0.379822	0.0001	0.0000	0.0730	59.4540	0.2105	57.2881
6	0.372320	0.0896	0.0000	0.0214	59.5437	0.2105	57.3095
7	0.277193	0.0000	37.8819	0.0000	59.5437	38.0924	57.3095
8	0.141679	0.6411	0.0000	0.1406	60.1847	38.0924	57.4501
9	0.134212	0.0000	0.0017	0.0000	60.1847	38.0941	57.4501
10	0.133808	12.9750	0.0000	2.8174	73.1598	38.0941	60.2675
11	0.108783	0.0000	0.8221	0.0000	73.1598	38.9162	60.2675
12	0.098867	0.0002	0.0000	0.0000	73.1599	38.9162	60.2676
13	0.074340	0.0000	3.0189	0.0000	73.1599	41.9351	60.2676
14	0.073492	0.0005	0.0000	0.0000	73.1605	41.9351	60.2676
15	0.054965	0.0000	0.0000	0.0000	73.1605	41.9351	60.2676
16	0.051327	0.0000	8.4374	0.0000	73.1605	50.3725	60.2676
17	0.038901	0.0000	0.0000	0.0001	73.1605	50.3725	60.2676
18	0.036907	0.0000	0.0000	0.0000	73.1605	50.3726	60.2676
19	0.027985	4.0423	0.0000	18.6019	77.2028	50.3726	78.8695
20	0.023442	0.0000	23.7160	0.0000	77.2028	74.0886	78.8695
21	0.020193	0.8574	0.0000	0.1843	78.0602	74.0886	79.0538
22	0.019280	0.0001	0.0002	0.0028	78.0603	74.0887	79.0566
23	0.006217	18.2668	0.0000	3.9236	96.3271	74.0887	82.9802
24	0.006198	0.6626	0.0003	3.2768	96.9897	74.0891	86.2570
25	0.005657	2.9847	0.0001	13.7171	99.9745	74.0892	99.9741

T O T A L   U N R E S T R A I N E D   M A S S   A N D   L O C A T I O N

DIRECTION	MASS	X	Y	Z
UX	302.354508	200.000000	-14.757525	.000000
UY	302.354508	200.000000	-14.757525	.000000
UZ	302.354508	200.000000	-14.757525	.000000

Design Step
6.3.4

Combination of Modes
[Division I-A, Article 4.5.5]

The Complete Quadratic Combination (CQC) Technique has been used to combine the modal results.

This combination accounts for the fact that the maximum response for each mode does not occur simultaneously. It also accounts for the potential coupling that can occur when two or more modes have essentially the same period.

Design Step 6.4 Determine Forces and Displacements in Transverse Direction

[Division I-A, Article 4.5]

Design Step 6.4.1 Results

The Multimode Spectral Method of analysis, Procedure 3, was used to determine the internal seismic forces, reactions, and displacements for earthquake loading parallel to the transverse axis of the bridge, "global z" direction.

The results of the analysis are given in Tables 5 and 6. Figure 18 provides a key to the force directions, and Figure 19 provides a key to the displacement directions listed in Table 6.

Table 5
Transverse Earthquake Forces and Moments

Support/Location		Transverse Earthquake Forces and Moments on Substructure: Weak Direction of Pier Shear (kips)	Weak Direction of Pier Moment (kip-ft)	Strong Direction of Pier Shear (kips)	Strong Direction of Pier Moment (kip-ft)	Axial (kips)
Abutment A - Bearings		46	0	88	9	8
Pier No. 1	Bearings	193	0	426	3,520	17
	Base of Wall	259	8,084	599	21,589	18
	Foundation	360	9,303	806	24,390	18
Pier No. 2	Bearings	193	0	426	3,520	17
	Base of Wall	259	8,085	599	21,589	18
	Foundation	360	9,305	806	24,390	18
Abutment B - Bearings		46	0	88	9	8

Design Step 6.4.1 (continued)

Table 6
Displacements for Transverse Earthquake (Feet)

Location	Abutment A		Pier No. 1		Pier No. 2		Abutment B	
Direction	Long	Trans	Long	Trans	Long	Trans	Long	Trans
Superstructure (Absolute)	0.036	0.115	0.036	0.106	0.036	0.106	0.036	0.115
Direction	Weak	Strong	Weak	Strong	Weak	Strong	Weak	Strong
Bearings (Relative across brig)	0.056	0.107	0.044	0.098	0.044	0.098	0.056	0.107
Substructure (Absolute, top of pier)	NA	NA	0.0078	0.0005	0.0078	0.0005	NA	NA

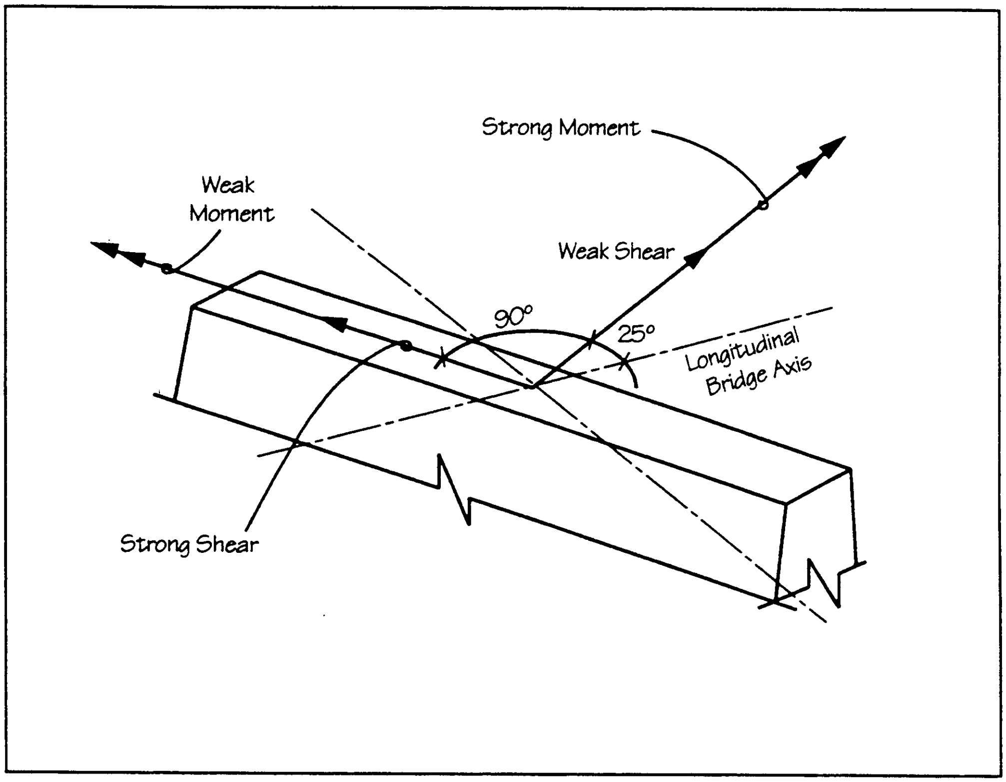

Figure 18 — Key to Force and Moment Directions

Design Step 6.4.1 (continued)

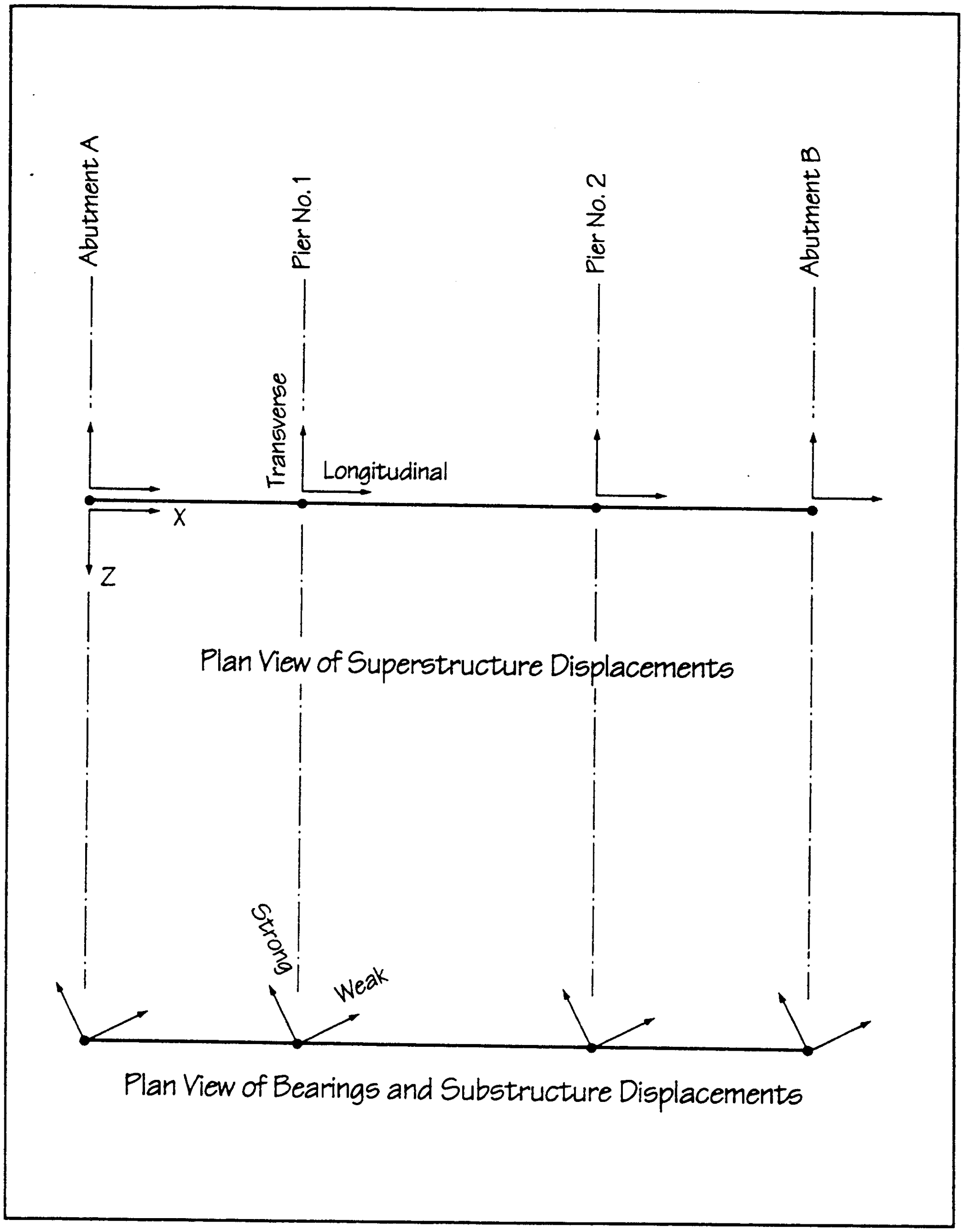

Figure 19 — Key to Displacement Directions

Design Step 6.4.2

Discussion of Foundation Shear Forces

The foundation shear forces listed in Table 5 are the sum of the modal results and a hand-calculated force that represents the inertial effect of the mass of the lower half of the footing. This extra force represents the inertial force that the modal analysis neglects since the bottom node of the model is fixed in all directions. See Figure 20.

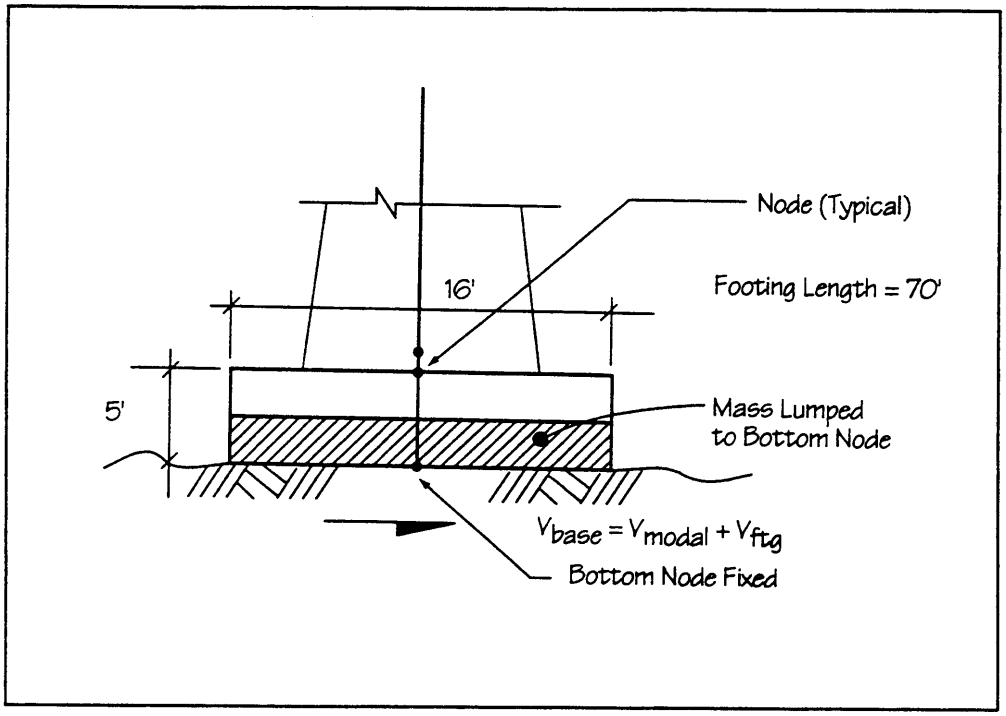

Figure 20 — Distribution of Mass to Footing Nodes

In the modal analysis tributary mass is lumped at the nearest node, and if that node is fixed, the inertial effect is lost. The fact that the analysis neglects this mass is evident upon comparison of the total mass of the structure, 328 kip sec^2/ft, and the total unrestrained mass listed in Table 4, 302 kip sec^2/ft. The difference is 26 kip sec^2/ft, which when multiplied by g and divided by two (because there are two piers) equals exactly half the footing weight, 420 kips.

In this example this force must be added since the lateral resistance provided by the ground is entirely due to the friction across the bottom of the footing. No soil is available to provide passive resistance against the side of the footing.

Design Step
6.4.2
(continued)

When footings are buried in the soil, their mass is often neglected in the seismic analysis. This is because only the inertial forces above those of the displaced soil need to be considered. These forces are due to the difference in the mass density of the soil and the footing concrete. Typically, the difference in density is small; thus the effects are often ignored.

The weight of the lower half of the footing is calculated first.

$H_{ftg} := \frac{5 \cdot ft}{2}$ Height of half of the footing

$W_{ftg} := 16 \cdot ft$ Width of footing

$L_{ftg} := 70 \cdot ft$ Length of footing

$\gamma_c := 0.150 \cdot \frac{kip}{ft^3}$ Density of concrete

$Wt_{ftg} := H_{ftg} \cdot W_{ftg} \cdot L_{ftg} \cdot \gamma_c$ $Wt_{ftg} = 420 \cdot kip$

Calculate the inertial force corresponding to this weight, given that the 2.5A amplification inherent in the Division I-A spectrum is applied.

$A := 0.15$ Acceleration coefficient

$V_{ftg} := Wt_{ftg} \cdot 2.5 \cdot A$ $V_{ftg} = 157.5 \cdot kip$

Resolve this force into the weak and strong directions of the footing, given that the earthquake acts in the transverse direction.

$S := 25 \cdot deg$ Skew angle

$V_{ftg\_w} := V_{ftg} \cdot \sin(S)$ $V_{ftg\_w} = 67 \cdot kip$

$V_{ftg\_s} := V_{ftg} \cdot \cos(S)$ $V_{ftg\_s} = 143 \cdot kip$

Design Step 6.4.2 (continued)

As shown in Figure 21, the foundation shear forces may be combined in the weak and strong directions of the pier. This process has been applied to both sets of foundation shear forces in Table 5.

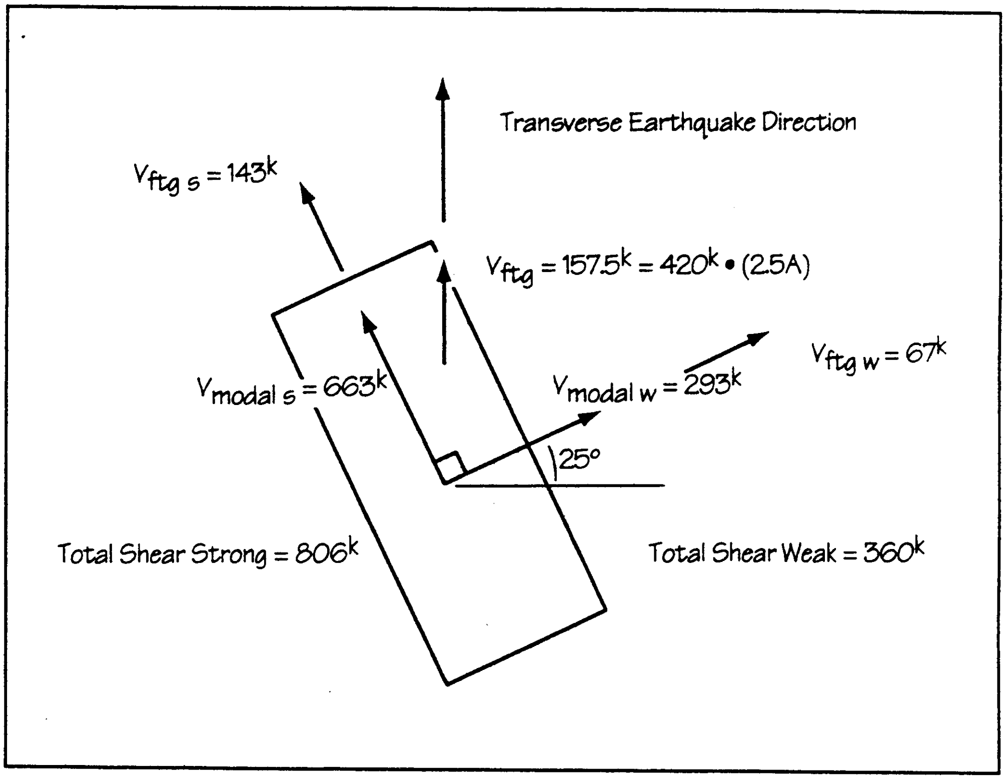

Figure 21 — Details of Foundation Shear Force

Design Step 6.5

Determine Forces and Displacements in Longitudinal Direction
[Division I-A, Article 4.5]

The Multimode Spectral Method of analysis was used to determine the internal seismic forces, reactions, and displacements for earthquake loading parallel to the longitudinal axis of the bridge, global x direction.

The results of the analysis are given in Tables 7 and 8. Again, Figure 18 provides a key to the force directions, and Figure 19 provides a key to the displacement directions. The same additional inertial force that was added to the transverse direction results was added to the longitudinal results.

Table 7
Longitudinal Earthquake Forces and Moments

Support/Location		Longitudinal Earthquake Forces and Moments on Substructure: Weak Direction of Pier: Shear (kips)	Weak Direction of Pier: Moment (kip-ft)	Strong Direction of Pier: Shear (kips)	Strong Direction of Pier: Moment (kip-ft)	Axial (kips)
Abutment A - Bearings		93	0	41	6	18
Pier No. 1	Bearings	415	0	202	1,104	37
	Base of Wall	556	17,364	282	9,240	37
	Foundation	771	19,983	378	10,569	37
Pier No. 2	Bearings	415	0	202	1,104	37
	Base of Wall	556	17,367	282	9,239	37
	Foundation	771	19,987	378	10,567	37
Abutment B - Bearings		93	0	41	6	18

Design Step 6.5 (continued)

Table 8
Displacements for Longitudinal Earthquake (Feet)

Location	Abutment A		Pier No. 1		Pier No. 2		Abutment B	
Direction	Long.	Trans.	Long.	Trans.	Long.	Trans.	Long.	Trans.
Superstructure (Absolute)	0.118	0.038	0.117	0.036	0.117	0.036	0.118	0.038
Direction	Weak	Strong	Weak	Strong	Weak	Strong	Weak	Strong
Bearings (Relative across brgs.)	0.113	0.049	0.095	0.047	0.095	0.047	0.113	0.049
Substructure (Absolute, top of pier)	NA	NA	0.017	0.0002	0.017	0.0002	NA	NA

Hand Check ✓ Check of Modal Analysis Forces and Discussion of Obtained Forces

a) Calculation of Longitudinal Base Shear by Hand

In general it is a good idea to check the forces obtained from the modal analyses by hand if possible. In this example, the presence of the elastomeric bearings produces vibration modes associated with the superstructure moving as a rigid body on the bearings and modes of the substructure (piers) moving independently as cantilevers. The periods of vibration of these two cases are well separated, since the pier modes have relatively short periods. Due to their short periods, the pier modes all have the same amplification factor per the Division I-A design equations, and this should make the hand check simple to perform. For this example there are some issues that complicate the simple check and affect the validity of the modal results. These are discussed in this section.

The first item required for the check is a force from the modal analysis that can be used as an indicator of the total response. In this case, the base shear developed over the bottom of the footing was selected. This value is used since all of the pier shear must be transferred between the footing and the rock below it. Any soil or loose rock around the footing is neglected since the footing would fail by sliding before any significant passive resistance from the soil could be developed.

Design Step 6.5 (continued)

The longitudinal earthquake loading case is used for the check. From Table 7 the base shear values in the weak and strong directions of the pier are 771 kips and 378 kips, respectively. The resultant of these two forces is 859 kips. This is the base shear value used for the check.

This shear resultant can be checked by calculating the force transferred through the bearings to the pier and adding the inertial force of the pier obtained by considering it a rigid body. This assumes that the superstructure and the bearings act as a single-degree-of-freedom system, which responds to the earthquake loading transmitted through rigid pier elements.

The hand check process is outlined below.

<u>Step 1.</u> Calculate the force transferred through the bearings.

First, calculate the loading coefficient for the superstructure.

Recall that the period of vibration in the weak direction was 0.863 second from the hand check of the periods. Then from Division I-A Equation 3-2

$A := 0.15$ Acceleration coefficient

$S := 1.0$ Soil coefficient

$T := 0.863$ Vibration period

$$C_{sm} := \frac{1.2 \cdot A \cdot S}{T^{\frac{2}{3}}} \qquad C_{sm} = 0.199$$

This is less than the cap value of 2.5A.

$2.5 \cdot A = 0.375$ Cap value for C_{sm}

Recall the weight of the superstructure and the stiffness of the bearings.

$W_{super} := 5540 \cdot kip$

$k_{pier} := 3744 \cdot \frac{kip}{ft}$ $\qquad k_{total} := 9136 \cdot \frac{kip}{ft}$

$k_{abut} := 824 \cdot \frac{kip}{ft}$

Design Step 6.5 (continued)

Calculate the total seismic deflection.

$$\Delta_s := \frac{W_{super} \cdot C_{sm}}{k_{total}} \qquad \Delta_s = 0.120 \cdot ft$$

This is very nearly equal to the 0.118 ft value listed in Table 8.

Force transmitted to the pier.

$$V_{brg} := \Delta_s \cdot k_{pier} \qquad V_{brg} = 451 \cdot kip$$

<u>Step 2.</u> Calculate the inertial force of the pier.

$W_{pier} := 2535 \cdot kip$ — Weight of pier, including the entire footing (taken from SAP results, or calculated from the volume of concrete)

$$V_{pier} := W_{pier} \cdot 2.5 \cdot A \qquad V_{pier} = 951 \cdot kip$$

To be consistent with the SAP90 input, an acceleration amplification of 2.5 was used for the pier. This was derived from Figure 17, which gives 2.5 times A for all periods less than 0.33 second. Considering the periods given in Table 4, all modes greater than Mode 6 should have the 2.5 amplification factor. Mode 10 is the first significant substructure mode.

<u>Step 3.</u> Combine forces to get an estimate of the pier base shear.

$$V_{base} := V_{brg} + V_{pier} \qquad V_{base} = 1401 \cdot kip$$

<u>Step 4.</u> Compare with the SAP results.

When this value is compared with the 859-kip value obtained from SAP, it is evident that the hand value is quite high. This discrepancy and its causes are discussed at length below.

Design Step 6.5 (continued)

b) Discussion of Forces from the Hand Check and from SAP90

General. One reason that the hand value greatly exceeds the modal value is that a statistical combination of modal results, the complete quadratic combination (CQC) method has been used. For modes with widely separated periods, such as the ones for this bridge in the principal directions, the CQC method reduces to a square root of the sum of the squares (SRSS) combination. Obviously a SRSS combination will give a somewhat lower value than will a direct summation, which is what the calculation in Step 3 above assumes.

The combination of modal results is one of two issues that complicates the simple hand check. The other issue is the shape of the spectral loading curve input into SAP90. These two issues are discussed below.

Issue 1 — Combination of the Modal Results. The SRSS or CQC methods are based on the modal responses each being amplified above the input ground motion, and thus the maximum response for each of the modes typically occurs at different times during the earthquake. In very stiff systems, the acceleration response in each of the very stiff modes is almost identical to the input earthquake ground accelerations, and thus the maximum response of each mode occurs almost simultaneously. The response is rather like that of a rigid block or brick that undergoes the same motion that the ground does. For such conditions, the SRSS or CQC combination underestimates the contributions from these very stiff modes. To compensate for this, a summation of the absolute value of each modal contribution can be used.

In this example, the pier modes would ideally be summed to give the rigid block response and then this response combined with the superstructure modes using CQC or SRSS methods. However, most programs do not allow such hybridization of the combination scheme, where some modal responses are summed and some are combined with the CQC method.

To match the hand check value obtained in Step 3, a SAP run was made using an absolute summation of modal responses. This run gave 1378 kips of base shear at each of the piers. This value is very close to the hand value of 1401 kips calculated above, and it is much larger than the 859-kip value obtained using a CQC combination. This illustrates the large effect that the combination method has on nearly rigid structures.

Design Step 6.5 (continued)

Issue 2 — Shape of the Input Spectrum Curve. Another issue that complicates the checking process is the spectral amplification that is applied to the very stiff modes. Spectral amplification refers to the increased acceleration that a structure may experience as a result of dynamic loading.

The relationship between amplification and period in the stiff period range may be seen in Figure 22. This figure shows the amplification used in Division I-A, Equation 3-2, which is shown by the horizontal line at 2.5. The figure also shows the smoothed spectral amplification line on which Equation 3-2 is based. This line is taken from Figure 10 of the Division I-A Commentary. Additionally, Newmark and Hall (1982) gave general rules for constructing amplification curves, and recommended beginning amplification at 33 hertz or 0.03 second (i.e., no amplification exists below a period of about 0.03 second). Their suggested relationship is also shown for very low periods.

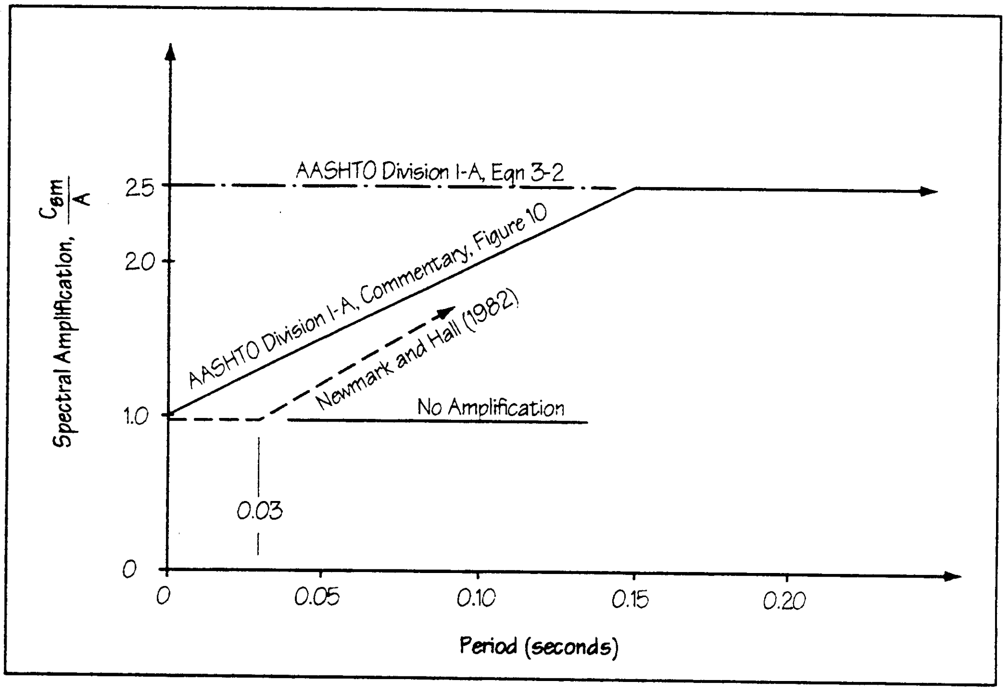

Figure 22 — Spectral Amplification versus Period

In setting up AASHTO's design requirements, it was decided that the amplification ascent from 1.0 to 2.5 could be conservatively replaced with a constant 2.5 value. The concern was that inelastic action, which will elongate the fundamental vibration period, could "drag" the response up the spectral loading curve. However, for a structure that responds

Design Step 6.5 (continued)

rigidly and may not yield at all, such as the piers in this bridge, this represents a 2.5 times increase in loading over what the structure may actually experience.

To illustrate the effect of the ascending amplification on the base shear, the curve given in Figure 10 of the Division I-A Commentary was used to define an input spectrum. SAP was then run and the modes again combined using an absolute summation. The base shear for the piers was 987 kips, which is substantially less than the 1378-kip value that was obtained using the full 2.5 times amplification.

978?.

The 978-kip value can be compared with a hand value based on no amplification. The 978-kip ascending amplification value should lie between the no amplification hand value and the 2.5 times amplification hand value (1401 kips).

Recalculate the base shear assuming no amplification of the pier response. Also, recall that the pier weight term includes the footing weight.

$V_{pier} := W_{pier} \cdot A \qquad V_{pier} = 380 \cdot kip$

Recall the shear through bearings.

$V_{brg} = 451 \cdot kip$

Calculate the base shear.

$V_{base} := V_{brg} + V_{pier} \qquad V_{base} = 831 \cdot kip$

It can be seen that the no amplification assumption produces a hand check value (831 kips) that is less than the modal ascending amplification value (978 kips). Additionally, the hand check value (831 kips) can be compared with the original multimodal analysis value (859 kips), and the conclusion is that the two numbers are remarkably close. However, these two numbers are based on entirely different input information, A for the hand value versus 2.5A for the multimodal value. Therefore, the 831-kip and 859-kip values are "apples" and "oranges" and should not be compared without recognizing this fact.

Design Step 6.5 (continued)

see pg 36

Impact of Issues 1 and 2. One purpose for performing these calculations is to gain confidence in the numbers that the spectral multimode analysis produces. There are then two questions to be answered by the checks and comparisons.

- Are there any evident numerical errors in the multimodal input?
- Are the multimodal results reasonable?

In this example it appears that no large numerical input problems exist, since the modal and hand check values are in the proper relation to one another. However, it is also evident that the modal combination method and the amplification factors have profound effects on the base shear results. In particular, the original modal result (859 kips) obtained by applying the Division I-A requirements is based on effects that offset one another; the CQC modal combination produces low shear contributions from the stiff piers and the 2.5 amplification factor produces increased shear contributions from the piers. The resulting base shear (859 kips) is, however, still larger than that calculated by hand assuming no amplification for the pier modes (831 kips). The hand value with no pier mode amplification can be thought of as a lower bound on the base shear, since the stiff piers will respond essentially as rigid blocks. Thus a reasonable "reality check" on the validity of the original modal results is: are they greater than the hand check with no amplification? In this example, they are; so the original modal results are reasonable.

The hand method with no pier amplification gives a base shear value (831 kips) that is less than the base shear from the original multimodal base shear (859 kips). Therefore, consider the modal forces adequate, and use them for design.

(c) Suggested Procedure for Handling this Modal Analysis Problem

Due to the offsetting effects of the modal combination method and the spectral amplification for very stiff structures, there is no guarantee that the modal values will be reasonable for every bridge with elastomeric bearings and very stiff piers. For this reason, the "reality check" made above is recommended. If the check exposes a modal result less than the hand value, two options are available.

- Scale the modal results up to correspond to at least the hand check value of base shear.

Design Step 6.5 (continued)

- Rerun the modal analysis using an absolute summation of modal results and the input spectrum amplification shown in Figure 10 of the Division I-A Commentary. This avoids the offsetting effects contained in the standard multimodal method for these type bridges. For this example, the base shear for this option would be (978 kips), which is a conservative value.

The main point to this discussion is: do not blindly trust the modal results. The designer should be aware of what effects various modal analysis options have on the final results. Once these effects are understood, the designer may then choose the level of conservatism that suits the situation at hand, and at the same time avoid having unconservative results.

DESIGN STEP 7 — DETERMINE DESIGN FORCES

Design Step 7.1 — Determine Nonseismic Forces

Design Step 7.1.1 — Determine Dead Load Forces

The dead load forces are summarized in Table 9 below. The forces shown in the table were determined using the same spine model of the bridge that was used for determining the seismic forces. The reported forces and moments are the totals for the connecting elements; thus, for instance, the bearings each support one-eighth of the forces shown because there are eight bearings at each pier.

Table 9
Dead Load Forces

Support/Location		Dead Load Forces and Moments on Substructure: Weak Direction of Pier — Shear (kips)	Weak Direction of Pier — Moment (kip-ft)	Strong Direction of Pier — Shear (kips)	Strong Direction of Pier — Moment (kip-ft)	Axial (kips)
Abutment A - Bearings		2	0	1	27	587
Pier No. 1	Bearings	7	0	1	65	2,165
	Base of Wall	7	270	1	98	3,858
	Foundation	7	307	1	103	4,698
Pier No. 2	Bearings	7	0	1	65	2,165
	Base of Wall	7	270	1	98	3,858
	Foundation	7	307	1	103	4,698
Abutment B - Bearings		2	0	1	27	587

It should be noted that the spine model may not give good estimates of the dead load forces, and for that reason a grid model is usually used to determine the dead load forces. This is particularly true for the superstructure of a skewed bridge. However, for the purposes of this example, the seismic model has also been used for the dead load analysis.

Design Step 7.2 Determine Seismic Forces

Design Step 7.2.1 Summary of Elastic Seismic Forces

As was discussed previously, the Multimode Spectral Method results are used to determine the modified design forces.

The full elastic seismic forces for earthquake loading along each of the principal axes (transverse and longitudinal) are shown in Tables 5 and 7, respectively.

Design Step 7.2.2 Combination of Orthogonal Seismic Forces [Division I-A, Article 3.9]

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 in load combinations LC1 and LC2 (without dead load).

The definition of LC1 and LC2 are as follows.

LC1 = 100 percent of the Longitudinal Analysis Results + 30 percent of the Transverse Analysis Results

LC2 = 30 percent of the Longitudinal Analysis Results + 100 percent of the Transverse Analysis Results

Note that all the forces in LC1 and LC2 are the full elastic seismic forces.

These forces are combinations using the full elastic seismic results and have not been modified by the R Factor yet. At this stage, the designer could elect to check for these forces combined with dead load, if other load cases such as stream flow control the size of the substructure.

For example, the weak axis shear across the bearings in Pier No. 1 for LC1 is derived as follows.

$$V = (1.0 * V_{LW}) + (0.3 * V_{TW})$$
$$V = (1.0 * 415) + (0.3 * 193) = 473 \text{ k}$$

The forces used in the calculations are listed in Tables 5 and 7. V_{LW} refers to the shear induced in the bearings in the weak pier direction under

Design Step 7.2.2 (continued)

longitudinal earthquake. Similarly, V_{TW} refers to the shear in the weak direction due to transverse earthquake loading. V_{LW} is taken from Table 7 and V_{TW} is from Table 5.

All other forces for the LC1 and LC2 loading combinations are calculated similarly. The orthogonal seismic force combinations are listed in the following tables.

- Table 10 - the elastomeric bearings at the piers (note that the shears reported in Table 10 are also used for the design of the cross frames at the piers)
- Table 11 - the base of the walls of the piers
- Table 12 - the foundations of the piers

Table 10
Orthogonal Seismic Force Combinations
LC1 and LC2 Bearings

		Bearings Forces and Moments on Substructure				
		Weak Direction of Pier		Strong Direction of Pier		
Support/Location		Shear (kips)	Moment (kip-ft)	Shear (kips)	Moment (kip-ft)	Axial (kips)
Pier No. 1	LC1	473	0	330	2,160	42
	LC2	318	0	486	3,851	28
Pier No. 2	LC1	473	0	330	2,160	42
	LC2	318	0	486	3,851	28

Design Step 7.2.2 (continued)

Table 11
Orthogonal Seismic Force Combinations LC1 and LC2
Base of the Pier Walls

Support/Location		Pier Wall Base Forces and Moments on Substructure				
		Weak Direction of Pier		Strong Direction of Pier		
		Shear (kips)	Moment (kip-ft)	Shear (kips)	Moment (kip-ft)	Axial (kips)
Pier No. 1	LC1	634	19,789	462	15,717	43
	LC2	426	13,293	684	24,361	29
Pier No. 2	LC1	634	19,793	462	15,716	43
	LC2	426	13,295	684	24,361	29

Table 12
Orthogonal Seismic Force Combinations
LC1 and LC2 Foundations

Support/Location		Pier Foundation Base Forces and Moments on Substructure				
		Weak Direction of Pier		Strong Direction of Pier		
		Shear (kips)	Moment (kip-ft)	Shear (kips)	Moment (kip-ft)	Axial (kips)
Pier No. 1	LC1	879	22,774	620	17,886	43
	LC2	591	15,298	919	27,561	29
Pier No. 2	LC1	879	22,779	620	17,884	43
	LC2	591	15,301	919	27,560	29

Design Step 7.3

Determine Design Forces

[Division I-A, Article 6.2]

For Seismic Performance Category B, the seismic design forces for structural members and connections are determined by dividing the elastic seismic forces by the appropriate Response Modification Factor R. The design forces for the foundations are determined by dividing the elastic seismic forces by half of the R Factor. The design forces obtained from Division I-A replace the Group VII load combination found in Table 3.22.1A of Division I. These forces are used in the seismic design of the various components of the bridge.

The design forces use the R Factor to "modify" or reduce the elastic seismic forces. This reduction is appropriate for structural systems that possess enough ductility to endure the inelastic demands likely to occur in the reduced strength system. As outlined in Section 1.1 of Division I-A, the design philosophy is to restrict inelastic effects and/or damage to parts of the bridge where such effects are readily detectable following a large earthquake. This implies that inelastic action should not occur in the foundations.

In SPC B the foundations are designed for twice the seismic force level as are the columns or piers that they support, due to the half R Factor. This requirement attempts to ensure that yielding occurs in the column or pier and not in the foundation. The approach for SPC B therefore differs from that applied to SPC C or D bridges where the probable column or pier plastic hinging forces are calculated for use in the design of adjacent members.

Design Step 7.3.1

Design Forces for Structural Members and Connections

[Division I-A, Article 6.2.1]

The Specification makes a distinction between the forces for members and connections verses the design forces for foundations calculated in Design Step 7.3.2. Use Equation (6-1) in Division I-A to calculate the maximum forces in each member.

$$\text{Group Load} = 1.0\ (D + B + SF + E + EQM)$$ Division I-A Eqn (6-1)

In this equation, forces B, SF, and E are buoyancy, stream flow, and earth forces, respectively. D and EQM forces are the dead load and earthquake forces, respectively. For this example the B, SF, and E forces are zero for the

Design Step 7.3.1 (continued)

design of the superstructure cross frames and the elastomeric bearings. The B, SF, and E forces are assumed to be zero for the design of the wall. Even though some small stream forces may be present when considering the forces acting at the base of the wall, the SF forces are not included here. The B, SF, and E forces will be considered for the design of the foundations.

If only the dead and earthquake loads are present then Equation 6-1 reduces to

Group Load = 1.0 (D + EQM)

Where:

EQM = (LC1 or LC2 forces) divided by R

a) Response Modification Reduction Factor, R
[Division I-A, Article 3.7, Table 3]

The R Factor is used to modify EQM and applies to specific forces for specific members. The decision of which R value to apply to each member is a critical one since the R values, to a great extent, affect the locations and magnitude of the expected damage.

In this example, R reduces the seismic wall moments. Recall that R was determined in Design Step 2.6, and a summary of the R values used to modify EQM is presented below.

R = 2.0 For design of the pier wall in its strong direction and in the wall's weak direction, provided that the wall is designed as a pier and not a column

R = 1.0 For connections that transfer forces between the superstructure and substructure (e.g. cross frames), except at the abutments and for the foundations

R = 0.8 For connections at the abutments, this factor is smaller than that used at the piers. This adds conservatism in the design of elements used to prevent the superstructure from falling off of the substructure at locations where the superstructure is discontinuous

The distinction between considering the wall in its weak direction as a pier or a column is based upon the amount of ductility available. If the wall is considered a column, then the Specification will require that the minimum plastic hinge zone confinement steel requirements be met. By meeting the

Design Step
7.3.1
(continued)

requirement, sufficient ductility will be provided to accommodate the larger inelastic demands that smaller design forces might produce. Thus the R Factor is taken as 3 if proper confinement is provided, and R is taken as 2 if not.

For this example, the wall will be designed as a pier in the weak direction. Although this produces larger design forces than designing it as a column, the benefit is not having to provide the confinement steel. The longitudinal steel in the wall will probably be controlled by minimum steel requirements since the wall is so large; thus flexural ductility should not be an issue. Lightly reinforced sections, particularly walls, are typically quite ductile.

b) Calculate the Design Forces with EQM

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

The design forces for the bearings at the piers and for the base of the piers are given in Tables 13 and 14. The R value used for each location is given above the table.

For example, at the base of the wall in Pier No. 2, the weak direction moment using LC1 is derived as follows. The result is given in Table 14.

$M = (D + EQ/R)$
$M = (270 + 19793/2) = 10166$ k-ft

All other forces in the tables are calculated similarly.

The R Factors have been applied to all the forces, including shear and axial forces, in accordance with the provisions of Division I-A for SPC B structures. However, such practice is unique to SPC B, since the probable shear forces and axial forces corresponding to full plastic hinging (development of plastic mechanisms in the substructure) are used for SPC C and D structures.

The designer should consider the implications of using the reduced design forces for shear and axial loads. If full plastic hinging forces are not used for the shear design of the columns or piers, then the possibility that the column is weaker in shear than in flexure exists. This means that a brittle shear failure could occur. If the designer wishes to avoid this possibility, several options are available: 1) apply the method outlined for SPC C and

Design Step 7.3.1 (continued)

D bridges in Article 7.2 of Division I-A, or 2) use the elastic shear force for design. Note that using the elastic forces does not prevent the column from being shear critical, it simply means that the elastic shear could be sustained without a shear failure. If an earthquake larger than the design earthquake occurred, a brittle shear failure could still conceivably occur.

Table 13
Design Forces for Structural Connections
Bearings at Piers

Group LC1 = 1.0*Dead Load + 1.0*LC1 / R

Group LC2 = 1.0*Dead Load + 1.0*LC2 / R

R = 1.0 Connection

Support/Location		Bearings: Forces and Moments on Substructure — Weak Direction of Pier: Shear (kips)	Weak Direction of Pier: Moment (kip-ft)	Strong Direction of Pier: Shear (kips)	Strong Direction of Pier: Moment (kip-ft)	Axial (kips)
Pier No. 1	LC1	480	0	331	2,225	2,207
	LC2	325	0	487	3,916	2,193
Pier No. 2	LC1	480	0	331	2,225	2,207
	LC2	325	0	487	3,916	2,193

Design Step
7.3.1
(continued)

Table 14
Design Forces for Structural Members
Base of Pier Walls

Group LC1 = 1.0*Dead Load + 1.0*LC1/R

Group LC2 = 1.0*Dead Load + 1.0*LC2/R

R = 2.0 Wall-Type Pier

Support/Location		Pier Wall Base Forces and Moments on Substructure				
		Weak Direction of Pier		Strong Direction of Pier		
		Shear (kips)	Moment (kip-ft)	Shear (kips)	Moment (kip-ft)	Axial (kips)
Pier No. 1	LC1	324	10,165	232	7,957	3,879
	LC2	220	6,917	343	12,279	3,872
Pier No. 2	LC1	324	10,166	232	7,956	3,879
	LC2	220	6,918	343	12,278	3,872

Design Step
7.3.2

Design Forces for Foundations
[Division I-A, Article 6.2.2]

Use Equation (6-2) in Division I-A to calculate the maximum forces in the wall pier foundations.

Group Load = 1.0 (D + B + SF + E + EQF) Division I-A Eqn (6-2)

For this example, the forces B and E will be added in the foundation design step using the appropriate free-body diagrams to illustrate the origins of the forces. The stream flow forces for this example are assumed to be zero. Therefore, in this section only, D and EQF forces are combined.

a) Response Modification Reduction Factor, R
[Division I-A, Article 6.2.2]

R = 1 for calculating the design forces in the foundation. This originates from the requirement that the foundation R Factor be half of the R Factor used for

Design Step 7.3.2 (continued)

the attached substructure element, in this case the pier wall. Because the wall is designed using R = 2 in both directions, the R for the foundations is 1.

The use of R/2 for the foundation design is unique to SPC B structures. The objective is to force inelastic action such as plastic hinges to form in the substructure elements above the foundations, while at the same time not requiring the designer to calculate plastic hinging forces. While this is a Specification provision, the designer should be aware that under certain conditions, e.g., when nonseismic issues control the substructure design, this provision may not ensure that inelastic effects are excluded from the foundation. In such instances, Principle No. 3 in Section 1.1 — *Purpose and Philosophy* — of the Specification may be assumed to control. This principle stipulates that damage resulting from large earthquakes should be "detectable and accessible," two constraints that are not met if damage occurs in the foundation. Therefore, the designer should check that this principle is met for the design.

b) Calculate the Design Forces with EQF

Table 15 summarizes the combination of D and EQF forces. These in conjunction with the appropriate B and E forces will comprise the foundation "Group VII" design forces.

For example, the overturning moment at the base of the footing for Pier No. 2 in the weak direction using LC1 is derived as follows.

$M = (D + EQ/R)$
$M = (307 + 22{,}779/1.0) = 23{,}086$ k-ft

Where the dead load moment D is from Table 9 and the earthquake moment EQ is from Table 12.

All other forces in the table are calculated similarly.

Design Step 7.3.2 (continued)

Table 15
Design Forces for Foundations

Group LC1 = 1.0*Dead Load + 1.0*LC1/R

Group LC2 = 1.0*Dead Load + 1.0*LC2/R

R = 1.0 Foundations

Support/Location		Pier Foundation Base Forces and Moments on Substructure: Weak Direction of Pier, Shear (kips)	Weak Direction of Pier, Moment (kip-ft)	Strong Direction of Pier, Shear (kips)	Strong Direction of Pier, Moment (kip-ft)	Axial (kips)
Pier No. 1	LC1	886	23,081	621	17,989	4,741
	LC2	599	15,605	920	27,664	4,727
Pier No. 2	LC1	886	23,086	621	17,987	4,741
	LC2	599	15,608	920	27,663	4,727

Design Step 7.4

Plastic Hinging Forces

The probable forces associated with plastic hinging do not have to be used to establish the design forces if the bridge is a SPC B structure. Therefore, these forces are not used in this phase of the design.

DESIGN STEP 8

SUMMARY OF DESIGN FORCES

The purpose of this section is to synthesize the various design forces applicable for SPC C and D designs as outlined in Section 7 of the Specification. For those two performance categories the design forces are controlled by either the elastic forces modified by the appropriate R Factor or the plastic hinging forces. In addition, design force levels for hold-down devices and other miscellaneous items are specified in Section 7. Thus this design step is intended to condense the various forces into controlling forces necessary for design of the bridge components.

Since SPC B designs do not consider plastic hinging forces, the force combinations given in Design Step 7 are used directly. For this reason, Design Step 8 may be skipped for SPC B designs.

DESIGN STEP 9

DETERMINE DESIGN DISPLACEMENTS
[Division I-A, Article 6.3]

Design Step 9.1

Minimum Support Length
[Division I-A, Article 6.3.1]

The bearing seats supporting the expansion ends of the bridge must provide a minimum support length at least N inches wide, and N is measured normal to the face of the abutment. See Figure 23.

$L := 400 \cdot ft$ Length of bridge between abutment seats

$H := 36 \cdot ft$ Average height of piers between abutments

$S := 25$ Skew

From Division I-A, Equation (6-3A)

$$N := \left(8 \cdot in + 0.02 \cdot L \cdot \frac{in}{ft} + 0.08 \cdot H \cdot \frac{in}{ft}\right) \cdot \left(1 + 0.000125 \cdot S^2\right)$$

$N = 1.696 \cdot ft$ or $N = 20.36 \cdot in$

The total seat width available must exceed the value of N. In this case, at least 20.5 inches must be provided. The total available seat width is 25 inches. Thus, plenty of seat width is available.

Design Step 9.1 (continued)

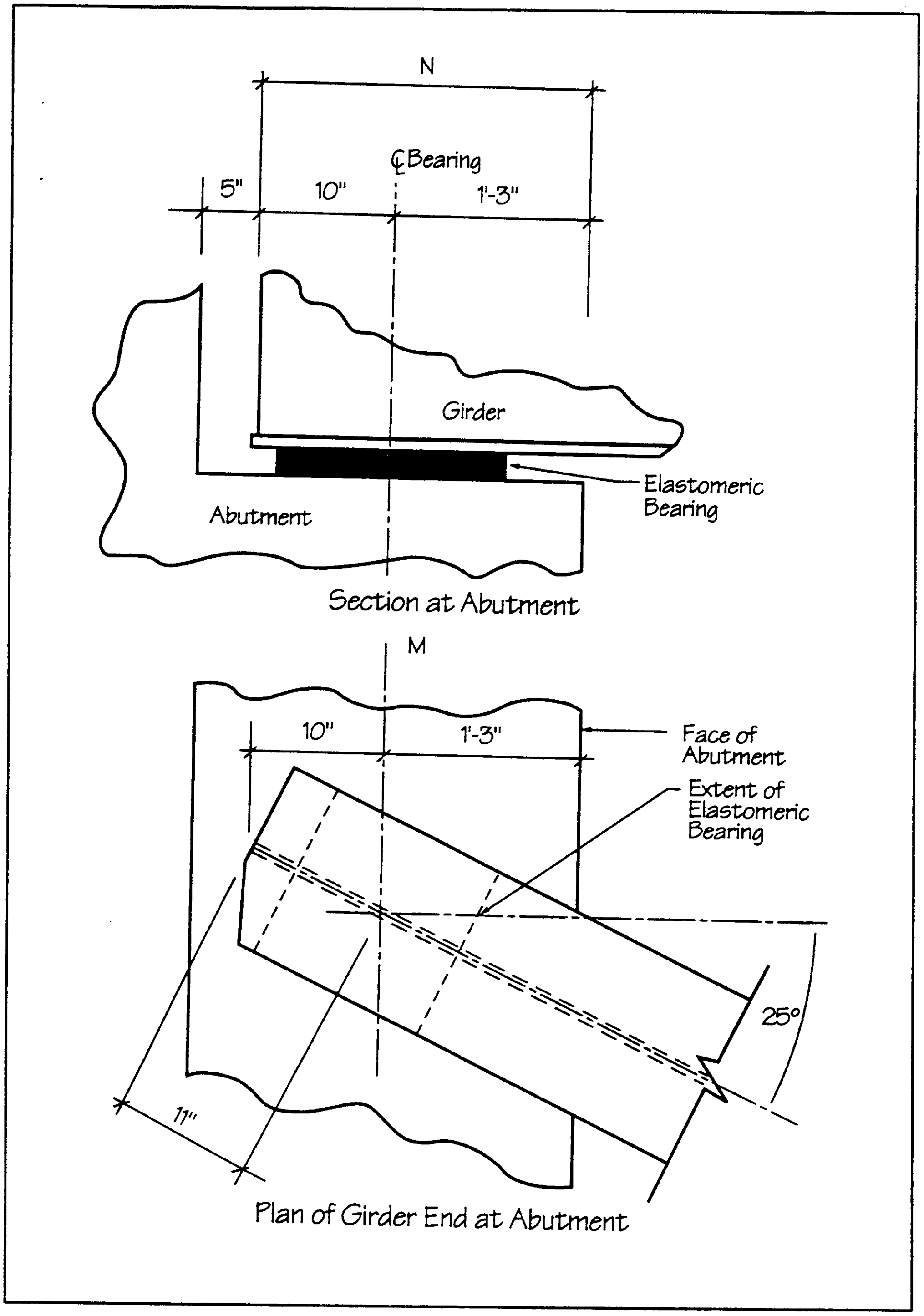

Figure 23 — Details of Girder Seat

Design Step 9.2

Design Displacements

The maximum displacements from the Multimode Spectral Method in the direction normal to the face of the abutments were given in Tables 6 and 8. The relative displacements across the bearings at the abutments are also equal to the absolute displacements at the abutments since the spring elements used to model the abutment elastomeric bearings were assumed fixed at the ends rather than connected to the superstructure.

From the table, the maximum displacements normal to the abutment face are 0.113 feet for the longitudinal earthquake and 0.056 feet for the transverse earthquake. The LC1 combination of these should be used for determining the maximum displacement. Thus the design displacement from the analysis is given by

$$\Delta = 0.113 + 0.3\,(\,0.056\,) = 0.130 \text{ feet}$$

DESIGN STEP 10 — DESIGN STRUCTURAL COMPONENTS

[Division I-A, Articles 6.5 and 6.6]

This section covers the design of critical superstructure and substructure elements, exclusive of the foundations, that resist seismic forces. For this example, only selected lateral force resisting systems will be discussed. These include the pier wall, bearings, and girder stops.

Design Step 10.1 — Pier Design

The Division I-A requirements for SPC B bridges do not directly address the design of piers. However, the philosophy of allowing more inelastic action in structural members that can better accommodate such action without severe damage is apparent in the two R Factors applicable to piers. As previously described, Division I-A requires an R of 2 in the strong direction of the pier, but it allows the designer to use an R of 3 in the weak direction if the pier is designed as a column in the weak direction. Otherwise an R of 2 should be used.

If the pier is designed as a column in the weak direction, then the detailing requirements for confinement in the plastic hinging zones must be met. These requirements are clearly defined for both SPC B and SPC C and D bridges in Division I-A, Sections 6.6.2 and 7.6.2, respectively. However, if the pier is not designed as a column in the weak direction, there are no special requirements for SPC B structures, although there are requirements for SPC C and D structures - Section 7.6.3, in which minimum reinforcement ratios and limiting shear stresses are defined.

In the absence of specific guidelines for SPC B pier walls, the requirements given in Section 7.6.3 could be used. For cases in SPC B where the seismic forces are low in relation to the forces that a given wall cross section may be reinforced to carry, the minimum steel specified in Section 7.6.3 may be considered a useful guide for design. In addition, Caltrans in its *Memo to Designers* 6-5 (1993) provides guidelines for design of walls in lower seismic risk zones, and use the same limiting reinforcement ratio as is given in Section 7.6.3.

Design Step 10.1.1 — Vertical or Longitudinal Reinforcement

The vertical reinforcement in the wall will be determined for the base of the wall.

Design Step 10.1.1 (continued)

a) Summary of Forces

Below is a summary of the design moments and axial forces for the piers at their bases. These are taken from Table 14 and the forces for both piers are, for all practical purposes, identical. For this reason, one design will suffice for both piers.

$P_1 := 3879 \cdot kip$	Axial load on pier for LC1
$M_{w1} := 10165 \cdot kip \cdot ft$	Moment in weak direction for LC1
$M_{s1} := 7957 \cdot kip \cdot ft$	Moment in strong direction for LC1
$P_2 := 3872 \cdot kip$	Axial load on pier for LC2
$M_{w2} := 6917 \cdot kip \cdot ft$	Moment in weak direction for LC2
$M_{s2} := 12279 \cdot kip \cdot ft$	Moment in strong direction for LC2

b) Minimum Steel for Pier Walls

As a first guess, the pier vertical reinforcement will be sized to produce a reinforcement ratio of 0.0025 on the gross cross section.

This is the minimum steel ratio required in Division I-A for walls in SPC C and D. However, this is also the minimum steel specified in Caltrans' *Memo to Designers 6-5* (1993) for both high seismic and low seismic zones for both horizontal and vertical steel. Other agencies, for instance the Washington State Department of Transportation (WSDOT, 1995), require steel areas equal to 0.011 times the thickness of the wall (in inches) to be used in each face in both directions per foot for temperature and shrinkage steel. This is equivalent to 0.0018 on the gross area. ACI 318-89 (1992) requires 0.0015 in the vertical direction and 0.0025 in the horizontal. In summary, various agencies have different philosophies regarding minimum steel in walls, and for the most part, durability issues are a primary concern. Thus the policies for the responsible agency should be consulted to establish appropriate minimum steel contents. In this case, it is rational to have vertical steel in at least the same ratio as the horizontal steel since the vertical bars provide the primary load resistance.

c) Design and Check of Pier

The clear distance to the horizontal bars, which will be the outermost bars in the pier, is taken as 2.5 inches. To meet the horizontal steel requirements, #8

Design Step 10.1.1 (continued)

bars are assumed. This gives a composite distance to the main vertical bars of 3.5 inches.

The program PCA COLUMN (PCA, 1993) was used to analyze the pier. This provides the flexibility to consider the biaxial effects that exist on the wall. The results of this analysis are shown in Figures 24 and 25, which show the comparisons between demand and design strength for load groups LC1 and LC2, respectively. The figures show the loading plotted on an interaction diagram, which is plotted for the applied loading angle. For instance, Figure 24 shows the interaction diagram taken at 38 degrees.

d) Calculation of the ϕ Factor

The figures show the strength interaction diagram inclusive of the understrength factor, ϕ. Because the bridge is a SPC B structure, the ϕ factors used are the same as for Division I. The ϕ for the load combination shown in Figure 24 is calculated below as an example of how to determine ϕ for a specific applied axial load. The equation shown can be derived from the wording of Article 8.16.1.2 , and the derivation is given in Wang and Salmon (1992).

$f_c := 4000 \cdot psi$	Concrete strength
$A_g := (66 \cdot ft) \cdot (6 \cdot ft)$	Gross cross-sectional area at base
$A_g := (66 \cdot ft) \cdot (6 \cdot ft)$	Gross cross-sectional area at base
$\phi P_n := P_u$	Equate the design strength with the required strength
$\phi := 0.9 - \dfrac{2.0 \cdot \phi P_n}{f_c \cdot A_g}$	$\phi = 0.87$

e) Discussion of Actual versus Required Strength for Seismic Loads

As evident in the figures, the minimum steel content of 0.0025 is more than adequate for the applied seismic loads. In fact, at the given axial load contour, the design moment strength ϕMn is well above the applied loading. Thus the actual strength Mn will be even larger than the applied loads require. The implication of this comparison is that inelastic action (yielding) of the pier in the weak direction will probably not occur. This inference is based on the fact that an R of 2 was used for determining the design loads for the pier, and the capacity is in excess of twice the demand.

Design Step
10.1.1
(continued)

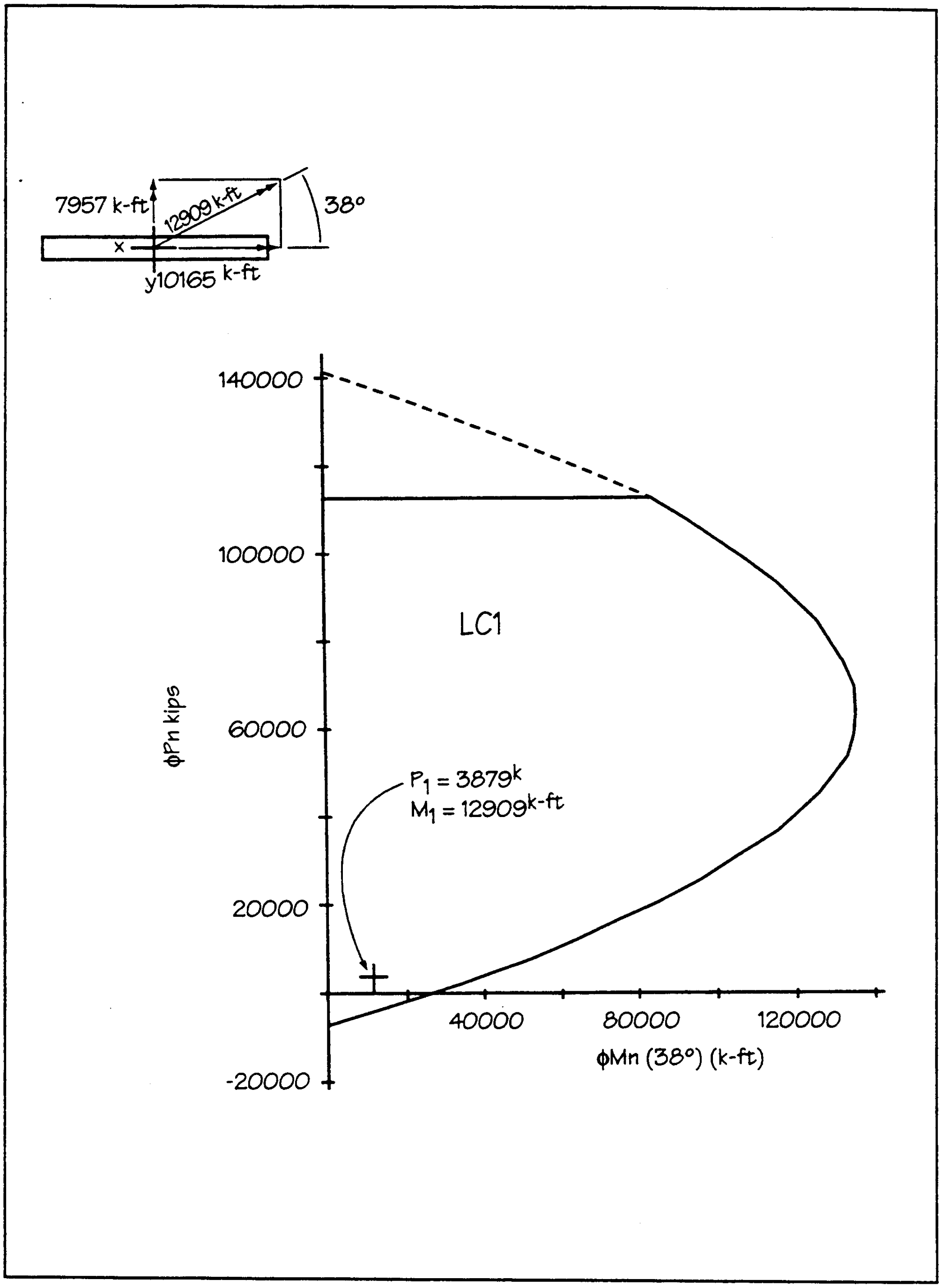

Figure 24 — Interaction Diagram for LC1/Base of Wall

Design Step
10.1.1
(continued)

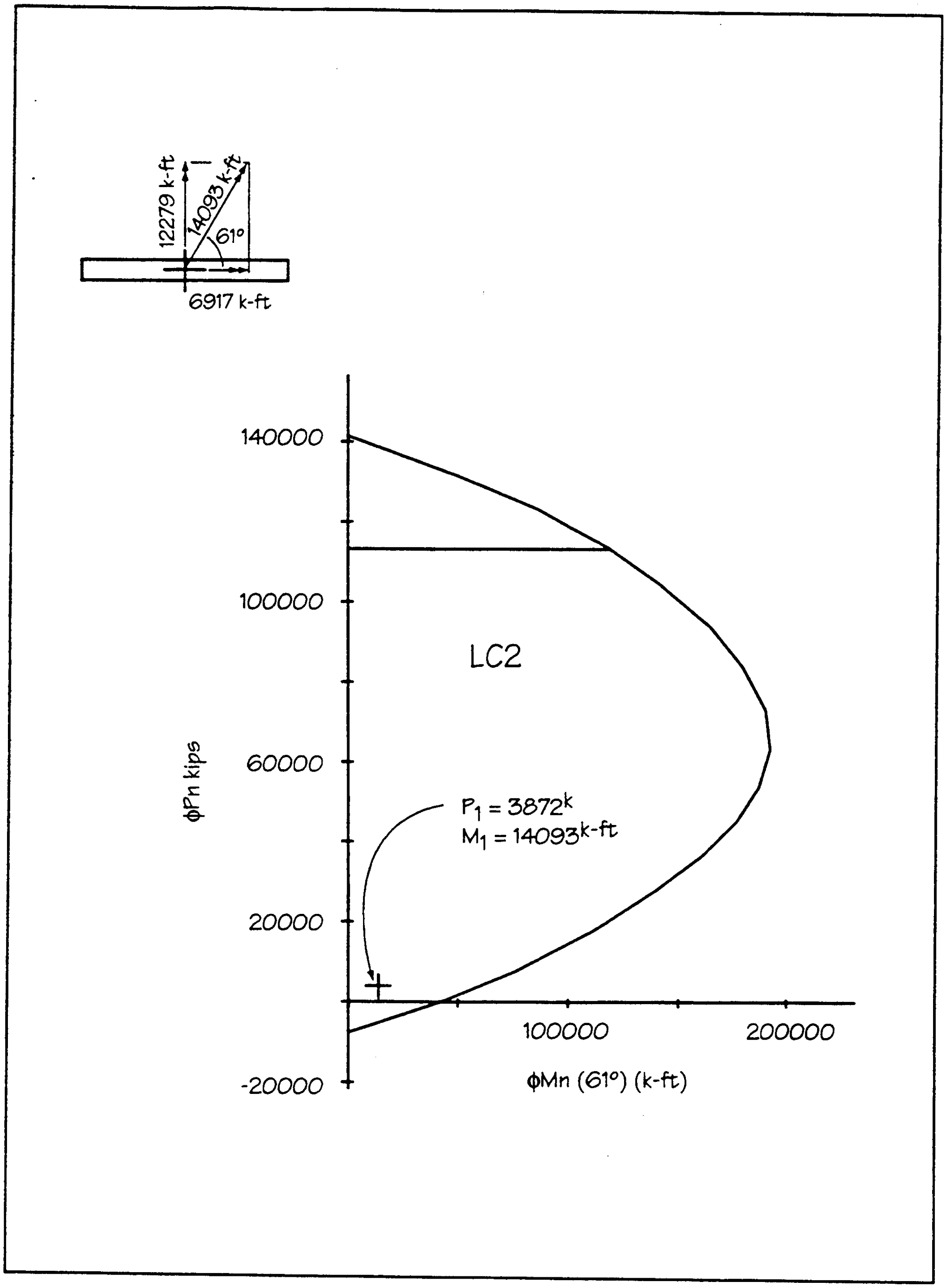

Figure 25 — Interaction Diagram for LC2/Base of Wall

Design Step 10.1.1 (continued)

Recall that the elastic axial earthquake forces were reduced by same R Factor as used for the moments. Rationally, for this bridge, it is difficult to justify reducing the axial forces. However, the elastic axial earthquake forces (as given in Table 11) are quite small, 43 and 29 kips for the LC1 and LC2 cases, respectively. It thus can be seen from the interaction diagram that shifting the axial load to correspond to elastic axial earthquake forces would not change the moment resistance, realistically, at all.

Design Step 10.1.2

Transverse Reinforcement

The limiting values for horizontal steel discussed in the previous step will be used for the design. The minimum ratio of steel to gross concrete area will be taken as 0.0025. This matches the Division I-A requirement for wall piers in SPC C and D, the ACI 318-89 (1992) requirements for walls, and the Caltrans' recommendations for high and low seismic areas.

Calculate the vertical spacing of bars assuming that the same bars are used on both faces of the wall. This calculation is made for the maximum thickness of the pier wall, 6 feet at the base.

For $\rho_h = 0.0025$, calculate the area steel A_h required per 12-inch height of wall.

$$A_h := 0.0025 \cdot (12 \cdot in) \cdot (6 \cdot ft) \qquad A_h = 2.16 \cdot in^2$$

Calculate the spacing for #9 bars.

$$A_b := 1.00 \cdot in^2$$ Area of a #9

$$s := \frac{(12 \cdot in) \cdot 2 \cdot A_b}{A_h} \qquad s = 11.1 \cdot in$$

Note that the factor 2 in the spacing equation accounts for the presence of bars on each face.

Design Step 10.1.2 (continued)

Calculate the spacing for #8 bars.

$A_b := 0.79 \cdot in^2$ Area of a #8

$s := \dfrac{(12 \cdot in) \cdot 2 \cdot A_b}{A_h}$ $s = 8.8 \cdot in$

The spacing that will work the most conveniently is that for the #8 bars. Therefore, use #8 bars spaced at 8 inches vertically on each face.

Because the wall is not designed as a column in the weak direction, the spacing limits for the transverse reinforcement do not apply. Division I-A, Section 7.6.3 limits the maximum spacing to 18 inches on center for SPC C and D structures. The selected spacing meets this requirement, even though the bridge is a SPC B bridge.

Caltrans limits the bar spacing in all walls to 12 inches in each direction, except that the vertical spacing of the horizontal bars in the plastic hinging zone should be 6 inches or less. These restrictions apply, provided that the wall is lightly loaded axially, which means that the axial load for LC1 and LC2 must be less than 40 percent of the balanced axial load, 0.4 P_b. In this example, the axial load for both LC1 and LC2 are well below this value. See Figures 24 and 25. Furthermore, the moment capacity of the wall is so much greater than the demand that the 6-inch spacing limit suggested by Caltrans will not be used.

Design Step 10.1.3

Cross Ties

The role of cross ties, those bars that pass through the thickness of the wall and hook around vertical and horizontal bars, is to restrain these bars from buckling in the event that the cover concrete is lost. The most critical bars for buckling will be the vertical bars. Loss of cover will typically only occur as a result of yielding in the plastic hinge zone that may develop during an earthquake. Such yielding will normally result from bending about the weak axis of the wall.

The amount of cross tie steel necessary in a wall is currently being researched. Experimental testing at the University of California, Irvine (Haroun, et. al., 1994) has suggested that the ductility available in walls loaded in flexure about their weak axes is not strongly dependent on the quantity of cross ties. In fact walls with no cross ties were capable of sustaining limited ductility demands.

Design Step
10.1.3
(continued)

Because a consensus on the amount of cross ties does not exist, a hybrid solution will be used in this example. Division I-A gives no guidance regarding cross tie quantities, and Caltrans suggests that cross ties should be spaced at 12-inch centers both horizontally and vertically in walls designed for lower seismic zones. Additionally, the Division I reinforced concrete tie requirements for columns can be used to derive cross tie limits that are rational, but not as severe as Caltrans' criteria. In Division I, Section 8.18.2.3.4 requires for columns that no longitudinal bar be more than 2 feet from a restrained bar. The tie spacing is also limited to 12 inches along the length of the member. This will be used for the example wall.

Use #4 cross ties spaced at 2 feet on center horizontally, and use the 8-inch vertical spacing of the horizontal bars. Additionally, shift the cross ties by 1 foot in adjacent horizontal rows. See Figure 26.

Note that the vertical spacing is 16 inches instead of the maximum 12-inch spacing recommended in Article 8.18.2.3.2 of Division I. However, the horizontal spacing is half of the 4-foot maximum that would be allowed by Article 8.18.2.3.4. It is therefore deemed acceptable for the ties to be arranged as shown, even though the exact letter of Division I is not met.

Design Step
10.1.3
(continued)

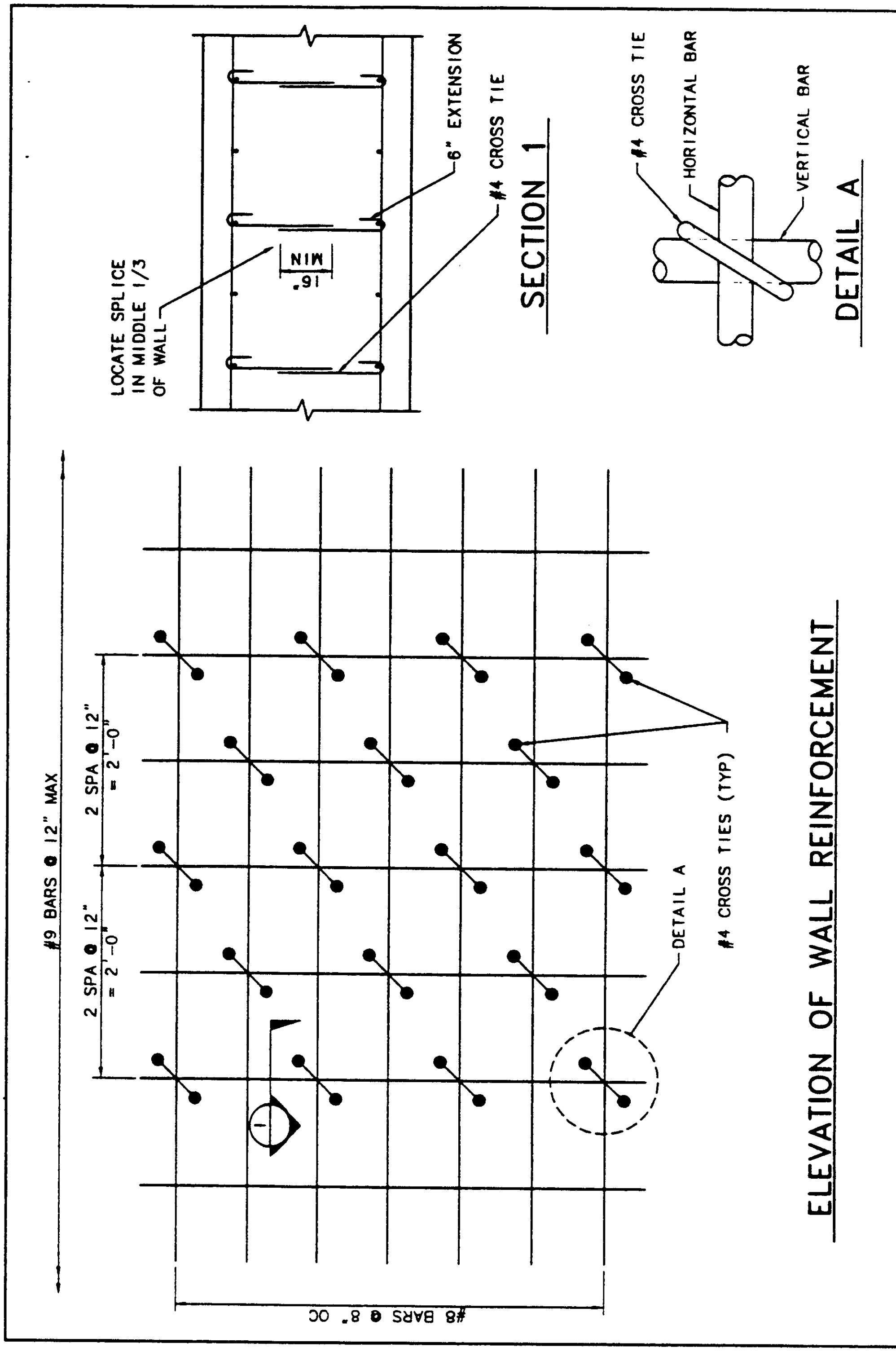

Figure 26 — Cross Tie Details

Design Step 10.1.4

Shear Stress Check

A shear check should be made on the wall. Division I-A, Section 7.6.3 for SPC C and D bridges specifies the allowable shear stress as

$$v_u := 2 \cdot \sqrt{f_c} + \rho_h \cdot f_y$$

Division I-A
Eqn (7-8)

Where:

ρ_h is the horizontal steel ratio and

f_y is the steel yield stress

Because biaxial loading is present, check the shear using the resultant shear demand and the first term of Equation (7-8) only. This will be conservative and should pose no problem for the large wall section.

Recall from Table 14 the weak and strong direction shears.

$V_{w1} := 324 \cdot kip$	Weak direction shear for LC1
$V_{s1} := 232 \cdot kip$	Strong direction shear for LC1
$V_{w2} := 220 \cdot kip$	Weak direction shear for LC2
$V_{s2} := 343 \cdot kip$	Strong direction shear for LC2

Resultant shear for LC1

$$V_{R1} := \sqrt{V_{w1}^2 + V_{s1}^2} \qquad V_{R1} = 398 \cdot kip$$

Resultant shear for LC2

$$V_{R2} := \sqrt{V_{w2}^2 + V_{s2}^2} \qquad V_{R2} = 407 \cdot kip$$

By inspection V_{R2} controls; therefore, use it to check shear.

Design Step 10.1.4 (continued)

Calculate the applied shear stress.

$$v_{R2} := \frac{V_{R2}}{(6 \cdot ft) \cdot (66 \cdot ft)} \qquad v_{R2} = 7 \cdot psi$$

$$v_u := 2 \cdot \sqrt{f_c} \cdot psi \qquad v_u = 126 \cdot psi$$

Recall:

$$f_c := 4000 \cdot psi$$

The applied shear is much less than the shear that the concrete alone can carry. By inspection the narrowest wall section will also be adequate for shear. Also, recall that the elastic shear forces were reduced by the R Factor to arrive at the design shear, and by doing so it was argued that it was then possible to have shear failure occur prior to flexural yielding. Obviously in this case, the applied shear stress 7 psi is so much smaller than the allowable shear stress 126 psi that the possibility of shear controlled behavior is nonexistent.

Design Step 10.1.5

Summary of Pier Reinforcement

Use 144 #9 bars vertically in the wall and distribute them evenly around the perimeter. Use 67 bars along each long face, and distribute the remaining bars along the ends of the pier. In the sloping end sections of the pier, terminate the vertical bars as necessary to accommodate the change in section. Because the vertical steel is controlled by the minimum reinforcement ratio, continue all steel in the center section (nonsloping section) to the top of the pier. See Figure 27.

Use #8 horizontal bars spaced at 8 inches and placed along each face. Use #8 hairpins on the ends of the wall to provide continuity of steel around the corners.

The horizontal bars may be spaced or sized differently in the upper portion of the pier, depending on the controlling loads from the bearings — particularly the bearings loading on the overhangs. Since gravity loading will control in the overhangs, they have not been designed in this example.

Use #4 cross ties as shown in Figure 26.

Design Step
10.1.5
(continued)

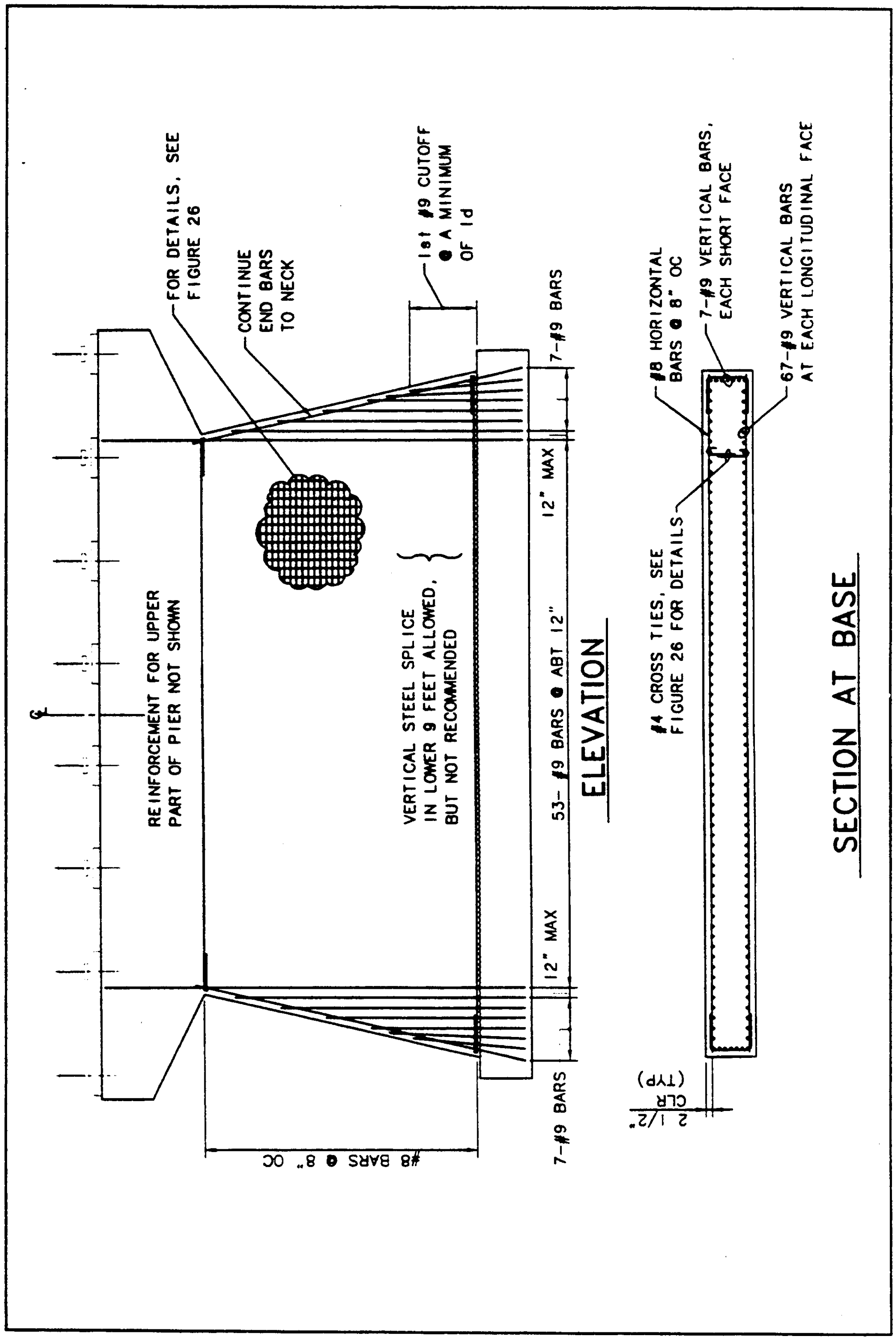

Figure 27 — Reinforcement in Lower Part of Pier Wall

Design Step 10.1.5 (continued)

The cross ties may be made up of two 180-degree hairpins as shown in the figure, or they may be a single piece with a 180-degree hook on one end and a 90-degree hook on the other. If the latter option is chosen, every other cross tie — horizontal and vertical — shall have its 90-degree hook placed on the opposite side of the wall.

For this wall, the spliced cross tie is preferred because the wall thickness tapers from base to top. Thus the spliced cross tie allows the same length ties to be used for more than one row by overlapping the cross tie splice more as the wall narrows.

Design Step 10.2

Pier Cross Frame Design

The design of the cross frames and diaphragms is not fully addressed in this example. Only a few general design comments are offered. The cross frames, which provide lateral load transfer of most of the superstructure inertial loads to the bearings, must be carefully detailed and designed to ensure the proper transfer of forces. This means that all the force transfer elements must be considered.

For instance, Figure 28 shows two different concepts for the diagonal bracing system. Figure 28(a) depicts a bracing system with the work point of the lower part of the system close to the lower flange of the girder. This arrangement provides better transfer of lateral forces than does that of Figure 28(b) in which the lateral forces must be carried through bending of the stiffener. Failures of this second type of system have occurred in recent earthquakes, most notably the Kobe Earthquake of January 1995 (SEAW, 1995).

The skew presents its own complications to the design of the cross frames. Since the bearing stiffeners at the piers and the abutments are used as gussets for the diagonal bracing, care should be taken to prevent undue eccentricity in the system. This means that the stiffeners should be oriented parallel to the skew and not perpendicular to the girder web. For this bridge, two bearing stiffeners on each side of the girder web are required at the piers. To accommodate this requirement and to avoid unnecessary eccentricities, the stiffeners should be configured as shown in Figure 29.

Design Step 10.2 (continued)

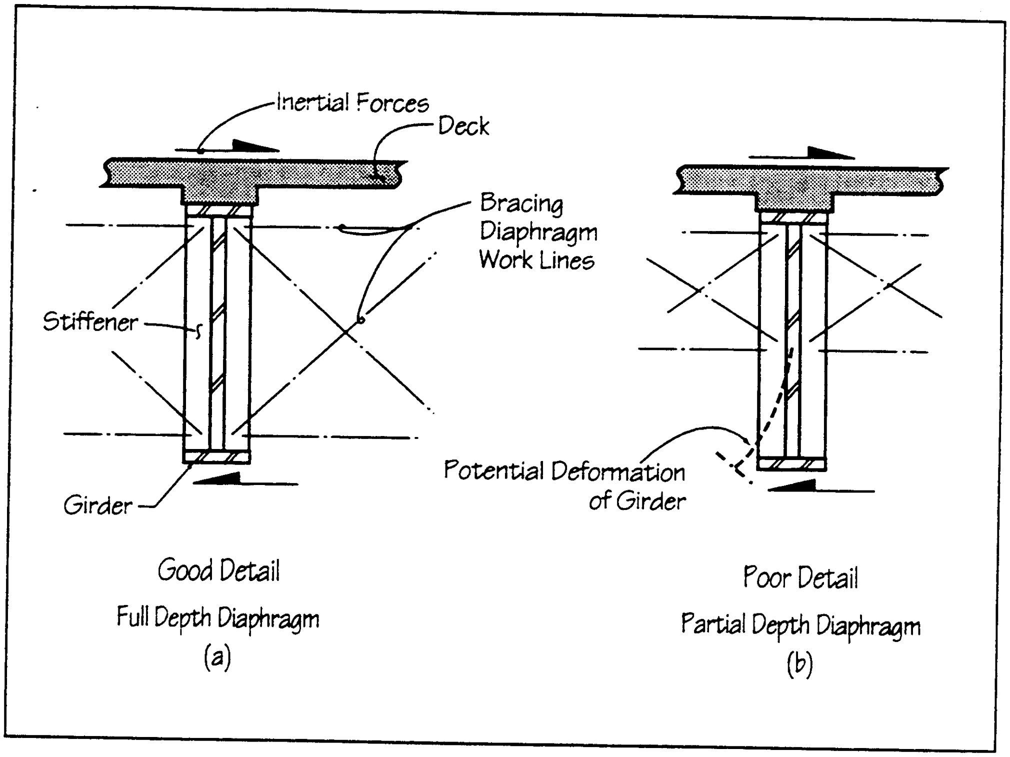

Figure 28 — Pier Cross Frame Details

Design Step 10.2 (continued)

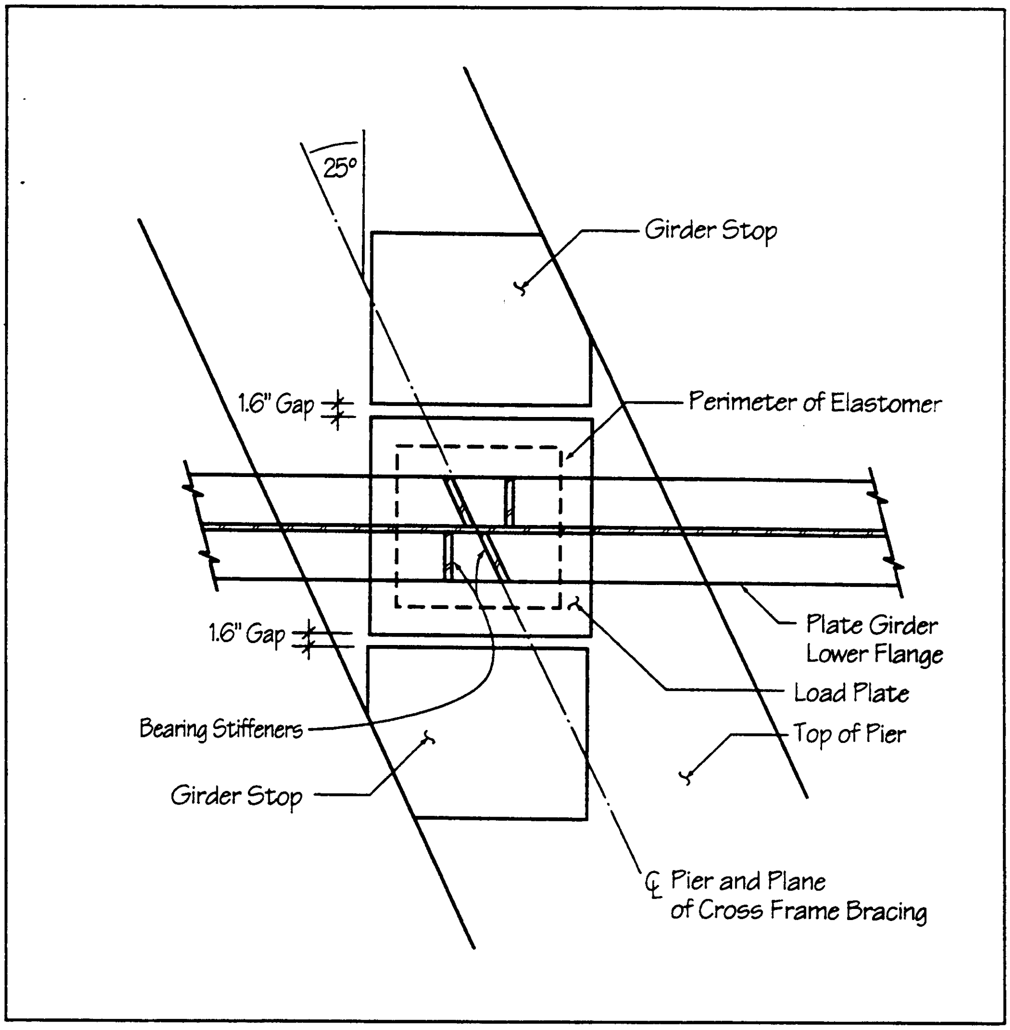

Figure 29 — Plan View of Bearing and Girder Stop

Design Step 10.3 — Connection Design

Design Step 10.3.1 — Longitudinal Linkage

Not applicable.

Design Step 10.3.2 — Hold Downs

Not applicable.

Design Step 10.3.3 — Connection of Superstructure to Substructure

a) Elastomeric Bearings

Background. The elastomeric bearings should be checked to determine whether they can sustain the strains that will be imposed under the combined dead and earthquake loading. The method that will be used for this check is that specified in Section 14.3 of the AASHTO Guide Specifications for Seismic Isolation Design (AASHTO, 1991). The reason that this document will be used is that neither the Division I nor the Division I-A Specifications address the design of elastomeric bearings under such combined loading.

The Division I-A Specification requires that the bearings have a large enough thickness such that the lateral displacements are less than half of the total thickness. This requirement is a service load provision and is not intended to apply to extreme loadings such as earthquakes, where some damage may be expected because the AASHTO Isolation Guide Specification does address such extreme loadings for elastomeric bearings. The method outlined in the Isolation Guide Specification will be used. Excerpts of the guide are provided in Appendix C.

The Isolation Guide Specification suggests that the shear strains due to compression, rotation, and shearing motions caused by dead and earthquake loadings be summed, and then this total shear strain should be less than about 75 percent of the elastomer's ultimate elongation at break.

This method is applied below, and the following steps to the calculations are followed.

1. Determine the deformations of the most critical bearings. Typically these will be the outermost bearings on the pier.

Design Step
10.3.3
(continued)

2. Calculate the shear strains due to compression of the bearing. This is based on the axial strain in the bearing via an equation in the Isolation Guide Specification.

3. Calculate the shear strains due to rotations. Again a relation is given by AASHTO.

4. Calculate the shear strains due to lateral deformations from dead and earthquake loading.

5. Add these and compare with the limiting strain. The ultimate elongation is taken as 400 percent for Durometer 50 elastomers per Section 18 of the AASHTO Division II Specification.

<u>Summary of Bearing Deformations.</u> The relative translational displacements across the bearings in both the weak and strong directions of the pier were given in Design Steps 6.3 and 6.4. However, for the check of the bearing deformation capacity, both the relative deflections and rotations across the bearings must be used. These deformations from the analysis are given in Table 16 for the dead, longitudinal earthquake, and transverse earthquake load cases. Note that the rotations are reported about axes as shown in Figure 30.

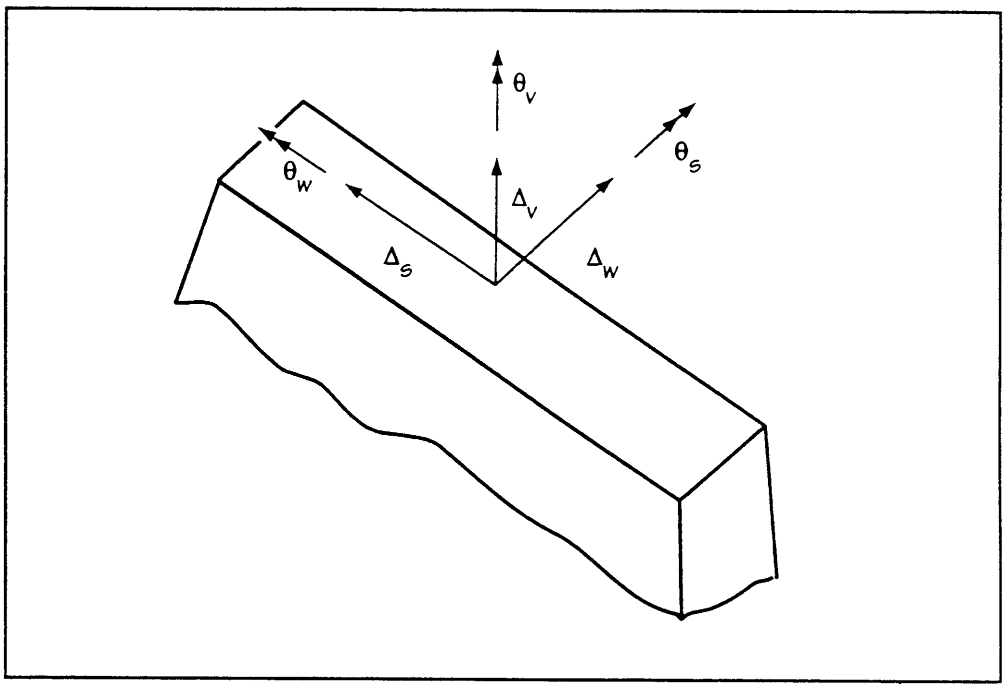

Figure 30 — Key to Relative Displacements Across Bearings

Design Step
10.3.3
(continued)

Table 16
Relative Deformations Across Pier Bearings

Dead Load Case Bearing Displacements at Piers					
Deflections (Feet)			Rotations (Radians)		
Weak Δwd	Strong Δsd	Vertical Δvd	Weak θwd	Strong θsd	Vertical θvd
0.00172	0.00021	0.00266	4.33E-04	1.88E-07	3.74E-06

Longitudinal Load Case Bearing Displacements at Piers					
Deflections (Feet)			Rotations (Radians)		
Weak Δwl	Strong Δsl	Vertical Δvl	Weak θwl	Strong θsl	Vertical θvl
0.09505	0.04674	4.54E-05	5.34E-04	2.47E-06	1.57E-05

Transverse Load Case Bearing Displacements at Piers					
Deflections (Feet)			Rotations (Radians)		
Weak Δwt	Strong Δst	Vertical Δvt	Weak θwt	Strong θst	Vertical θvt
0.04424	0.0946	2.14E-05	2.38E-04	9.90E-06	5.25E-05

Step 1. Critical Bearing Deformation.

For the check of the elastomeric bearing displacements, the outermost bearing is the most critical because both the weak direction translation and the vertical translation are amplified slightly by the rotations of the bearings. This effect is shown in Figure 31 for the weak direction translation; the vertical translation effect is similar. An example calculation for the longitudinal earthquake loading is given below.

Design Step
10.3.3
(continued)

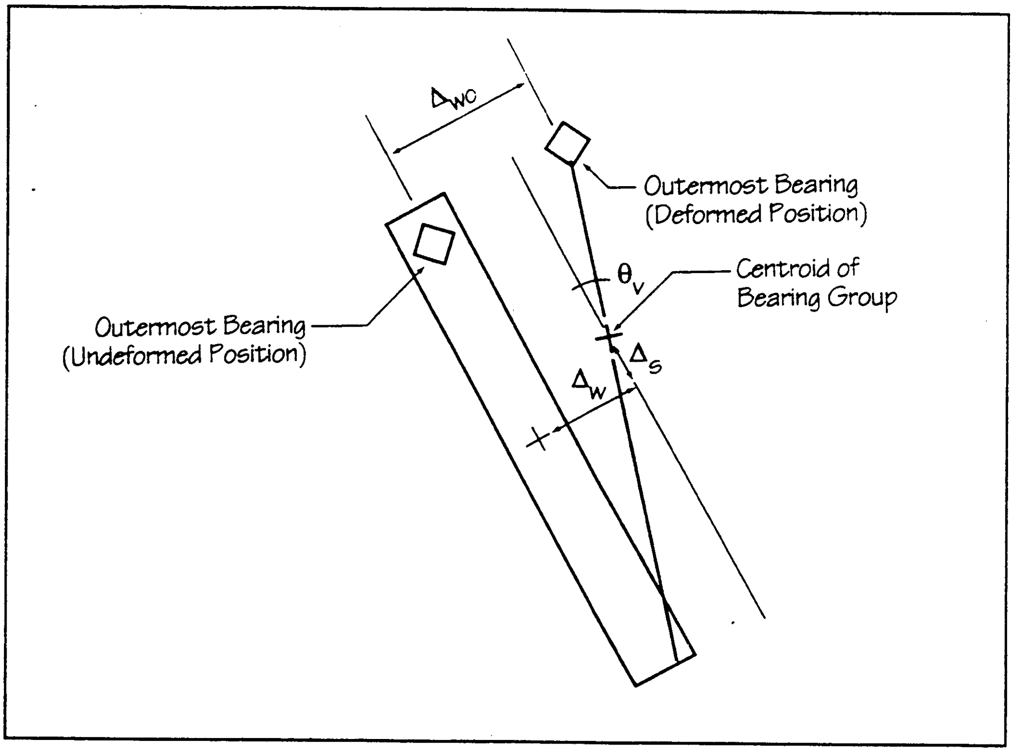

Figure 31 — Key to Outermost Bearing Deformations

$\Delta_{wl} := 0.09505 \cdot ft$	Deflection in weak direction
$\Delta_{sl} := 0.04674 \cdot ft$	Deflection in strong direction
$\Delta_{vl} := 4.54 \cdot 10^{-5} \cdot ft$	Deflection in vertical direction
$\theta_{wl} := 5.34 \cdot 10^{-4} \cdot rad$	Rotation about weak axis
$\theta_{sl} := 2.47 \cdot 10^{-6} \cdot rad$	Rotation about strong axis
$\theta_{vl} := 1.57 \cdot 10^{-5} \cdot rad$	Rotation about vertical axis

Calculate the deformations of the outermost bearings.

$L_o := 31.5 \cdot ft$	Distance from bearing group centroid to outermost bearing

Design Step 10.3.3 (continued)

$\Delta_{wlo} := \Delta_{wl} + \theta_{vl} \cdot L_o$ $\qquad \Delta_{wlo} = 0.096 \cdot ft$

$\Delta_{slo} := \Delta_{sl}$ $\qquad \Delta_{slo} = 0.047 \cdot ft$

$\Delta_{vlo} := \Delta_{vl} + \theta_{sl} \cdot L_o$ $\qquad \Delta_{vlo} = 1.23 \cdot 10^{-4} \cdot ft$

$\theta_{wlo} := \theta_{wl}$ $\qquad \theta_{wlo} = 5.34 \cdot 10^{-4} \cdot rad$

$\theta_{slo} := \theta_{sl}$ $\qquad \theta_{slo} = 2.47 \cdot 10^{-6} \cdot rad$

$\theta_{vlo} := \theta_{vl}$ $\qquad \theta_{vlo} = 1.57 \cdot 10^{-5} \cdot rad$

The deformations for the outermost bearings for the dead, longitudinal, and transverse earthquake load cases are given in Table 17.

The various displacements and rotations are then combined to form the LC1 and LC2 load case displacements. This combination occurs in the same fashion as for forces. The resulting deformations are given in Table 18.

Design Step
10.3.3
(continued)

Table 17
Relative Deformations Across Outermost Pier Bearings

Dead Load Case Displacements of Outer Pier Bearing					
Deflections (Feet)			Rotations (Radians)		
Weak Δwdo	Strong Δsdo	Vertical Δvdo	Weak θwdo	Strong θsdo	Vertical θvdo
0.00184	0.00021	0.00267	4.33E-04	1.88E-07	3.74E-06

Longitudinal Load Case Displacements Outer Pier Bearing					
Deflections (Feet)			Rotations (Radians)		
Weak Δwlo	Strong Δslo	Vertical Δvlo	Weak θwlo	Strong θslo	Vertical θvlo
0.09554	0.04674	1.23E-04	5.34E-04	2.47E-06	1.57E-05

Transverse Load Case Displacements of Outer Pier Bearing					
Deflections (Feet)			Rotations (Radians)		
Weak Δwto	Strong Δsto	Vertical Δvto	Weak θwto	Strong θsto	Vertical θvto
0.04589	0.0946	3.33E-04	2.38E-04	9.90E-06	5.25E-05

Design Step 10.3.3 (continued)

Table 18
Outermost Bearing Relative Deformations for LC1 and LC2

LC1 Loading/Bearing Displacements at Piers D + 1.0*Long. EQ + 0.3*Trans. EQ					
Deflections (Feet)			Rotations (Radians)		
Weak $\Delta w1$	Strong $\Delta s1$	Vertical $\Delta v1$	Weak $\theta w1$	Strong $\theta s1$	Vertical $\theta v1$
0.11115	0.07533	0.00289	1.04E-03	5.63E-06	3.52E-05

LC2 Loading/Bearing Displacements at Piers D + 0.30*Long. EQ + 1.0*Trans. EQ					
Deflections (Feet)			Rotations (Radians)		
Weak $\Delta w2$	Strong $\Delta s2$	Vertical $\Delta v2$	Weak $\theta w2$	Strong $\theta s2$	Vertical $\theta v2$
0.0764	0.10883	3.04E-03	8.31E-04	1.08E-05	6.10E-05

Step 2. Shear Strains Due to Compression.

The shear strains due to compression of the elastomeric bearings must be calculated for both LC1 and LC2 load cases.

In addition to the vertical deflection of the bearing, the shear strain depends on the total thickness T of the elastomer and the shape factor S, as defined in Section 14 of Division I.

$T := 1.125 \cdot in$ Total thickness of elastomer, excluding steel plate

$t_i := \frac{T}{2}$ Thickness of elastomer layers

$L := 21 \cdot in$ Length of bearing

$W := 21 \cdot in$ Width of bearing

Design Step 10.3.3 (continued)

Calculate the shape factor S

$$S := \frac{L \cdot W}{2 \cdot t_i \cdot (L + W)} \qquad S = 9.333$$

<u>Load Case, LC1</u>

Calculate the compressive or axial strain in the outer bearings.

$$\Delta_{v1} = 0.00289 \cdot ft$$

$$\varepsilon_{c1} := \frac{\Delta_{v1}}{T} \qquad \varepsilon_{c1} = 0.031$$

Use the expression given in Section 14.3.1 of the Isolation Guide Specification to calculate the shear strain that corresponds to this compressive strain.

$$\varepsilon_{sc1} := 6 \cdot S \cdot \varepsilon_{c1} \qquad \varepsilon_{sc1} = 1.7$$

<u>Load Case, LC2</u>

$$\Delta_{v2} = 0.00304 \cdot ft$$

$$\varepsilon_{c2} := \frac{\Delta_{v2}}{T} \qquad \varepsilon_{c2} = 0.032$$

$$\varepsilon_{sc2} := 6 \cdot S \cdot \varepsilon_{c2} \qquad \varepsilon_{sc2} = 1.8$$

<u>Step 3.</u> Shear Strains Due to Rotation.

Calculate the shear strains due to rotation of the bearings using the approach outlined in Section 14.3.4 of the Isolation Guide Specification.

Because rotation occurs about both axes, the resultant rotation could be used. However, inspection of the weak and strong axis rotations listed in Table 18 indicates that the rotations about the weak axis are much larger than that about the strong axis. Thus use the weak axis rotation directly to calculate the rotational shear strains.

Design Step 10.3.3 (continued)

Note that the weak axis rotation occurs at an angle that is skew to the bearings, since the bearings are square to the girders. The weak axis rotation could be broken into its transverse and longitudinal components and the rotation induced shear strains calculated for both directions. Then, the resultant strain would be calculated from the components. However, the final result is identical to that obtained from using the weak axis rotation directly. Thus this rotation will be used directly in the equations given in Section 14.3.4 of the Isolation Guide Specification.

$B := 21 \cdot in$ Bearing width in direction considered

Load Case, LC1

$\theta_{w1} = 0.00104 \cdot rad$

$$\varepsilon_{sr1} := \frac{B^2 \cdot \theta_{w1}}{2 \cdot t_i \cdot T} \qquad \varepsilon_{sr1} = 0.36$$

Load Case, LC2

$\theta_{w2} = 0.000831 \cdot rad$

$$\varepsilon_{sr2} := \frac{B^2 \cdot \theta_{w2}}{2 \cdot t_i \cdot T} \qquad \varepsilon_{sr2} = 0.29$$

Step 4. Shear Strain Due to Lateral Deformations.

Calculate the shear strains due to lateral deformations that result from dead and earthquake loading. For this calculation the resultant deformations from both translations in the weak and strong directions must be considered since the deflections in these directions are of the same order of magnitude. Use the expression given in Section 14.3.2 of the Isolation Guide Specification to calculate the shear strain.

Load Case, LC1

The resultant deflection is

$\Delta_{w1} = 0.111 \cdot ft \qquad \Delta_{s1} = 0.075 \cdot ft$

Design Step 10.3.3 (continued)

$\Delta_{r1} := \sqrt{\Delta_{w1}^2 + \Delta_{s1}^2}$ $\qquad \Delta_{r1} = 0.134 \cdot ft$

The shear strain is

$\varepsilon_{st1} := \dfrac{\Delta_{r1}}{T}$ $\qquad \varepsilon_{st1} = 1.4$

Load Case, LC2

$\Delta_{w2} = 0.076 \cdot ft$ $\qquad \Delta_{s2} = 0.109 \cdot ft$

$\Delta_{r2} := \sqrt{\Delta_{w2}^2 + \Delta_{s2}^2}$ $\qquad \Delta_{r2} = 0.133 \cdot ft$

$\varepsilon_{st2} := \dfrac{\Delta_{r2}}{T}$ $\qquad \varepsilon_{st2} = 1.4$

Step 5. Total Induced Shear Strains.

Calculate the total induced shear strains for the two load cases. Note that the maximum strains from the different effects may not occur at the same angle. However, they will be simply added in this case.

Load Case, LC1

Recall the shear strains:

$\varepsilon_{sc1} = 1.7$ $\qquad \varepsilon_{sr1} = 0.36$ $\qquad \varepsilon_{st1} = 1.4$

Calculate the total shear strain.

$\varepsilon_{sc1} + \varepsilon_{sr1} + \varepsilon_{st1} = 3.5$

Design Step 10.3.3 (continued)

Load Case, LC2

Recall the shear strains.

$\varepsilon_{sc2} = 1.8$ $\varepsilon_{sr2} = 0.29$ $\varepsilon_{st2} = 1.4$

Calculate the total shear strain.

$$\varepsilon_{sc2} + \varepsilon_{sr2} + \varepsilon_{st2} = 3.5$$

The minimum elongation of the elastomer at rupture is taken as 4 from Section 18 of Division II.

$$\varepsilon_u := 4.0$$

$$0.75 \cdot \varepsilon_u = 3$$

The ratio of the actual demand to the allowable shear strain for the outermost bearings is

$$\frac{\varepsilon_{sc1} + \varepsilon_{sr1} + \varepsilon_{st1}}{0.75 \cdot \varepsilon_u} = 1.2$$

This ratio indicates that the pier bearings are overstrained by 20 percent for LC1. The value for LC2 is not calculated, although by inspection it would be the same. Additionally, the shear strains have not been checked at the abutments, although the strains should be less there due to the greater thickness of elastomer.

Conclusions Regarding Bearing Deformations. Even though the bearings are overstrained somewhat, the total strain is still less than the ultimate. Some damage may be expected to occur to the bearings as a result of the high strains, and at this point the designer should decide whether damage to the bearings is acceptable. If it is not, then the bearing size should be adjusted and reanalyzed. If some damage is acceptable, then the size and the configuration of the bearings need not be revised.

Design Step
10.3.3
(continued)

It is essential to recognize that some damage is to be expected during the design earthquake shaking. In this case, if damage occurs to the bearings, the damage will likely be detectable. Thus it would be reasonable to accept the design as it is now, and require that the bearings be replaced in the event that earthquake induced damage occurs. This requirement implies that the bridge shall be designed and detailed such that the superstructure can be jacked upwards at the piers and the bearings replaced.

For the bearings to be removed and replaced without requiring relatively large lifting heights, some consideration should be given to the manner in which the bearings are connected to the plate girder and the pier. Figure 32 shows a potential detail for accommodating lifting. The bottom steel plate is comprised of an inner and outer plate that are machined to fit together to provide shear resistance, but are separate to allow replacement of the center section. Such a detail should only be used if uplift is not a possibility. A single-piece lower plate can be used, provided that allowance is made for lifting the bearing assembly over the bolts protruding from the pier.

The ability to replace the bearings is also dependent on providing jacking points on both the superstructure and substructure. The logical places to provide jacking points are beneath the girders on either side of the bearings or between the girders. The first option may require additional seat widths on top of the piers for positioning the jacks, and the second option may require special loading points to be included in the pier diaphragms to transfer the loads into the superstructure. Some agencies use plate diaphragms with stiffeners to accomplish this in lieu of the more traditional diagonal bracing.

Design Step
10.3.3
(continued)

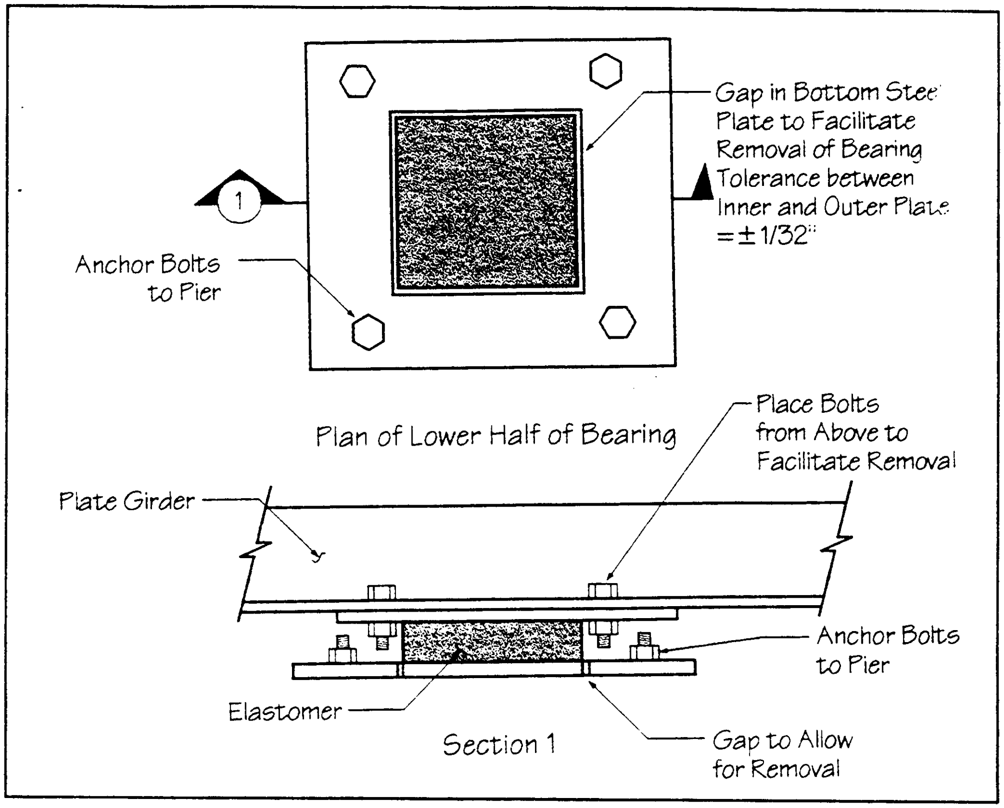

Figure 32 — Bearing Detail to Allow Replacement

b) Shear Blocks (Girder Stops)

General. In the previous design step it was shown that some damage to the elastomeric bearings at the piers may result from the high shear strains expected in the design earthquake. Since the elastomeric bearings are the only elements providing lateral load carrying capability in the transverse direction, it is prudent to provide a secondary or fail-safe load resisting system in this direction. There are a number of concepts that could be used to provide such restraint. For this bridge, shear blocks or girder stops are suggested, and these should be placed at both the piers and the abutments.

Design of Shear Blocks. Shear blocks are used for each of the six interior girders. The blocks are not used for the exterior girders due to space limitations on the outside edges of the girders. The stops are configured as shown in Figures 33 and 34. Since, as shown in the figures, the top loading plate for the elastomeric bearings is wider than the lower flange of the girder, the top load plate will be used to transfer load to the girder stops.

Design Step
10.3.3
(continued)

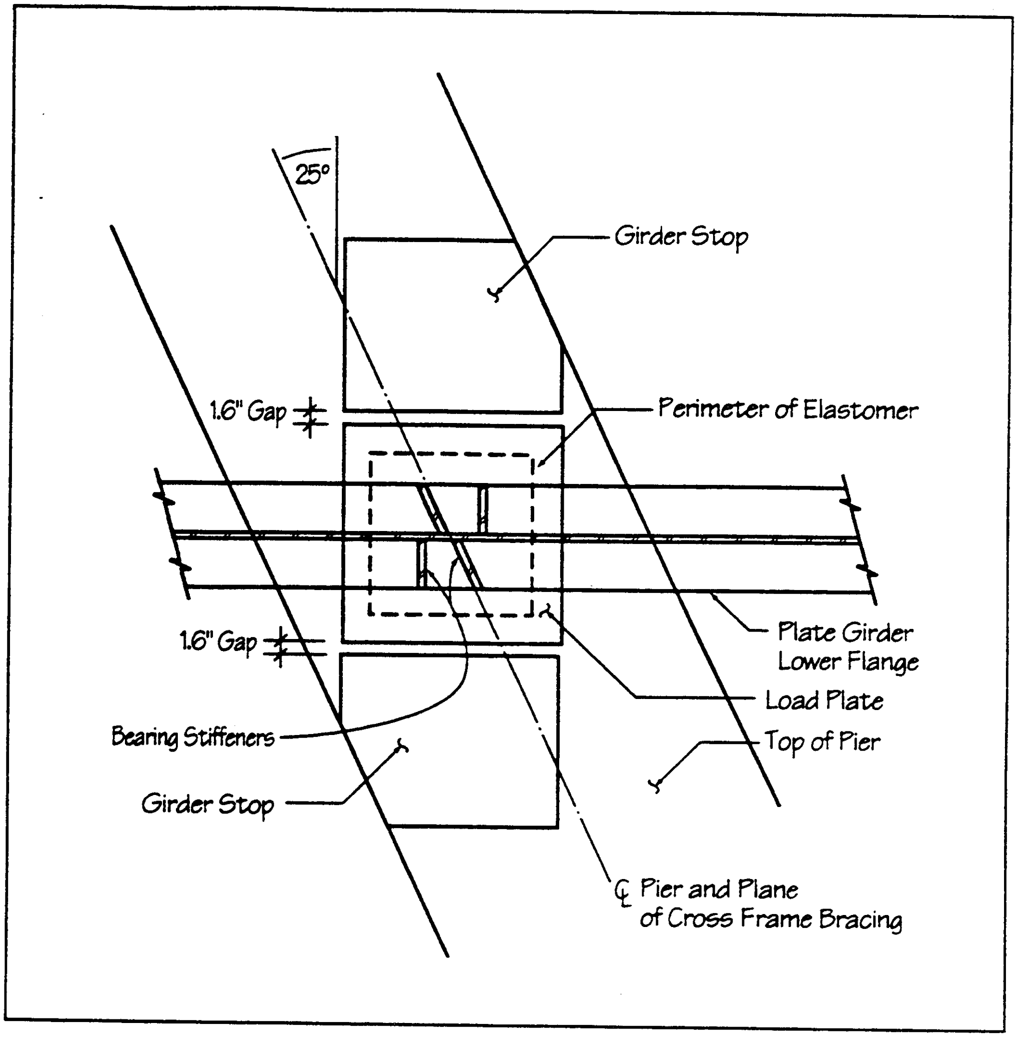

Figure 33 — Plan View of Bearing and Girder Stop

Design Step
10.3.3
(continued)

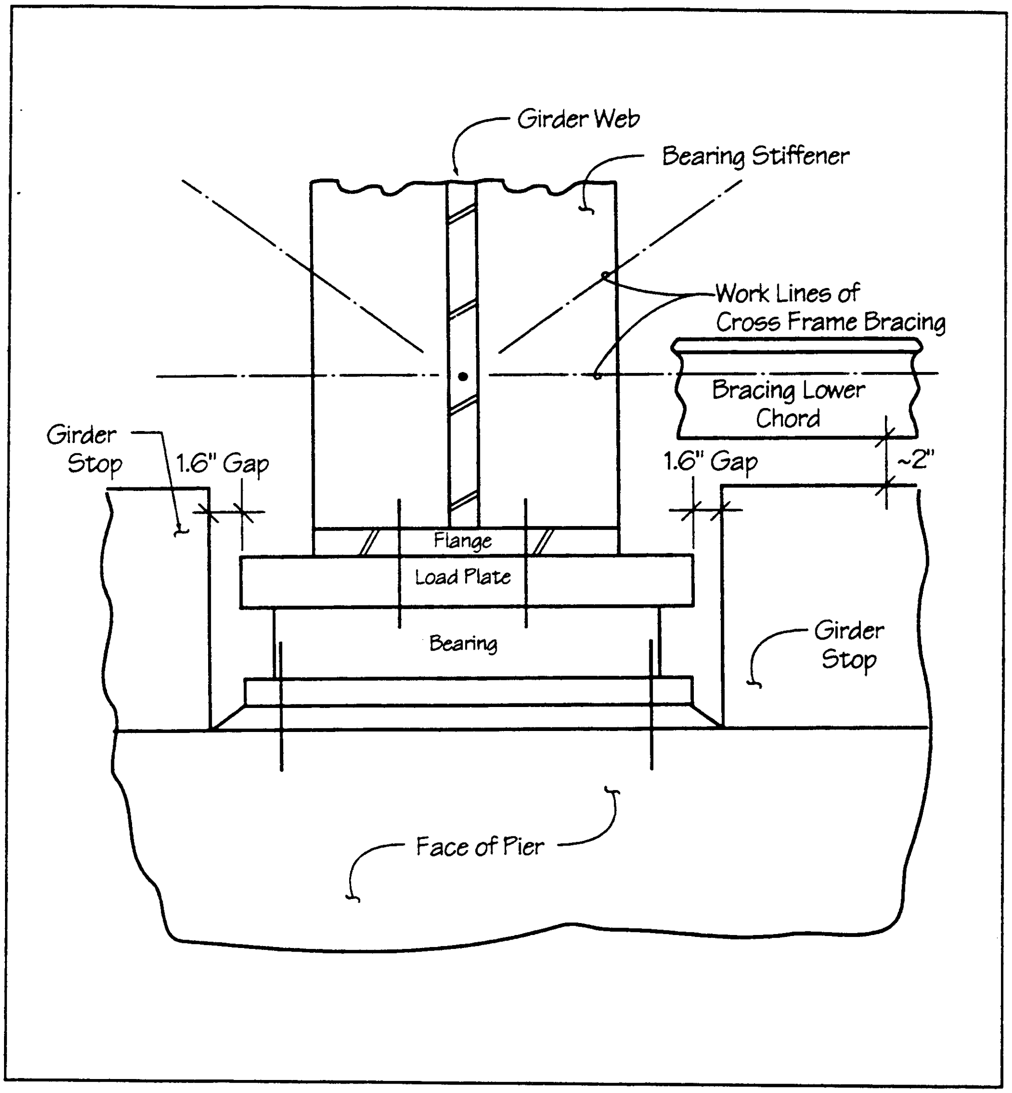

Figure 34 – Elevation of Bearing and Girder Stop

Design Step 10.3.3 (continued)

The girder stops are only back-up elements and, therefore, should not interfere with the functioning of the bearings for either normal service conditions or the design earthquake. The stops should engage the load plates at the design displacement in the transverse direction. This dimension is 1.6 inches and is determined by calculating the transverse displacement component from Table 18. This is shown below.

Calculate the transverse component of the bearing displacement; assume that the weak and strong direction displacements are resolved to add in the transverse direction. Use the displacements across the bearings listed in Table 18.

$S := 25 \cdot deg$

Load Case, LC1

$$\Delta_{gap1} := \Delta_{w1} \cdot \sin(S) + \Delta_{s1} \cdot \cos(S)$$

$$\Delta_{gap1} = 1.4 \cdot in$$

Load Case, LC2

$$\Delta_{gap2} := \Delta_{w2} \cdot \sin(S) + \Delta_{s2} \cdot \cos(S)$$

$$\Delta_{gap2} = 1.6 \cdot in$$

The design loads for the girder stops are taken as the same loads that the elastomeric bearings are required to carry, but prorated for the fact that six stops are available for the eight girders. Therefore, if the bearings fail completely, the girder stops can accept the redistributed forces. As with the gap displacement, the design force is based on the transverse force component. Using the bearing force to design the stops is a conservative approach, since it is unlikely that the bearings will lose their entire lateral resisting capability.

Additional Considerations. There is concern that multiple girder stops may "unbutton" during a large earthquake, because the superstructure does not always contact the stops simultaneously. Under such a condition, the stops contacted first may be overloaded and fail before other stops are loaded. Such behavior has been observed in past earthquakes, for instance at least one such failure was thought to have occurred in the 1995 Kobe Earthquake (SEAW, 1995). However for this bridge, the stops are fail-safe

Design Step 10.3.3 (continued)

elements that only function if the bearings fail. Additionally, the bridge is an SPC B structure in a relatively low seismic hazard zone. Thus, using individual girder stops should be acceptable.

If the designer does not wish to use multiple stops due to concerns about unbuttoning, then single stops may be used. For steel plate girder bridges, using single stops will make the diaphragm bracing much larger due to the larger forces that must be transferred. This effect should be considered in the design.

Design Step 10.3.4

Connection of Wall to Footing

Not addressed in this example.

DESIGN STEP 11

DESIGN FOUNDATIONS

[Division I-A, Article 6.4.2]

In this design example, only the spread footings under the piers are addressed, and in particular only the checks against overturning and against sliding are made.

Design Step 11.1

Design of Spread Footings (Under Piers)

[Division I-A, Article 6.4.2]

Design Step 11.1.1

Find Forces at Bottom of Footing

The forces at the base of the footing for dead loading were given in Table 9, and the earthquake induced forces for load cases LC1 and LC2 were given in Table 12. The earthquake forces do not include the effects of the rock fill over the piers or buoyancy effects.

The checks that are made for the piers are for overturning and sliding. Overturning in the weak direction controls over the strong direction; thus the weak direction is the only one checked. Sliding is checked using the resultant shear force at the foundation level.

Summary of Forces. Recall the following forces and moments that act at the bottom of the foundation.

$P_d := 4698 \cdot kip$ Dead load axial force

$P_1 := 43 \cdot kip$ Axial force for LC1

$P_2 := 29 \cdot kip$ Axial force for LC2

$V_{wd} := 7 \cdot kip$ Weak direction dead load shear

$V_{sd} := 1 \cdot kip$ Strong direction dead load shear

$V_{w1} := 879 \cdot kip$ Weak direction LC1 shear

$V_{s1} := 620 \cdot kip$ Strong direction LC1 shear

$V_{w2} := 591 \cdot kip$ Weak direction LC2 shear

$V_{s2} := 919 \cdot kip$ Strong direction LC2 shear

Design Step
11.1.1
(continued)

$M_{wd} := 307 \cdot kip \cdot ft$ Weak direction dead load moment

$M_{w1} := 22774 \cdot kip \cdot ft$ Weak direction LC1 moment

The dead load forces and the LC1 and LC2 forces listed must be augmented to account for buoyancy and overburden effects. The shear forces and moments, however, do not require adjustment.

Based on the foundation configuration shown in Figure 35, calculate the additional axial force acting at the base of the foundation due to the stone fill overburden, including buoyancy effects. 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.

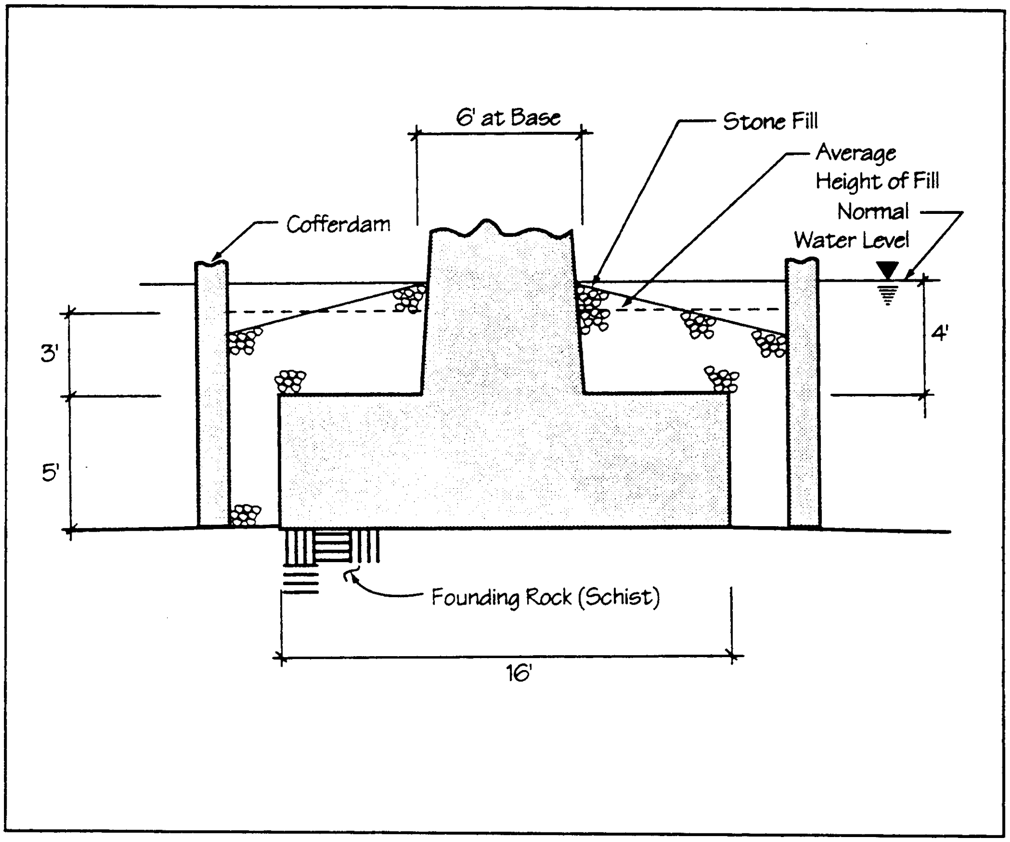

Figure 35 — Configuration of Pier Foundation

Design Step 11.1.1 (continued)

Overburden weight.

$$P_{sf} := (16\cdot ft - 6\cdot ft)\cdot(3\cdot ft)\cdot(70\cdot ft)\cdot\left(0.130\cdot\frac{kip}{ft^3}\right)$$

$$P_{sf} = 273\cdot kip$$

Buoyancy force.

Calculate the uplift force due to buoyancy assuming the water level corresponds to the normal level of 4 feet above the top of the footing.

$V_{ftg} := (16\cdot ft)\cdot(5\cdot ft)\cdot(70\cdot ft)$ 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

$$P_b := \left(V_{ftg} + V_{sf} + V_{stem}\right)\cdot 0.0624\cdot\frac{kip}{ft^3}$$

$$P_b = 579\cdot kip$$

Axial Force.

Calculate the adjusted axial force acting at base of foundation.

$$P_{LC1} := P_d - P_1 + P_{sf} - P_b$$

$$P_{LC1} = 4349\cdot kip$$

Note that the earthquake force comes from the spectral analysis, and therefore the sign is unknown. Thus the earthquake force is assumed to act upwards, the worse case for both overturning and sliding.

Design Step 11.1.1 (continued)

$$P_{LC2} := P_d - P_2 + P_{sf} - P_b$$

$$P_{LC2} = 4363 \cdot kip$$

Design Moment and Shear Forces.

Calculate the design moment to be used for the overturning check for the weak direction.

$$M_w := M_{wd} + M_{w1}$$ Weak direction driving moment

$$M_w = 23081 \cdot ft \cdot kip$$

Calculate the design shear forces to be used in the sliding check. These should be the resultant of the weak and strong direction forces for each load case.

$$V_{r1} := \sqrt{\left(V_{wd} + V_{w1}\right)^2 + \left(V_{sd} + V_{s1}\right)^2}$$

$$V_{r1} = 1082 \cdot kip$$ Shear for LC1

$$V_{r2} := \sqrt{\left(V_{wd} + V_{w2}\right)^2 + \left(V_{sd} + V_{s2}\right)^2}$$

$$V_{r2} = 1097 \cdot kip$$ Shear for LC2

Design Step 11.1.2

Check Stability of Footing — Overturning and Sliding

a) Check Overturning

Per Division I-A, Article 6.4.2(B), the footing can have a separation of the soil up to one-half of the contact area of the foundation under seismic loading (one-half uplift). This is only allowed under foundations not susceptible to loss of strength under cyclic loading.

To ensure that there is no more than one-half uplift on the footing, the eccentricity e must be less than $L_f/3$. Additionally, the soil at the contact edge of the footing must not fail. For this bridge the founding material is rock

Design Step 11.1.2 (continued)

(schist) with an ultimate bearing capacity of 50 ksf, and the material is not susceptible to loss of strength under cyclic loading.

One-Half Uplift.

The 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

$$\frac{L_f}{3} = 5.33 \cdot ft$$

The eccentricity of the axial load caused by the overturning moment can be calculated by

$$e := \frac{M_w}{P_{LC1}} \qquad e = 5.31 \cdot ft$$

The weak direction controlling moment is used in conjunction with the axial load from load case LC1. This is done because the controlling moment corresponds to LC1.

As is seen from the comparison of e with one-third of the footing dimension, the footing is just at the one-half uplift point under the action of the LC1 load case. This case controls over the LC2 case; thus the footing is adequate for the one-half uplift condition.

Contact Stress. The contact stress at the leading edge of the footing must also be checked to ensure that the founding material can sustain the stress corresponding to the critical overturning condition.

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.

$B_f := 70 \cdot ft$ Width of footing

Design Step 11.1.2 (continued)

$$q := \frac{2 \cdot P_{LC1}}{3 \cdot B_f \cdot \left(\frac{L_f}{2} - e\right)}$$

Maximum contact stress at edge of footing

$$q = 15.4 \cdot ksf$$

By inspection q is much less than the ultimate bearing capacity of 50 ksf. Thus, the footing width is adequate.

b) Check for Sliding Beneath the Footing

The check of sliding is made simply by comparing the ultimate sliding resistance with the driving force. For this footing, which is founded on a competent rock, the coefficient of friction may be taken as 0.8.

For LC1

$$V_{f1} := 0.8 \cdot P_{LC1}$$

Frictional sliding resistance for load case LC1

$$V_{f1} = 3479 \cdot kip$$

The driving force is

$$V_{r1} = 1082 \cdot kip$$

Because the resistance is larger than the driving force the footing is adequate for sliding for LC1.

For LC2

$$V_{f2} := 0.8 \cdot P_{LC2}$$

Frictional sliding resistance for load case LC2

$$V_{f2} = 3490 \cdot kip$$

Design Step 11.1.2 (continued)

The driving force is

$$V_{r2} = 1097 \cdot kip$$

Because the resistance is larger than the driving force, the footing is adequate for sliding for LC2.

DESIGN STEP 12 — DESIGN ABUTMENTS

Not addressed in this example.

DESIGN STEP 13 — DESIGN SETTLEMENT SLABS

Not applicable.

DESIGN STEP 14 — REVISE STRUCTURE

Not required.

DESIGN STEP 15 — SEISMIC DETAILS

DETAILS SUMMARY

Several details emphasizing the seismic issues discussed in this example are included within this section. Many of the sketches shown in this section have been introduced and discussed in greater detail in the previous design sections. The details are repeated here as a summary.

Wall Reinforcement Detail (Figure 36)

The vertical and horizontal reinforcement is based on the minimum steel specified for walls in SPC C and D. This is a reasonable amount of minimum steel. Although very little, if any, inelastic action is expected in the wall, the vertical steel splice with the foundation starter bars should not be made at the base of the wall. Even moving the splice up slightly can improve the seismic performance of the wall.

Cross Tie Details (Figure 37)

Cross ties should be provided throughout the wall. No consensus exists regarding the density of ties to use for walls. For cases where there is likely to be little inelastic action, an arrangement such as shown in the figure should be sufficient. This arrangement is based on the AASHTO Division I requirements.

Bearing Detail to Allow Replacement (Figure 38)

If replacement of the bearings is anticipated after either a major earthquake or after their normal service life, proper allowance must be made to remove the bearings without excessive lifting of the bridge. One viable arrangement for this bridge is shown in the figure. Such a detail may be used provided that there is no uplift at the bearings.

Girder Stop Details (Figures 39 and 40)

Individual girder stops, placed next to all the interior girders provide positive restraint from excessive transverse movement. The configuration of the stops renders them effective only in the event the elastomeric bearings exceed their design displacement.

Additionally, Figure 39 shows how the bearing stiffeners, to which the diaphragm connects, should be oriented along the skew to avoid eccentricity problems.

DETAILS SUMMARY
(continued)

Pier Diaphragm Details (Figures 41 and 42)

Conventional diagonal cross frame bracing can be used, but the frames should extend over the full height of the girders at the piers and abutments. If replacement of the bearings is an issue, then the diaphragms must be designed to carry the loads induced by lifting of the bridge. An alternate solid diaphragm design is shown for those cases where the lifting jacks must be placed under the diaphragm.

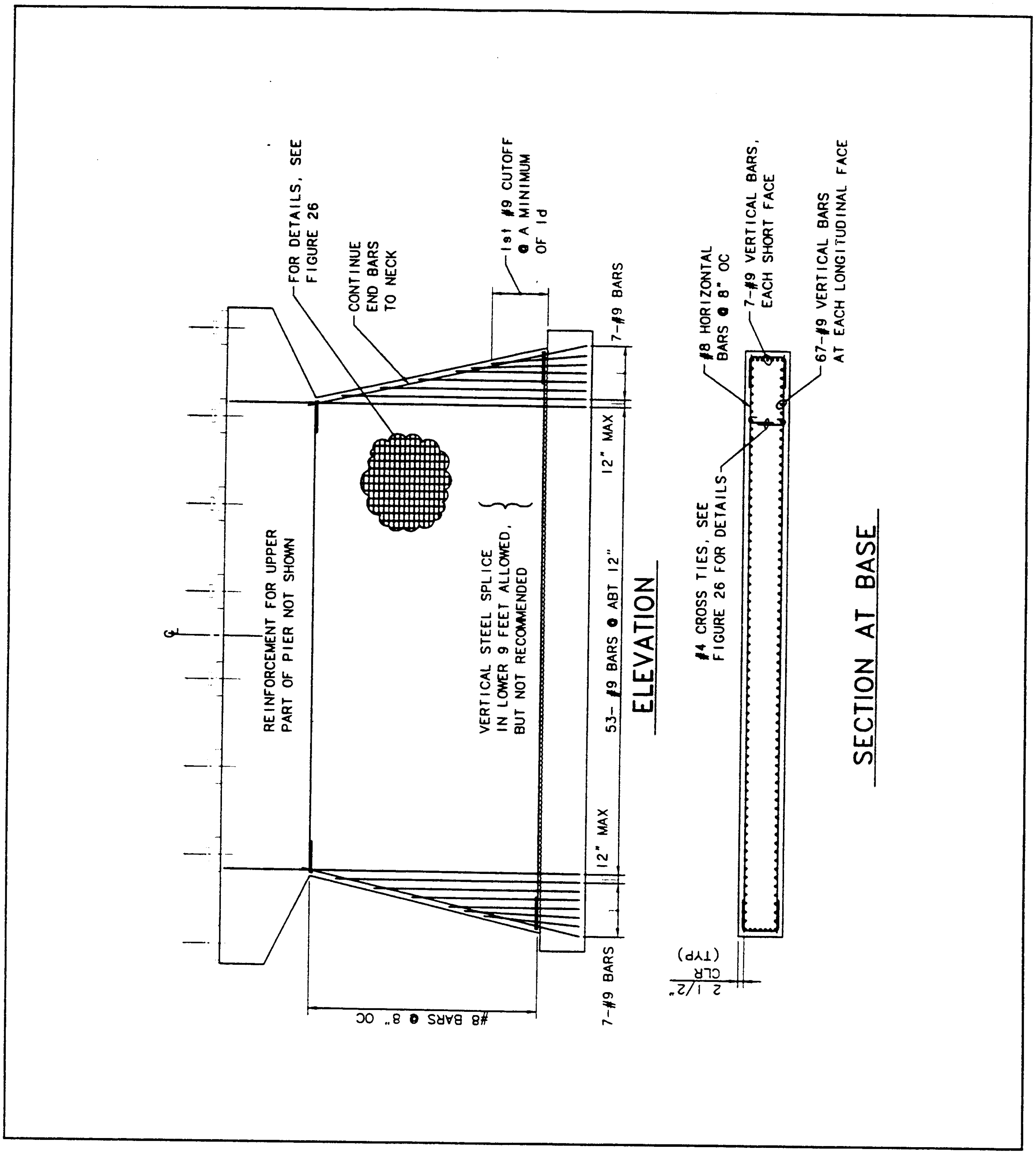

Figure 36 — Wall Reinforcement Detail

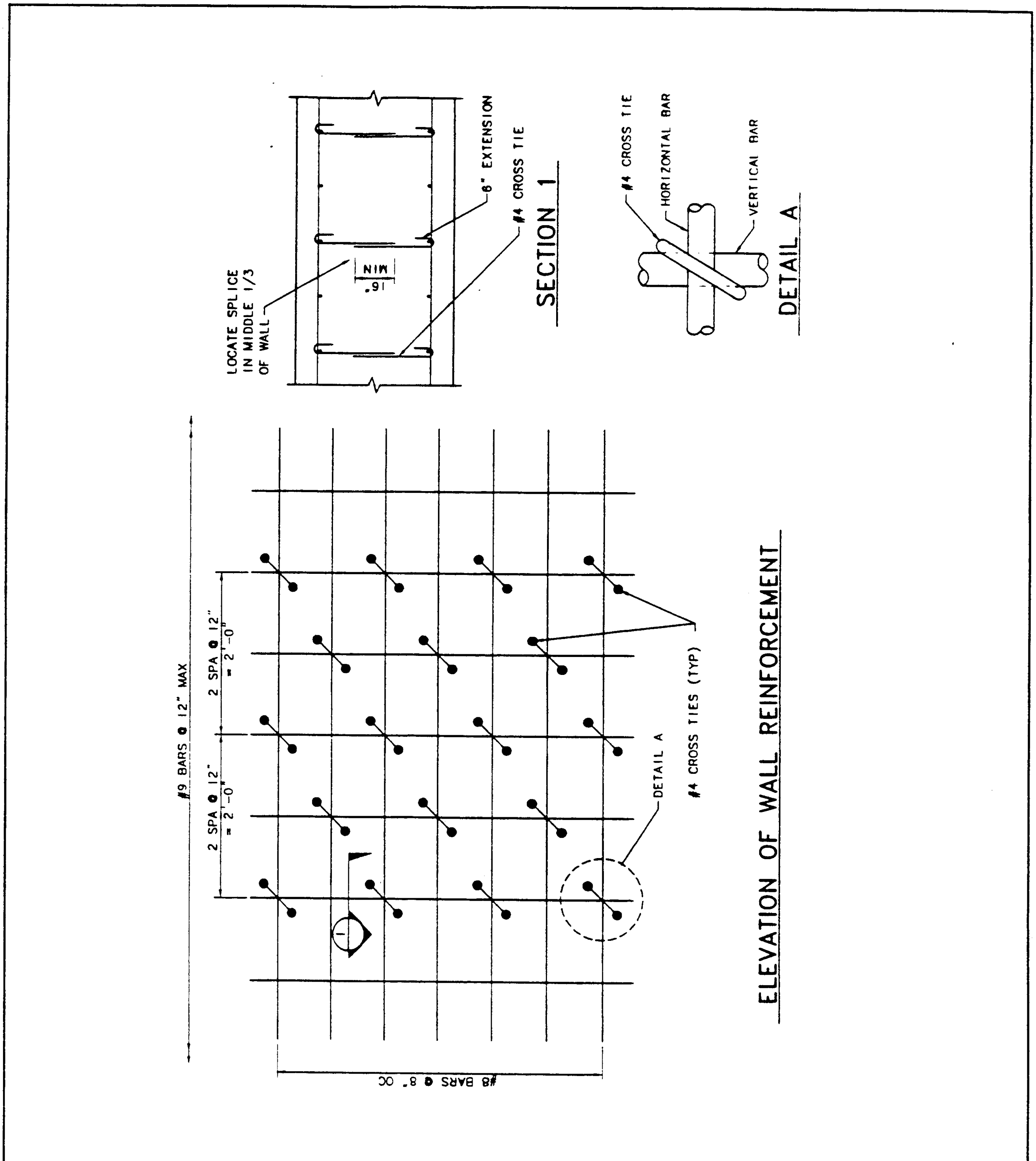

Figure 37 — Cross Tie Details

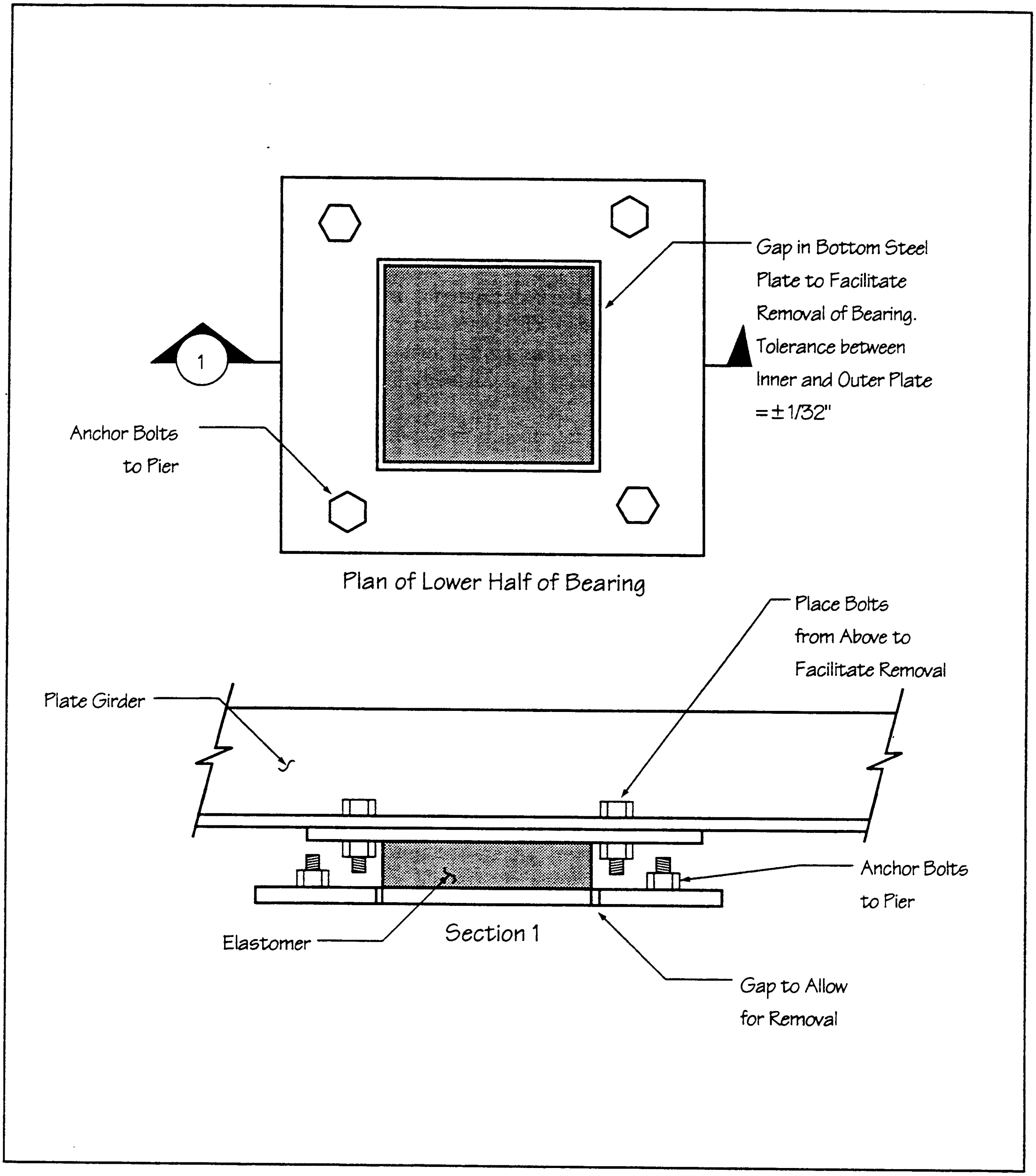

Figure 38 — Bearing Replacement Detail

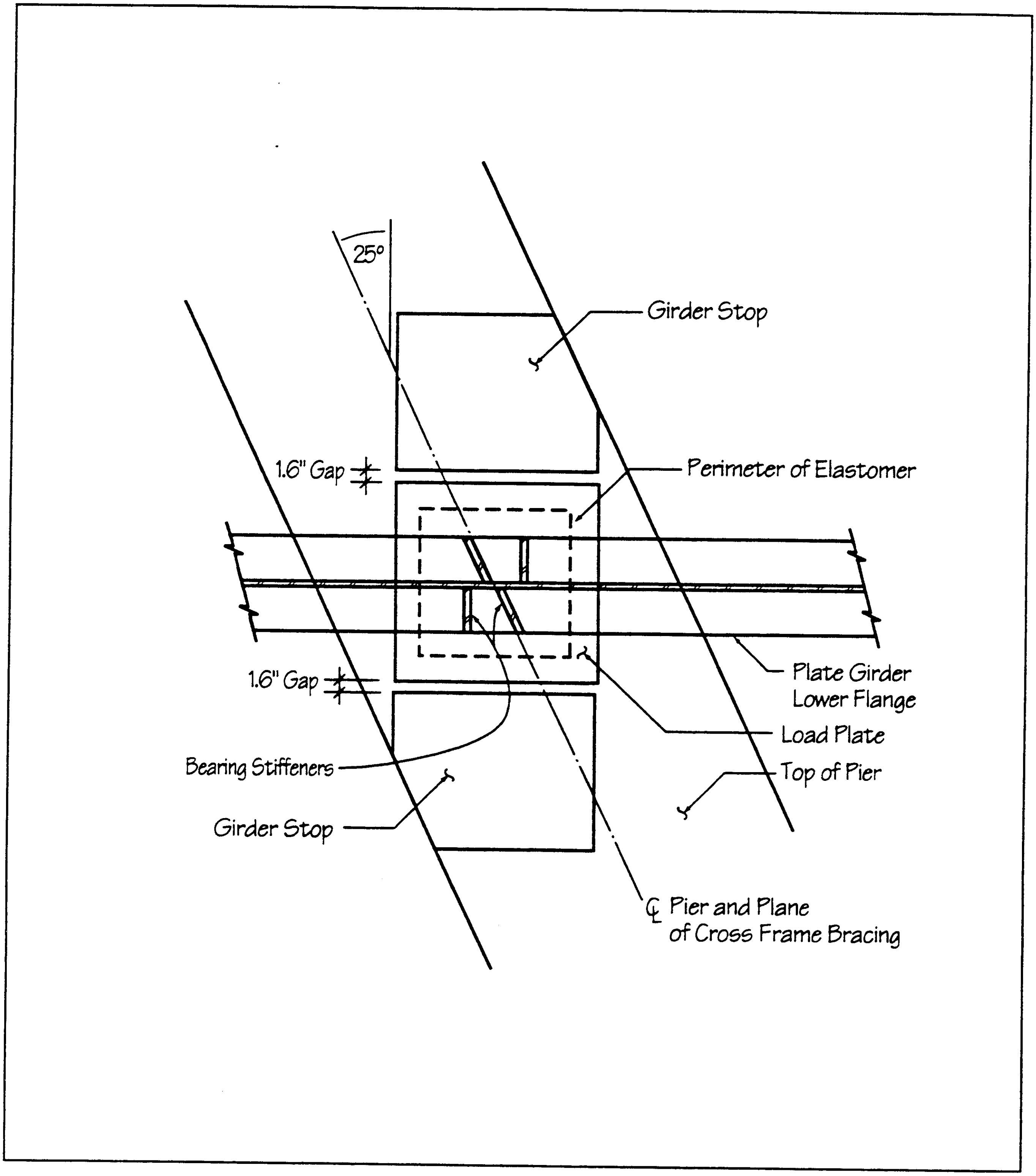

Figure 39 — Plan of Girder Stop Detail

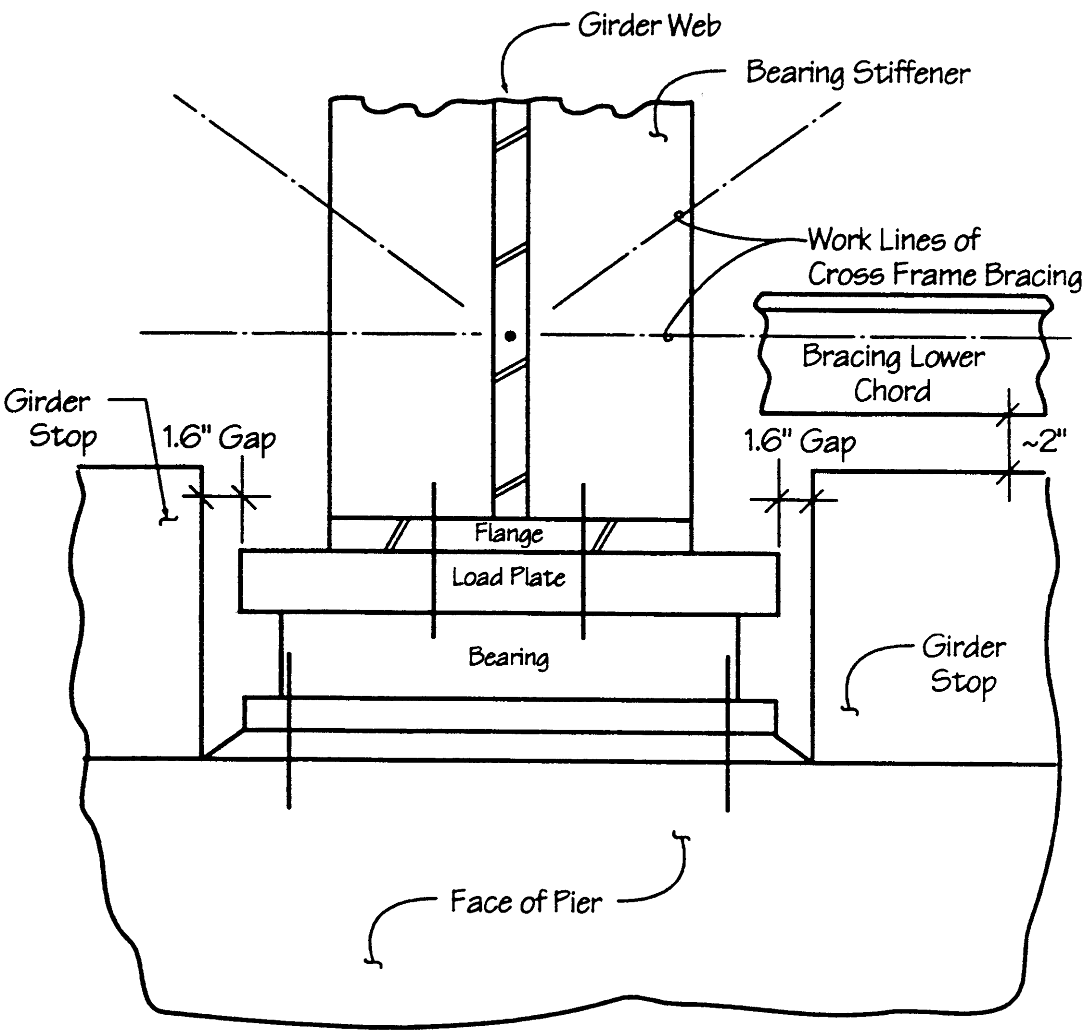

Figure 40 — Elevation of Girder Stop Detail

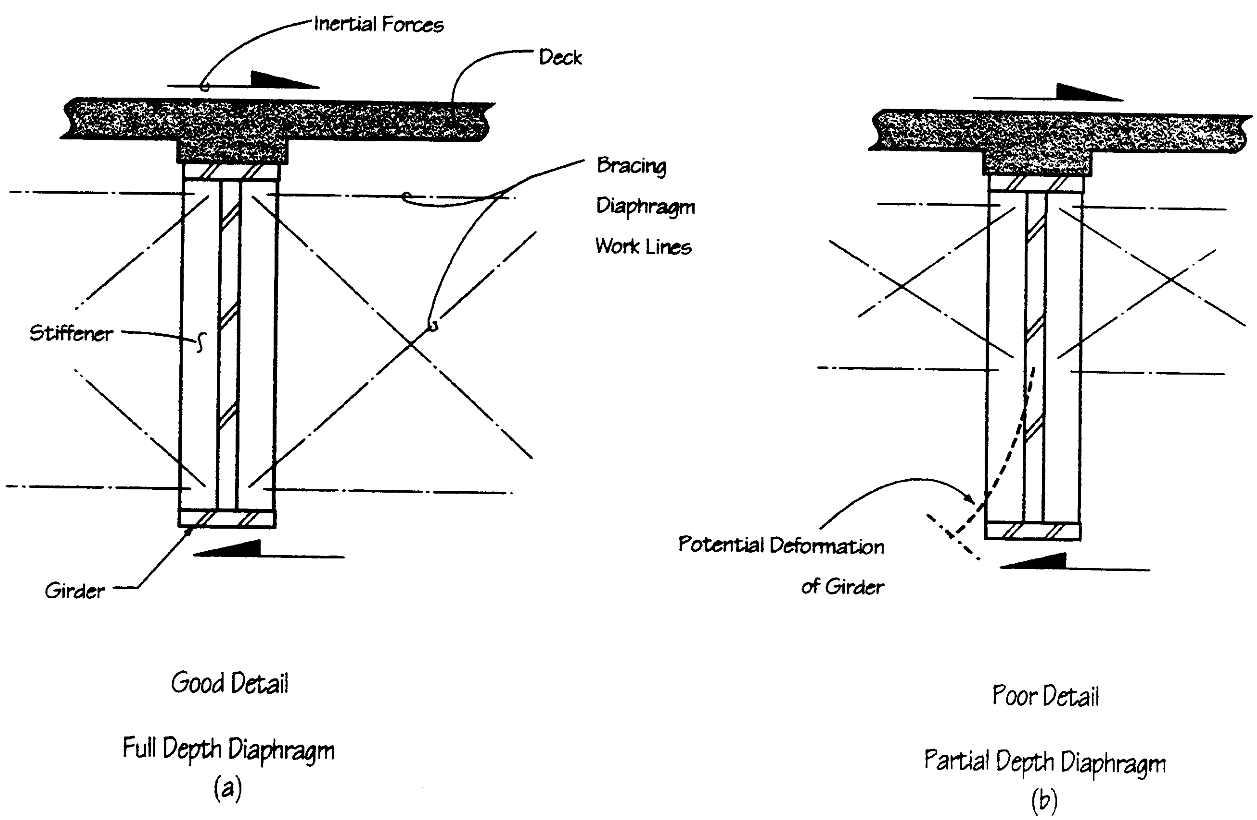

Figure 41 – Conventional Diaphragm Bracing Detail

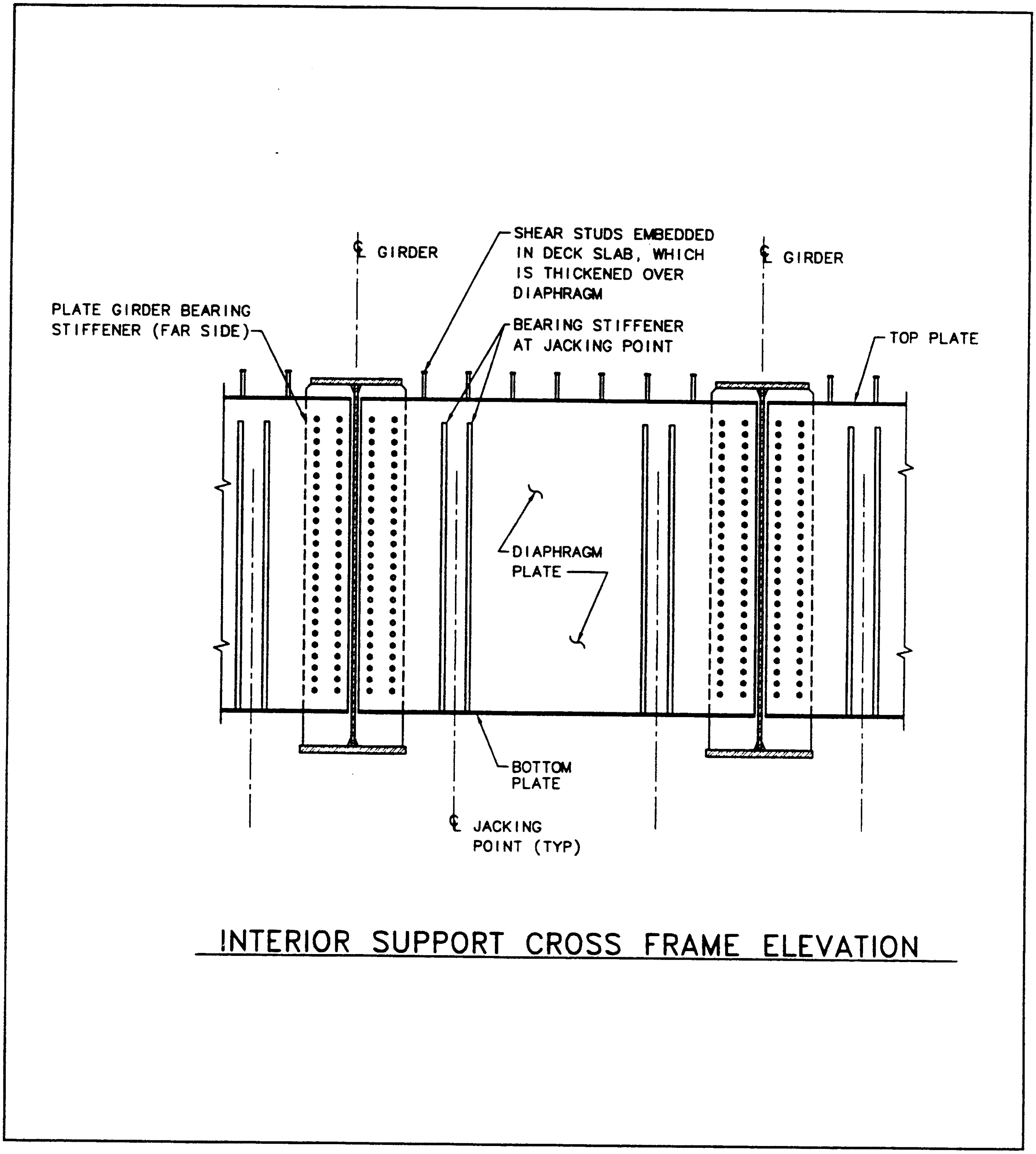

Figure 42 — Solid Plate Diaphragm Detail

Section IV
Closing Statements

SECTION IV — CLOSING STATEMENTS

SEISMIC PERFORMANCE

The seismic performance of this bridge is highly dependent on the manner in which the superstructure is connected to the piers and abutments. Conventional steel bearings, both fixed and sliding, were considered, but the more common elastomeric bearings were selected for the connections. These bearings provide both the longitudinal and transverse restraint for the superstructure, although this restraint is coupled with the flexibility of the bearings. Due to this flexibility, the vibration periods are lengthened, and the inertial forces that are developed are smaller than if rigid restraints had been used. The sizes of the bearings are based on that required for thermal movements alone, and they have not been sized as seismic isolation bearings.

The modal analysis of this structure is not entirely straightforward due to the concentration of mass near the bottoms of the piers. If the base shear force transferred between the pier and the rock is to be correctly estimated, the number of modes used must be carefully selected. The simple hand calculations of the base shear are easy to perform and are quite accurate for this bridge. However the modal analysis requires more modes than the Division I-A Specification leads one to believe is necessary. If the normal modes of vibration are used, nearly 60 modes are required to accurately estimate the seismic forces. On the other hand, if Ritz vectors are used, then only 25 vectors are required. The use of Ritz vectors is relatively new, although some analysis programs do provide them as an alternative to normal modes. The Ritz vector approach was used in this example.

The design of the pier for the seismic load cases results in the use of minimum vertical and horizontal reinforcement. The final piers are shown to be strong enough to resist the earthquake forces without developing any inelastic action. This is the combined result of the ice loading controlled pier size and the use of minimum steel. In the absence of the ice loading constraints, the pier could be reduced significantly in size and still easily resist the seismic forces.

The probable weak link in the substructure is the stability of the piers in a rocking mode. This means that the wall piers would tend to overturn prior to developing plastic hinges at the base of the wall. In this case, such a failure mode is not necessarily undesirable since pier overturning would occur only for larger than design level earthquakes, and complete loss of

SEISMIC PERFORMANCE
(continued)

longitudinal stability of the structure would be prevented by the abutments. Although the design of the abutments is not considered in this example, some thought should be given to the potential loading from longitudinal superstructure movement.

The use of elastomeric bearings had the effect of substantially reducing the magnitude of the forces transferred to the substructure. Due to the relatively low acceleration level, the displacements that the bearings are required to accommodate is not excessive. The elastomeric bearings experience deformation levels that are slightly larger than the allowable strain levels specified in the AASHTO Guidelines for Seismic Isolation Design. The slight overstrain is deemed acceptable provided that a fail-safe transverse restraint system, in the form of girder stops, is supplied. In fact, it is prudent to include such a fail-safe system regardless of the strain levels in the bearings. However, if the seismic design is cycled once more, the elastomeric bearing thicknesses could be increased to better the accommodate the seismic movements.

Section V
References

SECTION V

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Appendix A
Geotechnical Data

APPENDIX A GEOTECHNICAL DATA

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.

SOIL PROFILE TYPE

Type I — Rock at the ground surface.

SITE ACCELERATION

0.15g — Taken from AASHTO seismicity map.

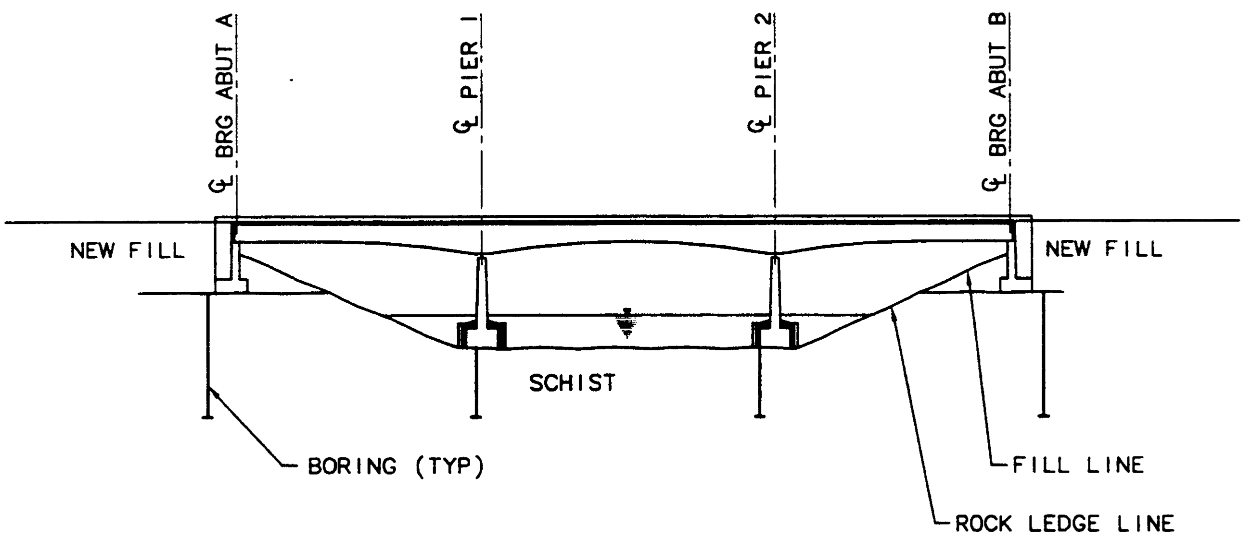
FOUNDATION DESIGN PARAMETERS

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.



LOCATION OF FOUNDATION BORINGS

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

Appendix B
SAP90 V6.0 Beta Input

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FHWA BRIDGE NO 2 / SAP90 (BETA VERSION) INPUT FILE

SYSTEM

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COORDINATE


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            X=1      Y=0   Z=1


JOINT
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802 X=  31.0 Y=   -1.38 Z= 0.0
803 X=  62.0 Y=   -1.41 Z= 0.0
804 X=  93.0 Y=   -1.70 Z= 0.0
811 X= 124.0 Y=   -2.48 Z= 0.0
812 X= 162.0 Y=   -1.60 Z= 0.0
813 X= 200.0 Y=   -1.38 Z= 0.0
814 X= 238.0 Y=   -1.60 Z= 0.0
821 X= 276.0 Y=   -2.48 Z= 0.0
822 X= 307.0 Y=   -1.70 Z= 0.0
823 X= 338.0 Y=   -1.41 Z= 0.0
824 X= 369.0 Y=   -1.38 Z= 0.0
831 X= 400.0 Y=   -1.38 Z= 0.0

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511 X= 124.0 Y= -14.375 Z=0.0
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621 X= 276.0 Y= -9.375  Z=0.0
521 X= 276.0 Y= -14.375 Z=0.0
421 X= 276.0 Y= -19.375 Z=0.0
321 X= 276.0 Y= -45.375 Z=0.0
322 X= 276.0 Y= -45.175 Z=0.0
221 X= 276.0 Y= -50.375 Z=0.0

LOCAL
  ADD=211,221,10 CSYS=PIER
  ADD=801,831,30 CSYS=PIER
  ADD=611,711,100 CSYS=PIER
  ADD=621,721,100 CSYS=PIER

RESTRAINT
  ADD=211,221,10 DOF=U1,U2,U3,R1,R2,R3
```

```
SPRING
   CSYS=PIER
   ADD=801 U1=824.0 U2=148000.0 U3=824.0 R1=6.29E7 R2=3.50E5 R3=0
   ADD=831 U1=824.0 U2=148000.0 U3=824.0 R1=6.29E7 R2=3.50E5 R3=0

MATERIAL

  NAME=S801   TYPE=ISO M=0.166/32.2  W=0.166  IDES=C ;INCLUDES WEIGHT OF X-FRAMES,
OVERLAY, BARRIERS & ETC
    E=519000  U=0.18  A=6.0E-06
  NAME=S803   TYPE=ISO M=0.166/32.2  W=0.166  IDES=C
    E=519000  U=0.18  A=6.0E-06
  NAME=S804   TYPE=ISO M=0.162/32.2  W=0.162  IDES=C
    E=519000  U=0.18  A=6.0E-06
  NAME=S811   TYPE=ISO M=0.143/32.2  W=0.143  IDES=C
    E=519000  U=0.18  A=6.0E-06
  NAME=S812   TYPE=ISO M=0.163/32.2  W=0.163  IDES=C
    E=519000  U=0.18  A=6.0E-06
  NAME=RIGID  TYPE=ISO M=0.0         W=0.0    IDES=C
    E=519000  U=0.18  A=6.0E-06
  NAME=SUB    TYPE=ISO M=0.150/32.2  W=0.150  IDES=C
    E=519000  U=0.18  A=6.0E-06


SECTION

  NAME=S801 MAT=S801 A= 81.0 I=296,36207 J=8.1
  NAME=S803 MAT=S803 A= 81.3 I=311,36353 J=8.1
  NAME=S804 MAT=S804 A= 84.3 I=473,37607 J=8.1
  NAME=S811 MAT=S811 A=104.0 I=996,45988 J=8.1
  NAME=S812 MAT=S812 A= 83.4 I=417,37206 J=8.1
  NAME=S7 MAT=RIGID SH=R T=40.,740.
  NAME=S6 MAT=SUB   SH=R T= 4.00,74.0
  NAME=S5 MAT=SUB   SH=R T= 4.28,74.0
  NAME=S4 MAT=SUB   SH=R T= 4.56,54.0
  NAME=S3 MAT=SUB   SH=R T= 6.00,66.0
  NAME=S2 MAT=SUB   SH=R T=16.00,70.0

NLPROP
  NAME=N6 TYPE=GAP M=0.0 W=0.0
  DOF=U1 KE=813000.0
  DOF=U2 KE=  4320.0
  DOF=U3 KE=  4320.0
  DOF=R1 KE= 1.84E6
  DOF=R2 KE= 3.46E8
  DOF=R3 KE= 0.0


FRAME
  CSYS=0
  801 J=801,802 SEC=S801                PLANE13=+Z
  802 J=802,803 SEC=S801,S803 EIVAR=3,1 PLANE13=+Z
  803 J=803,804 SEC=S803,S804 EIVAR=3,1 PLANE13=+Z
  804 J=804,811 SEC=S804,S811 EIVAR=3,1 PLANE13=+Z
  811 J=811,812 SEC=S811,S812 EIVAR=3,1 PLANE13=+Z
  812 J=812,813 SEC=S812,S801 EIVAR=3,1 PLANE13=+Z
  813 J=813,814 SEC=S801,S812 EIVAR=3,1 PLANE13=+Z
```

```
  814 J=814,821 SEC=S812,S811 EIVAR=3,1 PLANE13=+Z
  821 J=821,822 SEC=S811,S804 EIVAR=3,1 PLANE13=+Z
  822 J=822,823 SEC=S804,S803 EIVAR=3,1 PLANE13=+Z
  823 J=823,824 SEC=S803,S801 EIVAR=3,1 PLANE13=+Z
  824 J=824,831 SEC=S801                PLANE13=+Z

  CSYS=PIER
  211 J=211,311 SEC=S2              PLANE13=+Z
  311 J=311,312 SEC=S3              PLANE13=+Z
  312 J=312,411 SEC=S3,S4 EIVAR=1,1 PLANE13=+Z
  411 J=411,511 SEC=S4,S5 EIVAR=1,1 PLANE13=+Z
  511 J=511,611 SEC=S5,S6 EIVAR=1,1 PLANE13=+Z
  711 J=711,811 SEC=S7              PLANE13=+Z

  221 J=221,321 SEC=S2              PLANE13=+Z
  321 J=321,322 SEC=S3              PLANE13=+Z
  322 J=322,421 SEC=S3,S4 EIVAR=1,1 PLANE13=+Z
  421 J=421,521 SEC=S4,S5 EIVAR=1,1 PLANE13=+Z
  521 J=521,621 SEC=S5,S6 EIVAR=1,1 PLANE13=+Z
  721 J=721,821 SEC=S7              PLANE13=+Z

NLLINK
  CSYS=PIER
  611 J=611,711 NLP=N6 PLANE13=+Z
  621 J=621,721 NLP=N6 PLANE13=+Z

LOAD
  CSYS=0
  NAME=DL
    TYPE=GRAVITY ELEM=FRAME
       ADD=* UY=-1

  NAME=TL
    TYPE=TEMPERATURE ELEM=FRAME
       ADD=801,804,1,821,10   T=10


MODES

  TYPE=RITZ  N=25 ;


FUNCTION

  NAME=S1 NPL=1
    0.0     2.50
    0.333   2.50
    0.4     2.21
    0.5     1.90
    0.6     1.69
    0.7     1.52
    0.8     1.39
    0.9     1.29
    1.0     1.20
    1.2     1.06
    1.4     0.96
```

```
      1.6     0.88
      1.8     0.81
      2.0     0.76
      2.5     0.65
      3.0     0.58
      3.5     0.52
      4.0     0.48
     10.0     0.26
    100.0     0.06

SPEC

  CSYS=0
  NAME=EQLONG    MODC=CQC    DAMP=0.05
    ACC=X FUNC=S1   SF=32.2*0.15*1.0
  NAME=EQTRAN    MODC=CQC    DAMP=0.05
    ACC=Z FUNC=S1   SF=32.2*0.15*1.0
```

Appendix C
Excerpts from AASHTO Isolation Guide

Guide Specifications for Seismic Isolation Design

June 1991

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(1) The value of k_{min} shall be based on the minimum Effective Stiffnesses of individual Isolator Units as established by the cyclic tests of Section 13.2c(2) at a displacement amplitude equal to the Design Displacement.

(2) The value of k_{max} shall be based on the maximum Effective Stiffnesses of individual Isolator Units as established by the cyclic tests of Section 13.2c(2) at a displacement amplitude equal to the Design Displacement.

(b) Equivalent Damping. The Equivalent Viscous Damping ratio (β) of the Isolation System shall be calculated as:

$$\beta = \frac{1}{2\pi} \times \frac{\text{TotalArea}}{\Sigma\, k_{max} d_i^2} \qquad (7)$$

where the Total Area shall be taken as the sum of the areas of the hysteresis loops of all Isolators and the hysteresis loop area of each Isolator shall be taken as the minimum area of the three hysteresis loops established by the cyclic tests of Section 13.2c(2) at a displacement amplitude equal to the Design Displacement.

14. Elastomeric Bearings

14.1 General

The following shall be considered supplemental to Article 14.1 of the AASHTO *Standard Specifications for Highway Bridges*.

Elastomeric bearings utilized in implementing seismic isolation design shall be designed by the procedures and specifications given in the following Subsections. Additional test requirements for seismic isolation bearings are given in Section 15. The design procedures are based on service loads excluding impact. The elastomeric bearings must be reinforced using integrally bonded steel reinforcement. Fabric reinforcement is not permitted.

14.2 Definitions

The following shall be considered in addition to those given in Article 14.2.1 of the AASHTO *Standard Specifications*:

A_b = Bonded area of rubber;

A_r = Reduced net bonded area of rubber, $A_b(1 - \Delta/B)$;

B = Plan dimension in loaded direction of rectangular bearing or diameter of circular bearing;

d_i = Lateral displacement under earthquake loads as specified in Section 2;

E = Modulus of elasticity of elastomer;

K = Material constant;

P = Maximum vertical load resulting from the combination of dead load plus live load (including seismic live load, if applicable) using a γ factor of 1;

ε_{eq} = Shear strain due to d_i, the seismic design displacement;

ε_{sc} = Shear strain due to vertical loads;

ε_{sh} = Shear strain due to maximum horizontal displacement resulting from creep, post-tensioning, shrinkage, and thermal effects computed between the installation temperature and the least favorable extreme temperature;

ε_{sr} = Shear strain due to imposed rotation;

ε_c = Compression strain in bearing due to vertical loads;

ε_u = Minimum elongation-at-break of rubber;

θ = Rotation imposed on bearing;

Δ = Shear deflection in the bearing.

Note: The term t in the definition of S in Article 14.2.1 of the AASHTO *Standard Specifications* should be designated as t_i.

14.3 Shear Strain Components for Isolation Design

The various components of shear strain in the bearing are computed as follows:

14.3.1 Shear strain (ε_{sc}) due to compression by vertical loads is given by

$$\varepsilon_{sc} = 6S\varepsilon_c$$

where

$$\varepsilon_c = \frac{\Delta_c}{\Sigma t_i} = \frac{\Delta_c}{T} = \frac{P}{A_r E(1 + 2K\,S^2)}$$

The effects of creep of the elastomer shall be added to the instantaneous compressive deflection, Δ_c, when considering long-term deflections. They are not to be included in the calculation of Section 14.5. Long-term deflections shall be computed from information relevant to the elastomer compound used if it is available. If not, the values given in Article 14.2.2 of the AASHTO *Standard Specifications* shall be used as a guide.

14.3.2 Shear strain (ε_{sh}) due to imposed lateral displacement is given by

$$\varepsilon_{sh} = \frac{\Delta_s}{T}$$

where $T = \Sigma t_i$, the sum of the thicknesses of the deformable rubber layers.

14.3.3 Shear strain (ε_{eq}) due to earthquake-imposed displacement is given by

$$\varepsilon_{eq} = \frac{d_i}{T}$$

14.3.4 Shear strain (ε_{sr}) due to rotation is given by

$$\varepsilon_{sr} = \frac{B^2\theta}{2t_i T}$$

14.4 Limiting Criteria for Allowable Vertical Loads

The allowable vertical load on an elastomeric isolation bearing is not specified explicitly. The limits on vertical load are governed indirectly by limitations on the equivalent shear strain in the rubber due to different load combinations and to stability requirements. The permissible shear strain in the rubber is expressed as φ times the minimum specified elongation-at-break (ε_u). The value of φ is dependent on the load combination under consideration.

14.5 Service Load Combinations

The following two criteria shall be satisfied for service loads which include dead load plus live load, thermal, creep, shrinkage and rotation.

14.5.1 $0.5\ \varepsilon_u \geq \varepsilon_{sc} + \varepsilon_{sh} + \varepsilon_{sr}$

and

14.5.2 $0.33\ \varepsilon_u \geq \varepsilon_{sc}$

In no case shall 0.5 ε_u exceed 5.0.

14.6 Seismic Load Combinations

The following criterion shall be satisfied for seismic loads which include dead load and seismic live load, seismic design displacements and rotation.

$$0.75\ \varepsilon_u > \varepsilon_{sc} + \varepsilon_{eq} + \varepsilon_{sr}$$

14.7 Stability Against Overturning

Elastomeric isolation bearings shall be shown either by test or analysis to be capable of resisting 1.2D + E or 0.8D – E where D is the dead load and E is any vertical load resulting from earthquake effects at seismic design displacements and defined in Section 12.3.

C10 and C11. Design Forces for SPC B, C, and D

Design forces for a seismically isolated bridge are obtained using the same load combinations as for a conventionally designed bridge. For bridges designated SPC B, foundation design forces are determined based on one-half the R value used for column design; they need not be greater than the elastic forces. This is consistent with the foundation design procedure for conventionally designed bridges.

For bridges designated C or D, the foundation design forces need not exceed the elastic forces nor the forces resulting from plastic hinging in the columns.

C12. Other Requirements

C12.1 Non-Seismic Lateral Forces

Since an element of flexibility is an essential part of an isolation system (Introduction to Commentary), it is important that the isolation system also provide sufficient rigidity to resist more frequently occurring wind and braking loads. This requires an elastic restraint system with higher initial stiffness than the element of flexibility (see Figure C4). Limits on displacements resulting from non-seismic loads need to be satisfactory to the Design Engineer.

C12.2 Lateral Restoring Force

The basic premise of these seismic isolation design provisions is that the energy dissipation of the system can be expressed in terms of equivalent viscous damping and the stiffness by an effective linear stiffness. The requirement of this section provides the basis for which this criteria is met.

Systems that do not meet this requirement are not excluded; however, the analysis requirements (Section 7.1) and vertical load stability requirements are more stringent.

C12.3 Vertical Load Stability

This section provides minimum requirements for the design of the Isolation System. The detailed design requirements of the system will be dependent on the type of system. The multipliers of 1.5 and 3.0 on the total design displacement are based on a design response spectra corresponding to a 475-year return period event. If a maximum credible response spectra is used for the design of the isolation, these multipliers are reduced to 1.1 and 2.2, respectively. In some of the low seismic risk areas ($A < 0.25$) of the United States, a multiplier of 2.0 and 4.0 may be appropriate since a longer return period event (2,400 years) may be up to two times greater than the 475-year event.

C13. Required Tests of Isolation System

The code requirements are predicated on the fact that the isolation system design is based on tested properties of prototype isolators. This section provides a comprehensive set of tests to both establish the design properties of the system and then determine the adequacy of the tested properties. Systems that have been previously tested with this specific set of tests on similar type and size of isolator units do not need to have these tests repeated. Design properties must therefore be based on manufacturers' pre-approved or certified test data. Extrapolation of design properties from tests of similar type and size of isolator units is permissible.

C14. Elastomeric Bearings

Elastomeric bearings which are used for seismic isolation will be subjected to earthquake induced displacements (d_i) and must therefore be designed to safely carry the vertical loads at these displacements. Since earthquakes are infrequently occurring events, the factors of safety required under these circumstances will be different from those required for more frequently occurring loads.

Since the primary design parameter for earthquake loading is the displacement (d_i) of the bearing, the design procedures must be capable of incorporating this displacement in a logical consistent manner. The requirements of Section 14.2 of the AASHTO *Standard Specifications* limit vertical loads by use of a limiting compressive stress and therefore do not have a mechanism for including the simultaneous effects of seismic displacements. The British Specifications BE 1/76 and BS 5400 recognize that shear strains are induced in reinforced bearings by both compression and shear deformation. In these codes, the sum of these shear strains is limited to a proportion of the elongation-at-break of the rubber. The proportion (1/2 or 1/3 for service load combinations and 3/4 for seismic load combinations) is a function of the loading type.

Since the approach used in BE 1/76 and BS 5400 incorporates shear deformation as part of the criteria, it can be readily modified for seismic isolation bearings. The design requirements given are based on the appropriate modifications to BE 1/76 and BS 5400. It is assumed that the displacements (d_i) due to earthquake loads have been determined by the provisions of Section 7 of these *Guide Specifications*.

The more conservative aspects of BE 1/76 and BS 5400 have been used. For example, BS 5400 requires the summation of compression, thermal and rotational shear strains and requires this to be less than 5.0. BE 1/76 requires the summation of only the compression and thermal shear strains and requires this to be less than $\varepsilon_u/2$. These requirements require the summation of the three different shear strains with a limit of $\varepsilon_u/2$ where $\varepsilon_u/2$ may not exceed 5.0.

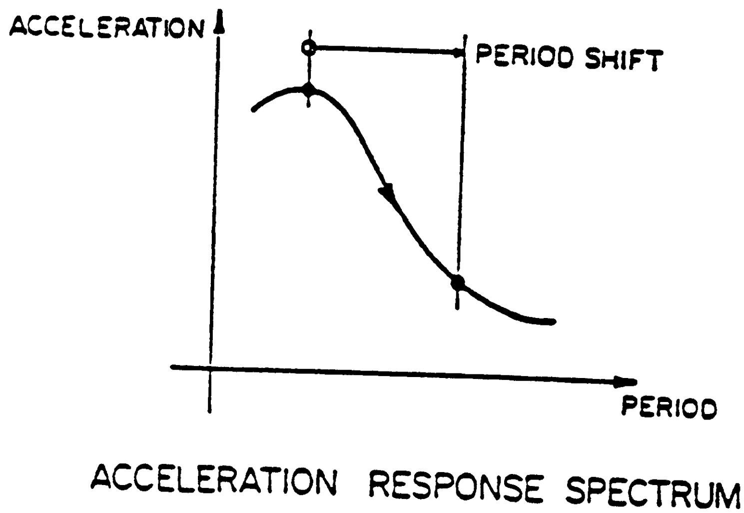

Figure C1.
Idealized Force Response Curve

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