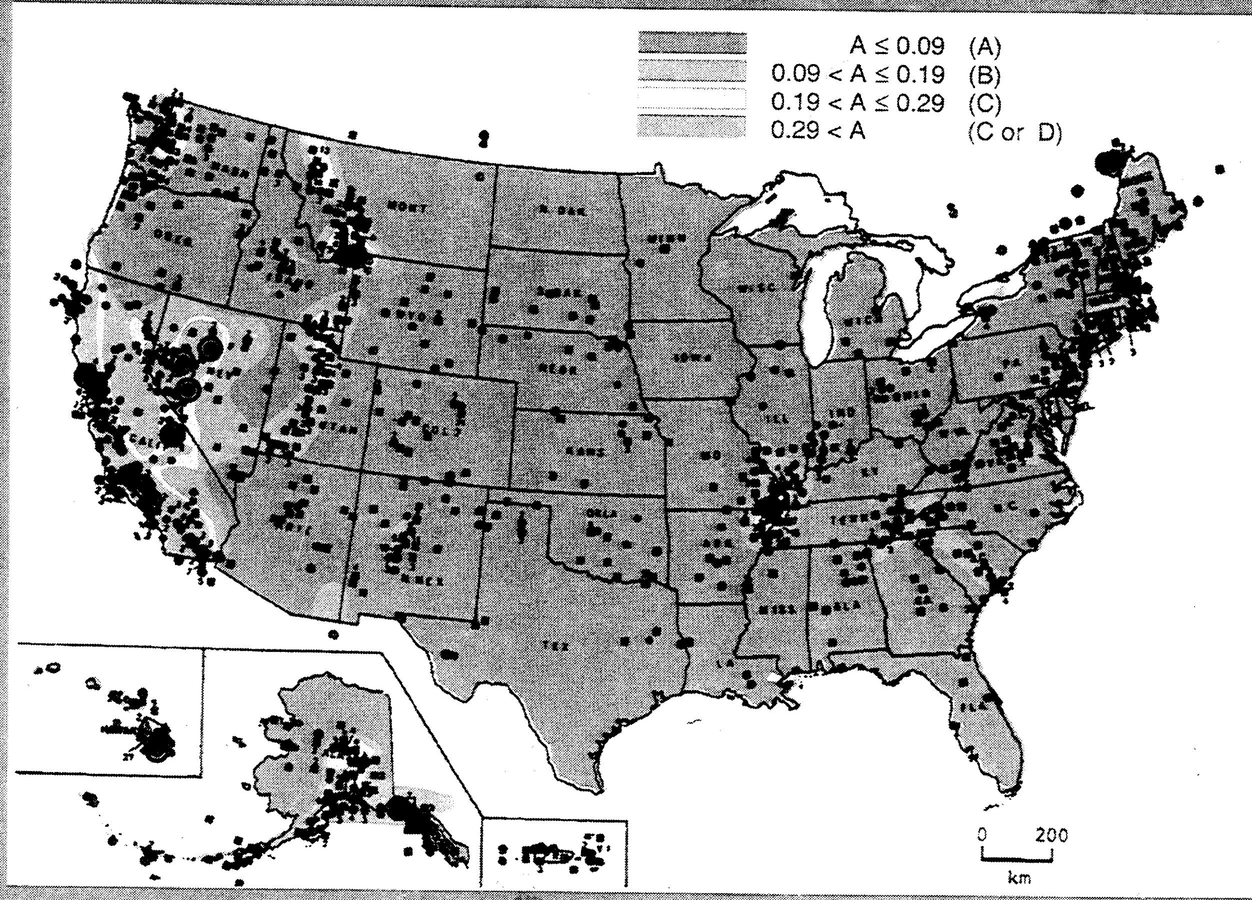
U.S. Department
of Transportation
Federal Highway
Administration

October 1996

Seismic Design of Bridges
Design Example No. 5
Nine-Span Viaduct
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. 5

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Steering Group

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PLEASE NOTE

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Section I
Introduction

PURPOSE OF DESIGN EXAMPLE

This is the fifth 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")

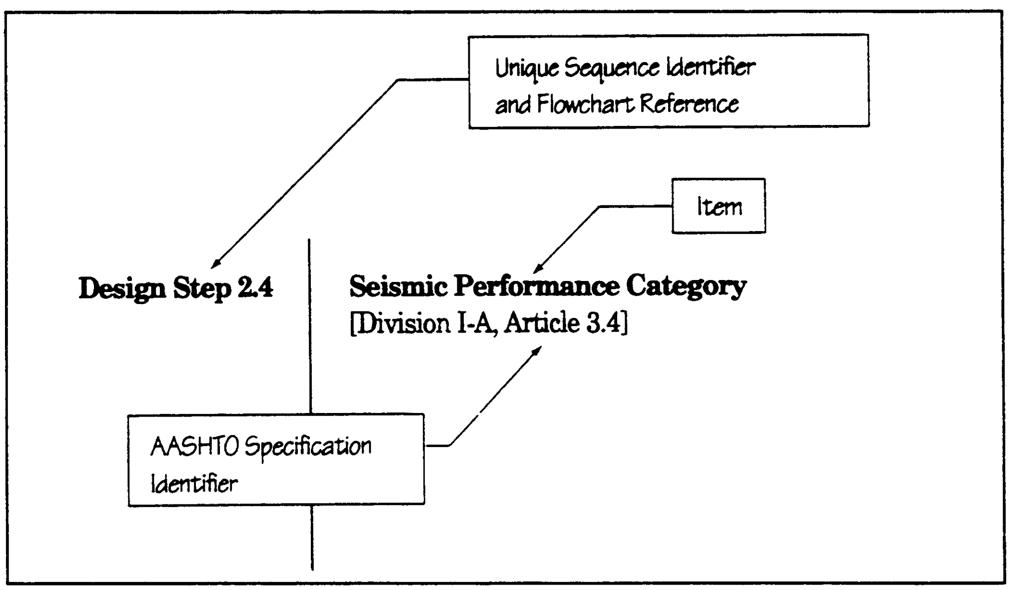
Standard Specifications for Highway Bridges, Division I-A, Seismic Design, American Association of State Highway and Transportation Officials, Inc., 15th Edition, as amended by the Interim Specifications-Bridges-1995.

FLOWCHARTS AND DESIGN STEPS

This fifth 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.

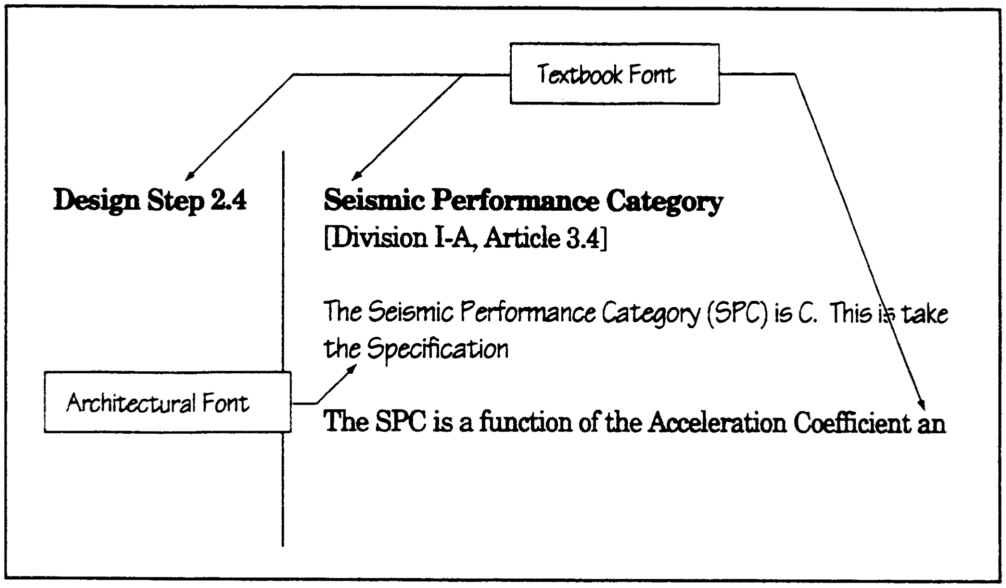
An example is shown below.



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.

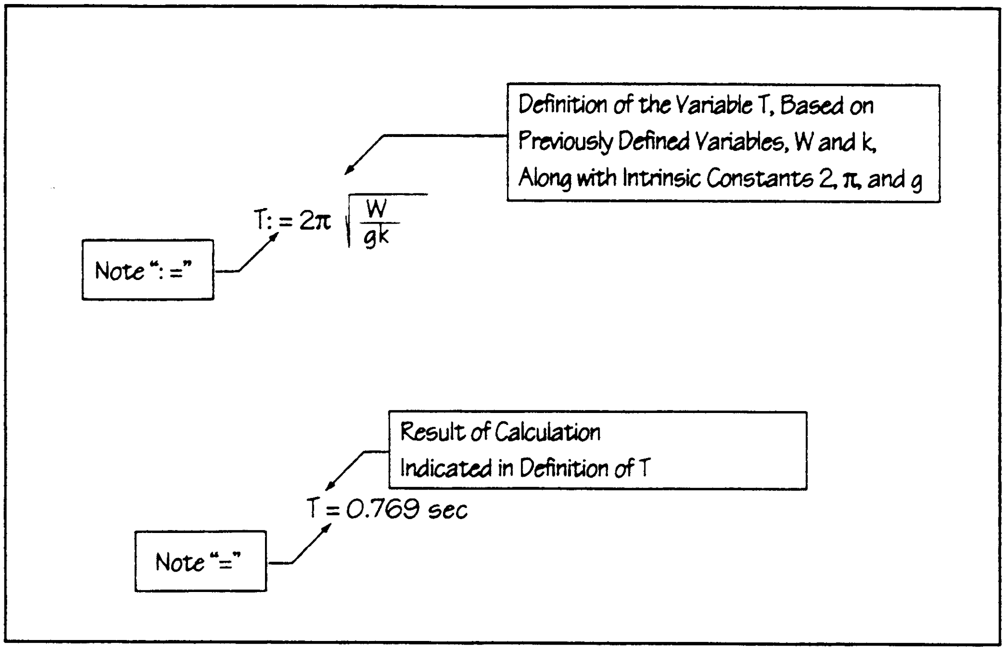


USE OF MATHCAD®

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.

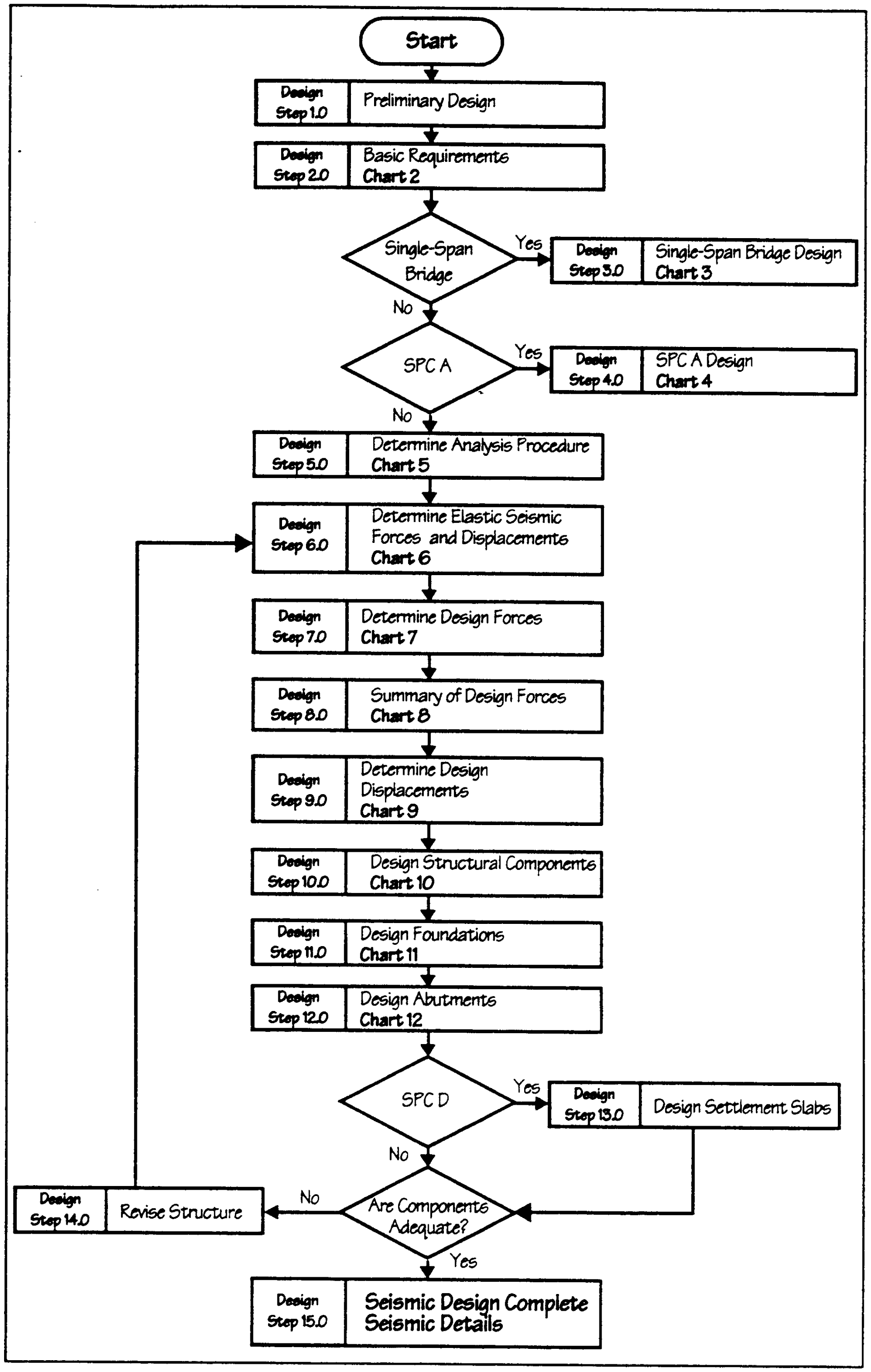


Note that Mathcad® carries the full precision of the variables throughout the calculations, even though the listed result of a calculation is rounded off. Thus, hand-calculated checks made using intermediate rounded results may not yield the same result as the number being checked.

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

Section II
Flowcharts

FLOWCHARTS



Main Flowchart — Seismic Design AASHTO Division I-A

FLOWCHARTS
(continued)

Key to Detailed Flowcharts

FLOWCHARTS
(continued)

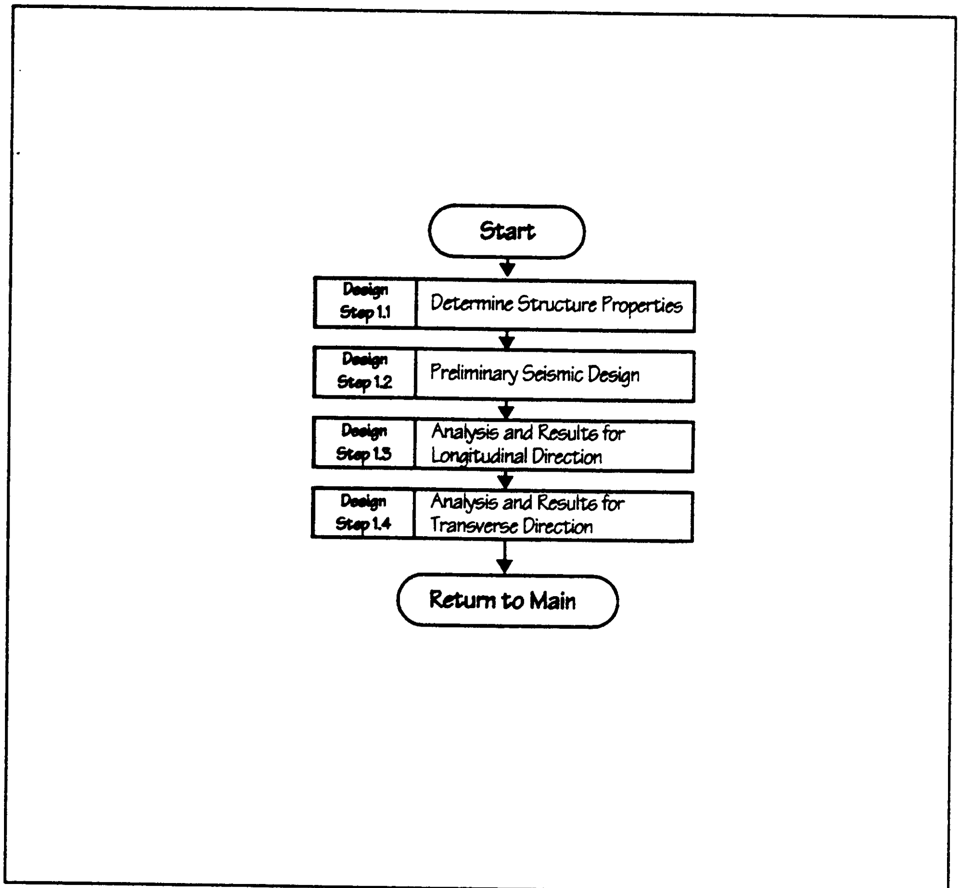


Chart 1 — Preliminary Design

FLOWCHARTS
(continued)

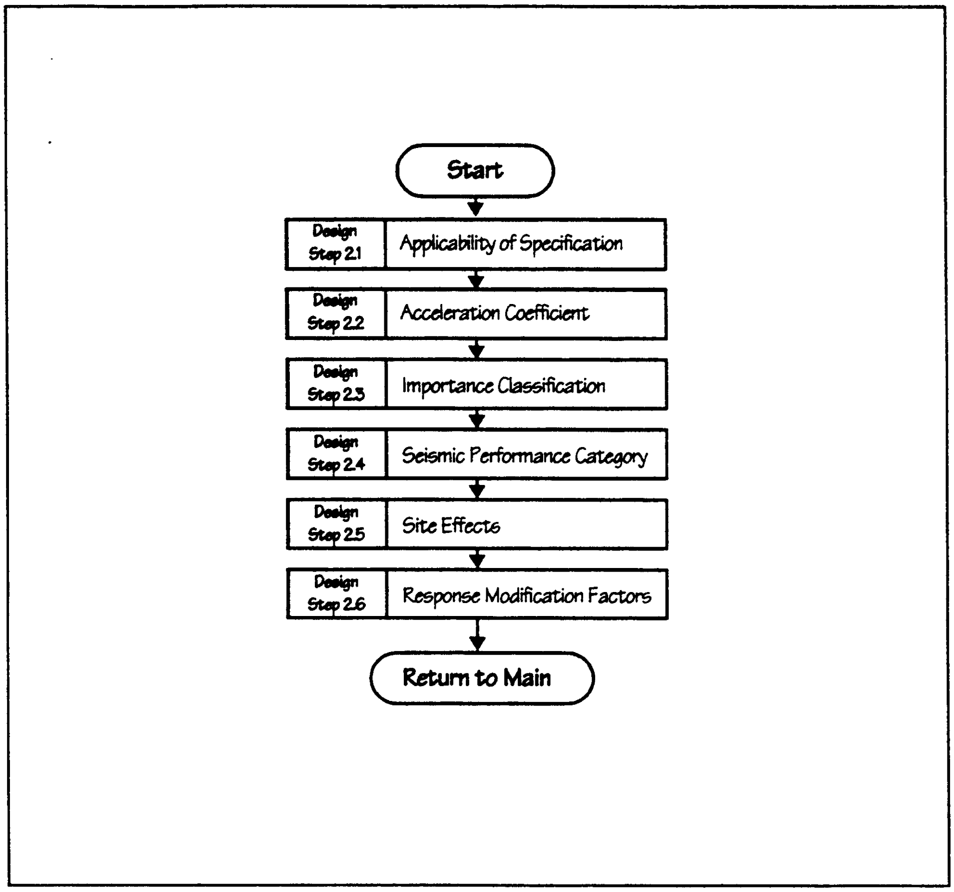


Chart 2 — Basic Requirements

FLOWCHARTS
(continued)

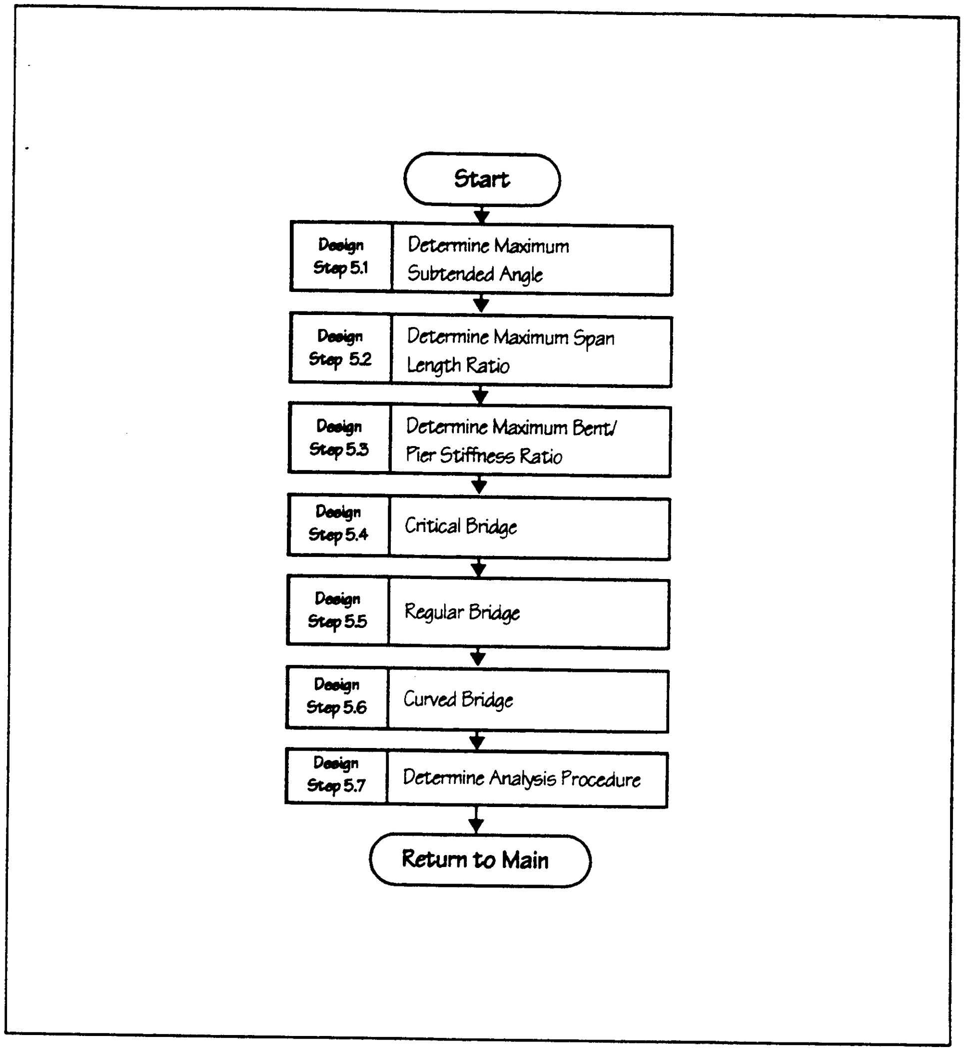


Chart 5 — Determine Analysis Procedure

FLOWCHARTS
(continued)

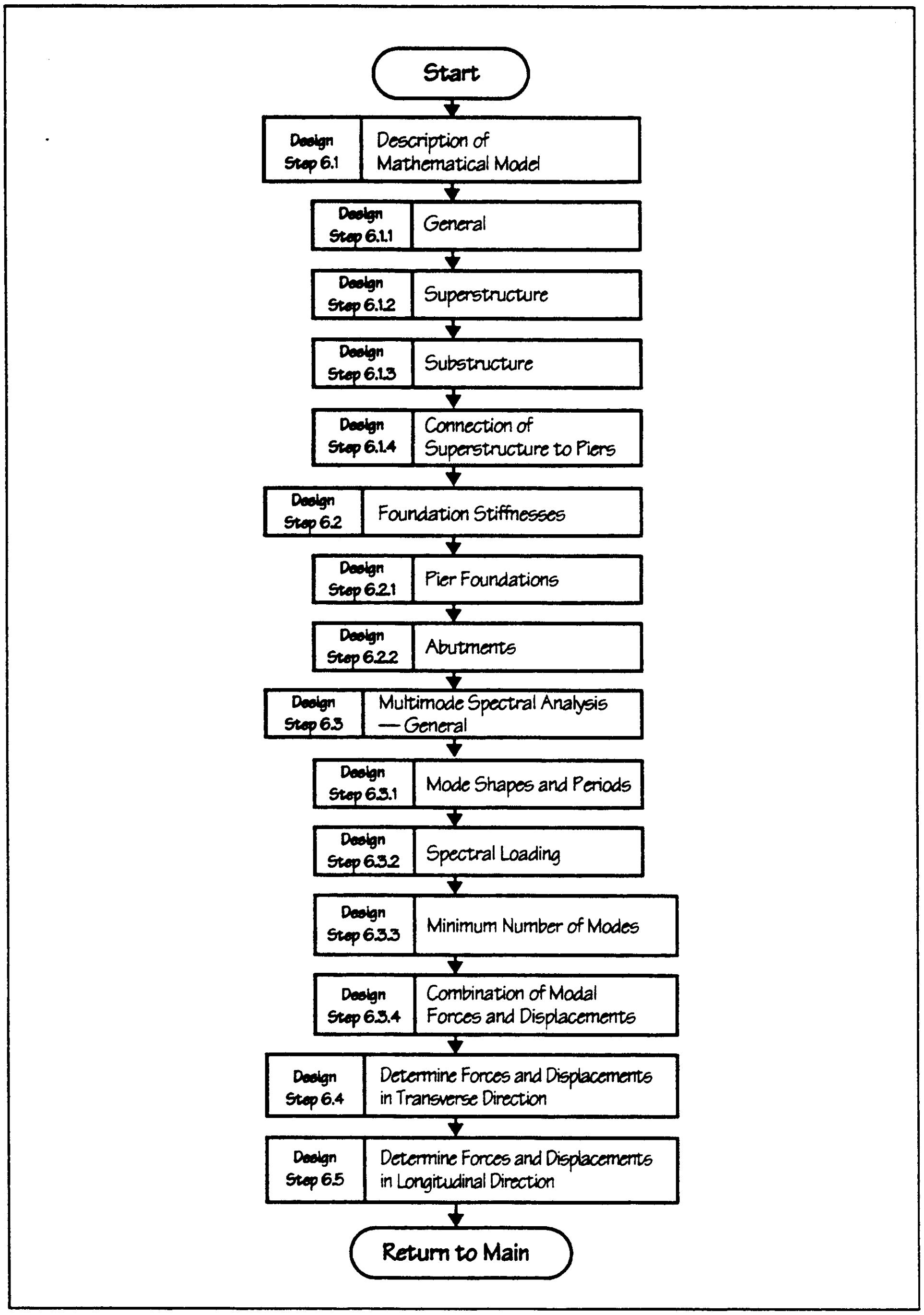


Chart 6 — Determine Elastic Seismic Forces and Displacements

FLOWCHARTS
(continued)

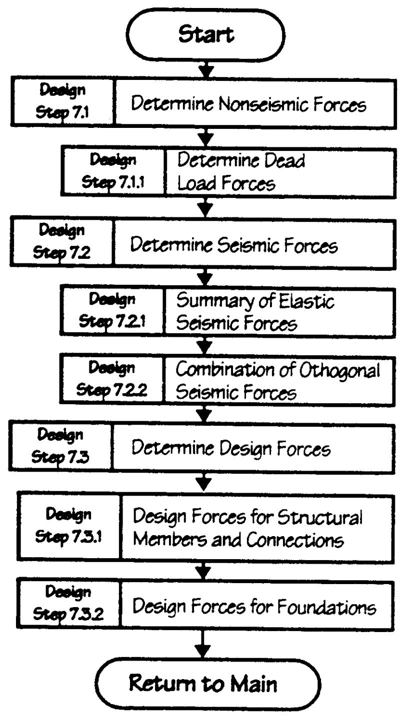


Chart 7 — Determine Design Forces (SPC B)

FLOWCHARTS
(continued)

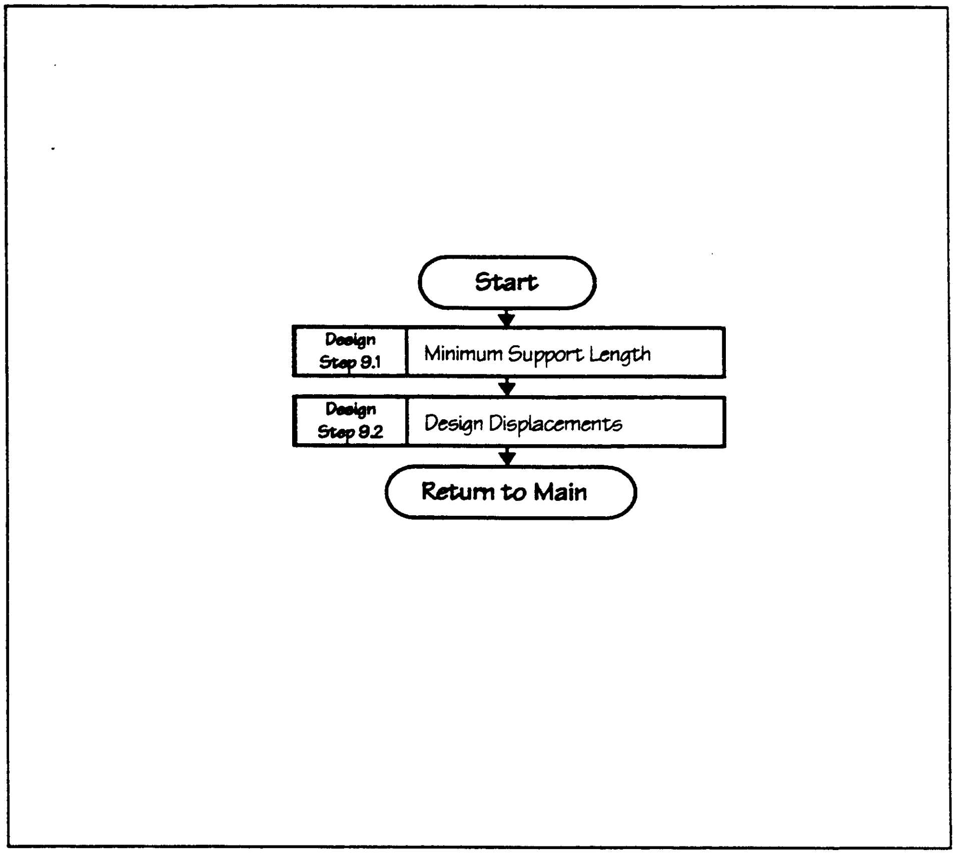


Chart 9 — Determine Design Displacements

FLOWCHARTS
(continued)

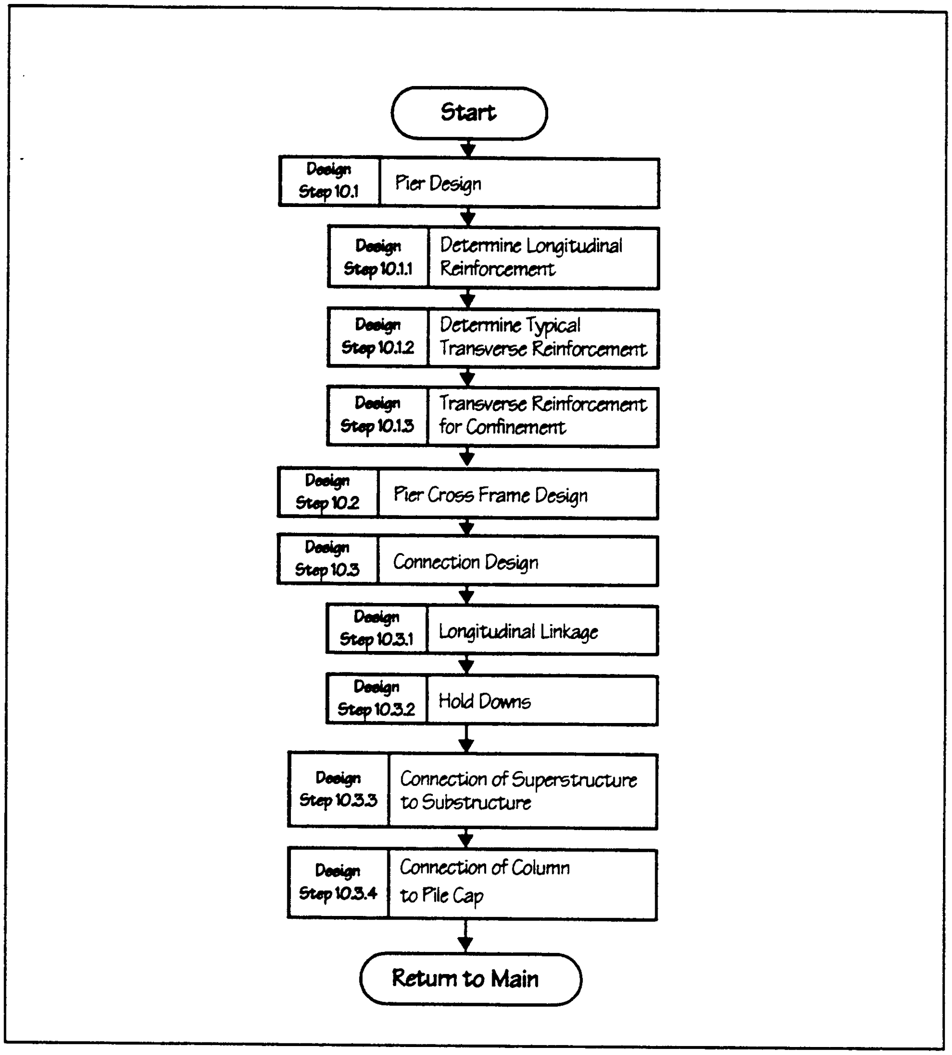


Chart 10 — Design Structural Components

FLOWCHARTS
(continued)

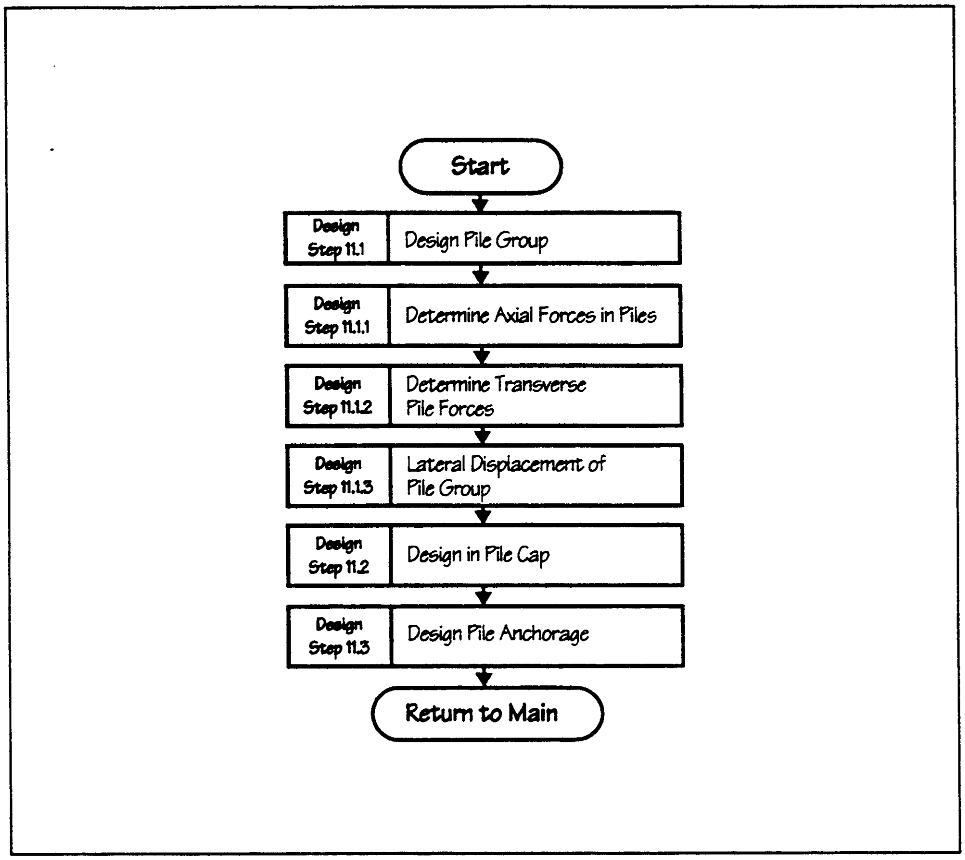


Chart 11 — Design Foundations

Section III
Analysis and Design

SECTION III — ANALYSIS AND DESIGN

DATA

The bridge is to be built across a large river and flood plain in the inland Pacific Northwest in a seismic zone with an acceleration coefficient of 0.15g. The subsurface conditions were derived from borings drilled along the bridge alignment. Soils consist of coarse alluvial flood deposits overlying volcaniclastic sediments. The alluvial deposits are approximately 50 feet deep and consist of very dense sand, gravel, and cobbles. The volcaniclastic sediments consist of very dense/hard silt (very soft tuff). Geotechnical information for the bridge site is provided in Appendix A.

The configuration of the bridge has nine spans totaling 1488 feet and consisting of two units: a four-span tangent (Unit 1) and a five-span with a 1300-foot radius curve (Unit 2). The superstructure is composed of four steel plate girders with a composite cast-in-place concrete deck. The substructure elements, seat-type abutments, and single-column intermediate piers are all cast-in-place concrete supported on steel H-piles. All substructure elements are oriented normal to the centerline of the bridge. Figure 1 (a to d) provides details of the configuration.

Because the bridge crosses the flood plain and main channel of a sizable river, it is assumed that the column size of the intermediate piers is not controlled by seismic loading. Flow issues and ice loading have dictated the size requirements for the pier columns. The configuration of intermediate piers is shown in Figure 1c.

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 Specification-Bridges-1995.

FEATURES

ISSUES EMPHASIZED FOR THIS EXAMPLE

- Preliminary Seismic Design
- Multiple Unit Behavior
- Deck Force Transfer to Piers Through Steel Cross Frames
- SPC B Effects on Single-Column Pier Design
- Steel Pile Design

BRIDGE DATA
(continued)

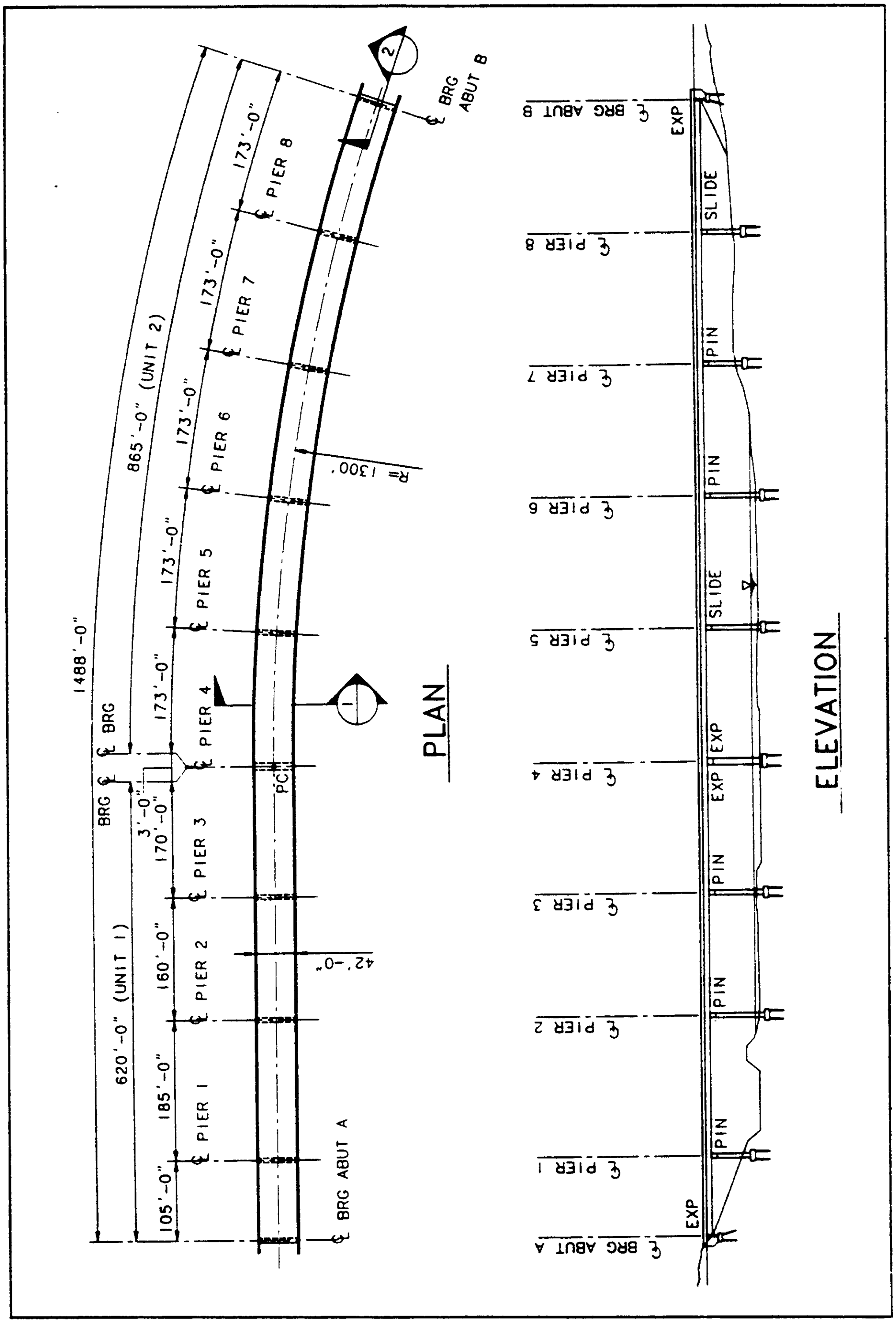


Figure 1a — Bridge No. 5 - Plan and Elevation

BRIDGE DATA
(continued)

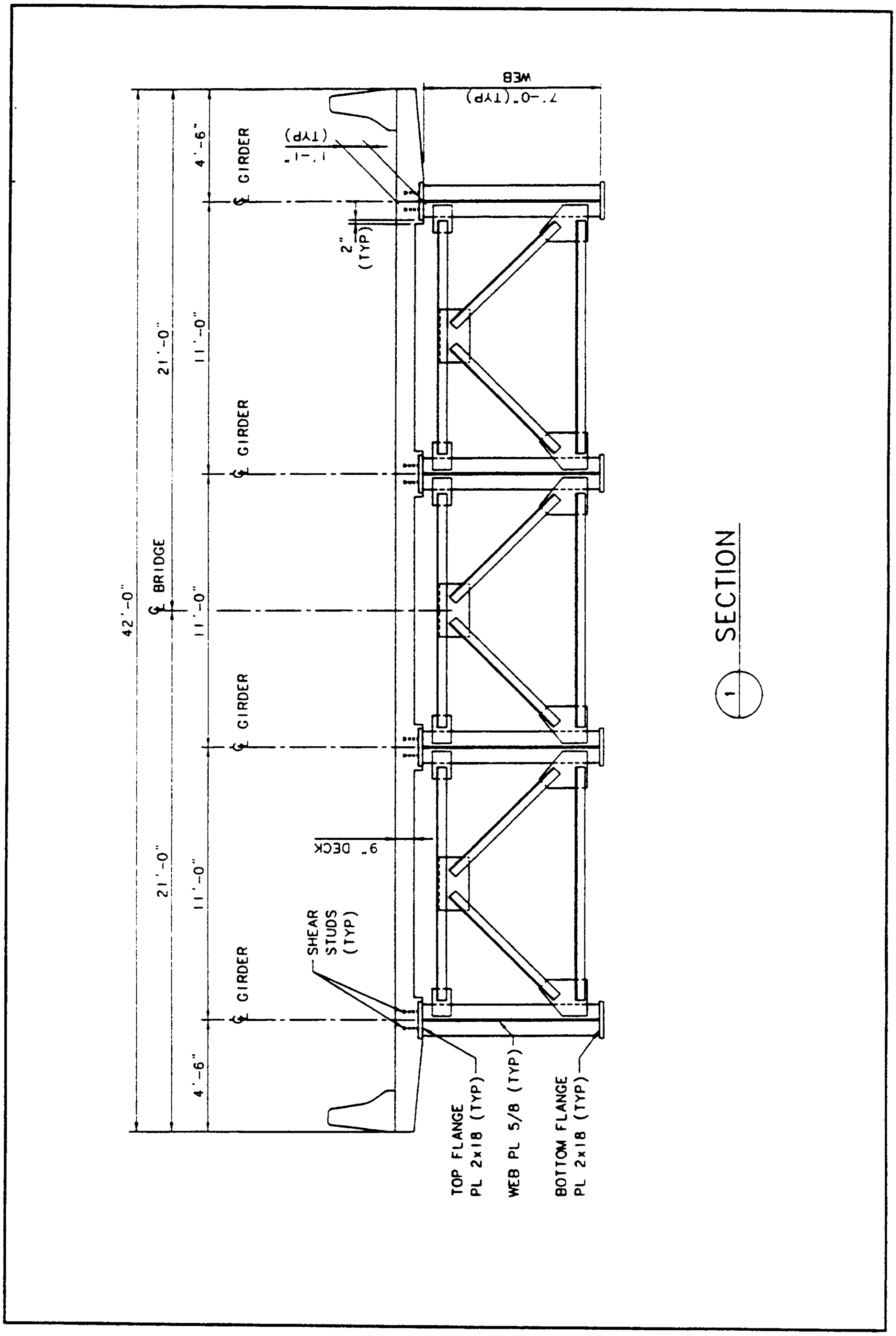


Figure 1b — Bridge No. 5 - Typical Cross Section

BRIDGE DATA
(continued)

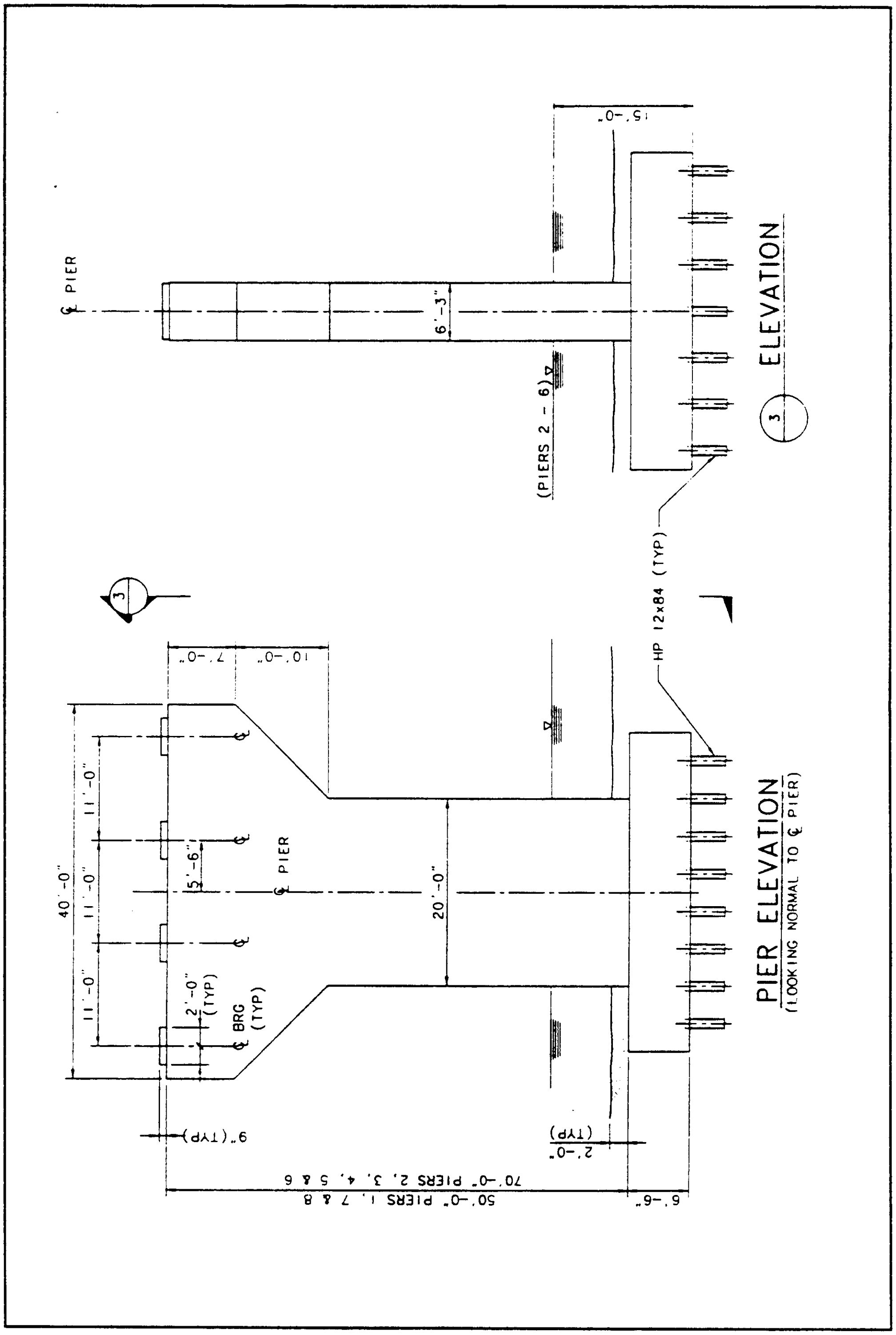


Figure 1c — Bridge No. 5 - Intermediate Pier Elevations

BRIDGE DATA
(continued)

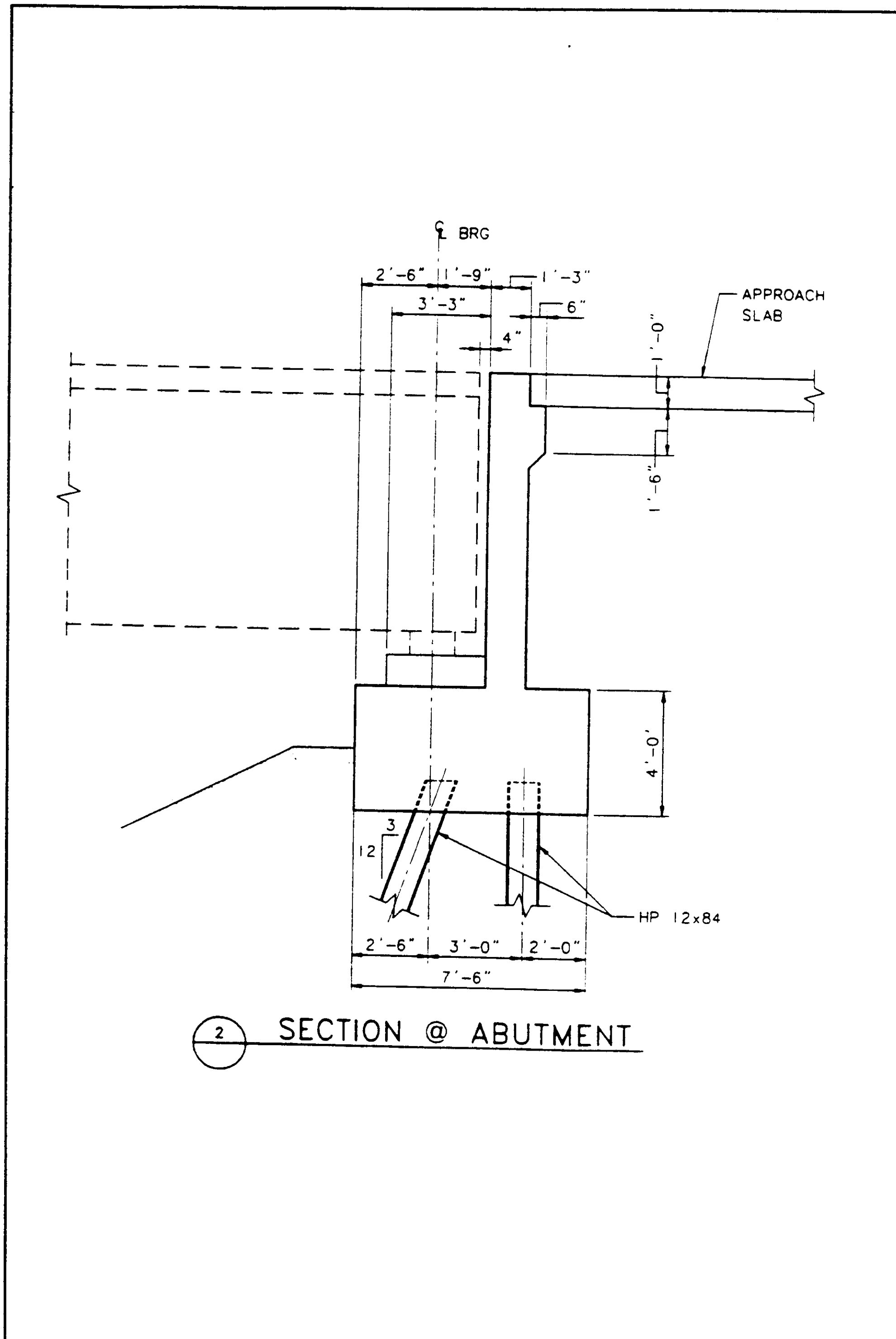


Figure 1d — Bridge No. 5 - Seat-Type Abutment

SOLUTION

DESIGN STEP 1 — PRELIMINARY DESIGN

Preliminary design is the first step in refining the design. Its purpose is to obtain reasonable sizes for elements in the structure without spending a great deal of effort. The emphasis for this example is seismic forces, though data presented here assumes that similar steps have been made for static analysis and design for other forces and effects, i.e., dead loads, live loads, temperature, ice, scour, etc.

The preliminary seismic design of the bridge has been completed and selected calculations are shown in this section. The following assumptions were used for preliminary seismic design. Effects of the structure's curvature are ignored for preliminary seismic design.

In the longitudinal direction, the pinned intermediate pier columns (Pier Nos. 1, 2, and 3 in Unit 1, and Pier Nos. 6 and 7 in Unit 2) are assumed to resist the entire longitudinal seismic force. The seat-type abutments and the expansion joint at Pier No. 4 will accommodate significant motion in the longitudinal direction and will provide restraint in the transverse direction. The two units of the bridge are assumed to act independently for longitudinal motion. This behavior is illustrated in Figure 2.

In the transverse direction, the structure is assumed to act as a two-rigid link system pivoting at the abutments with maximum transverse displacement at Pier No. 4. All of the intermediate piers and abutments are assumed to participate in resisting the transverse seismic force. This behavior is illustrated in Figure 3.

In both the transverse and longitudinal directions, the column bases are considered fixed against rotation at the bottom of the pile cap to account for expected lack of foundation flexibility. The moment of inertia of the column was assumed to be that of the full cross section, "Igross." This assumed fixed base condition using the gross cross section of the column should provide an upper bound to the foundation stiffness. A stiffer system will have shorter periods of vibration and higher values of C_{sm}, the elastic response coefficient, as shown in Figure 22. This results in higher, conservative levels of seismic forces for preliminary design.

DESIGN STEP 1
(continued)

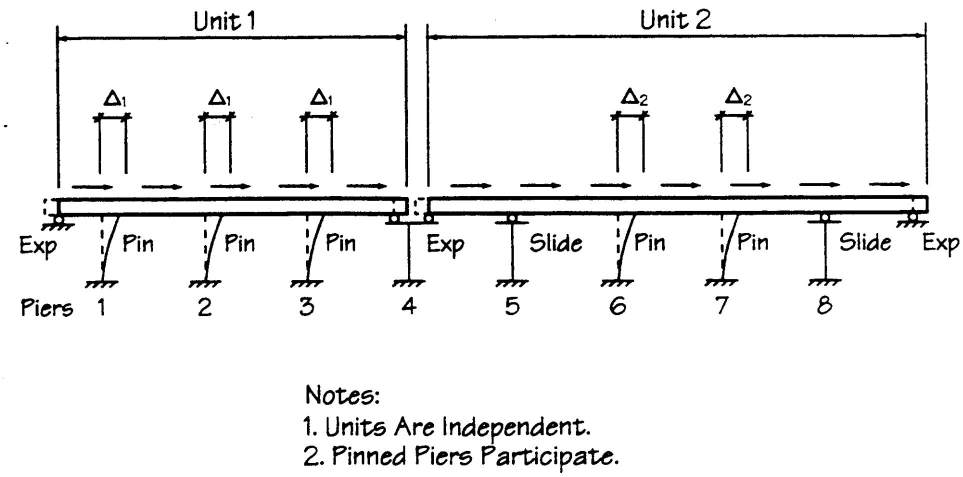


Figure 2 — Longitudinal Seismic Behavior

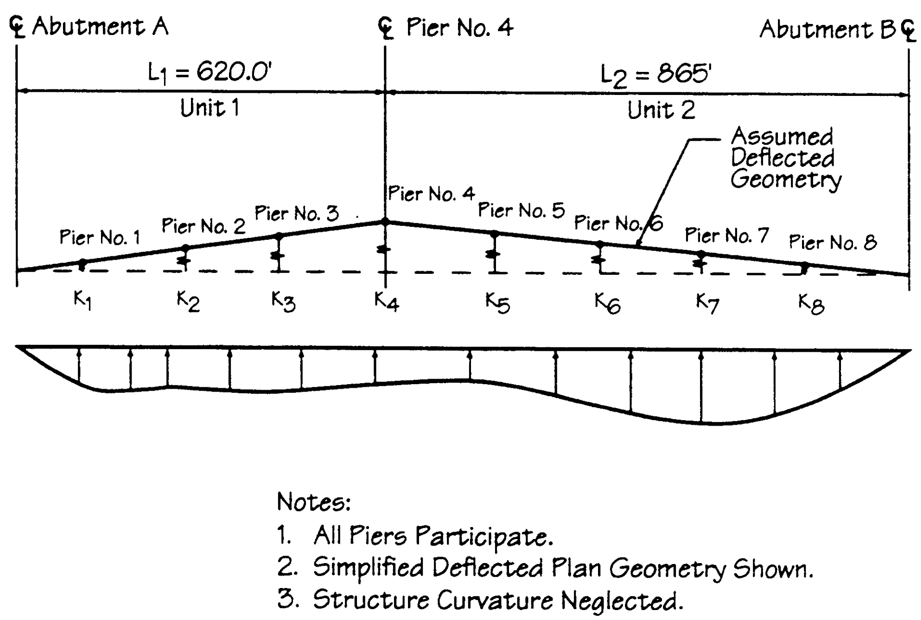


Figure 3 — Transverse Seismic Behavior

DESIGN STEP 1
(continued)

Conventional types of pinned bearings are assumed at the pinned piers to transfer both longitudinal and transverse seismic forces to the substructure through anchor bolts. At the sliding piers (Pier Nos. 5 and 8) and the expansion locations (Pier No. 4 and abutments) several types of bearings could be used to accommodate the expected displacements. Elastomeric bearings with provision for sliding between the bearing and the girder under large displacements would work, such as polytetraflouralethylene (PTFE) against a sliding surface (stainless steel). The transverse restraint would be provided by girder stops to transfer transverse seismic forces to Pier Nos. 4, 5, and 8 and the abutments.

Design Step 1.1

Determine Structure Properties

Properties of the structure are calculated in order to perform preliminary seismic design and provide input to the computer model for final analysis. The properties that are not computed are assumed to be taken from a previously performed preliminary design for static loads.

SUPERSTRUCTURE

Properties of the superstructure and its elements are shown below. The superstructure area and moments of inertia include the concrete deck, the girder webs, and both flanges with steel transformed to concrete using a modular ratio, n = 8.

$L := 1488 \cdot ft$ — Overall length of bridge

$L_1 := 620 \cdot ft$ — Length of Unit 1

$L_2 := 865 \cdot ft$ — Length of Unit 2

$A_d := 60 \cdot ft^2$ — Cross-sectional area of superstructure and deck (steel transformed to concrete with modular ratio, $n = E_s/E_c = 8$)

$I_{zd} := 518 \cdot ft^4$ — Moment of inertia of superstructure about a horizontal axis (steel transformed to concrete with modular ratio, n=8)

$I_{yd} := 9003 \cdot ft^4$ — Moment of inertia of superstructure about a vertical axis (steel transformed to concrete with modular ratio, n=8)

$f_c := 4000 \cdot psi$ — Compressive strength of concrete

$\gamma_{conc} := .150 \cdot \frac{kip}{ft^3}$ — Unit weight of concrete

Design Step 1.1 (continued)

$t_{deck} := 9 \cdot in$ — Thickness of concrete deck

$b_{deck} := 42 \cdot ft$ — Width of concrete deck

The torsional constant of the superstructure is calculated using only the deck. The contribution to torsional resistance offered by warping of the steel sections has been neglected. The torsional constant J is calculated as

$$J := \frac{b_{deck} \cdot t_{deck}^{3}}{3}$$

$J = 5.906 \cdot ft^4$ — Torsional constant of superstructure

$E_c := 3600 \cdot ksi$ — Young's Modulus of concrete (based on Division I, Article 8.7.1)

Weights for the superstructures are calculated with the following.

$w_{slab} := 5.0 \cdot \frac{kip}{ft}$ — Weight of concrete deck and girder pads

$w_{steel} := 1.9 \cdot \frac{kip}{ft}$ — Weight of steel plate girders and cross frames

$w_{miscl} := 2.4 \cdot \frac{kip}{ft}$ — Weight of barriers, stay-in-place metal forms, and future overlay

$$w_{super} := w_{slab} + w_{steel} + w_{miscl}$$

$w_{super} = 9.3 \cdot \frac{kip}{ft}$ — Weight per foot of superstructure

Design Step 1.1 (continued)

For each of the units, compute the total superstructure weight.

$W_{1super} := L_1 \cdot w_{super}$

$W_{1super} = 5766 \cdot kip$ Superstructure weight for Unit 1

$W_{2super} := L_2 \cdot w_{super}$

$W_{2super} = 8045 \cdot kip$ Superstructure weight for Unit 2

SUBSTRUCTURE

The 6-foot 3-inch by 20-foot pier columns have moments of inertia and cross-sectional areas at the column base as given below. The columns have a varying width at the top as shown in Figure 1c.

The columns are supported by steel H-piles and concrete pile caps that have been preliminary sized at 28 feet square by 6-foot 6-inch thick.

$d_{long} := 6.25 \cdot ft$ Column base dimension in the longitudinal direction

$d_{trans} := 20.0 \cdot ft$ Column base dimension in the transverse direction

$A := \left(d_{long}\right) \cdot \left(d_{trans}\right)$

$A = 125 \cdot ft^2$ Cross-sectional area of column base

The total weight for calculation of the period in the longitudinal direction will include the weight for the top one-half of the pinned columns that participate for each unit because a lumped mass analysis is used for the preliminary longitudinal seismic analysis. From the dimensions shown in Figure 1c, and γ conc = 0.150 kip/ft^3

For the 50-foot piers (1 and 7)

$W_{p50} := 690 \cdot kip$ per pier

Design Step 1.1 (continued)

For the 70-foot piers (2, 3, and 6)

$W_{p70} := 880 \cdot kip$ per pier

Moments of inertia for the column base to be used in computing the intermediate pier stiffnesses.

$$I_{long} := \left(d_{trans}\right) \cdot \frac{\left(d_{long}\right)^3}{12}$$

$I_{long} = 407 \cdot ft^4$

Pier column base moment of inertia in the longitudinal direction

$$I_{trans} := \left(d_{long}\right) \cdot \frac{\left(d_{trans}\right)^3}{12}$$

$I_{trans} = 4167 \cdot ft^4$

Pier column base moment of inertia in the transverse direction

Design Step 1.2

Preliminary Seismic Design

Simplified approaches are used for quick hand analyses for preliminary design in both longitudinal and transverse directions. Fundamental periods of the structure are obtained, and the associated forces are computed for preliminary design and sizing of the substructure elements. For preliminary seismic design, the effects of the structure's curvature are ignored.

Assume a SOIL PROFILE TYPE I. The Site Coefficient (S) is from Division I-A, Article 3.5.1, Table 2 (see Design Step 2.5).

$S := 1.0$

The Acceleration Coefficient (A) is provided in the introduction to Design Step 1 (see Design Step 2.2 for further discussion).

$A := 0.15$

Calculate intermediate pier stiffnesses for each direction. For preliminary design, ignore the stiffness of the foundation and assume that the piers are fixed at the bottom of the pile cap. The top of the pier is free to translate and rotate. From P/Δ for a cantilever beam, use $k = 3*EI / H^3$.

For Pier Nos. 1, 7, and 8, the pier height from the top to the bottom of the footing is 50 feet + 6 feet 6 inches = 56 feet 6 inches.

$H_{50} := 56.5 \cdot ft$

In the longitudinal direction

$$K_{50long} := \frac{3 \cdot E_c \cdot I_{long}}{H_{50}^3}$$

$K_{50long} = 3509 \cdot \frac{kip}{ft}$ Pier Nos. 1, 7, and 8

Design Step 1.2 (continued)

In the transverse direction

$$K_{50trans} := \frac{3 \cdot E_c \cdot I_{trans}}{H_{50}^{3}}$$

$$K_{50trans} = 35928 \cdot \frac{kip}{ft}$$ Pier Nos. 1, 7, and 8

For Pier Nos. 2, 3, 4, 5, and 6, the pier height from the top to the bottom of the footing is 70 feet + 6 feet 6 inches = 76 feet 6 inches.

$$H_{70} := 76.5 \cdot ft$$

In the longitudinal direction

$$K_{70long} := \frac{3 \cdot E_c \cdot I_{long}}{H_{70}^{3}}$$

$$K_{70long} = 1413 \cdot \frac{kip}{ft}$$ Pier Nos. 2 through 6

In the transverse direction

$$K_{70trans} := \frac{3 \cdot E_c \cdot I_{trans}}{H_{70}^{3}}$$

$$K_{70trans} := \frac{3 \cdot E_c \cdot I_{trans}}{H_{70}^{3}}$$

$$K_{70trans} = 14474 \cdot \frac{kip}{ft}$$ Pier Nos. 2 through 6

Design Step 1.3

Analysis and Results for Longitudinal Direction

LONGITUDINAL DIRECTION

Assume that the units are independent as discussed previously. See Figure 2. The superstructure is assumed to act rigidly and all pinned columns for the unit have the same longitudinal displacement, Δ. Compute the total weight and stiffness for each unit for use in calculation of the longitudinal period.

For the longitudinal direction, a simple Single Degree of Freedom (SDOF) system analysis using a single mass (W) and spring (K = stiffness of columns) is used here. This is considered to be a reasonable approach for longitudinal response of straight, continuous bridges by FHWA (1987), ***Seismic Design and Retrofit Manual.***

For Unit 1, include half of the column top weight for the participating pinned piers (Pier Nos. 1, 2, and 3) to compute the total weight.

$$W_{1tot} := W_{1super} + W_{p50} + 2 \cdot W_{p70}$$

$$W_{1tot} = 8216 \cdot kip$$

Compute the total longitudinal stiffness for Unit 1 with the springs for Pier Nos. 1, 2, and 3 acting in parallel.

$$K_{1long} := K_{50long} + 2 \cdot K_{70long}$$

$$K_{1long} = 6336 \cdot \frac{kip}{ft}$$ Longitudinal stiffness for Unit 1

Compute the period from Division I-A, Equation (4-3).

$$T_{1long} := 2 \cdot \pi \cdot \sqrt{\frac{W_{1tot}}{K_{1long} \cdot g}} \qquad T_{1long} = 1.26 \cdot sec$$

Design Step 1.3 (continued)

For Unit 2, include half of the column top weight for the participating pinned piers (Piers 6 and 7) to compute the total weight.

$$W_{2tot} := W_{2super} + W_{p50} + W_{p70}$$

$$W_{2tot} = 9615 \cdot kip$$

Compute the total longitudinal stiffness for Unit 2 with the springs for Pier Nos. 6 and 7 acting in parallel.

$$K_{2long} := K_{50long} + K_{70long}$$

$$K_{2long} = 4922 \cdot \frac{kip}{ft}$$ Longitudinal stiffness for Unit 2

Compute the period from Division I-A, Equation (4-3).

$$T_{2long} := 2 \cdot \pi \cdot \sqrt{\frac{W_{2tot}}{K_{2long} \cdot g}} \qquad T_{2long} = 1.55 \cdot sec$$

These preliminary periods can now be used to compute seismic shears for the longitudinal direction and the columns and foundations could be checked quickly to see if dimensions are adequate and reinforcing within code limits. Later in Design Step 6.3.1, the preliminary design periods are compared with those obtained from the computer analysis as a check.

For example, the longitudinal column shear for Pier No. 7 is computed as shown in the following. Note that there are conversion units in the numerator of Equation (3-1) to yield the dimensionless constant, C_s.

Calculate the elastic seismic response coefficient for Unit 2 in the longitudinal direction to determine the longitudinal seismic shear at Pier No. 7.

$$C_s := \frac{1.2 \cdot A \cdot S}{T_{2long}^{\frac{2}{3}}} \cdot sec^{\frac{2}{3}}$$ Division I-A, Equation (3-1)

Design Step 1.3 (continued)

$C_s = 0.135$ Elastic response coefficent, $< 2.5*A$ ($= 0.375$), okay

The value of 2.5*A represents the upper limit of C_S per Division I-A, Article 3.6.1.

The total longitudinal shear force at the top of the columns resisted by Unit 2 displaced as a rigid body of lumped mass is computed as

$$F_{2long} := C_s \cdot W_{2tot}$$

$$F_{2long} = 1293 \cdot kip$$

Distributing this force to the pinned piers (6 and 7) in relation to their respective longitudinal stiffnesses, the shear force at Pier No. 7 is

$$F_{7long} := \left(\frac{K_{50long}}{K_{2long}}\right) \cdot F_{2long}$$

$F_{7long} = 922 \cdot kip$ Longitudinal seismic shear at Pier No. 7 from preliminary design

Preliminary longitudinal seismic shear forces for other piers may be computed similarly and are shown in Figure 4. These preliminary forces were used to check the substructure members for size and reinforcing. The results from the Multimode Spectural Analysis are shown in Table 6 for comparison.

The longitudinal displacement associated with the previously computed preliminary seismic force may be calculated for Unit 2.

$$\Delta_2 := \frac{F_{2long}}{K_{2long}} \qquad \Delta_2 = 3.153 \cdot in$$

The longitudinal displacement for Unit 1 may be computed similarly. This information is useful at a preliminary design stage because it provides the designer with an estimate of the structure's expected seismic motion.

Design Step 1.3 (continued)

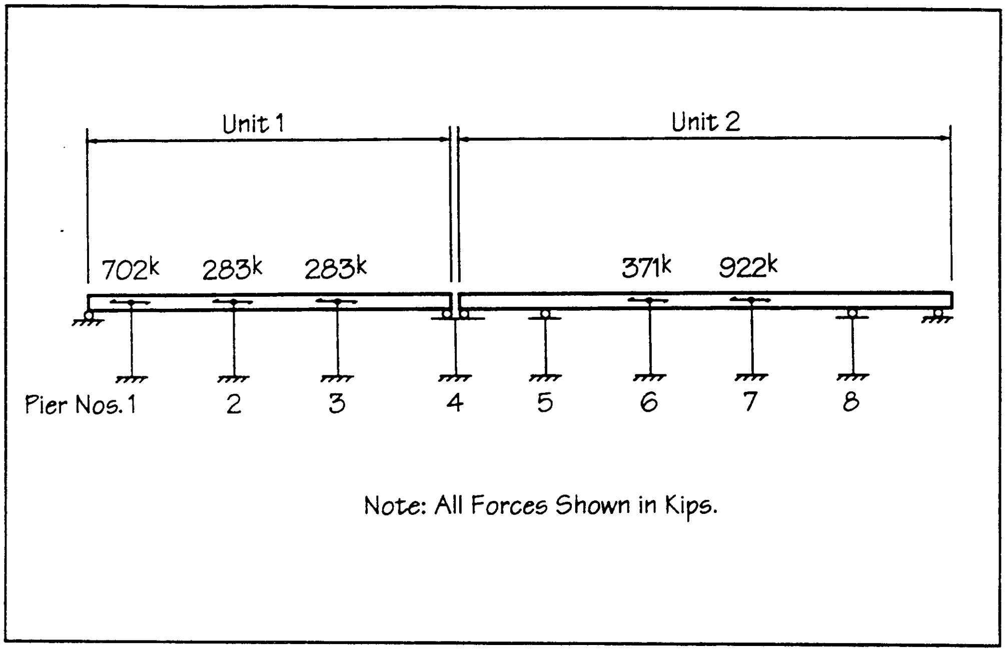


Figure 4 - Longitudinal Seismic Shears

Design Step 1.4

Analysis and Results for Transverse Direction

TRANSVERSE DIRECTION

Assume that the units act as a two-rigid link system pivoting at the abutments with a maximum transverse displacement at Pier No. 4 as shown in Figure 5. A Generalized Coordinate Method is used based upon the simplified geometry shown in the figure. All piers and the abutments resist the seismic force in the transverse direction.

Application of the Generalized Coordinate Method for transverse displacement of a bridge structure is presented in FHWA (1981), *Seismic Design of Highway Bridges - Workshop Manual.* A more complete discussion of the Generalized Coordinate Method may be found in a structural dynamics text, such as Clough and Penzien (1993).

The reliability of this method depends on the ability to predict and define the structure's mode shape. The effective application of this technique also requires that one mode dominate in the direction under consideration.

The generalized coordinate is at the hinge between Units 1 and 2, which occurs at Pier No. 4. At this location, $\psi_i = \Delta_{max} = 1.0$.

The generalized stiffness is given by

$$K_{gen} = \text{Sum}\ (\ K_i\ \psi_i^{\ 2})$$

and the generalized mass (or weight) is given by

$$W_{gen} = \text{Sum}\ (\ W_i\ \ \psi_i^{\ 2}\ )$$

Where ψ_i is a shape factor at each pier relative to the maximum or general coordinate and is a function of x / L as shown in Figure 5. The assumed maximum or unit displacement ($\psi_4 = \Delta_{max} = 1.0$) occurs at the hinge at Pier No. 4.

Design Step 1.4 (continued)

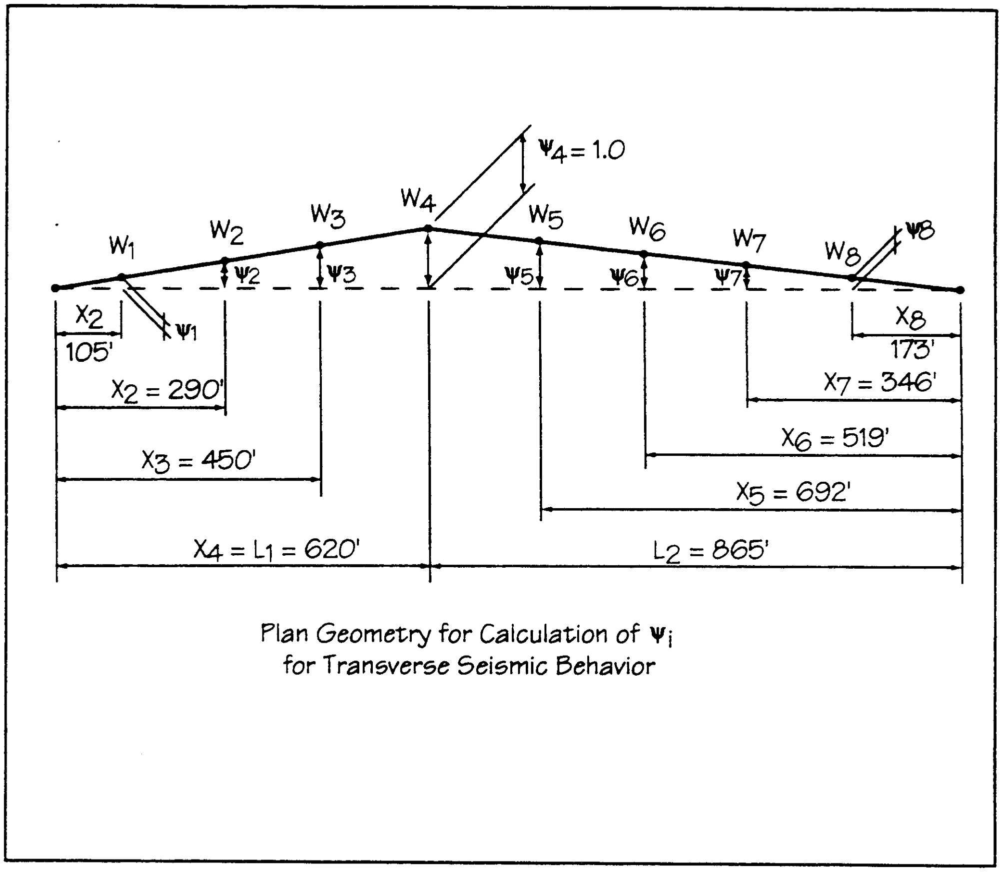


Figure 5 — Plan Geometry for Calculation of ψ_i

From Figure 5

$L_1 := 620 \cdot ft$ Length of Unit 1

$x_1 := 105 \cdot ft$

$x_2 := 290 \cdot ft$

$x_3 := 450 \cdot ft$

$x_4 := 620 \cdot ft$

$L_2 := 865 \cdot ft$ Length of Unit 2

$x_5 := 692 \cdot ft$

Design Step 1.4 (continued)

$x_6 := 519 \cdot ft$

$x_7 := 346 \cdot ft$

$x_8 := 173 \cdot ft$

Compute the ψ_i terms for each intermediate pier location.

$\psi_1 := \frac{x_1}{L_1}$ $\psi_1 = 0.169$ at Pier No. 1

$\psi_2 := \frac{x_2}{L_1}$ $\psi_2 = 0.468$ at Pier No. 2

$\psi_3 := \frac{x_3}{L_1}$ $\psi_3 = 0.726$ at Pier No. 3

$\psi_4 := \frac{L_1}{L_1}$ $\psi_4 = 1$ at Pier No. 4 (maximum)

$\psi_5 := \frac{x_5}{L_2}$ $\psi_5 = 0.8$ at Pier No. 5

$\psi_6 := \frac{x_6}{L_2}$ $\psi_6 = 0.6$ at Pier No. 6

$\psi_7 := \frac{x_7}{L_2}$ $\psi_7 = 0.4$ at Pier No. 7

$\psi_8 := \frac{x_8}{L_2}$ $\psi_8 = 0.2$ at Pier No. 8

Design Step 1.4 (continued)

The pier stiffnesses for the transverse direction have already been computed. The K_i terms for the generalized expressions are

$K_1 := K_{50trans}$ $\quad K_1 = 3.593 \cdot 10^4 \cdot \frac{kip}{ft}$

$K_2 := K_{70trans}$ $\quad K_2 = 1.447 \cdot 10^4 \cdot \frac{kip}{ft}$

$K_3 := K_{70trans}$ $\quad K_3 = 1.447 \cdot 10^4 \cdot \frac{kip}{ft}$

$K_4 := K_{70trans}$ $\quad K_4 = 1.447 \cdot 10^4 \cdot \frac{kip}{ft}$

$K_5 := K_{70trans}$ $\quad K_5 = 1.447 \cdot 10^4 \cdot \frac{kip}{ft}$

$K_6 := K_{70trans}$ $\quad K_6 = 1.447 \cdot 10^4 \cdot \frac{kip}{ft}$

$K_7 := K_{50trans}$ $\quad K_7 = 3.593 \cdot 10^4 \cdot \frac{kip}{ft}$

$K_8 := K_{50trans}$ $\quad K_8 = 3.593 \cdot 10^4 \cdot \frac{kip}{ft}$

The generalized stiffness for the system is

$$K_{gen} := \sum_{i=1}^{8} K_i \cdot (\psi_i)^2$$

$K_{gen} = 4.796 \cdot 10^4 \cdot \frac{kip}{ft}$ *Generalized stiffness*

Design Step 1.4 (continued)

Note that this stiffness does not include any contribution from the superstructure because there is a discontinuity of the superstructure's stiffness at Pier No. 4 due to the expansion joint between the two units.

Compute the generalized mass (or weight) term from the tributary superstructure weights at each pier plus the column top weight computed previously.

$span_1 := 105 \cdot ft$ $\quad span_4 := 170 \cdot ft$ $\quad span_7 := 173 \cdot ft$

$span_2 := 185 \cdot ft$ $\quad span_5 := 173 \cdot ft$ $\quad span_8 := 173 \cdot ft$

$span_3 := 160 \cdot ft$ $\quad span_6 := 173 \cdot ft$ $\quad span_9 := 173 \cdot ft$

$$W_1 := w_{super} \cdot \frac{span_1 + span_2}{2} + W_{p50} \qquad W_1 = 2039 \cdot kip$$

$$W_2 := w_{super} \cdot \frac{span_2 + span_3}{2} + W_{p70} \qquad W_2 = 2484 \cdot kip$$

$$W_3 := w_{super} \cdot \frac{span_3 + span_4}{2} + W_{p70} \qquad W_3 = 2415 \cdot kip$$

$$W_4 := w_{super} \cdot \frac{span_4 + span_5}{2} + W_{p70} \qquad W_4 = 2475 \cdot kip$$

$$W_5 := w_{super} \cdot \frac{span_5 + span_6}{2} + W_{p70} \qquad W_5 = 2489 \cdot kip$$

$$W_6 := w_{super} \cdot \frac{span_6 + span_7}{2} + W_{p70} \qquad W_6 = 2489 \cdot kip$$

$$W_7 := w_{super} \cdot \frac{span_7 + span_8}{2} + W_{p50} \qquad W_7 = 2299 \cdot kip$$

Design Step 1.4 (continued)

$$W_8 := w_{super} \cdot \frac{span_8 + span_9}{2} + W_{p50} \qquad W_8 = 2299 \cdot kip$$

The generalized mass (or weight) for the system is

$$W_{gen} := \sum_{i=1}^{8} W_i \cdot (\psi_i)^2$$

$W_{gen} = 7298 \cdot kip$ Generalized mass (or weight)

Compute the transverse period.

$$T_{trans} := 2 \cdot \pi \cdot \sqrt{\frac{W_{gen}}{K_{gen} \cdot g}} \qquad T_{trans} = 0.43 \cdot sec$$

Preliminary design forces for transverse seismic were computed from the assumed shape of the structure deflection. The maximum transverse deflection (Δ_{max}) is computed at Pier No. 4 and used to compute the transverse forces using the transverse pier stiffnesses and the deflections at each pier. An example calculation for the transverse shear at Pier No. 7 is shown here.

Calculate the elastic seismic response coefficient for the structure in the transverse direction to determine the transverse seismic shears acting at the top of the intermediate piers.

$$C_s := \frac{1.2 \cdot A \cdot S}{T_{trans}^{\frac{2}{3}}} \cdot sec^{\frac{2}{3}}$$

Division I-A
Eqn (3-1)

$C_s = 0.315$

Elastic response coefficient
< 2.5*A (= 0.375) O.K.

Design Step 1.4 (continued)

At Pier No. 4, compute the maximum transverse deflection.

$$\Delta_{max} := \frac{C_s \cdot W_{gen}}{K_{gen}} \qquad \Delta_{max} = 0.0479 \cdot ft$$

Transverse deflections at the other pier locations may be computed as $\Delta = \psi_i \Delta_{max}$. At Pier No. 7

$$\Delta_7 := \psi_7 \cdot \Delta_{max} \qquad \Delta_7 = 0.019 \cdot ft$$

The transverse force associated with this deflection and transverse stiffness for Pier No. 7 may be computed.

$$F_{7trans} := \Delta_7 \cdot K_7$$

$F_{7trans} = 689 \cdot kip$ — Transverse seismic shear at Pier No. 7 from preliminary design

Preliminary transverse seismic shear forces for other piers may be computed similarly and are shown in Figure 6. These preliminary forces were used to check the substructure members for size and reinforcing. The results from the Multimode Spectral Method are shown in Table 4 for comparison.

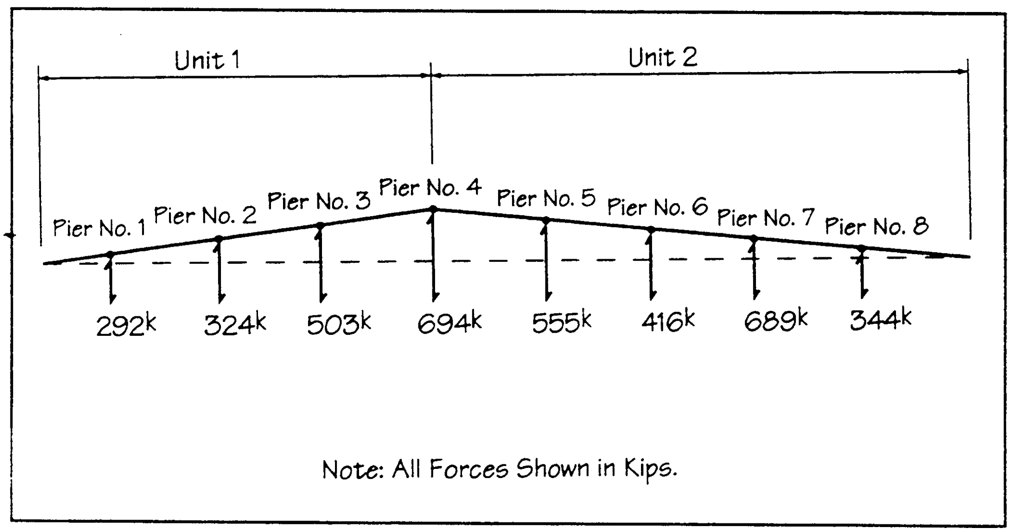


Figure 6 — Transverse Seismic Shears

DESIGN STEP 2 — BASIC REQUIREMENTS

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

The configuration of the bridge is nine spans totaling 1488 feet and consisting of two units: four spans (Unit 1) and five spans (Unit 2). The bridge features a steel plate girder superstructure with cast-in-place concrete deck and reinforced concrete substructure. Thus, the Specification applies.

Design Step 2.2 — Acceleration Coefficient
[Division I-A, Article 3.2]

For this example, the Acceleration Coefficient (A) is 0.15 (normally taken from Figure 3 of the Specification).

A = 0.15

A site investigation by a qualified geotechnical engineer or seismic hazard assessment specialist may be used to develop accurate acceleration data. Such an investigation is required if the structure is near an active fault, if long-duration earthquakes are expected, or if design for a long return period is required due to great importance of the structure. In addition, some agencies may require acceleration coefficients that are different than those given in the AASHTO Specification.

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

The SPC is a function of the Acceleration Coefficient and the Importance Classification.

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 is used since it corresponds to stable deposits of sands and gravels less than 200 feet deep overlying rock.

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

$S = 1.0$

A geotechnical investigation may be made by qualified professionals to establish site-specific seismic response information (e.g., site-specific response spectra). This is typically done on a site-by-site basis. In some cases, State Departments of Transportation (DOTs) have developed representative spectra for soil types and seismic hazards in their jurisdiction. These are then used in lieu of the information in Article 3.5. Lacking such specific information, the structural engineer should decide whether to have site-specific information generated or use the approach given in this section.

Design Step 2.6

Response Modification Factors
[Division I-A, Article 3.7]

Since 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.

$R = 3.0$ For the substructure since single-column piers are used

$R = 1.0$ For the superstructure to intermediate pier connection (bearings)

These factors will be used to ensure that inelastic effects are restricted to elements that can be designed to provide reliable, ductile response that can be inspected after an earthquake to assess damage and that can be repaired relatively easily. The foundations do not fit this constraint and thus will be designed not to experience inelastic effects.

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 curved in the horizontal plane. The 856-foot-long Unit 2 has a curve radius of 1300 feet.

Calculate the subtended angle.

$R := 1300 \cdot ft$ — Radius of curvature

$S := 865 \cdot ft$ — Length of arc

$$\Delta := \frac{S}{R}$$

$\Delta = 38 \cdot deg$ — Subtended angle in plan

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

Compute the maximum span length ratio from span-to-span, i.e., for adjacent spans.

$L_{max} := 185 \cdot ft$ — Maximum span length (Span 2)

$L_{min} := 105 \cdot ft$ — Minimum span length (Span 1)

$$Span_{ratio} := \frac{L_{max}}{L_{min}}$$

$Span_{ratio} = 1.76$ — Maximum span length ratio, greater than 1.5

Design Step 5.3

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

Using the transverse pier stiffnesses computed in Design Step 1, compute the maximum bent/pier stiffness ratio from span-to-span, i.e., for adjacent piers, excluding abutments.

$K_{1trans} := 35928 \cdot \frac{kip}{ft}$ Maximum pier stiffness

$K_{2trans} := 14474 \cdot \frac{kip}{ft}$ Minimum pier stiffness

$$Stiffness_{ratio} := \frac{K_{1trans}}{K_{2trans}}$$

$Stiffness_{ratio} = 2.48$ Maximum bent/pier stiffness ratio, greater than 2

Design Step 5.4

Critical Bridge
[Division I-A, Article 4.2.3]

Assume that the bridge is not critical.

If the bridge is large, expensive, required to function immediately following the design earthquake, or geometrically complex, then the Specification recommends that Time-History Method (Procedure 4) be used to analyze the structure.

Design Step 5.5

Regular Bridge
[Division I-A, Article 4.2]

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

The bridge is not regular because the span length ratio and bent/pier stiffness ratios are exceeded, and there are more than six spans.

Design Step 5.6

Curved Bridge
[Division I-A, Article 4.2.2]

A curved bridge may be analyzed as if it were straight provided all of the requirements of Article 4.2.2 are satisfied.

The bridge has a subtended angle in plan and is greater than 30°; therefore, the bridge must be analyzed using the actual curved geometry.

Design Step 5.7

Analysis Procedure
[Division I-A, Article 4.2]

Because this bridge is not a single-span bridge and it is not a SPC A bridge, a detailed seismic analysis is required. Table 4 of the Specification is used to select the minimum analysis requirements.

From Table 4 of the Specification, the Multimode Spectral Method (Procedure 3) must be used because this bridge is not regular and has more than six spans.

This is the minimum method that can be used. The Time-History Method (Procedure 4) could be used in lieu of Procedure 3.

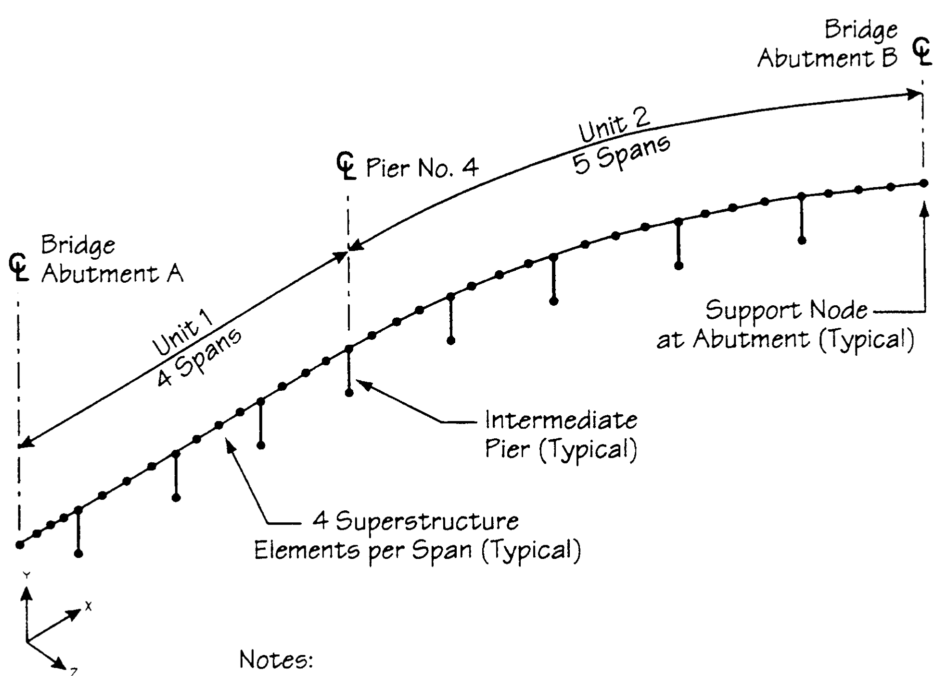
For this example, Procedure 3 is used for the analysis.

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 BETA 6.00 (CSI, 1994) was used for the linear elastic analyses. The model used is shown in Figure 7 and includes a single line of elements for the superstructure and a single line of vertical elements for each of the intermediate piers. A copy of the SAP90 input file for the analyses is provided in Appendix B.



Notes:
1. For Span Lengths, See Figure 1a.
2. For Details of Pier Column Elements, See Figure 8.
3. For Spring Details at Piers, See Figure 11.
4. For Details of Abutment Supports, See Figure 16.

Figure 7 — Structural Model of Bridge

Design Step 6.1.2

Superstructure

a) Geometry

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

As shown in Figure 7, the superstructure has been collapsed into a single line of 3-D frame elements that follow the horizontal geometry of the bridge centerline. As discussed in Design Step 5.6, the bridge must be analyzed using the actual curved geometry. This "stick" model is used solely for the determination of seismic forces for this example. For some structures, such as multicolumn bents, this type of model may not give the correct forces for other loadings, such as dead loads. However, substructure dead load forces obtained from a "stick" model for the type of bridge in this example would generally be acceptable, though dead load distribution to the superstructure elements would require more exact analysis by girder line or grid. Springs are used to support the structure. The determination of the foundation spring stiffnesses is discussed in Design Step 6.2.

Enough nodes must be used along the length of the superstructure to accurately characterize the response and forces. The mass of the structure will be lumped by SAP90 at the nodes, which is typical of most dynamic analyses programs. For a uniform cross section such as this one and relatively large radius curvature (greater than 800 feet), nodes at the quarter points are sufficient. Moments of inertia and torsional stiffness of the superstructure are based on uncracked cross-sectional properties.

b) Properties

The properties of the elements were presented previously in Design Step 1, Preliminary Design. Since the superstructure is a composite of steel and concrete, these properties are transformed to equivalent concrete properties. The density used for the modal analysis has been adjusted to include additional dead loads from traffic barriers, wearing surface overlay, and stay-in-place metal forms. The total weight of these additional dead loads is 2.4 kips per lineal foot of superstructure.

Design Step 6.1.2 (continued)

The centroid of the superstructure has been located 8 feet above the top of the pier to account for the height of the bearings and leveling pedestal. The connection of the superstructure to the pier is made in the SAP90 model with rigid link elements shown in Figure 8 as the top elements of the piers.

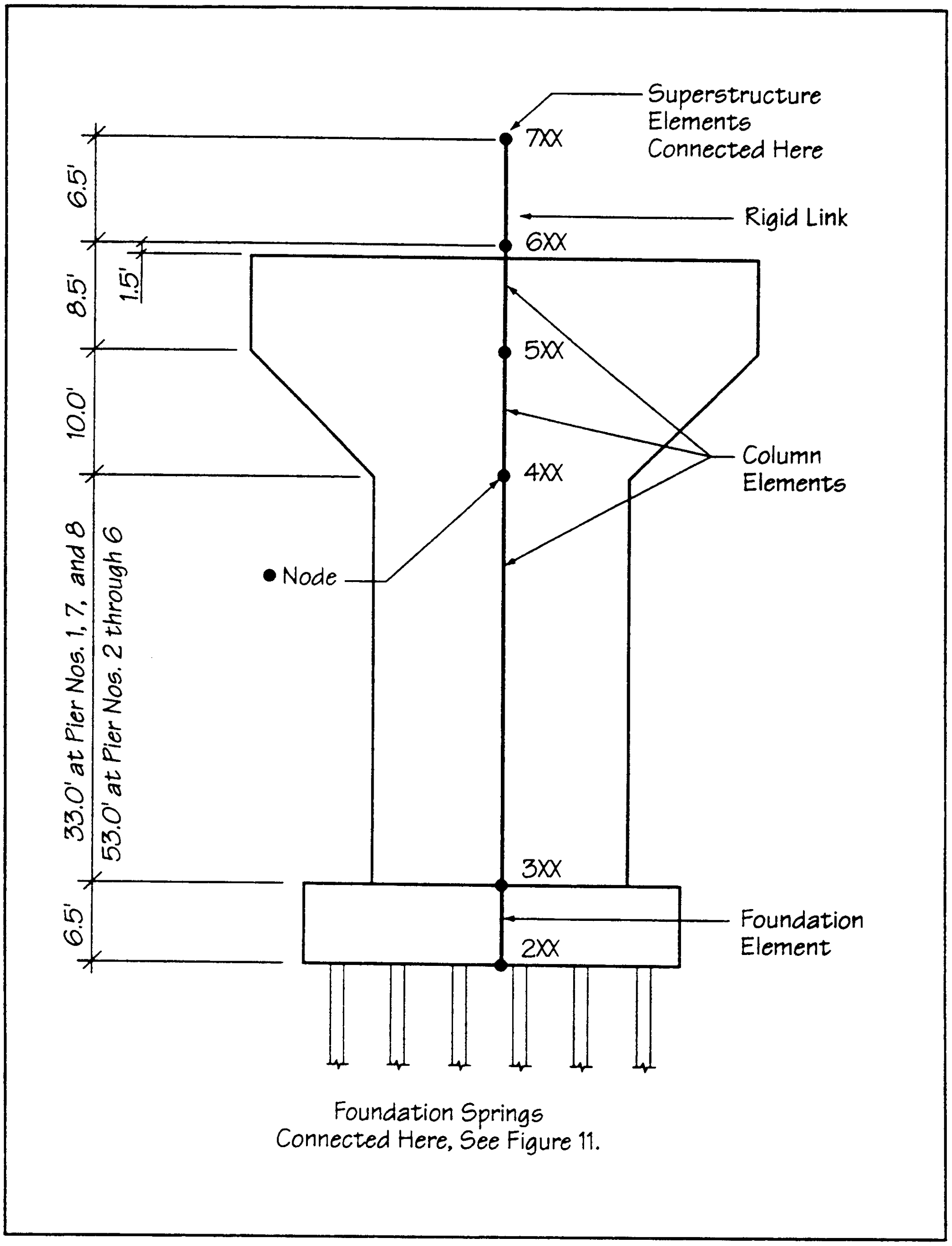


Figure 8 — Details of Pier Column Elements

Design Step 6.1.3

Substructure

The intermediate piers are modeled with 3-D frame elements that represent the individual columns. Figure 8 shows the relationship between the actual pier and the "stick" model of 3-D frame elements. Three elements were used for the column between the top of footing and the bearings. This was to account for the varying cross section near the top of the column since SAP90 handles members with varying cross sections by interpolating between the member end nodes. For this model, the moments of inertia and torsional properties of the columns are based on an uncracked section. Foundation springs are connected to the node (2xx) at the base of the pile cap. There are no elements to model the abutments, only support nodes as shown in Figure 7.

Design Step 6.1.4

Connection of Superstructure to Piers

In the actual structure, internal forces are transferred between the superstructure and the pier through the bearings. In the seismic model, the superstructure forces are transferred at the single point where the superstructure and pier intersect. At pinned piers, node 6xx (in Figure 8) transfers shears from the superstructure in all directions, and is released for moment in the longitudinal direction. At Pier Nos. 4, 5, and 8 which are free to move longitudinally, only transverse shears are transferred.

Figure 9 shows modeling details for the connection at the top of Pier No. 4, which is the location of the expansion joint between Unit 1 and Unit 2. If the ends of the adjacent superstructure elements are connected directly to Node 741 and these element ends are released for longitudinal translation and rotation, the node (741) is still attached to the top of the rigid link and will receive the tributary mass from each end of the attached superstructure. (Lumped mass for spectral analysis is discussed further in Design Step 6.3.) This will result in longitudinal shears being transmitted to Pier No. 4 though the superstructure is free to move longitudinally there and should transfer no shear.

To model the behavior at the expansion joint correctly, three coincident nodes are defined at the top of the rigid link. The two additional nodes (741A and 741B) are used to define connectivity, which will result in correct forces for Pier No. 4. The end of the superstructure element from Unit 1 is connected to one of the nodes (741A), the end of the superstructure from Unit 2 is connected to another of the nodes (741B), and the third node (741) is connected to the top of the rigid link of the pier column elements. Local coordinate systems and release constraints of

Design Step
6.1.4
(continued)

each of the three nodes are defined. This prevents the column top node (741) from picking up lumped mass from the adjacent superstructure elements in the longitudinal direction, for which the structure is free to move. Instead of coincident nodes, a short element could have been defined at the two ends of the superstructure elements adjacent to Node 741. The ends of these short elements adjacent to Node 741 would then be released for translation and rotation longitudinally in their respective local directions to model the superstructure ends at the expansion joint, and the lumped mass from the short elements transmitted to Pier No. 4 would be very small.

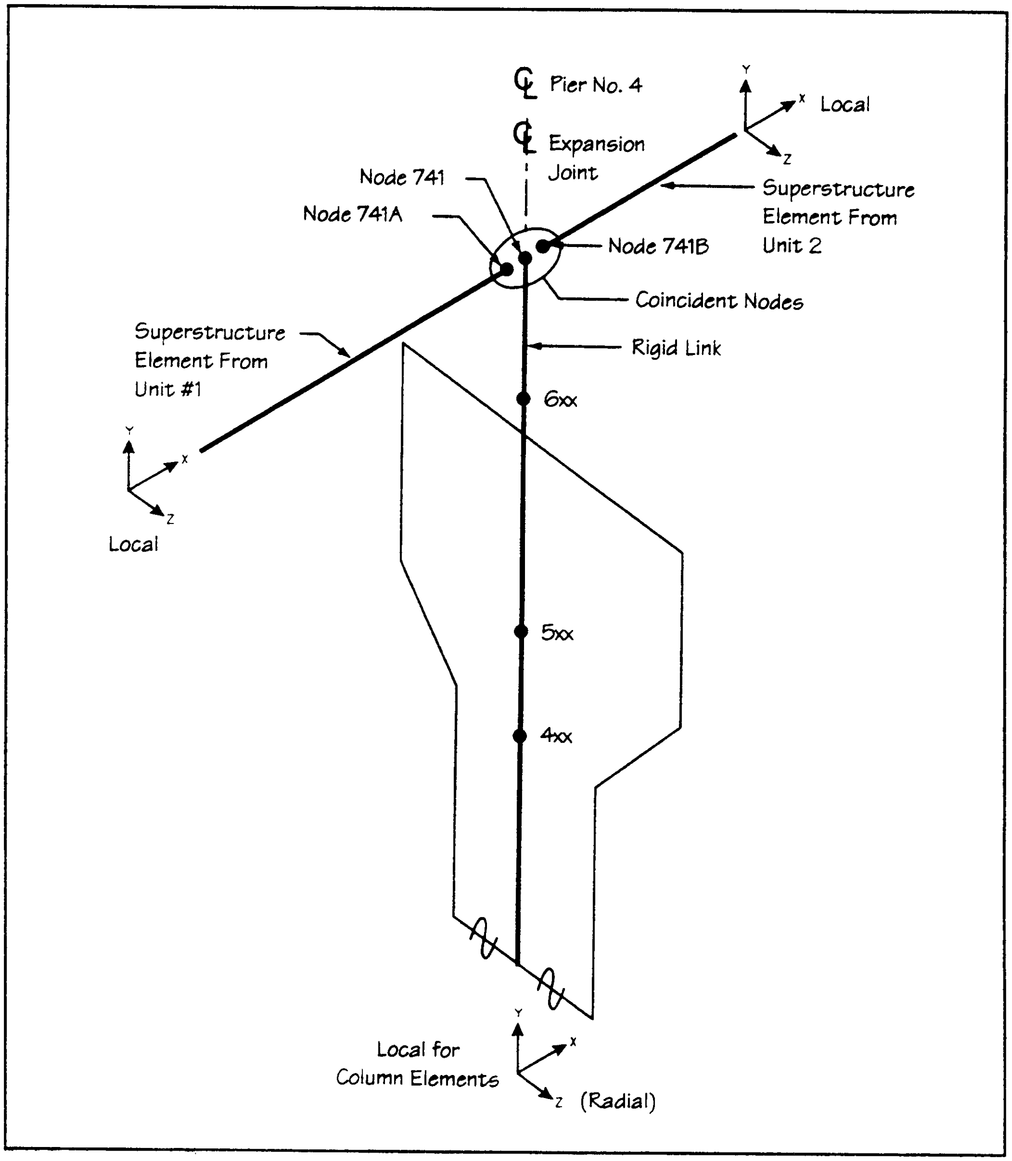


Figure 9 — Details at Pier No. 4 Expansion Joint

Design Step 6.1.4 (continued)

Modeling details for connections at the tops of Pier Nos. 5 and 8 are shown in Figure 10. These piers have sliding bearings to allow unrestrained longitudinal motion. Since the superstructure is continuous, it is not necessary to provide coincident nodes as with Pier No. 4 in order to provide correct modeling for longitudinal forces. Translational and rotational releases are provided at the top end of the rigid link element. The direction for the releases is in the local column coordinate system, and so is oriented tangential to the point of curvature at the center of the pier as shown in Figure 10.

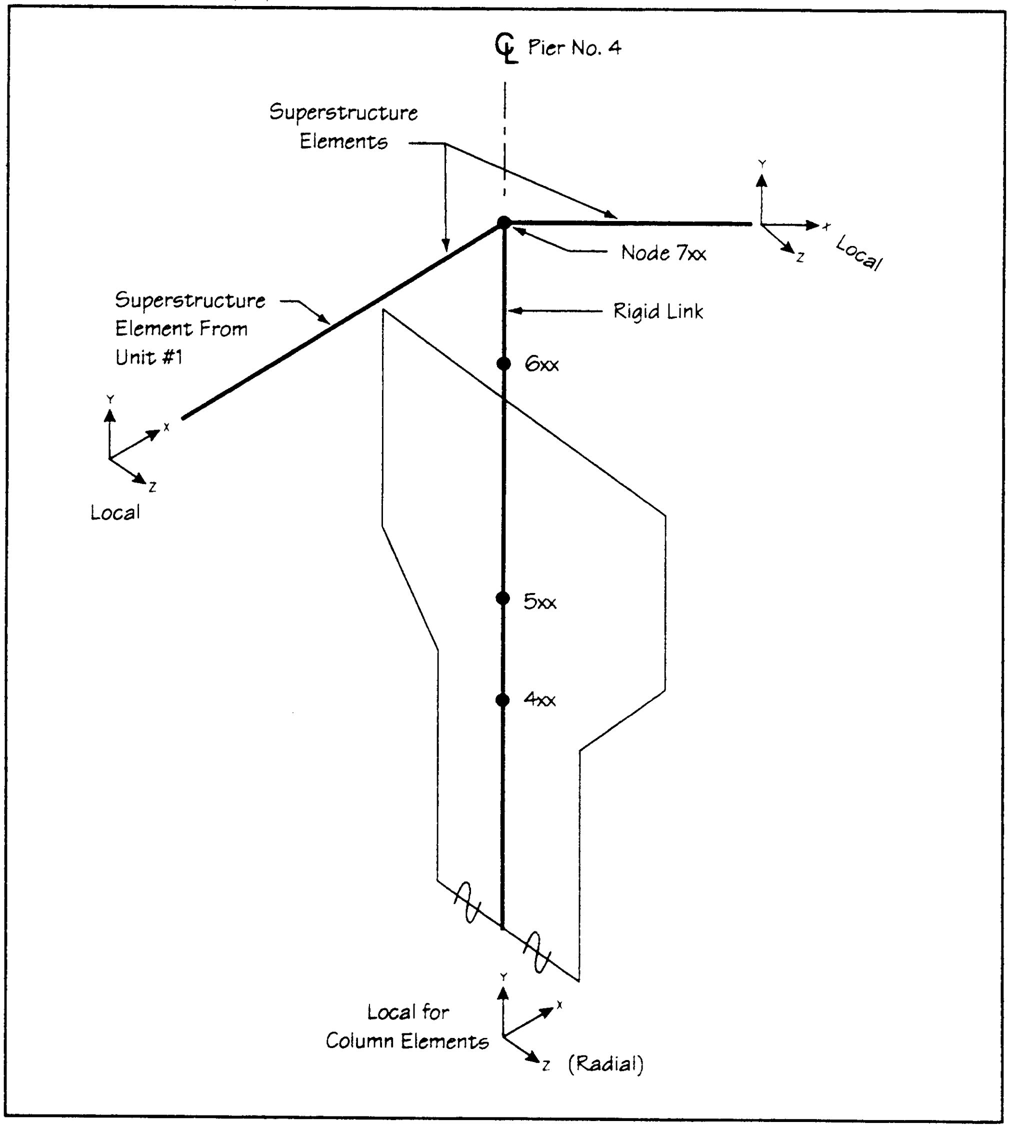


Figure 10 — Details at Pier Nos. 5 and 8 Sliding Bearings

Design Step 6.2

Foundation Stiffnesses

Design Step 6.2.1

Pier Foundations

The intermediate pier foundations were modeled with equivalent spring stiffnesses for the pile group. Figure 11 shows details of the spring supports. For this example, all of the intermediate piers use the same foundation springs. The spring stiffnesses are developed for the local pier support coordinate geometry and are input into the SAP90 model with the same orientation as the local pier columns. Note that the local axes for the spring support nodes are identified differently (i.e., x, y, and z are not the same) in Figure 11 than the local axes of the column elements.

For a program that can only accommodate global directions for spring releases, the stiffnesses computed here would require some transformation from local to global coordinate geometry for input into the model.

Establishing meaningful soil stiffnesses for bridge pile foundations is a complex problem that is often simplified to linear springs for static or modal analyses. There are several methods available for establishing spring constants for use in a seismic analysis. Generally, the steps are

- Obtain the stiffness of a single pile, axially and laterally.
- Combine the stiffnesses of individual piles to obtain the group pile stiffnesses.
- Determine if any additional stiffness contribution from the pile cap (footing) should be included or if the flexibility of the pile cap contributes to reducing the stiffness of the pile group.
- Combine any contribution of the pile cap with the pile group stiffnesses to obtain the final foundation springs.

Judgment is necessary to determine the stiffness contribution of the pile cap or footing. This largely depends on how confident the designer feels about the assumed soil stiffnesses, strengths, and interaction with the pile cap. If soils are weak, have liquefaction potential, or may not be in full contact with the pile cap due to scour or settlement, then the stiffness contribution from the pile cap interacting with the supporting soil should be neglected. Generally, soil contribution under a pile cap is not included

Design Step
6.2.1
(continued)

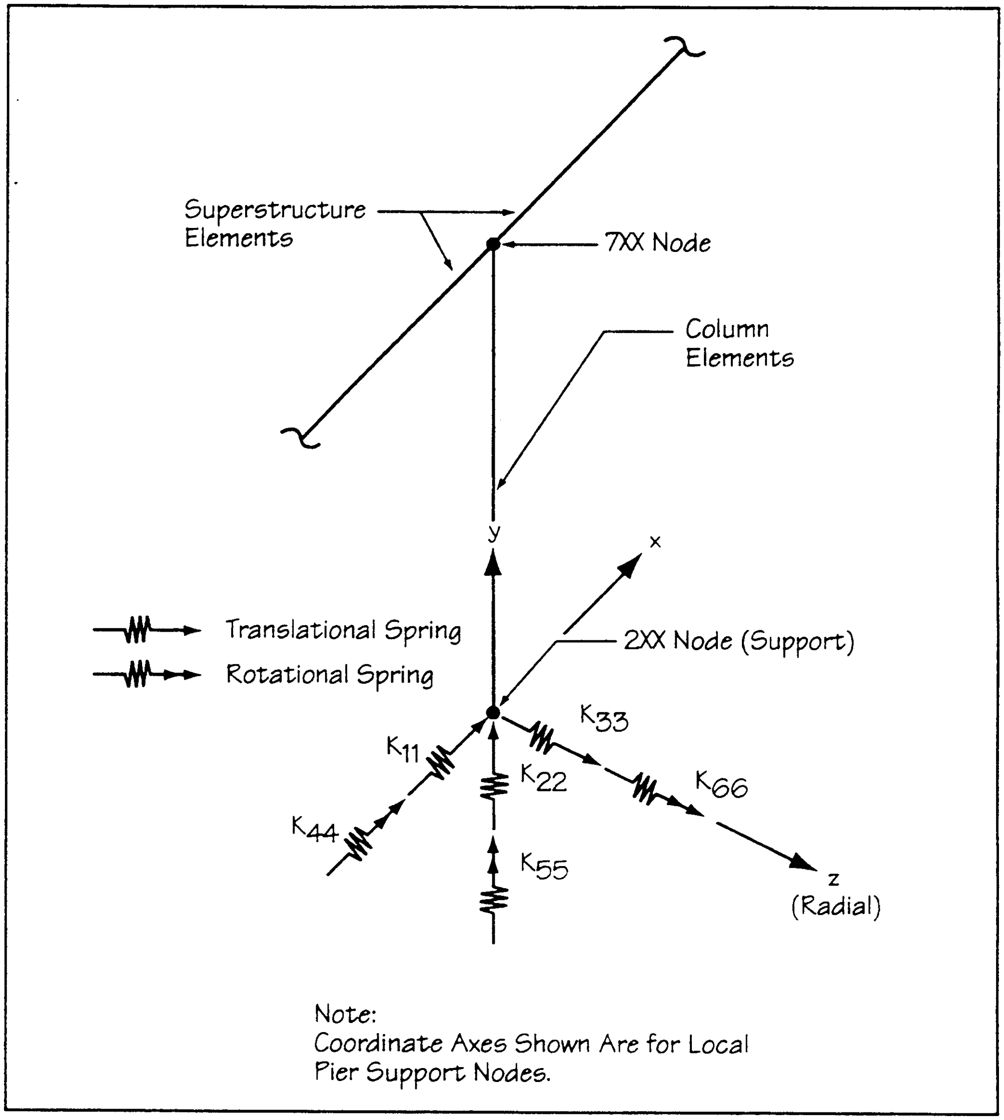


Figure 11 — Details of Supports for Spring Foundation Model

because it is assumed that soil will settle away from the cap. Piles are usually required in poor soil conditions where settlement or liquefaction is expected. In the case of liquefiable soils, downdrag on the piles may add to the vertical load and reduce the pile capacity. Proper consideration of the soil effects requires close coordination between the structural and geotechnical engineers.

For this example, the piers are located in the flood plain of a large river. With the potential for scour and loss of contact of soil around and beneath the pile cap, only the stiffness of the pile group is considered in computing the equivalent springs to model the foundation. Thus, resulting forces at

Design Step 6.2.1 (continued)

the foundation level will only be applied to the pile group to determine design loads to the piles. Flexibility of the pile cap is neglected.

If it is desirable to include any stiffness contribution of the pile cap, a number of methods may be used to calculate equivalent springs for the pile cap and its soil interaction. One way to compute linear springs is to use an elastic subgrade (or half-space) approach as described in the *Seismic Design and Retrofit Manual for Highway Bridges*, FHWA (1987). Additionally, if pile cap stiffnesses are included in the total foundation stiffness, the resulting forces at the foundation level should be properly apportioned between the pile group and the pile cap, which may result in unconservatively low levels of force for design of the pile group if there is any soil settlement or failure.

In order to investigate the effects of varying the foundation stiffnesses for the model, two SAP90 runs were made. The first run used foundation springs computed from the pile group only (lower stiffness). The second run included the pile group stiffness plus the full elastic half-space contribution of the pile cap acting as a spread footing (higher stiffness) in order to provide an upper bound for the foundation spring stiffnesses. Since the relative stiffness of the foundation to the stiffness of the pier column is very large, whether the pile group is considered alone or the contribution of the pile cap is included, the resulting forces for design of the piers and foundations did not vary significantly, generally less than 5 percent. The designer should keep in mind that trends for the sensitivity for resulting levels of force in the structure under seismic loading, with variation of the foundation spring stiffnesses, are dependent upon the relative stiffnesses within the individual structure and are unique to that structure. Generally, any reasonable development of spring stiffnesses will produce acceptable results. The sensitivity of varying bridge foundation stiffnesses has been studied by Cook, et. al., (1995).

The pier foundation stiffnesses used in the model for producing final design forces are the stiffnesses of the pile group only without any stiffness contribution from the soil below the pile cap or contribution of flexibility of the cap itself. A rigid cap was assumed.

a) Determine Single Pile Axial Stiffness

The piles used for the foundation are all 40 feet long, HP 12 x 84. It is assumed that the piles are end bearing and skin friction is neglected in calculation of the axial stiffness.

Design Step 6.2.1 (continued)

For HP 12 x 84

$A := 24.6 \cdot in^2$ Cross-sectional area of pile

$I_{ps} := 650 \cdot in^4$ Moment of inertia about the strong axis

$I_{pw} := 213 \cdot in^4$ Moment of interia about the weak axis

$L := 40.0 \cdot ft$

$E := 29000 \cdot ksi$ Young's Modulus for steel

Calculate the axial stiffness by assuming an axial displacement of 1 inch and computing the axial load associated with that displacement.

$\Delta := 1 \cdot in$

$P := \frac{\Delta \cdot A \cdot E}{L}$ From the relationship $\Delta = PL/AE$

$P = 1.486 \cdot 10^3 \cdot kip$

Therefore, the vertical stiffness

$k_{pv} := \frac{P}{\Delta}$

$k_{pv} = 1.783 \cdot 10^4 \cdot \frac{kip}{ft}$ For a single pile

b) Determine Single Pile Lateral Stiffnesses

There are different methods available to evaluate lateral pile load-deformation behavior, which is a complex relationship of pile deformation and the reaction of the surrounding soil, which may be nonlinear. Computer programs such as COM624 and LPILE are commonly used to obtain load displacement curves. This requires input of soil material properties, such as modulus of subgrade reaction and soil shear strength, along with the properties of the pile. For this example, a linear approximation will be

Design Step
6.2.1
(continued)

used such as described in NAVFAC (1986), *Foundations and Earth Structures, Design Manual 7.02.*

Group action should be considered when pile spacing in the direction of loading is less than six- to eight-pile diameters (D). Because typically spacing may be 4D to 5D, the subgrade reaction modulus should be reduced to account for the increased deflection of a pile in a group versus a pile acting alone. From NAVFAC (1986), *Foundations and Earth Structures, Design Manual 7.02,* Table 1 gives values for reduction factors if specific values have not been supplied in the geotechnical report.

Table 1
Group Effect Reduction Factors

Pile Spacing in Direction of Loading D = Pile Diameter	**Subgrade Reaction Reduction Factor R**
8D	1.00
6D	0.70
4D	0.40
3D	0.25

From: NAVFAC DM7.02 (1986)

For very dense sand, assume a coefficient of variation of lateral subgrade reaction with depth, f from Figure 12, taken from NAVFAC (1986) *Design Manual 7.02*. For group effect, use a reduction (efficiency) Factor R to reduce the effective subgrade reaction.

$$f := 50 \cdot \frac{tons}{ft^3}$$ From Figure 12

$$R := 0.65$$ From Table 1

Design Step 6.2.1 (continued)

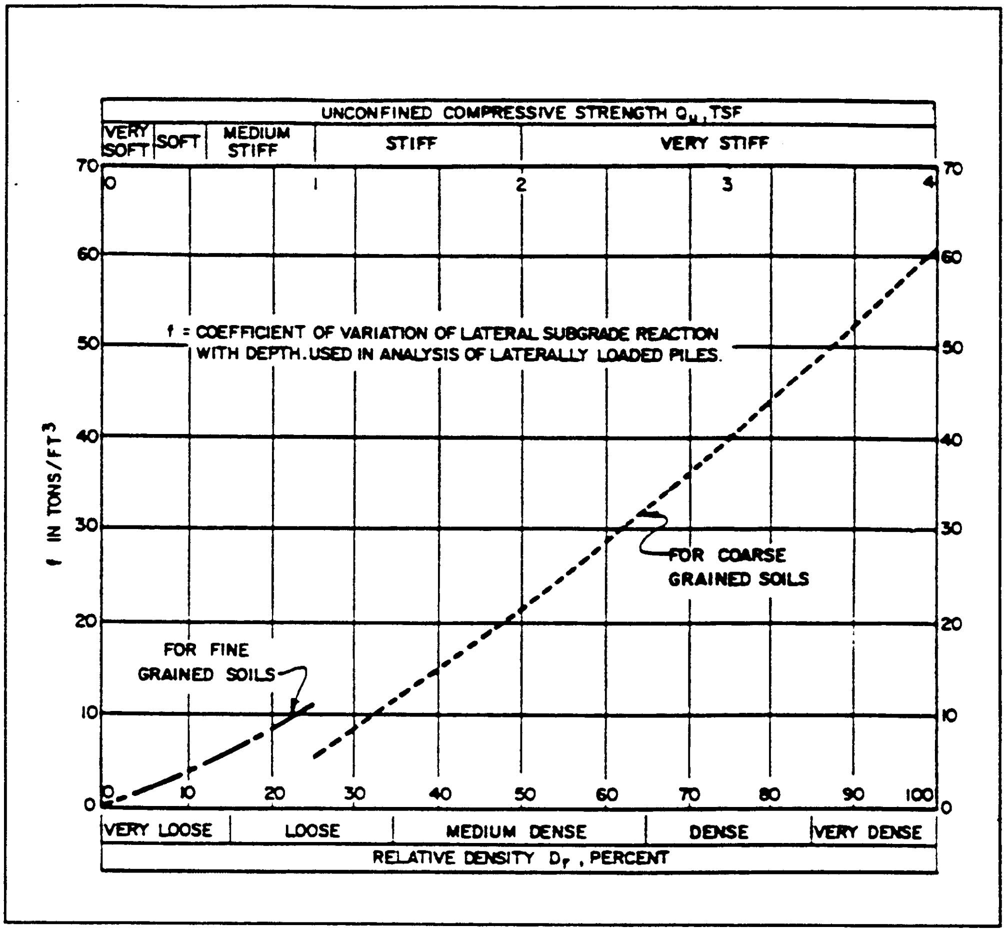


Figure 12 — Coefficient of Variation of Subgrade Reaction

Note: This reduction factor reflects an easy design iteration of the pile group layout and is higher than what would be obtained from Table 1 for pile spacings of 4D to 5D as shown in Figure 14. Due to an oversight, this factor was correct for the computation of springs presented here. Subsequent recalculation of the spring values using a lower reduction factor indicates there would be negligible change to the structure's resulting design forces. The sensitivity of varying foundation stiffness was discussed previously in this design step.

$f_e := f \cdot R$

$f_e = 0.038 \cdot \frac{kip}{in^3}$ Effective coefficient of variation of lateral subgrade reaction in kip/in³

Design Step 6.2.1 (continued)

Compute the relative stiffness factor (T) for each direction from Figure 12 taken from NAVFAC (1986) *Design Manual 7.02.*

$$T_{ps} := \left(\frac{E \cdot I_{ps}}{f_e}\right)^{\frac{1}{5}}$$

$T_{ps} = 55 \cdot in$ — Relative stiffness factor for the strong direction of the pile

$$T_{pw} := \left(\frac{E \cdot I_{pw}}{f_e}\right)^{\frac{1}{5}}$$

$T_{pw} = 44 \cdot in$ — Relative stiffness factor for the weak direction of the pile

Compute the ratios L/T for each direction and use Figure 13 (assuming a pinned pile head condition) to determine the deflection coefficient F_δ for a depth Z equal to zero.

$$Strong := \frac{L}{T_{ps}} \qquad Strong = 8.7$$

$$Weak := \frac{L}{T_{pw}} \qquad Weak = 10.9$$

From Figure 13, the curves for L/T ratios for 5 to 10 are the same for a value of Z = 0, at the top of the pile. Therefore, the deflection coefficient will be assumed the same for both strong and weak axes of the pile.

$F_\delta := 2.25$ — Deflection coefficient

Design Step
6.2.1
(continued)

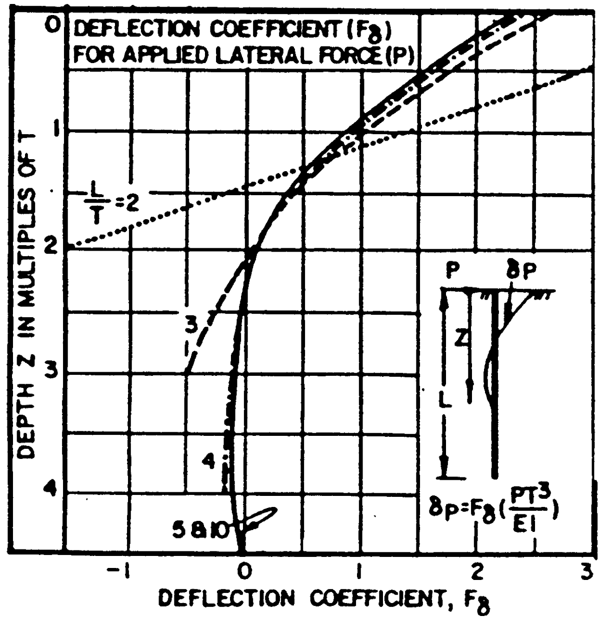


Figure 13 — Deflection Coefficient, Fδ

Calculate the lateral load P for a deflection δ_p at the top of the pile in each direction from the following relationship.

$$\delta_{ps} := F_\delta \cdot \left(\frac{P \cdot T_{ps}^{\,3}}{E \cdot I_{ps}}\right)$$ See Figure 13

For a deflection of 1 inch in the strong direction

$$\delta_{ps} := 1 \cdot in$$

$$P_s := \frac{\delta_{ps} \cdot E \cdot I_{ps}}{F_\delta \cdot T_{ps}^{\,3}} \qquad P_s = 50 \cdot kip$$

Therefore, the translational stiffness in the strong (x) direction

$$k_{ps} := \frac{P_s}{\delta_{ps}} \qquad k_{ps} = 606 \cdot \frac{kip}{ft}$$ For a single pile

Design Step 6.2.1 (continued)

For a deflection of 1 inch in the weak direction

$\delta_{pw} := 1 \cdot in$

$P_w := \dfrac{\delta_{pw} \cdot E \cdot I_{pw}}{F_\delta \cdot T_{pw}^3}$ $\qquad P_w = 32 \cdot kip$

Therefore, the translational stiffness in the weak (z) direction

$k_{pw} := \dfrac{P_w}{\delta_{pw}}$ $\qquad k_{pw} = 388 \cdot \dfrac{kip}{ft}$ $\qquad$ For a single pile

c) Determine Pile Group Stiffnesses

In order to obtain springs for the pile group, some arrangement of piles must be assumed. From a preliminary design step, vertical and lateral loads on the pier foundation would give the designer an estimate of the number of piles required. For this example, all of the intermediate pier foundation springs in the model will be the same. (For a bridge with significantly varying spans and pier loads, different springs should be computed as required.) The assumed pile arrangement has 44 piles and is shown in Figure 14.

Figure 15 shows the six spring directions (three translational and three rotational) that will be calculated for the group. The cross coupling stiffnesses between lateral translation and rocking rotation have been neglected for this example.

$N_p := 44$ $\qquad$ Number of piles in the group

<u>Step 1.</u> Calculate Vertical Stiffness (y translation) of Pile Group.

$k_{22} := k_{pv} \cdot N_p$

$k_{22} = 7.847 \cdot 10^5 \cdot \dfrac{kip}{ft}$

Design Step 6.2.1 (continued)

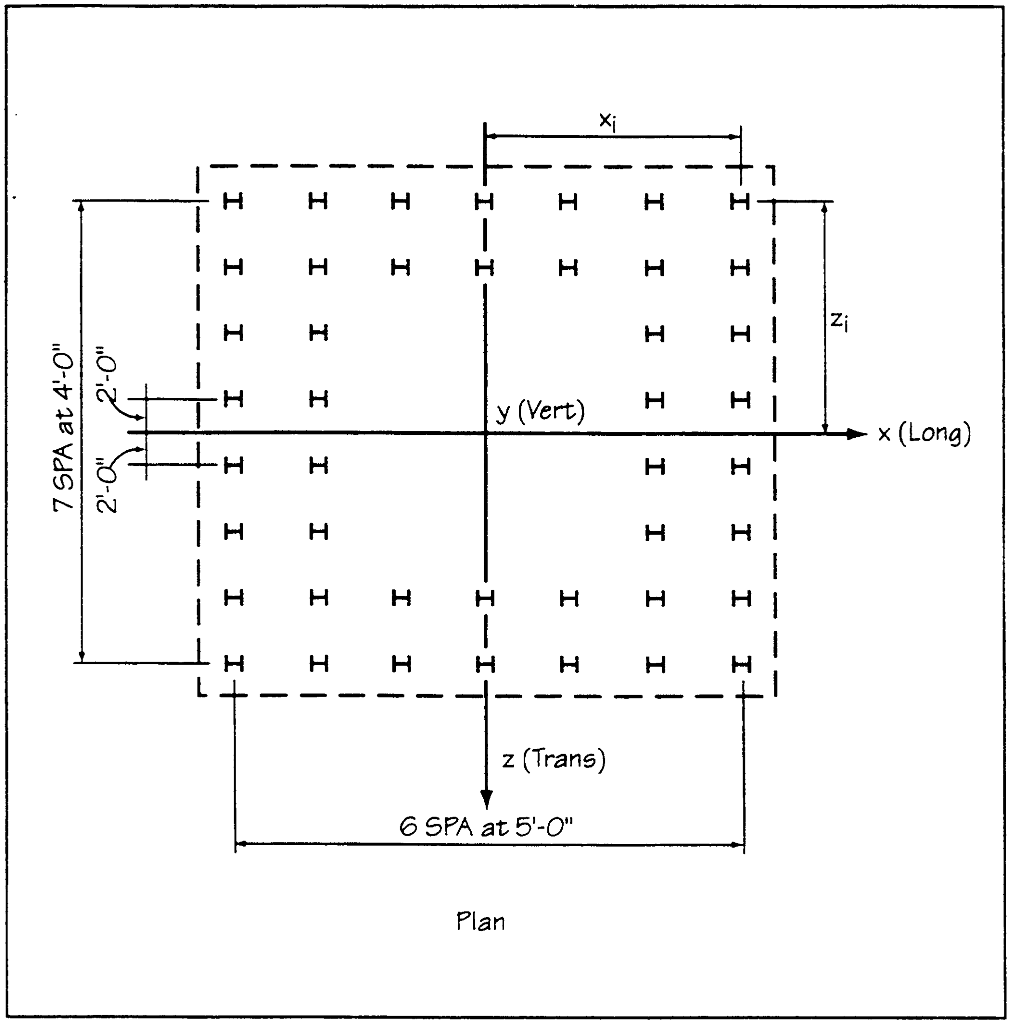


Figure 14 — Pile Layout

Design Step
6.2.1
(continued)

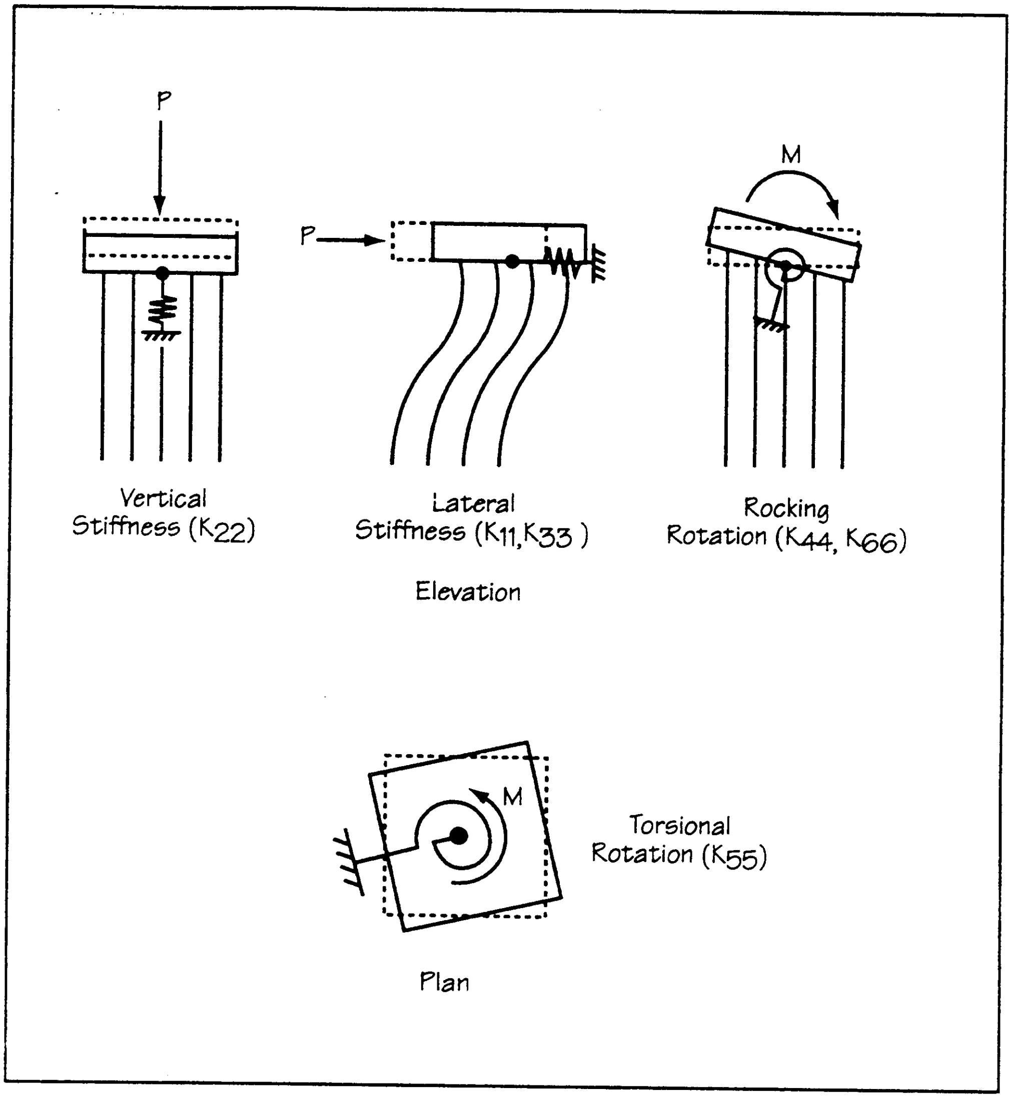


Figure 15 — Foundation Springs

Design Step 6.2.1 (continued)

Step 2. Calculate Lateral Stiffness (x translation) of Pile Group.

$$k_{11} := k_{ps} \cdot N_p$$

$$k_{11} = 2.67 \cdot 10^4 \cdot \frac{kip}{ft}$$

Step 3. Calculate Lateral Stiffness (z translation) of Pile Group.

$$k_{33} := k_{pw} \cdot N_p$$

$$k_{33} = 1.71 \cdot 10^4 \cdot \frac{kip}{ft}$$

The rotational springs require calculation that is dependent upon the pile layout. Torsional resistance from the pile group is computed from the lateral resistances of the sum of the piles in the group much like forces resisting a torque on a bolt group. (The torsional resistance of a single pile is considered equal to zero.)

Step 4. Calculate Torsional Stiffness (y rotation) of Pile Group.

For a single pile

$k_{ps} = 606 \cdot \frac{kip}{ft}$ Translation in the x direction

$k_{pw} = 388 \cdot \frac{kip}{ft}$ Translation in the y direction

To obtain the group stiffness, take the summation of the translational stiffnesses for each direction times the square of the distance component to the pile from the center of the pile group.

$$k_{55} := \sum_{i=1}^{n} k_{ps} \cdot z_i^2 + \sum_{i=1}^{n} k_{pw} \cdot x_i^2$$

Design Step 6.2.1 (continued)

Two piles each have the following distances from the pile group center.

$x_1 := 0.0 \cdot ft \qquad z_1 := 10 \cdot ft$

$x_2 := 0.0 \cdot ft \qquad z_2 := 14 \cdot ft$

Four piles each have the following distances from the pile group center.

$x_3 := 5 \cdot ft \qquad z_3 := 10 \cdot ft$

$x_4 := 5 \cdot ft \qquad z_4 := 14 \cdot ft$

$x_5 := 10 \cdot ft \qquad z_5 := 2 \cdot ft$

$x_6 := 10 \cdot ft \qquad z_6 := 6 \cdot ft$

$x_7 := 10 \cdot ft \qquad z_7 := 10 \cdot ft$

$x_8 := 10 \cdot ft \qquad z_8 := 14 \cdot ft$

$x_9 := 15 \cdot ft \qquad z_9 := 2 \cdot ft$

$x_{10} := 15 \cdot ft \qquad z_{10} := 6 \cdot ft$

$x_{11} := 15 \cdot ft \qquad z_{11} := 10 \cdot ft$

$x_{12} := 15 \cdot ft \qquad z_{12} := 14 \cdot ft$

Summing the torsional pile resistance by rows for both weak and strong axes of the piles.

$$k_{1w} := 2 \cdot k_{pw} \cdot \left(x_1^2 + x_2^2\right)$$

$$k_{1s} := 2 \cdot k_{ps} \cdot \left(z_1^2 + z_2^2\right)$$

$$k_{2w} := 4 \cdot k_{pw} \cdot \left(x_3^2 + x_4^2\right)$$

$$k_{2s} := 4 \cdot k_{ps} \cdot \left(z_3^2 + z_4^2\right)$$

Design Step 6.2.1 (continued)

$$k_{3w} := 4 \cdot k_{pw} \cdot \left(x_5^2 + x_6^2 + x_7^2 + x_8^2\right)$$

$$k_{3s} := 4 \cdot k_{ps} \cdot \left(z_5^2 + z_6^2 + z_7^2 + z_8^2\right)$$

$$k_{4w} := 4 \cdot k_{pw} \cdot \left(x_9^2 + x_{10}^2 + x_{11}^2 + x_{12}^2\right)$$

$$k_{4s} := 4 \cdot k_{ps} \cdot \left(z_9^2 + z_{10}^2 + z_{11}^2 + z_{12}^2\right)$$

Summing up for the entire pile group.

$$k_{55} := k_{1w} + k_{1s} + k_{2w} + k_{2s} + k_{3w} + k_{3s} + k_{4w} + k_{4s}$$

$$k_{55} = 4.798 \cdot 10^6 \cdot kip \cdot \frac{ft}{rad}$$

The rocking rotational resistance in each direction of the pile group is computed from the sum of the moment resistance of the piles as a function of their vertical stiffness and distance squared from the axis of rotational stiffness, which for a symmetrical pile group is the center of the pile group. Resisting moments at the connection of the individual piles and the pile cap are disregarded. This method also assures elastic pile behavior and no soil resistance.

Step 5. Calculate Rocking Rotational Stiffness (x axis) of Pile Group.

For a single pile

$$k_{pv} = 1.783 \cdot 10^4 \cdot \frac{kip}{ft}$$

About the x axis, the distance from the rotation center and the number of piles is

$z_1 := 2 \cdot ft$ 8 piles

$z_2 := 6 \cdot ft$ 8 piles

Design Step 6.2.1 (continued)

$z_3 := 10 \cdot ft$ 14 piles

$z_4 := 14 \cdot ft$ 14 piles

$$k_{44} := k_{pv} \cdot \left[\left(8 \cdot z_1^{\,2}\right) + \left(8 \cdot z_2^{\,2}\right) + \left(14 \cdot z_3^{\,2}\right) + \left(14 \cdot z_4^{\,2}\right) \right]$$

$$k_{44} = 7.962 \cdot 10^{7} \cdot kip \cdot \frac{ft}{rad}$$

<u>Step 6.</u> Calculate Rocking Rotational Stiffness (z axis) of Pile Group.

About the z axis, the distance from the rotation center and the number of piles is as follows. (Note that four piles have $x = 0$ and don't contribute.)

$x_1 := 5 \cdot ft$ 8 piles

$x_2 := 10 \cdot ft$ 16 piles

$x_3 := 15 \cdot ft$ 16 piles

$$k_{66} := k_{pv} \cdot \left[\left(8 \cdot x_1^{\,2}\right) + \left(16 \cdot x_2^{\,2}\right) + \left(16 \cdot x_3^{\,2}\right) \right]$$

$$k_{66} = 9.631 \cdot 10^{7} \cdot kip \cdot \frac{ft}{rad}$$

<u>Step 7.</u> Summary of Pile Group Spring Stiffness.

$k_{11} = 2.67 \cdot 10^{4} \cdot \frac{kip}{ft}$ Translation, x axis

$k_{22} = 7.85 \cdot 10^{5} \cdot \frac{kip}{ft}$ Translation, y (vertical) axis

$k_{33} = 1.71 \cdot 10^{4} \cdot \frac{kip}{ft}$ Translation, z axis

$k_{44} = 7.96 \cdot 10^{7} \cdot kip \cdot \frac{ft}{rad}$ Rotation, x axis (rocking)

Design Step
6.2.1
(continued)

$$k_{55} = 4.8 \cdot 10^{6} \cdot kip \cdot \frac{ft}{rad}$$ Rotation, y axis (torsion)

$$k_{66} = 9.63 \cdot 10^{7} \cdot kip \cdot \frac{ft}{rad}$$ Rotation, z axis (rocking)

Use these pile group springs to model the foundation stiffnesses in the Multimode Spectral Method. These are input into the SAP90 Model in the local pier support node coordinate systems as shown in Figure 11.

Design Step 6.2.2

Abutments

The abutments were modeled with a combination of full restraints (vertical translation and superstructure torsional rotation) and an equivalent spring stiffness (transverse translation) as shown in Figure 16. The transverse translational spring stiffness is based upon the stiffnesses of the individual pile stiffnesses used for the intermediate piers. The spring value for the abutments is a ratio of the number of abutment piles (assumed to be 12, see Figure 17) to the number of intermediate pier piles times the value of the transverse translational spring (K_{33}) used at the intermediate piers. Other degrees of freedom at the abutment support nodes are released. Since SAP90 allows for springs and releases relative to the local coordinate geometry, the longitudinal direction at the abutment nodes is oriented along the axis of the superstructure element connected at that node. The transverse direction is perpendicular to the longitudinal direction in the global x-z plane.

The model allows longitudinal response that is unrestrained at the abutment. A gap between the end of the superstructure and the abutment backwall that is larger than the expected seismic displacement must be included if no longitudinal force is to be developed, see Figure 18. Depending on the site acceleration coefficient, soil conditions, and bridge configuration, this gap may be a reasonable size to accommodate available expansion joint configurations, or it could be too small.

In such a case, the longitudinal movement would be unrestrained until the superstructure came into contact with the abutment backwall. Then a longitudinal force would develop. This effect can be modeled and is described in the *Seismic Design and Retrofit Manual for Highway Bridges*, FHWA (1987).

The ends of the superstructure are restrained against translation in the transverse direction at the abutments by girder stops. The forces resulting from this restraint are passed through the girder stops into the abutment to be resisted by the pile group.

Torsional response of the superstructure is fully restrained in the model by the abutments.

Design Step
6.2.2
(continued)

The support node locations at the abutments are at the intersection of the superstructure work line (at the centroid of the superstructure) and the centerline of the bearings. The abutment restraints and transverse spring act at these nodes that are oriented in the local superstructure element coordinate geometry.

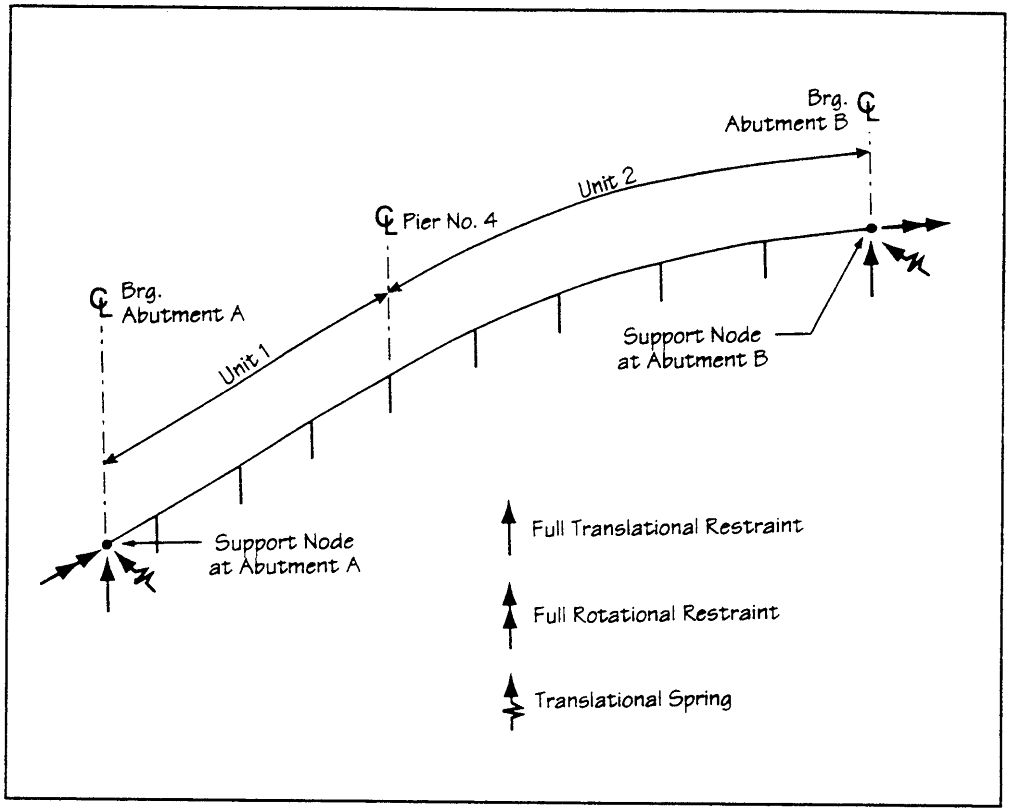


Figure 16 — Details of Abutment Supports

Design Step
6.2.2
(continued)

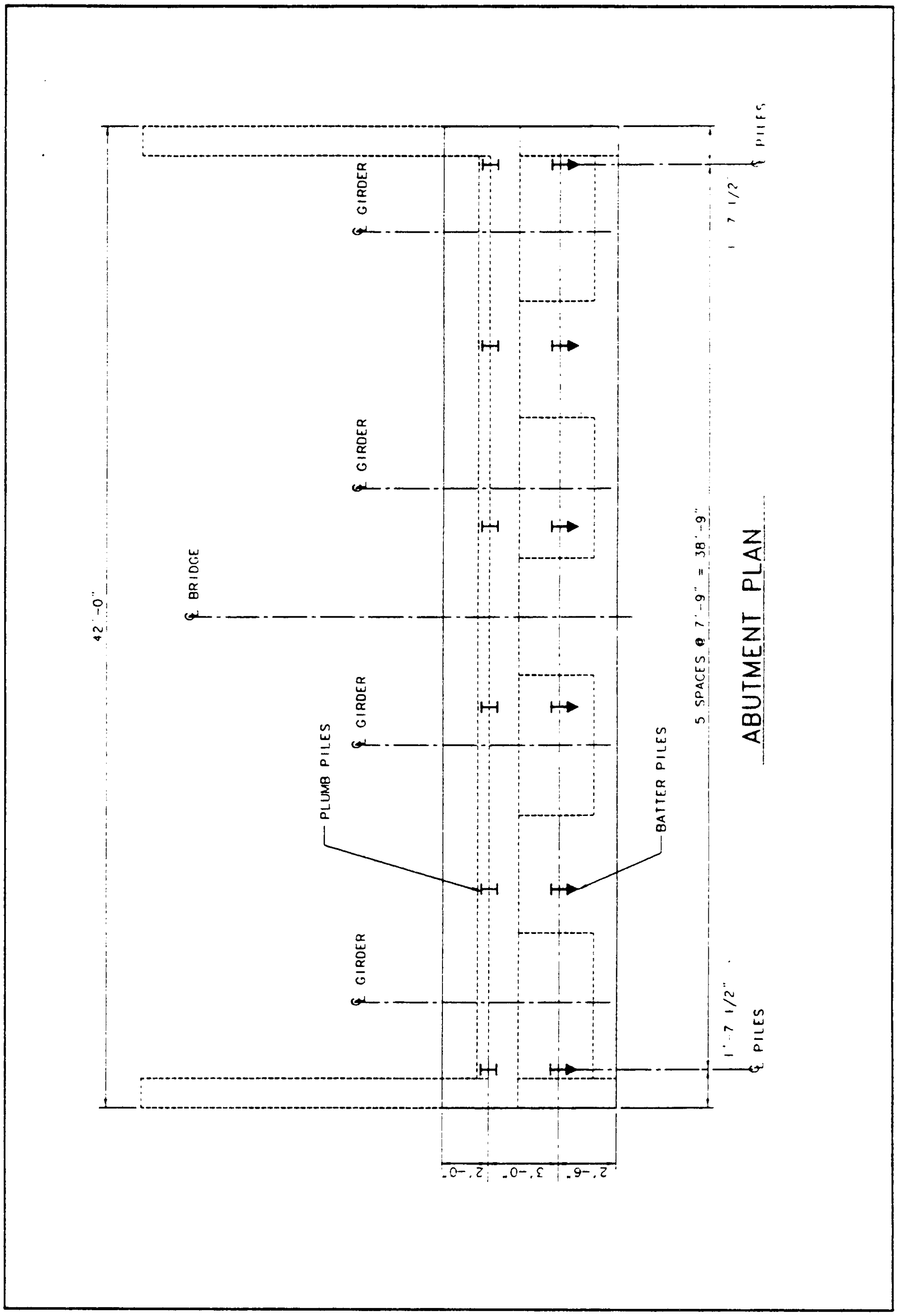


Figure 17 — Abutment Plan

Design Step
6.2.2
(continued)

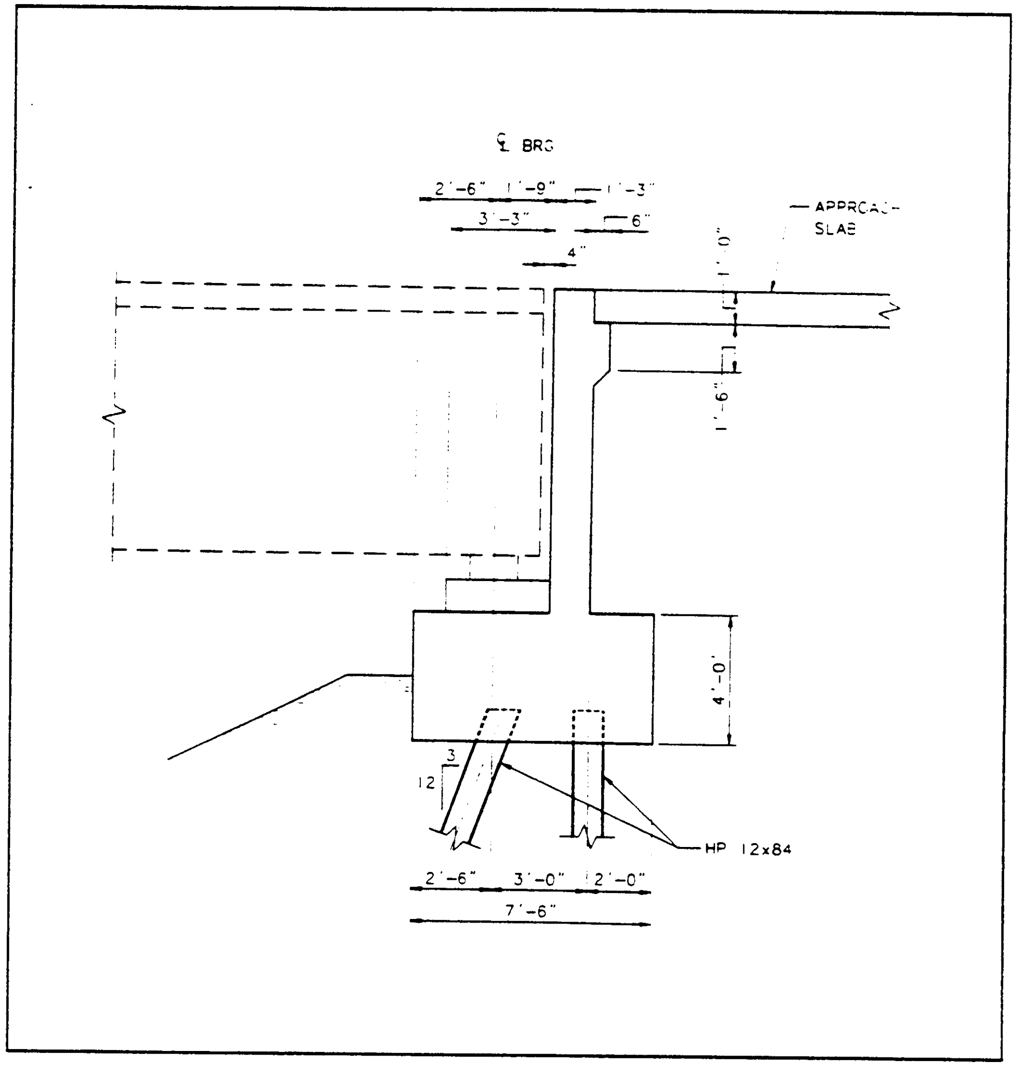


Figure 18 — Section at Abutment

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 discretized using four elements per span and elements at each pier dimension transition as discussed previously. Thirty-six vibration modes were included in the multimodal spectral analysis, which involves the superposition of individual modal responses to estimate the overall structural seismic response.

The SAP90 program (or most any other dynamic spectral analysis program) lumps the tributary mass of each element to the adjacent nodes. Spring elements that provide foundation flexibility are massless. SAP90 determines the vibration periods and shapes for each of the vibration modes of the structure. The number of modes is dependent on the number of masses, the number of constrained degrees of freedom, and the number of foundation restraints for the system. Enough modes have to be specified so that the modal superposition to determine forces and displacements is accurate. Typically the modes are numbered sequentially from the longest period to the shortest.

The natural periods of vibration for the bridge are shown in Table 2 for the first 36 modes. Figures 19, 20, and 21 show three selected modes for the structure. Figures 19 and 20 show the modes associated with the fundamental periods in the longitudinal direction for Unit 2 and Unit 1, respectively. The longitudinal periods for these modes (first and second) are 1.52 seconds for Unit 2 and 1.21 seconds for Unit 1. Figure 21 shows the third mode that is the first significant mode in the transverse direction, i.e., has translation of the majority of the piers in the same direction. The period for the third mode is 0.80 second.

Hand Check ✓ Check Fundamental Period in the Longitudinal Direction

As a check, compare the longitudinal periods from the multimode analysis with those calculated in Design Step 1, Preliminary Design.

From Preliminary Design (Step 1.3), the calculated longitudinal period was 1.55 seconds for Unit 2 and 1.26 seconds for Unit 1. In the preliminary design, the foundations were fixed at the base of the pile cap. The values of the longitudinal periods are quite close (1.55 versus 1.52 seconds for Unit 2 and 1.26 versus 1.21 seconds for Unit 1). This suggests that assuming the

Design Step 6.3.1 (continued)

foundations are fixed at the base of the pile cap, the longitudinal stiffness closely approximated the longitudinal foundation spring stiffnesses used in the modal analysis.

Table 2
Modal Periods and Frequencies

PROGRAM SAP90, VERSION BETA6.00 FILE:EXAM5.OUT
FHWA BRIDGE NO. 5

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

MODE	PERIOD (TIME)	FREQUENCY (CYC/TIME)	FREQUENCY (RAD/TIME)	EIGENVALUE (RAD/TIME)**2
1	1.517657	0.658911	4.140057	17.140073
2	1.206924	0.828553	5.205950	27.101917
3	0.802425	1.246222	7.830247	61.312764
4	0.748225	1.336496	8.397452	70.517201
5	0.748225	1.336496	8.397452	70.517192
6	0.746454	1.339668	8.417381	70.852297
7	0.744797	1.342647	8.436100	71.167789
8	0.680350	1.469831	9.235219	85.289270
9	0.654901	1.526949	9.594104	92.046835
10	0.597015	1.675001	10.524340	110.761722
11	0.568440	1.759202	11.053391	122.177449
12	0.504153	1.983523	12.462843	155.322457
13	0.489787	2.041705	12.828412	164.568157
14	0.462418	2.162546	13.587677	184.624971
15	0.445179	2.246286	14.113830	199.200187
16	0.445174	2.246313	14.113998	199.204946
17	0.391640	2.553366	16.043274	257.386632
18	0.340580	2.936169	18.448492	340.346865
19	0.323410	3.092050	19.427922	377.444158
20	0.322856	3.097357	19.461270	378.741040
21	0.306128	3.266602	20.524667	421.261940
22	0.256582	3.897389	24.488020	599.663123
23	0.233949	4.274430	26.857035	721.300317
24	0.233057	4.290803	26.959907	726.836609
25	0.231134	4.326489	27.184132	738.977032
26	0.231034	4.328370	27.195953	739.619857
27	0.230329	4.341609	27.279133	744.151116
28	0.228764	4.371318	27.465803	754.370344
29	0.225047	4.443523	27.919478	779.497267
30	0.216346	4.622222	29.042275	843.453712
31	0.216346	4.622220	29.042265	843.453163
32	0.211280	4.733052	29.738643	884.386910
33	0.206355	4.846029	30.448497	927.110987
34	0.204352	4.893525	30.746922	945.373221
35	0.194483	5.141843	32.307149	1043.752
36	0.191911	5.210740	32.740044	1071.910

Design Step 6.3.1 (continued)

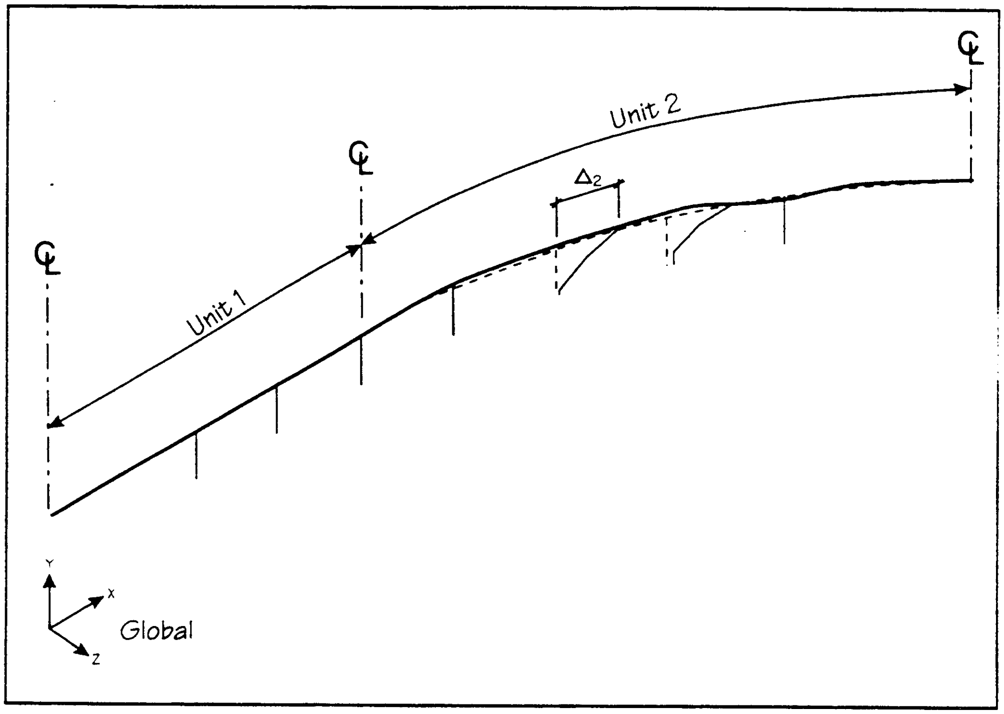


Figure 19 – Deformed Shape for Mode 1

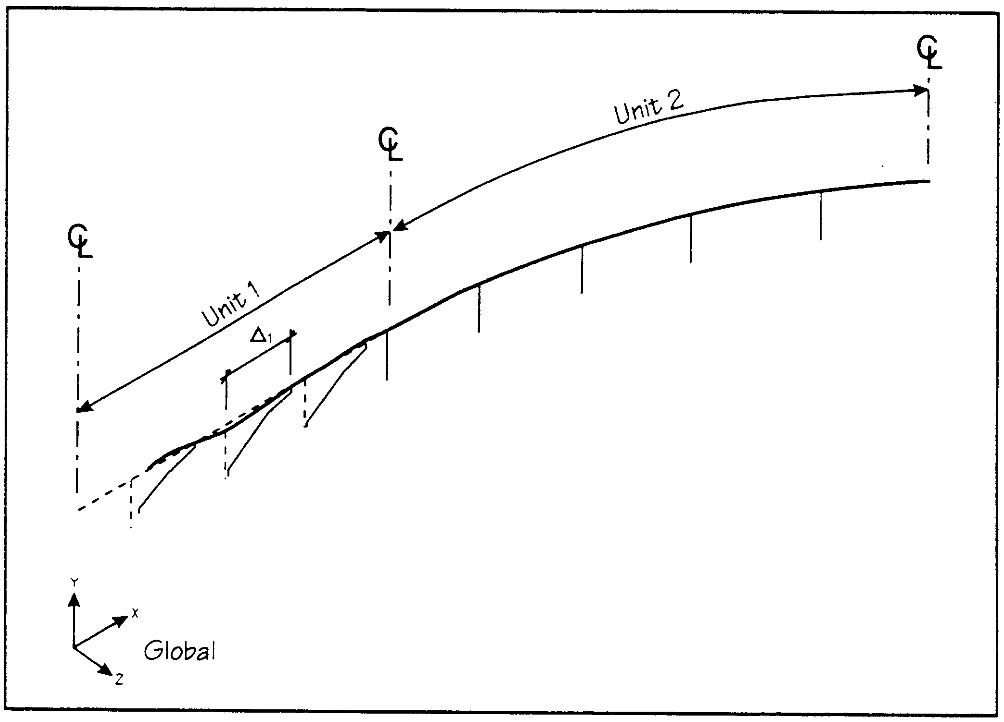


Figure 20 – Deformed Shape for Mode 2

Design Step
6.3.1
(continued)

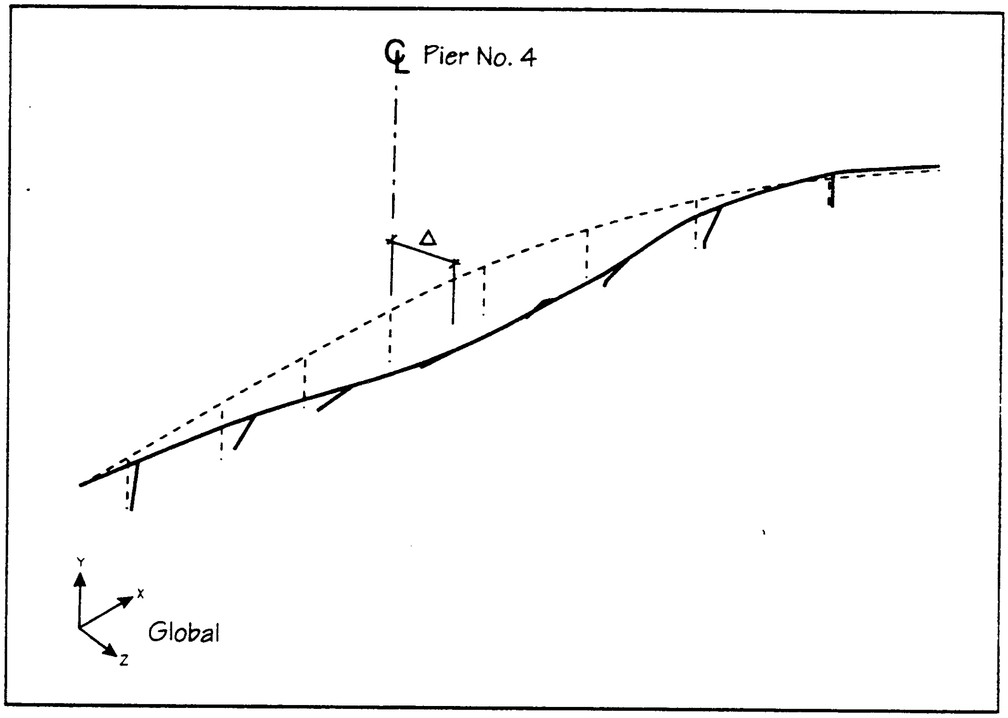


Figure 21 — Deformed Shape for Mode 3

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 22. The curve shown in the figure is given by the equation for C_{sm}, the elastic seismic response coefficient below.

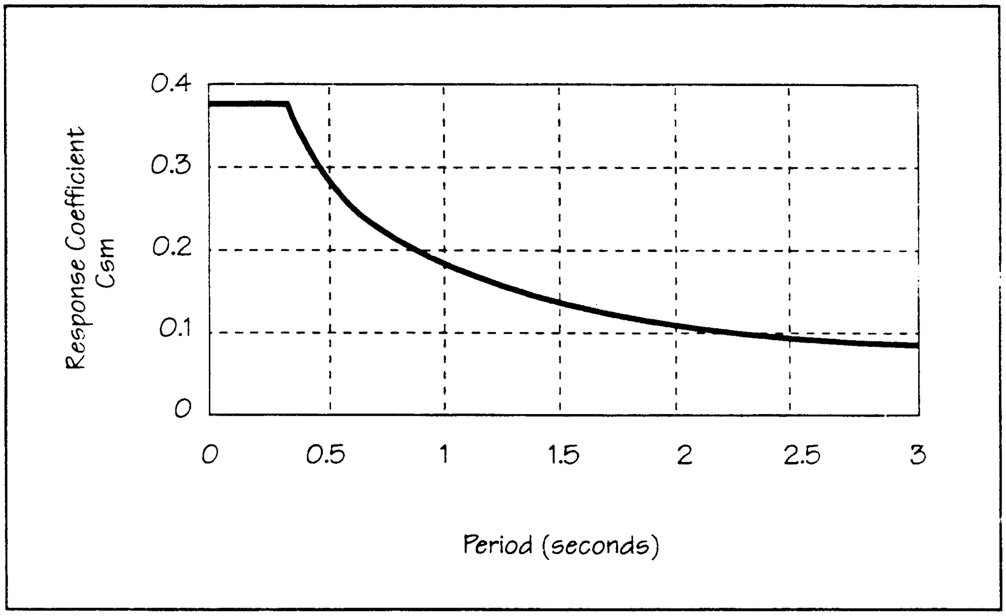


Figure 22 — Relationship Between Elastic Seismic Response Coefficient and Period

$$C_{sm}(T_m) := \frac{1.2 \cdot A \cdot S}{T_m^{\frac{2}{3}}} < 2.5 \cdot A$$

Division I-A
Eqn (3-2)

Where:

A is the acceleration coefficient
S is the site coefficient
T_m is the period of the mth mode of vibration

A design response spectrum must be input to provide loading for the model. This spectrum is specified in Article 3.6.2 of the Specification, and it applies in both the transverse and longitudinal directions.

Design Step 6.3.2 (continued)

For this example, the longitudinal direction (EQlong) is along a straight line, which connects the node at Abutment A with the node at Abutment B. Because of the structure's plan curvature, this direction is in between the tangent direction of Unit 1 and the chord direction of the curve of Unit 2. The transverse direction (EQtrans) is applied at 90 degrees to the longitudinal direction. See Figure 23 for plan directions of the applied earthquake loading.

The spectrum is defined as a function of period T by Equation 3-2 of Division I-A with the upper limit of two-and-a-half times A, which C_{sm} need not exceed. 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. (C_{sm} is the modal analysis version of C_s.) The range must cover the entire range of expected periods for the structure.

Figure 22 and Equation 3-2 are based on 5 percent damping.

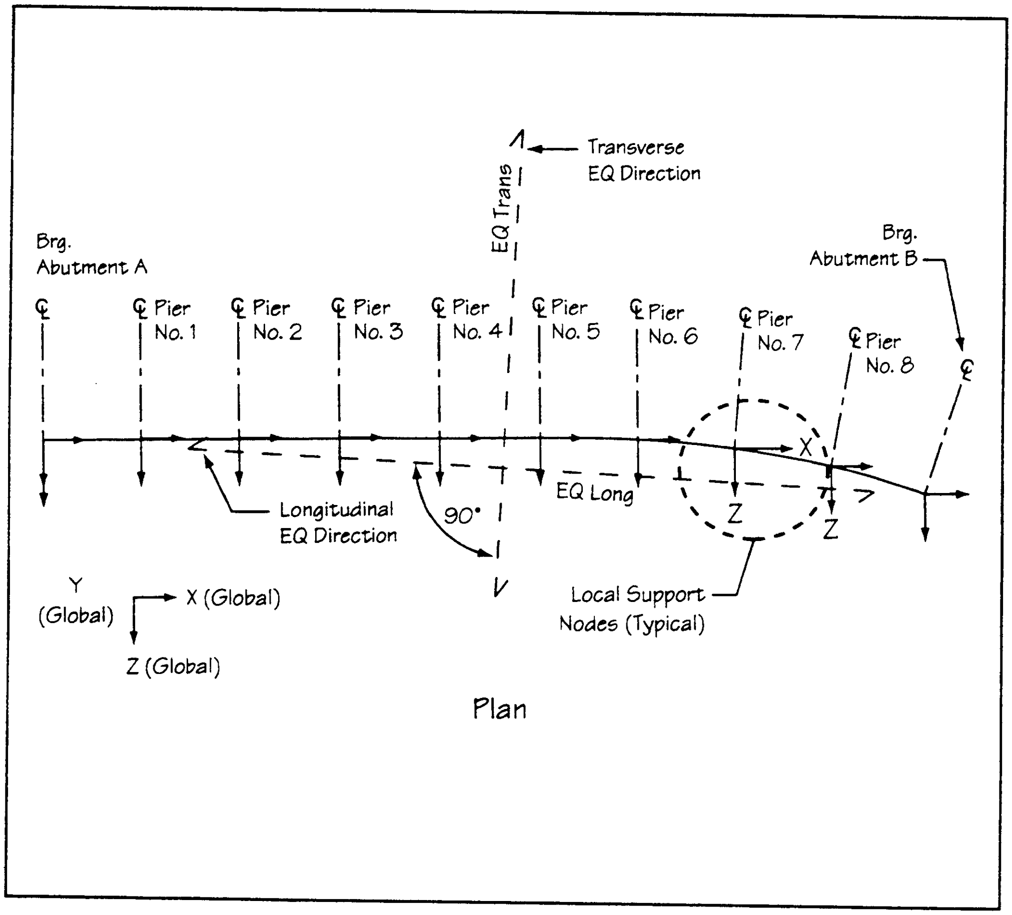


Figure 23 — Earthquake Loading Directions

Design Step 6.3.3

Minimum Number of Modes
[Division I-A, Article 4.5]

Thirty-six modes have been included to provide an accurate estimate of the response and internal forces. Note that 36 modes is more than three times the number of spans or maximum of 25 as given in Article 4.5.4 of the Specification in order to obtain at least 90 percent mass participation for each of the principal directions of applied loading.

As discussed previously, enough modes have to be specified so that the modal superposition to determine forces and displacements is sufficiently accurate.

One way of assessing how many modes are sufficient to characterize response is to ensure that the percentage of mass that participates in each mode in each direction is at least 90 percent of the total for each of the directions of the applied loading. In this example, there is no loading applied in the vertical (y) direction, and having 90 percent minimum mass participation in that direction is not critical. However, the designer should not rely only on mass participation. Mode shapes should be inspected to determine that important masses, such as all of the substructure elements, are excited by the selected modes.

Results from the multimode analysis are given in Table 3. The three columns under Individual Mode (percent) show the participating mass in each direction for each mode. The next three columns under Cumulative Sum (percent) show the cumulative participating mass in each direction. The result is that 31 modes are required to obtain more than 90 percent of the mass in each of the two plan directions (x and z), which are the directions of the applied lateral loading.

Design Step
6.3.3
(continued)

In Table 3

UX = Longitudinal Direction
UY = Vertical Direction
UZ = Transverse Direction

Table 3
Modal Participating Mass

PROGRAM SAP90, VERSION BETA6.00 FILE:EXAM5.OUT

FHWA BRIDGE NO. 5

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)			CUMULATIVE SUM (PERCENT)		
		UX	UY	UZ	UX	UY	UZ
1	1.517657	29.3365	0.0001	3.0720	29.3365	0.0001	3.0720
2	1.206924	30.4339	0.0005	0.0000	59.7704	0.0006	3.0720
3	0.802425	0.3700	0.0006	60.0941	60.1404	0.0011	63.1661
4	0.748225	7.5502	0.0000	0.0625	67.6905	0.0011	63.2287
5	0.748225	1.0236	0.0000	0.0136	68.7141	0.0011	63.2423
6	0.746454	1.0358	0.0947	0.0400	69.7499	0.0958	63.2822
7	0.744797	0.0805	0.6990	0.0567	69.8304	0.7949	63.3389
8	0.680350	1.6075	0.0000	14.2497	71.4379	0.7949	77.5885
9	0.654901	0.0022	0.0000	0.0205	71.4401	0.7949	77.6090
10	0.597015	2.9385	0.0000	4.6694	74.3786	0.7949	82.2784
11	0.568440	0.0113	1.7409	0.0000	74.3899	2.5358	82.2784
12	0.504153	0.0038	2.3373	0.0000	74.3937	4.8732	82.2784
13	0.489787	0.1215	0.0003	4.7125	74.5152	4.8734	86.9909
14	0.462418	0.0007	0.0030	0.0000	74.5159	4.8764	86.9909
15	0.445179	3.3345	0.0000	1.1579	77.8504	4.8764	88.1488
16	0.445174	0.1508	0.0002	0.4261	78.0012	4.8766	88.5749
17	0.391640	0.0138	0.0013	0.0031	78.0150	4.8779	88.5780
18	0.340580	0.0212	0.0000	0.5894	78.0363	4.8779	89.1674
19	0.323410	0.0171	0.0024	0.0030	78.0534	4.8803	89.1704
20	0.322856	0.0001	16.6257	0.0001	78.0535	21.5060	89.1705
21	0.306128	0.0054	7.9300	0.0000	78.0589	29.4360	89.1705
22	0.256582	0.0000	0.0000	0.0489	78.0589	29.4361	89.2195
23	0.233949	0.0010	0.0000	1.2420	78.0599	29.4361	90.4614
24	0.233057	0.0780	0.0000	6.1700	78.1378	29.4361	96.6314
25	0.231134	0.0420	0.0269	0.0000	78.1799	29.4629	96.6314
26	0.231034	0.0484	0.0003	0.0037	78.2282	29.4632	96.6351
27	0.230329	2.8202	0.0005	0.3042	81.0484	29.4637	96.9393
28	0.228764	5.7128	0.0002	0.0000	86.7612	29.4640	96.9393
29	0.225047	0.0619	0.0001	0.1896	86.8231	29.4640	97.1289
30	0.216346	2.3639	0.0000	0.0522	89.1870	29.4640	97.1811
31	0.216346	3.5549	0.0000	0.0005	92.7420	29.4640	97.1816
32	0.211280	0.0808	0.0000	0.1340	92.8228	29.4641	97.3156
33	0.206355	2.8918	0.7484	0.0000	95.7145	30.2125	97.3156
34	0.204352	2.2508	0.0008	0.4787	97.9653	30.2133	97.7943
35	0.194483	0.0001	0.0000	0.6491	97.9654	30.2133	98.4434
36	0.191911	0.4414	3.9573	0.0000	98.4068	34.1706	98.4434

Design Step 6.3.4

Combination of Modal Forces and Displacements
[Division I-A, Article 4.5]

The response of the model in each of the calculated modes must be superimposed to estimate the overall response. Since all the modal maximum responses do not occur simultaneously, a simple summation of the modal absolute values is not appropriate. Most programs use either the Square Root of the Sum of the Squares (SRSS) Method or the Complete Quadratic Combination (CQC) Method. The simplest is the SRSS method, and it is adequate when the modal periods are well spaced. When the periods are quite close, coupling between modal response can occur, and the CQC method should be used. This method accounts for coupling between modes, preserves the signs of the cross-modal terms, and is based on random vibrational fundamentals. Most programs now have the CQC method as an option. The method requires very little additional run time for most models and should be used exclusively to eliminate the judgment of what constitutes closely spaced periods. The default combination method for SAP90 is CQC.

Design Step 6.4

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

Using the Multimode Spectral Method, perform a transverse analysis. Transverse analysis means that the input response spectrum was assigned to the transverse direction, and in this case no longitudinal or vertical spectra were used. The longitudinal and transverse directions for the application of loading for the structure are described in Design Step 6.3.2 and are shown in Figure 23. The longitudinal direction is along a straight line that connects the node at Abutment A with the node at Abutment B. The transverse direction is applied at 90 degrees to the longitudinal direction.

The analysis program handles all the calculations, including the modal combinations. In this case, 36 modes were used to characterize the response. This number was kept constant for all the analyses.

The results are given in Table 4. The SAP90 input file for this analysis is EXAM5. Shown in the table are forces and moments for the intermediate piers, which are the focus of the design process for this example. Directions for forces and moments are shown in Figure 24 and are oriented along the local coordinate system for the column elements.

Displacements are given in Table 5 for both transverse and longitudinal analysis. Figure 25 shows directions for the displacements that are in the global coordinate system.

Hand Check ✓ Check Transverse Column Shear Forces

As a check, compare the preliminary values computed in Design Step 1.4, Figure 6 with the average values of the column top transverse shears shown in Table 4.

For example, from Figure 6 in Design Step 1, the transverse column shear for Pier No. 7 is 689 kips. The transverse column top shear for Pier No. 7 from Table 4 is 391 kips.

Other shears may be similarly compared. All of the preliminary transverse column shears are higher than those from the modal analysis.

Design Step 6.4
(continued)

Table 4
Response for Transverse Direction (EQtrans)

Support/Location		Forces and Moments - EQtrans: Longitudinal Shear (kips)	Longitudinal Moment (kip-ft)	Transverse Shear (kips)	Transverse Moment (kip-ft)	Axial (kips)
Pier No. 1	Column Top	119	0	259	1,679	9
	Column Base	140	6,972	384	19,908	9
	Foundation	147	7,892	445	22,723	10
Pier No. 2	Column Top	40	0	315	2,046	6
	Column Base	64	4,279	509	35,925	7
	Foundation	84	4,707	580	39,446	7
Pier No. 3	Column Top	40	0	406	2,634	3
	Column Base	65	4,282	632	44,870	3
	Foundation	83	4,709	712	49,295	4
Pier No. 4	Column Top	6	0	472	3,066	7
	Column Base	50	2,958	753	53,373	7
	Foundation	70	3,277	846	58,753	7
Pier No. 5	Column Top	0	0	458	2,941	6
	Column Base	15	917	702	50,052	6
	Foundation	22	1,017	788	54,935	6
Pier No. 6	Column Top	37	0	337	2,167	6
	Column Base	52	3,493	543	38,385	6
	Foundation	58	3,837	613	42,217	6
Pier No. 7	Column Top	108	0	391	2,524	5
	Column Base	125	6,138	539	28,593	5
	Foundation	132	6,936	602	32,492	5
Pier No. 8	Column Top	0	0	289	0	6
	Column Base	109	4,455	448	23,128	6
	Foundation	130	5,302	516	26,474	6

Design Step 6.4 (continued)

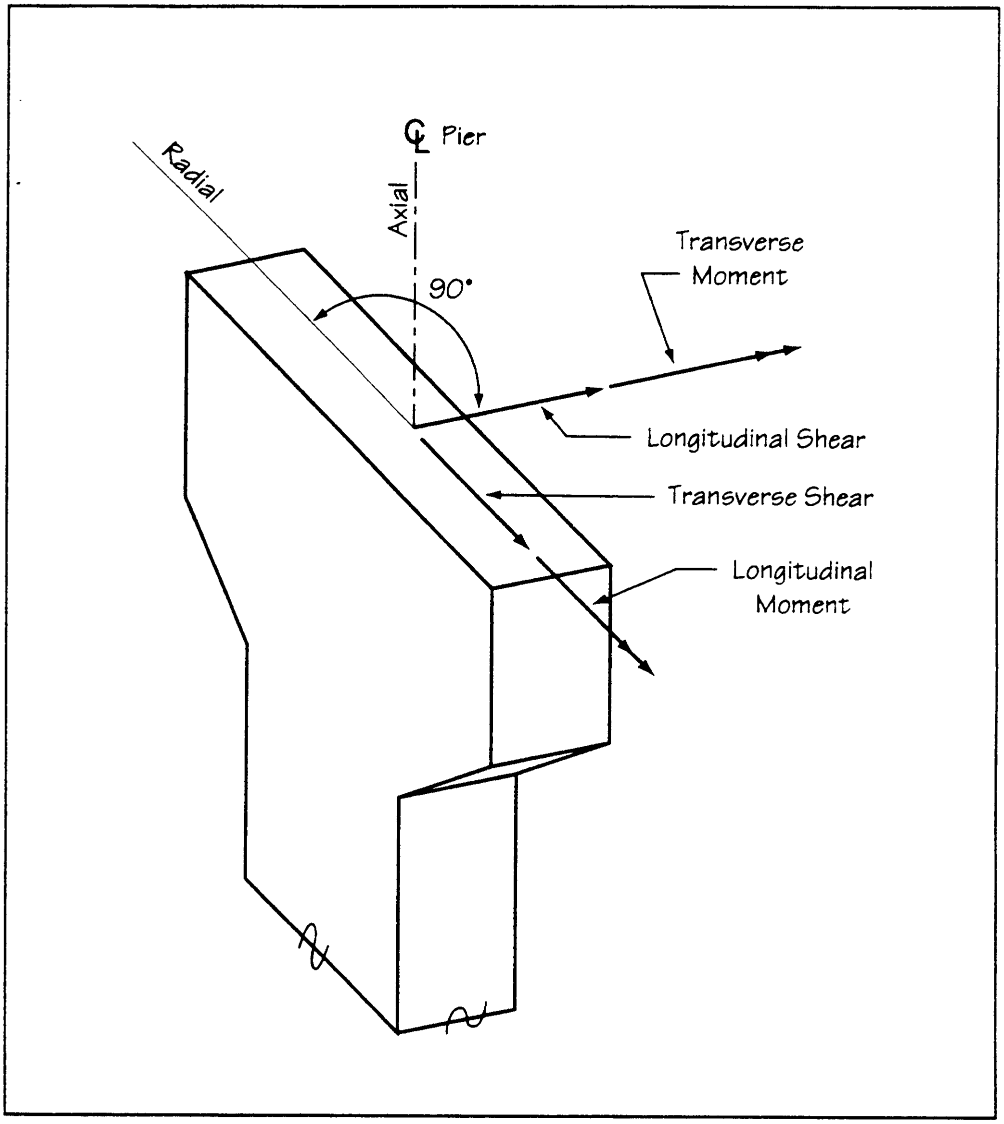


Figure 24 — Key to Force and Moment Directions

Design Step 6.4 (continued)

Table 5
Displacements

Support/Location		DISPLACEMENTS EQtrans Global X (ft)	EQtrans Global Z (ft)	EQlong Global X (ft)	EQlong Global Z (ft)
Abutment A	Superstructure	0.0394	0.0327	0.2022	0.0059
Pier No. 1	Superstructure	0.0394	0.0563	0.2019	0.0114
	Foundation	0.0058	0.0280	0.0297	0.0056
Pier No. 2	Superstructure	0.0397	0.1095	0.2036	0.0243
	Foundation	0.0036	0.0363	0.0187	0.0080
Pier No. 3	Superstructure	0.0399	0.1368	0.2047	0.0284
	Foundation	0.0036	0.0441	0.0185	0.0092
Pier No. 4	Superstructure - Unit 1	0.0400	0.1636	0.2054	0.0263
	Superstructure - Unit 2	0.0486	0.1636	0.2679	0.0263
	Foundation	0.0032	0.0518	0.0163	0.0087
Pier No. 5	Superstructure	0.0480	0.1488	0.2654	0.0314
	Foundation	0.0010	0.0487	0.0166	0.0058
Pier No. 6	Superstructure	0.0433	0.1139	0.2565	0.0630
	Foundation	0.0024	0.0379	0.0211	0.0061
Pier No. 7	Superstructure	0.0375	0.0772	0.2413	0.1023
	Foundation	0.0053	0.0369	0.0364	0.0097
Pier No. 8	Superstructure	0.0383	0.0657	0.2254	0.2388
	Foundation	0.0052	0.0320	0.0148	0.0128
Abutment B	Superstructure	0.0408	0.0354	0.2645	0.0161

Design Step 6.4 (continued)

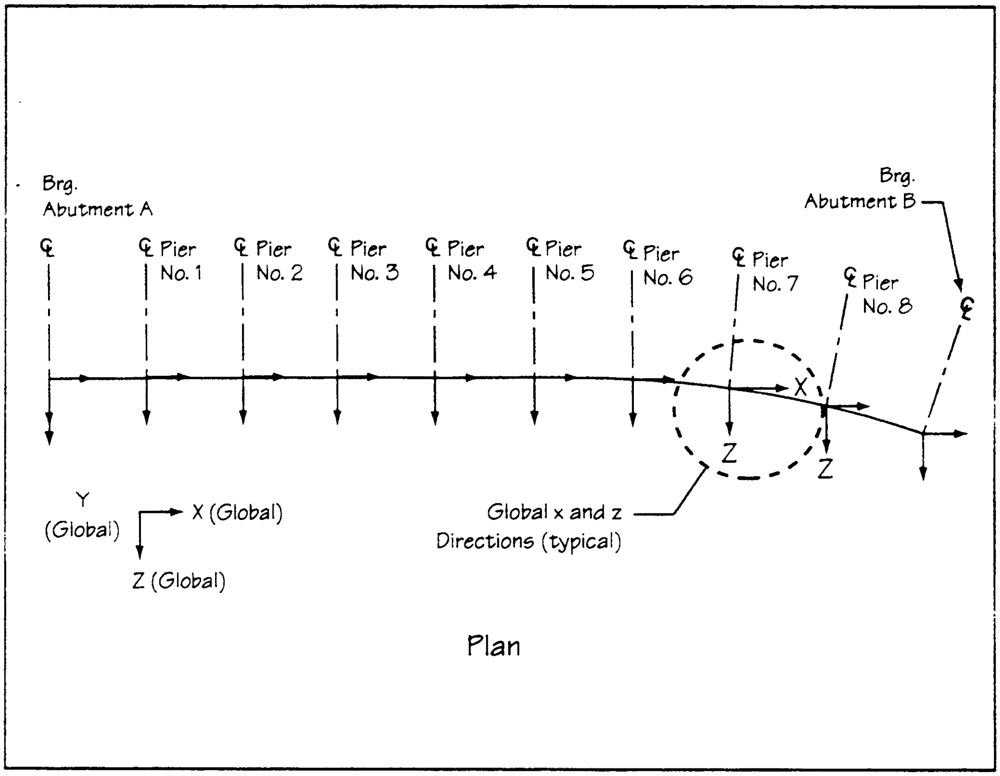


Figure 25 — Key to Displacement Directions

Design Step 6.5

Determine Forces and Displacements in Longitudinal Direction

[Division I-A, Article 4.5]

Perform the analysis for loading in the longitudinal direction.

The resulting forces and moments at the intermediate piers for the spectral analysis in the longitudinal direction are given in Table 6. (Refer to Figure 24 for force and moment directions.) The SAP90 input file for this analysis is EXAM5.

Displacements for both transverse and longitudinal analyses are given in Table 5. Figure 25 shows the global displacement directions.

Hand Check ✓ Check Longitudinal Column Shear Forces

As a check, compare the preliminary values computed in Design Step 1.3, Figure 4 with the column top longitudinal shears shown in Table 6.

For example, from Figure 4 in Design Step 1, the longitudinal column shear for Pier No. 7 is 922 kips. The longitudinal column top shear for Pier No. 7 from Table 6 is 827 kips.

Other shears may be similarly compared. All of the preliminary longitudinal column shears are higher than those from the modal analysis.

Design Step 6.5 (continued)

Table 6
Response for Longitudinal Direction (EQlong)

Support/Location		Forces and Moments - EQlong: Longitudinal Shear (kips)	Longitudinal Moment (kip-ft)	Transverse Shear (kips)	Transverse Moment (kip-ft)	Axial (kips)
Pier No. 1	Column Top	612	0	54	353	46
	Column Base	716	35,767	78	4,078	48
	Foundation	756	40,484	89	4,645	49
Pier No. 2	Column Top	205	0	69	450	33
	Column Base	331	21,950	113	7,955	35
	Foundation	431	24,143	128	8,744	36
Pier No. 3	Column Top	203	0	82	530	16
	Column Base	331	21,963	131	9,284	17
	Foundation	428	24,157	149	10,213	17
Pier No. 4	Column Top	0	0	73	467	10
	Column Base	254	15,171	122	8,524	10
	Foundation	359	16,812	140	9,398	10
Pier No. 5	Column Top	0	0	54	344	17
	Column Base	258	15,429	83	5,865	17
	Foundation	365	17,100	95	6,470	17
Pier No. 6	Column Top	311	0	56	370	35
	Column Base	413	28,134	88	6,230	36
	Foundation	502	30,862	99	6,857	36
Pier No. 7	Column Top	827	0	93	604	18
	Column Base	912	46,054	137	7,172	18
	Foundation	943	52,034	156	8,185	19
Pier No. 8	Column Top	0	0	113	734	30
	Column Base	308	12,606	178	9,178	30
	Foundation	368	14,999	206	10,517	30

DESIGN STEP 7 — DETERMINE DESIGN FORCES

INTRODUCTION

Under seismic loading, the bridge behaves much differently in the longitudinal direction than it does in the transverse direction. In the longitudinal direction, the bridge is free to slide at the abutments and Pier Nos. 4, 5, and 8. All the longitudinal seismic load is, therefore, taken by the pinned intermediate pier columns.

In the transverse direction, all of the intermediate piers and the abutments participate in resisting the load. The pier columns are very strong in the transverse direction relative to the longitudinal direction.

For this example, only one of the piers and its pile foundation will be designed. Pier No. 7 has been selected because it has the highest full elastic seismic longitudinal shears and moments, which is the weak axis direction for the column.

According to the Specification, design for plastic hinging forces need not be performed for SPC B. Division I-A, Article 6.2.2, presently allows the designer to use a value of R/2 for foundation design where R is the Response Modification Factor for the substructure (column or pier) to which the foundation is attached. At the same time, the Commentary of Division I-A, Article C6.2, warns the designer that forces larger than the R/2 design forces may be transferred to the foundation. This depends on the strength of the columns and piers, which should be investigated by the designer. A rational approach would be to compare magnitudes of the plastic hinging forces in the column or pier with the full elastic seismic forces to determine the final forces for design of the foundation. This would reduce the possibility of inelastic behavior from occurring in the foundation for the design seismic event. Gajer and Wagh (1994 and 1995) offer further discussion on this topic.

This example provides an opportunity to examine the forces used for design of foundations in SPC B.

Design Step 7.1 — Determine Nonseismic Forces

Design Step 7.1.1 — Determine Dead Load Forces

The dead load forces are summarized in Table 7 for all of the intermediate piers.

Design Step
7.1.1
(continued)

Table 7
Dead Load Forces

Support/Location		Forces and Moments - Dead Load				
		Longitudinal		Transverse		
		Shear (kips)	Moment (kip-ft)	Shear (kips)	Moment (kip-ft)	Axial (kips)
Pier No. 1	Column Top	12	0	0	0	1540
	Column Base	12	603	0	0	2758
	Foundation	12	679	0	0	3368
Pier No. 2	Column Top	19	0	0	0	1567
	Column Base	19	1360	0	0	3160
	Foundation	19	1484	0	0	3770
Pier No. 3	Column Top	7	0	1	9	1686
	Column Base	7	523	1	101	3280
	Foundation	7	570	1	109	3889
Pier No. 4	Column Top	0	0	3	750	1238
	Column Base	0	0	3	558	2832
	Foundation	0	0	3	541	3442
Pier No. 5	Column Top	0	0	3	502	1825
	Column Base	0	0	3	269	3419
	Foundation	0	0	3	248	4028
Pier No. 6	Column Top	9	0	0	347	1534
	Column Base	9	659	0	353	3128
	Foundation	9	719	0	354	3737
Pier No. 7	Column Top	9	0	1	356	1536
	Column Base	9	475	1	405	2755
	Foundation	9	535	1	411	3364
Pier No. 8	Column Top	0	0	2	508	1824
	Column Base	0	0	2	436	3043
	Foundation	0	0	2	426	3652

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.

A summary of the full elastic seismic forces for an earthquake at Pier No. 7 along each of the principal directions (both transverse and longitudinal) is shown in Table 8, which contains results from Tables 4 and 5.

Table 8
Full Elastic Seismic Forces

Support/Location		Full Elastic Seismic Forces and Moments: Longitudinal Shear (kips)	Longitudinal Moment (kip-ft)	Transverse Shear (kips)	Transverse Moment (kip-ft)	Axial (kips)
Pier No. 7 EQlong	Column Top	827	0	93	604	18
	Column Base	912	46,054	137	7,172	18
	Foundation	943	52,034	156	8,185	19
Pier No. 7 EQtrans	Column Top	108	0	391	2,524	5
	Column Base	125	6,138	539	28,593	5
	Foundation	132	6,936	602	32,492	5

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). See Table 9 for a summary of these forces.

Design Step 7.2.2 (continued)

The definition of LC1 and LC2 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 design for these forces combined with dead load if other load cases, such as stream flow, control the size of the substructure.

A sample calculation of the longitudinal column base moment for LC1 at Pier No. 7 is derived as follows.

$$M = (1.0 * M_{EQlong}) + (0.3 * M_{EQtrans})$$
$$M = (1.0 * 46{,}054) + (0.3 * 6138) = 47{,}895 \text{ k-ft}$$

All other forces in Table 9 are similarly calculated.

Table 9
Orthogonal Seismic Force Combinations
LC1 and LC2

LC1 = 1.0*EQlong + 0.3*EQtrans
LC2 = 0.3*EQlong + 1.0*EQtrans

		Pier No. 7 Forces and Moments				
		Longitudinal		Transverse		
	Load Case	Shear (kips)	Moment (kip-ft)	Shear (kips)	Moment (kip-ft)	Axial (kips)
Column Top	LC1	859	0	210	1,361	20
	LC2	356	0	419	2,705	10
Column Base	LC1	950	47,895	299	15,750	20
	LC2	399	19,954	580	30,745	10
Foundation	LC1	983	54,115	337	17,933	21
	LC2	415	22,546	649	34,948	11

Design Step 7.3

Determine Design Forces

For design of members and foundations, the design forces in Table 9 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. (Note that Table 9 values may require the inclusion of earth pressure, stream flow, and buoyancy forces as applicable.)

The seismic design forces use the R Factor in modifying the elastic seismic forces. Looking at the entire bridge as a system, the intent of the Specification is to prevent inelastic action from occurring in the foundation.

There is a distinction between design forces for a) structural members and connections and b) foundations.

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 seismic design forces for members and connections versus the seismic 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.

Group Load = 1.0 (D + B + SF + E + EQM) — Division I-A Eqn (6-1)

For this example, forces B, SF, and E are assumed zero, only D and EQM forces are combined. (Buoyancy forces, B, will be included during design of the foundation and added in Design Step 11.) The equation 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.

Design Step 7.3.1 (continued)

In this example, R reduces the full elastic seismic column forces, but does not reduce the full elastic seismic lateral shear force on the connection of the superstructure to the intermediate piers. 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 = 3.0 For forces in single-column piers
R = 1.0 For connection of column to superstructure

b) Calculate the Design Forces with EQM

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

Table 10 summarizes the design forces. The R value used for each force is given in the table.

For example, at Pier No. 7, the longitudinal column base moment using LC1 (Group LC1) is derived as follows.

$$M = (D + EQ/R)$$
$$M = (475 + 47{,}895 / 3) = 16{,}440 \text{ k-ft}$$

All other forces in Table 10 are similarly calculated.

The R Factors have been applied to all of the forces, including shear and axial forces, in accordance with the provisions of Division I-A, Article 6.2.1 for SPC B. This application of R Factors is unique to SPC B. In SPC C and D, the probable shear forces and axial forces corresponding to full plastic hinging (development of plastic mechanisms in the substructure) are used.

However, for SPC B, the designer should consider the implications of using the reduced design forces for shear and axial loads as presently allowed by the code. If full plastic hinging forces are not used for the shear design of the columns, then the possibility exists that the column is weaker in shear than in flexure and a brittle shear failure could occur. To avoid the possibility of this undesirable mode of failure, these options are available: 1) apply the method outlined for SPC C and D bridges in Division I-A, Article 7.2, or 2) use the full elastic seismic shear forces for design.

Design Step 7.3.1 (continued)

Note that using the full elastic seismic forces does not prevent the column from being shear critical, it simply means that the calculated design-level elastic shear could be sustained without a shear failure. For an earthquake larger than the design earthquake occurred, a brittle shear failure could conceivably still occur.

Table 10
Design Forces — Members and Connections

Group LC1 = 1.0*Dead Load + 1.0*LC1/R   R = 1.0 Column Top (Connection)
Group LC2 = 1.0*Dead Load + 1.0*LC2/R   R = 3.0 Column Base

		Pier No. 7 Design Forces and Moments				
		Longitudinal		Transverse		
Location	Load Case	Shear (kips)	Moment (kip-ft)	Shear (kips)	Moment (kip-ft)	Axial (kips)
Column Top R = 1.0	Group LC1	868	0	211	1,717	1,556
	Group LC2	365	0	420	3,061	1,546
Column Base R = 3.0	Group LC1	326	16,440	101	5,655	2,762
	Group LC2	142	7,126	194	10,653	2,758

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 bent column foundations.

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

For this example, forces B, SF, and E are assumed zero; only D and EQF forces are combined. The equation reduces to

Group Load = 1.0 (D + EQF)

Design Step
7.3.2
(continued)

Where, for foundation structures in SPC B, EQF = (LC1 or LC2 forces) divided by R/2, where R is the Response Modification Factor for the substructure (column or pier) to which the foundation is attached. In this example for the design of Pier No. 7, R = 3.

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

Effectively, for the design of the foundation, R = 3/2 = 1.5. Use this for calculating the design forces in the foundation.

b) Calculate the Foundation Design Forces with EQF

Table 11 summarizes the values of EQF design forces using R = 1.5.

For example, at Pier No. 7, the longitudinal foundation moment using LC1 is derived as follows.

M = (D + EQ/R)
M = (535 + 54,115 / 1.5) = 36,612 k-ft

All other forces in Table 11 are similarly calculated.

Table 11
Design Forces for Foundations with R = 1.5

Group LC1 = 1.0*Dead Load + 1.0*LC1/R   R = 1.5 Foundation
Group LC2 = 1.0*Dead Load + 1.0*LC2/R

		Pier No. 7 Foundation Design Forces and Moments				
		Longitudinal		Transverse		
Location	Load Case	Shear (kips)	Moment (kip-ft)	Shear (kips)	Moment (kip-ft)	Axial (kips)
Foundation	Group LC1	664	36,612	225	12,366	3,378
	Group LC2	286	15,566	434	23,709	3,371

Table 12 shows the design example foundation forces calculated using R = 1.0 for comparison with the values of Table 11.

Design Step 7.3.2 (continued)

Table 12 summarizes the values of EQF design forces using R = 1.0.

For example, at Pier No. 7, the longitudinal foundation moment using LC1 is derived as follows.

$$M = (D + EQ/R)$$
$$M = (535 + 54{,}115) = 54{,}650 \text{ k-ft}$$

All other forces in Table 12 are similarly calculated.

Table 12
Design Forces for Foundations with R = 1.0

Group LC1 = 1.0*Dead Load + 1.0*LC1/R
Group LC2 = 1.0*Dead Load + 1.0*LC2/R

R = 1.0 Foundation

		Pier No. 7 Foundation Design Forces and Moments				
		Longitudinal		Transverse		
Location	Load Case	Shear (kips)	Moment (kip-ft)	Shear (kips)	Moment (kip-ft)	Axial (kips)
Foundation	Group LC1	992	54,650	338	18,344	3,385
	Group LC2	424	23,081	650	35,359	3,375

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.

Because SPC B designs presently do not consider plastic hinging forces, the force combinations given in Design Step 7 are used directly. Design Step 8 is skipped for this example.

As discussed in the Introduction to Design Step 7, the forces used for design of the foundation will be examined. Consideration of the capacity of the pier column to transfer forces to the foundation is presented in Design Step 10.1.1(c) and foundation design with R = 1.0 forces is presented in Design Step 11.1.1.

DESIGN STEP 9 — DETERMINE DESIGN DISPLACEMENTS

Design Step 9.1 — Minimum Support Length

[Division I-A, Article 6.3.1]

The bearing seats supporting the expansion ends of the bridge at the abutments and Pier No. 4 must provide a minimum support length at least N inches wide. See Figure 26 for condition at Pier No. 4 and Figure 27 for condition at the abutments.

Pier No. 4 will have to accommodate support lengths for both Units 1 and 2.

$L_1 := 622 \cdot ft$ — Length of Unit 1

$H_1 := 70 \cdot ft$ — Average height of columns between expansion joints for Unit 1

$L_2 := 867 \cdot ft$ — Length of Unit 2

$H_2 := 60 \cdot ft$ — Average height of columns between expansion joints for Unit 2

$S := 0$ — Skew

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

$$N_1 := \left(8 \cdot in + 0.02 \cdot L_1 \cdot \frac{in}{ft} + 0.08 \cdot H_1 \cdot \frac{in}{ft}\right) \cdot \left(1 + 0.000125 \cdot S^2\right)$$

$N_1 = 2.17 \cdot ft$ — Support length required for Unit 1

$$N_2 := \left(8 \cdot in + 0.02 \cdot L_2 \cdot \frac{in}{ft} + 0.08 \cdot H_2 \cdot \frac{in}{ft}\right) \cdot \left(1 + 0.000125 \cdot S^2\right)$$

$N_2 = 2.51 \cdot ft$ — Support length required for Unit 2

Design Step 9.1 (continued)

As can be seen from Figure 26, with 6 inches provided between the end of superstructure and the centerline of the pier, the pier width of 6 feet 3 inches is sufficient to accommodate the support length requirements at Pier No. 4.

Abutment B will have to accommodate support length for Unit 2. The calculation for N is the same as performed above for N_2.

$N = 2.51 \cdot ft$

The support length provided at the abutments of 2 feet 11 inches is sufficient. The support length for Abutment A is the same as calculated for N and is less than for Abutment B.

Design Step 9.2

Design Displacements

The superstructure displacements from Design Step 6, Table 5 for the global X (longitudinal direction) are given as

at Abutment A, longitudinal displacement = 0.202 ft (= 2.4 inches)
at Pier No. 4, Unit 1 longitudinal displacement = 0.205 ft (= 2.5 inches)
at Pier No. 4, Unit 2 longitudinal displacement = 0.268 ft (= 3.2 inches)
at Abutment B, longitudinal displacement = 0.265 ft (= 3.2 inches)

These displacements are compatible with the gaps provided for longitudinal motion as shown in Figures 26 and 27.

Design Step 9.2 (continued)

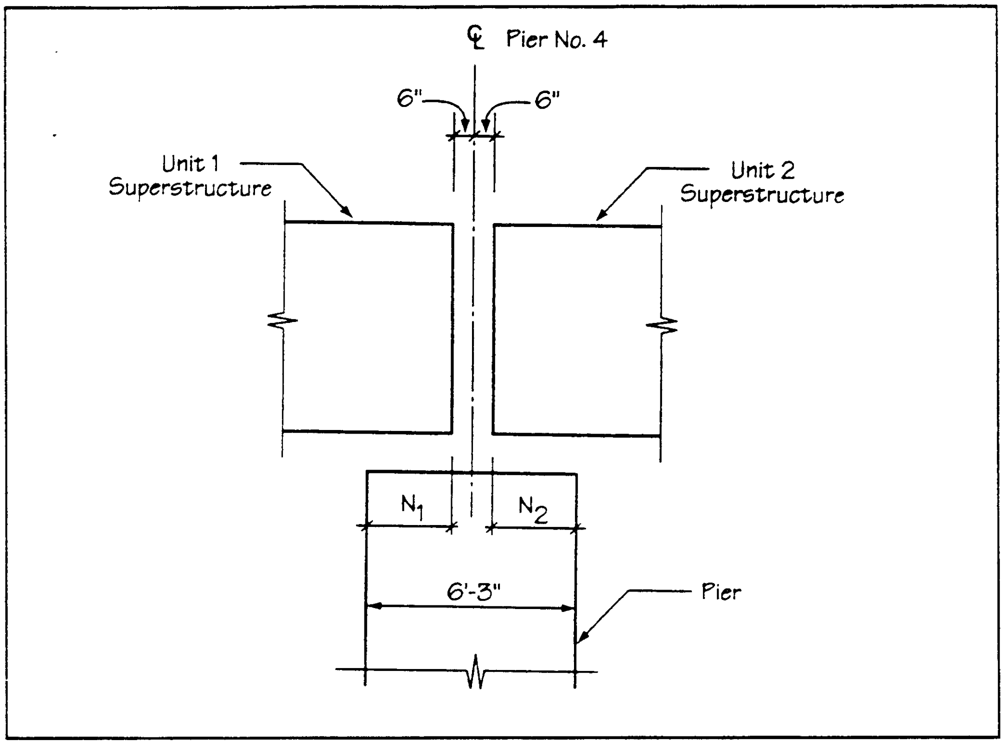


Figure 26 — Minimum Support Lengths at Pier No. 4

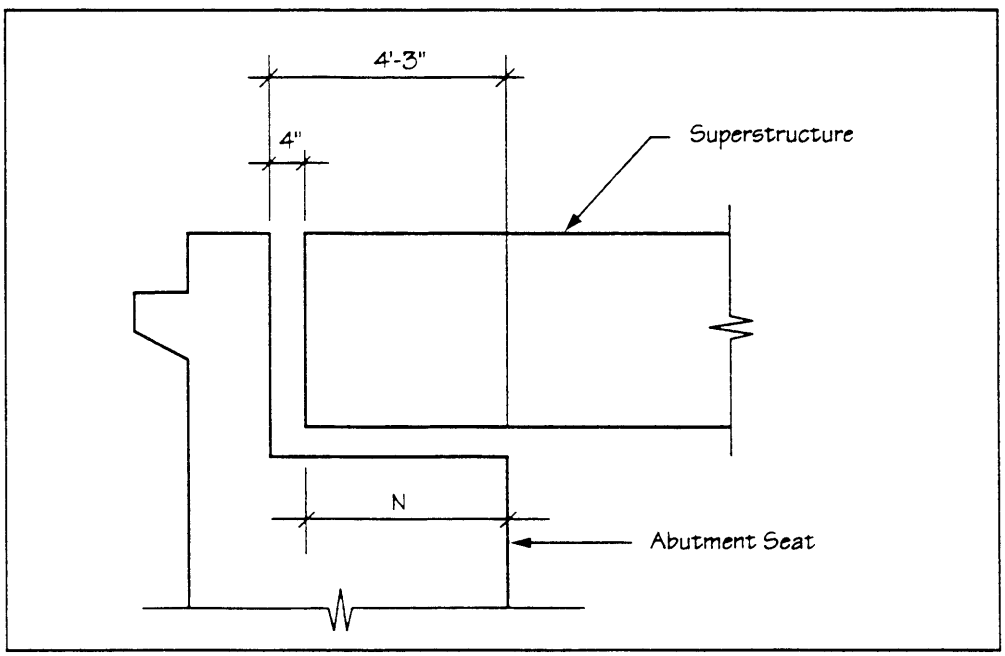


Figure 27 — Minimum Support Lengths at Abutments

DESIGN STEP 10

DESIGN STRUCTURAL COMPONENTS

This section concentrates on the critical components that resist the seismic forces. As discussed in Design Step 7, only structural components for Pier No. 7 will be designed for this example.

Design Step 10.1

Pier Design

Because this example features single-column piers, the design of the columns will be addressed in this section.

For essential bridges in SPC B, the designer may wish to consider the column design requirements for SPC C and D in Division I-A, Section 7 to enhance the column ductility capacity.

Basic column data, see Figure 28 for details.

$f_c := 4000 \cdot psi$ Concrete strength

$f_{yh} := 60 \cdot ksi$ Yield strength of hoop reinforcing

$b_{long} := 75 \cdot in$ Column base dimension in the longitudinal direction

$b_{trans} := 240 \cdot in$ Column base dimension in the transverse direction

$A_g := b_{long} \cdot b_{trans}$

$A_g = 18000 \cdot in^2$ Gross area of column base

Dimensions of concrete core at the column base, measured to the outside of the transverse tie reinforcement. Assume a 3-inch clear cover to #7 ties.

$h_{clong} := b_{long} - 2 \cdot (3 \cdot in)$

$h_{clong} = 69 \cdot in$ Longitudinal core dimension

Design Step 10.1 (continued)

$h_{ctrans} := b_{trans} - 2 \cdot (3 \cdot in)$

$h_{ctrans} = 234 \cdot in$ Transverse core dimension

$A_c := h_{clong} \cdot h_{ctrans}$

$A_c = 16146 \cdot in^2$ Area of concrete core

Design Step 10.1.1

Determine Longitudinal Reinforcement

a) Summary of Controlling Column Design Forces from Design Step 7.3, Table 10 **[Division I-A, Article 6.2.1]**

From LC1 at Pier No. 7 column base (with R = 3.0)

$P_{u1} := 2762 \cdot kip$

$M_{u1long} := 16440 \cdot kip \cdot ft$ $V_{u1long} := 326 \cdot kip$

$M_{u1trans} := 5655 \cdot kip \cdot ft$ $V_{u1trans} := 101 \cdot kip$

From LC2 at Pier No. 7 column base (with R = 3.0)

$P_{u2} := 2758 \cdot kip$

$M_{u2long} := 7126 \cdot kip \cdot ft$ $V_{u2long} := 142 \cdot kip$

$M_{u2trans} := 10653 \cdot kip \cdot ft$ $V_{u2trans} := 194 \cdot kip$

b) Minimum Column Reinforcing

Check the column with minimum longitudinal reinforcing of 1 percent of the gross concrete area per Division I, Article 8.18.1.1. Use #11 bars for the longitudinal reinforcement. Assume the reinforcement is spaced nearly equally around the perimeter as shown in Figure 28. The two load cases above are plotted on the interaction diagrams as shown in Figures 29 and 30. A ϕ factor of 0.7 is used for a tied column per Division 1, Article 8.16.1.2.2. Interaction diagrams shown are developed from PCACOL program, PCA (1993).

Design Step
10.1.1
(continued)

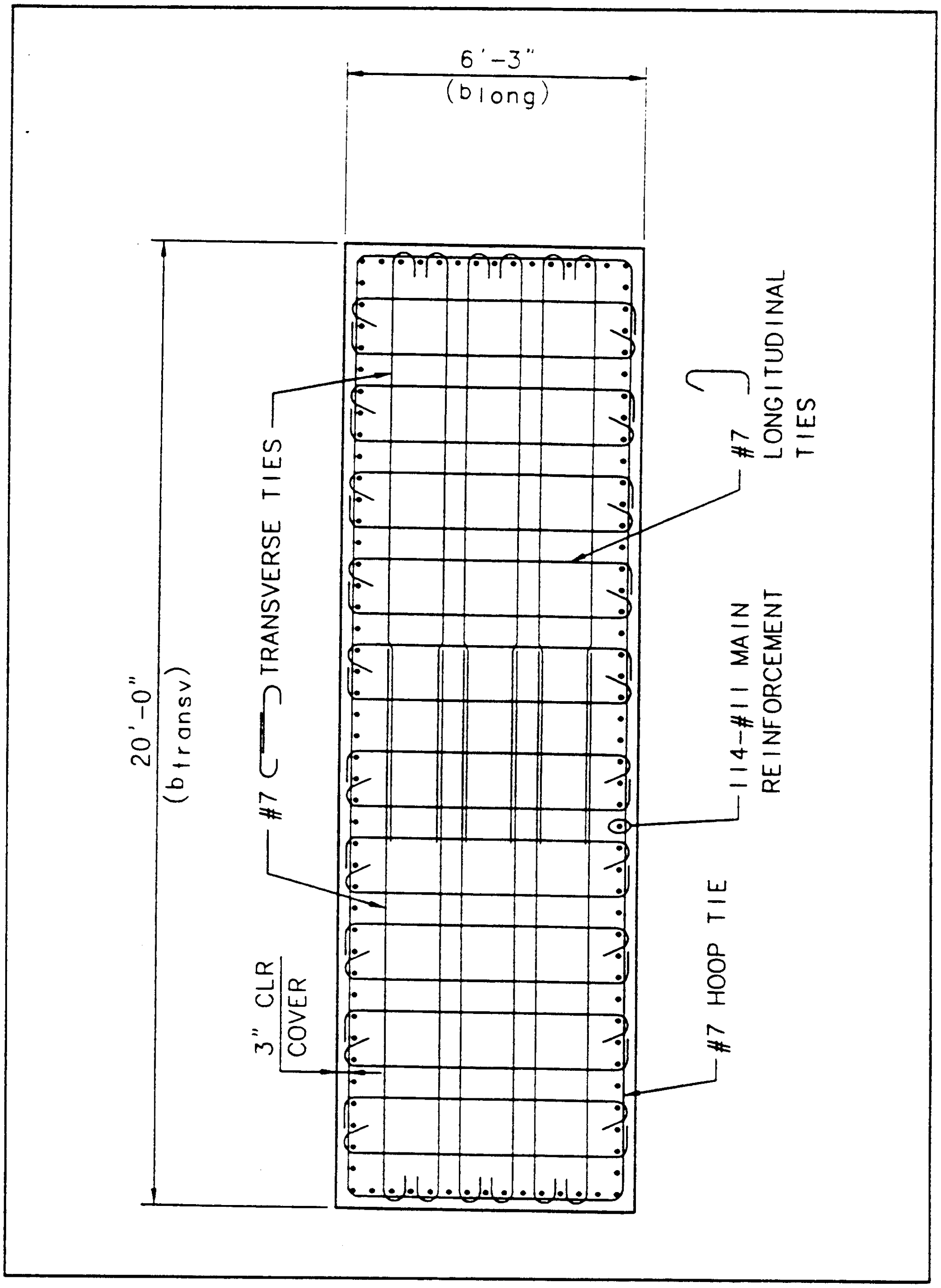


Figure 28 — Column Cross Section at Base

Design Step
10.1.1
(continued)

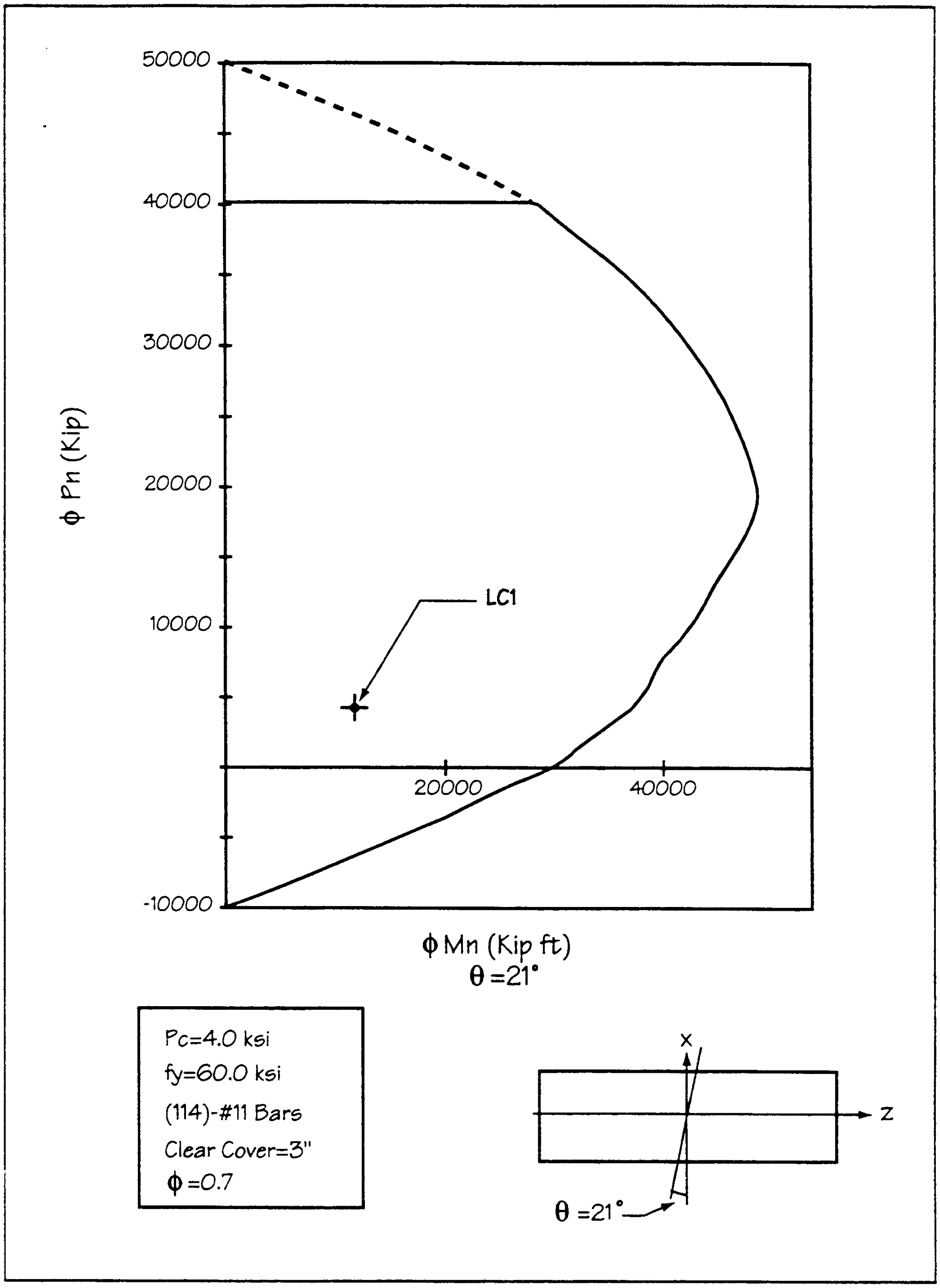


Figure 29 — Interaction Diagram for LC1

Design Step
10.1.1
(continued)

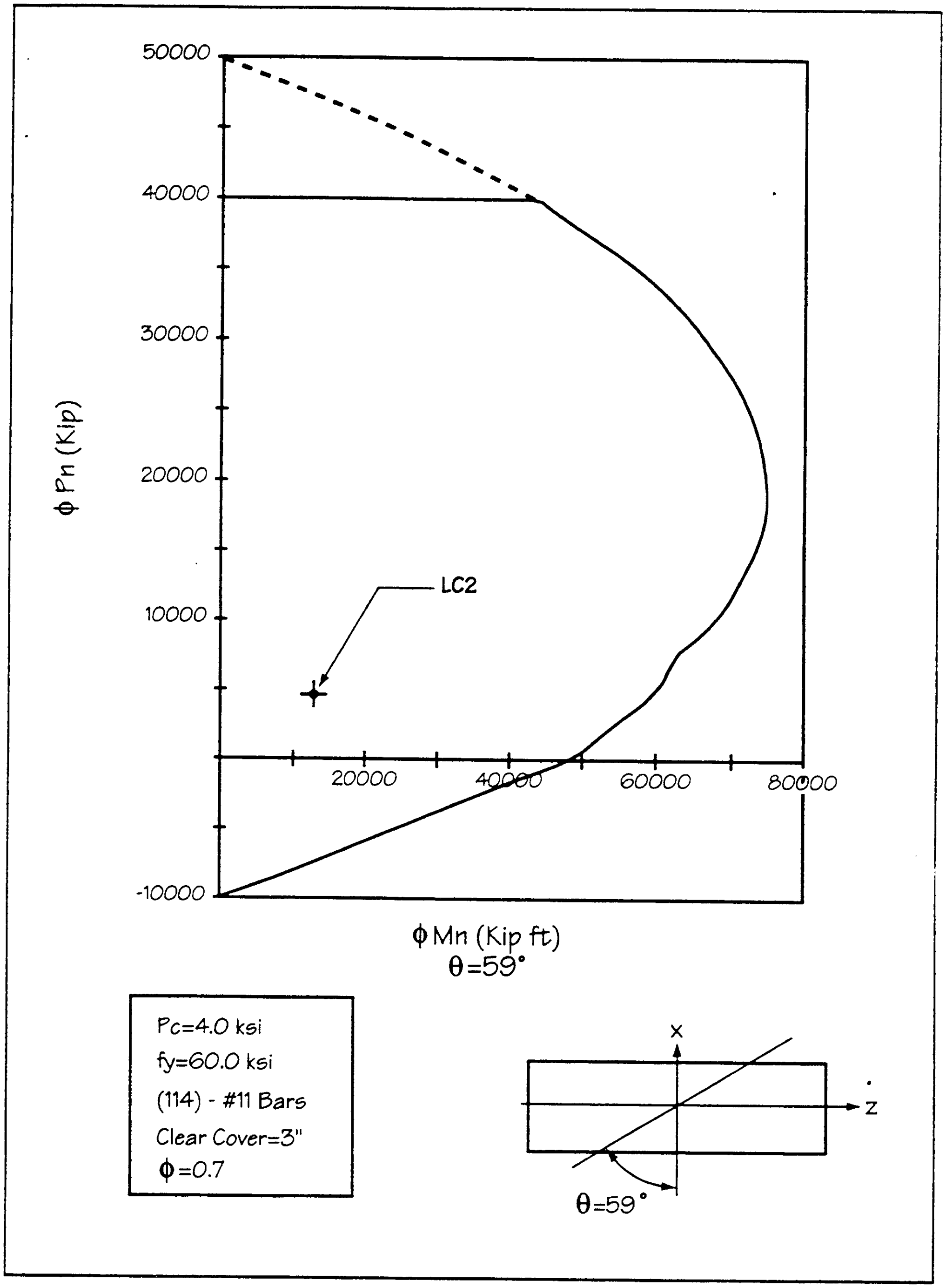


Figure 30 — Interaction Diagram for LC2

Design Step 10.1.1 (continued)

Because the forces for both load cases plot inside the capacity curve for the column with 1 percent steel, this reinforcement is sufficient.

$A_{st} := 0.01 \cdot A_g$

$A_{st} = 180 \cdot in^2$ Use 114 #11 bars ($A_{st} = 177.84\ in^2$)

Arrange the reinforcement with 15 #11 along each longitudinal face and 42 #11 along each transverse face as shown in Figure 28. Check the clear spacing between bars on both faces.

$d_{b11} := 1.55 \cdot in$ Outside diameter for #11 bar

$d_{b7} := 0.96 \cdot in$ Outside diameter for #7 bar

$$s_{clear} := \frac{b_{long} - 2 \cdot \left(3 \cdot in + d_{b7} + \frac{d_{b11}}{2}\right)}{14} - d_{b11}$$

$s_{clear} = 3.13 \cdot in$ Clear spacing between bars > 1.5*d_{b11} okay

$$s_{clear} := \frac{b_{trans} - 2 \cdot \left(3 \cdot in + d_{b7} + \frac{d_{b11}}{2}\right)}{43} - d_{b11}$$

$s_{clear} = 3.81 \cdot in$ Clear spacing between bars > 1.5*d_{b11} okay

c) Determine Column Overstrength Plastic Moment Capacities

As discussed in the Introduction to Design Step 7, presently design for plastic hinging forces need not be performed for SPC B. However, in order to properly evaluate the magnitude of the forces used for design of

Design Step
10.1.1
(continued)

the foundations in Design Step 11 to avoid the possibility of foundation understrength, the column's overstrength plastic moment capacites need to be computed.

Using the longitudinal reinforcement pattern from Design Step 10.1.1(b), a column interaction diagram was developed with $\phi = 1$. The diagram as shown in Figure 31 is plotted for biaxial nominal moment capacities of the column base for an axial load from LC1, of $P_{u1} = 2762$ kips. From the plot, the maximum column nominal capacities are determined for each of the principal directions.

$M_{nlong} := 37200 \cdot kip \cdot ft$ Nominal longitudinal moment capacity

$M_{ntrans} := 118800 \cdot kip \cdot ft$ Nominal transverse moment capacity

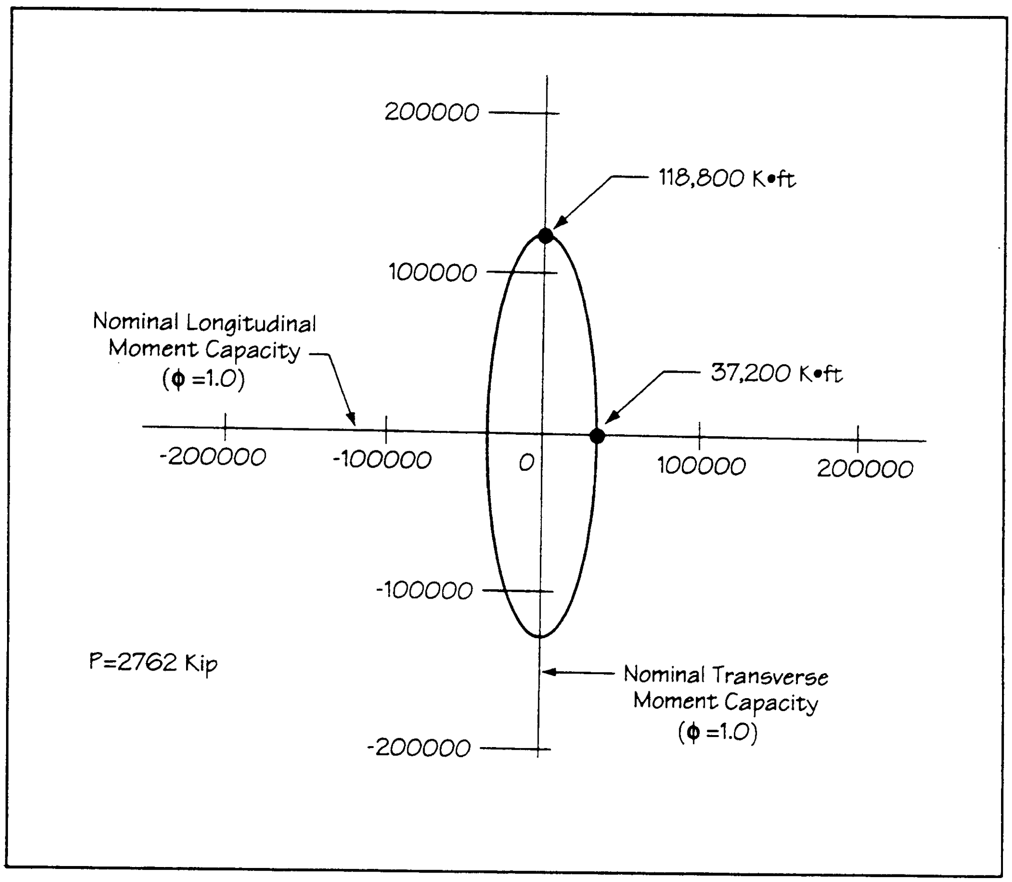


Figure 31 — Nominal Moment Capacities

Design Step 10.1.1 (continued)

Compute the plastic overstength moment capacity from the nominal capacities using $\phi = 1.3$ for reinforced concrete per Division I-A, Article 7.2.2(A).

$\phi := 1.3$ Strength reduction factor for plastic overstrength

Longitudinal moment

$M_{plong} := \phi \cdot M_{nlong}$

$M_{plong} = 48360 \cdot kip \cdot ft$ Longitudinal overstrength plastic moment capacity

Transverse moment

$M_{ptrans} := \phi \cdot M_{ntrans}$

$M_{ptrans} = 154440 \cdot kip \cdot ft$ Transverse overstrength plastic moment capacity

Compute the full-elastic seismic moments for Pier No. 7 column base using the dead load moments from Table 7 and the LC1 orthogonal seismic force combination moments from Table 9 with R = 1.0.

$M_{slong} := 475 \cdot kip \cdot ft + 47895 \cdot kip \cdot ft$

$M_{slong} = 48370 \cdot kip \cdot ft$ Full elastic seismic longitudinal moment for LC1

$M_{strans} := 405 \cdot kip \cdot ft + 15750 \cdot kip \cdot ft$

$M_{strans} = 16155 \cdot kip \cdot ft$ Full elastic seismic transverse moment for LC1

Design Step
10.1.1
(continued)

Compare the magnitudes of the full-elastic seismic moments with the overstrength plastic moment capacities for the column base. In the longitudinal direction, the overstrength capacity is nearly equal to the full elastic seismic moment. For the transverse direction, the overstrength capacity is more than 9.5 times the magnitude of the full-elastic seismic moment. This shows that full-elastic seismic forces can be carried by the column elastically without hinging of the column. Therefore, full-elastic seismic forces can be transmitted to the foundation. This is discusssed further in Design Step 11.

Design Step 10.1.2

Determine Typical Transverse Reinforcement
[Division I, Article 8.16.6]

From the previous step, it was shown that the column can carry the full elastic seismic forces without hinging, particularly in the transverse direction. As discussed in Design Step 7.3.1(b), if plastic hinging forces are not used for the design of the column, then the possibility that the column is weaker in shear than in flexure exists and a brittle shear failure could occur. This possibility can be avoided by using the full elastic seismic shear forces for design. Therefore, recompute the column shear forces used for design with R = 1.0 (versus R = 3.0 as used in Table 10).

$R := 3.0$ Response Modification Reduction Factor

Design shear values from Table 10 with $R = 3.0$

$V_{u1long} = 326 \cdot kip$ From LC1

$V_{u2trans} = 194 \cdot kip$ From LC2

Recompute design shear values with $R = 1.0$.

$V_{ulong} := V_{u1long} \cdot R$

$V_{ulong} = 978 \cdot kip$ Design shear for longitudinal direction

$V_{utrans} := V_{u2trans} \cdot R$

$V_{utrans} = 582 \cdot kip$ Design shear for transverse direction

The required shear strength of the Section V_n must be the following in each direction.

$\phi := 0.85$ Strength reduction factor for shear design
[Division I, Article 8.16.1.2.2]

Design Step 10.1.2 (continued)

Longitudinal direction

$$V_{nlong} := \frac{V_{ulong}}{\phi}$$

Division I
Eqn (8-46)

$$V_{nlong} = 1151 \cdot kip$$

Transverse direction

$$V_{ntrans} := \frac{V_{utrans}}{\phi}$$

Division I
Eqn (8-46)

$$V_{ntrans} = 685 \cdot kip$$

Compute the effective depth d in each direction assuming 3-inch cover to #7 transverse reinforcing.

$$d_c := 3 \cdot in + d_{b7} + \frac{d_{b11}}{2}$$

$$d_c = 5 \cdot in$$

$$d_{long} := b_{long} - d_c \qquad d_{long} = 70 \cdot in$$

$$d_{trans} := b_{trans} - d_c \qquad d_{trans} = 235 \cdot in$$

Shear strength provided by concrete for each direction.

$$V_{clong} := 2 \cdot \sqrt{f_c} \cdot b_{trans} \cdot d_{long}$$

Division I
Eqn (8-51)

$$V_{clong} := 2125 \cdot kip$$

Design Step
10.1.2
(continued)

$$V_{ctrans} := 2 \cdot \sqrt{f_c} \cdot b_{long} \cdot d_{trans}$$

Division I
Eqn (8-51)

$$V_{ctrans} := 2229\,kip$$

Since for both directions, $V_c > V_n$, provide minimum shear reinforcement per Division I, Article 8.19.1.

$s := 12 \cdot in$ Minimum spacing of ties

$$A_{vlong} := \frac{50 \cdot b_{trans} \cdot s}{f_{yh}}$$

$A_{vlong} := 2.4 \cdot in^2$ Provide 12 #4 in longitudinal direction

$$A_{vtrans} := \frac{50 \cdot b_{long} \cdot s}{f_{yh}}$$

$A_{vtrans} := 0.75 \cdot in^2$ Provide 4 #4 in transverse direction

The above determination of transverse shear reinforcing is for the typical section, not in the end regions that have special confinement requirements examined in the next Design Step. For this example, the above provisions for minimum shear reinforcing are not strictly applicable because V_u does not exceed one-half of the shear strength provided by the concrete, ϕV_c, for the transverse direction. Article 8.18.2.3 provisions for ties will also need to be satisfied.

Because the longitudinal reinforcement is #11 bars (not bundled), #4 ties are acceptable. The spacing of ties shall not exceed 12 inches. With the requirement that no longitudinal bar shall be more than 2 feet from a restrained bar on either side, check the minimum number of ties.

$No_{long} := \dfrac{b_{trans}}{2 \cdot ft}$ $No_{long} = 10$ < than 12 #4 ties computed above, use 12 ties

$$No_{trans} := \frac{b_{long}}{2 \cdot ft} \qquad No_{trans} = 3$$

< than 4 #4 ties computed above, use 4 ties

Design Step 10.1.3

Determine Transverse Reinforcement for Confinement [Division I-A, Article 6.6.2]

The core of the column must be confined by ties in the expected plastic hinge regions. For this example having wide single-column piers with pinned, sliding, or expansion bearings at the top, the column base is the only end region where plastic hinging is expected. Therefore, the column end region transverse confinement requirements will only apply to the bottom of the columns.

The total gross sectional area (A_{sh}) of rectangular hoop (stirrup) reinforcement for a rectangular column is the greater of that required by Equations (6-6) or (6-7) in Division I-A where

$A_c = 16146 \cdot in^2$ — Area of concrete core

$A_g = 18000 \cdot in^2$ — Gross area of column

$f_c = 4000 \cdot psi$ — Concrete strength

$f_{yh} = 60 \cdot ksi$ — Yield strength of hoop reinforcing

$a := 6 \cdot in$ — Maximum vertical spacing of hoops

$h_{clong} = 69 \cdot in$ — Longitudinal core dimension

$h_{ctrans} = 234 \cdot in$ — Transverse core dimension

For ties in the longitudinal direction

$$A_{sh} := 0.30 \cdot a \cdot h_{ctrans} \cdot \frac{f_c}{f_{yh}} \cdot \left(\frac{A_g}{A_c} - 1 \right) \qquad \text{Equation (6-6)}$$

$$A_{sh} = 3.22 \cdot in^2$$

Design Step 10.1.3 (continued)

$$A_{sh} := 0.12 \cdot a \cdot h_{ctrans} \cdot \frac{f_c}{f_{yh}}$$ Equation (6-7)

$A_{sh} = 11.23 \cdot in^2$ <==== Controls, provide 19 #7 bars
$A_{sh} = 11.40\ in^2$

For ties in the transverse direction

$$A_{sh} := 0.30 \cdot a \cdot h_{clong} \cdot \frac{f_c}{f_{yh}} \cdot \left(\frac{A_g}{A_c} - 1\right)$$ Equation (6-6)

$A_{sh} = 0.95 \cdot in^2$

$$A_{sh} := 0.12 \cdot a \cdot h_{clong} \cdot \frac{f_c}{f_{yh}}$$ Equation (6-7)

$A_{sh} = 3.31 \cdot in^2$ <==== Controls, provide 6 #7 bars
$A_{sh} = 3.60\ in^2$

Following the recommended tie details shown in the Commentary of Division I-A, if alternate bars of the main #11 reinforcement are tied, then the tie bar pattern would be as shown in Figure 28. This provides longitudinal tie reinforcement of 22 #7 bars and transverse tie reinforcement of 8 #7 bars.

For this example, plastic hinging confinement reinforcing was computed for both longitudinal and transverse directions (as shown in Figure 28) although the column will likely remain elastic in the transverse direction. The designer may, therefore, wish to use the tie requirements from Design Step 10.1.2 for the transverse ties throughout the column height.

Extent of column "End Region" at the bottom of the column is the maximum of the following three criteria per Division I-A, Article 6.6.2(B).

a. Maximum cross-sectional column dimension (at base) = 20 feet
b. $H_{clr}/6$ = 50 feet/6 = 8.33 feet 1/6 x clear height of column
c. 18 inches (minimum)

Design Step 10.1.3 (continued)

Because the column's strength in the transverse direction is many times in excess of the demand, it is unlikely that the column will hinge in this direction. Therefore, the extent of the transverse reinforcement for confinement provided will be controlled by criteria b because this is greater than the minimum cross-sectional column base dimension of 6 feet 3 inches.

Therefore, extend the transverse confinement reinforcing region 8 feet 4 inches up from the top of the pile cap. Extend the transverse confinement reinforcement one-half of this dimension (4 feet 2 inches) into the bottom connection (pile cap) per Division I-A, Article 6.6.2(B). See Figure 32.

The connection of the column to the pile cap is shown in Design Step 10.3.4.

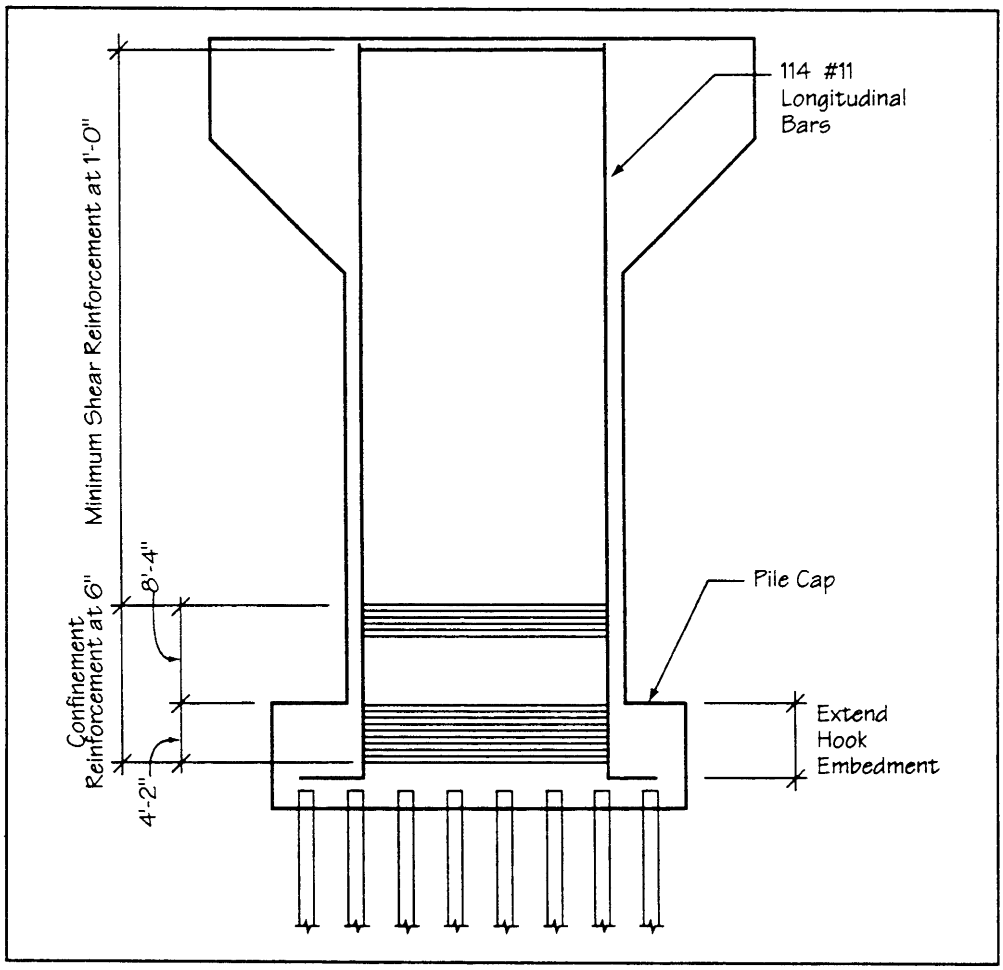


Figure 32 — Column Reinforcement Details

Design Step 10.2

Pier Cross Frame Design

Lateral forces from the acceleration of the mass of the superstructure are transferred to the substructure through steel cross frames to the bearings at the top of the pier. The lateral forces from the deck slab are transferred to the steel girders through shear studs along the top flanges of the girders and from the girders to the cross frames at the piers. For this structure, the cross frames at the piers consist of AASHTO M 270, Grade 50W steel k-bracing as shown in Figure 33.

Design forces are the column top connection shear forces from Design Step 7, Table 10. The transverse moments from Table 10 are not included in the design forces. These moments are not significant and have little effect on the anchor bolt forces. Additionally, the dead load forces are not included for determination of the most critical anchor bolt forces.

This example features the force transfer from the deck to the cross frame at Pier No. 7. Members and connections in the cross frame will not be designed in detail. Seismic forces transferred to the bearings will be computed and the anchor bolts will be checked in Design Step 10.3.3.

Design shear forces from Step 7, Table 10 for the top of Pier No. 7 (R = 1).

For LC1

$$V_{u1long} := 868 \cdot kip$$

$$V_{u1trans} := 211 \cdot kip$$

For LC2

$$V_{u2long} := 365 \cdot kip$$

$$V_{u2trans} := 420 \cdot kip$$

Because the bearings at Pier No. 7 are pinned, the longitudinal force will be transferred directly through the pin to the bearing anchor bolts. The transverse force will be distributed by the deck slab to the girders and k-brace cross frames to the bearing anchor bolts. The k-brace diagonals are at 45 degrees to the horizontal. Distribute the applied transverse shear from LC2, as the critical case for transverse loading, to the bracing as shown in Figure 34(a).

Design Step 10.2 (continued)

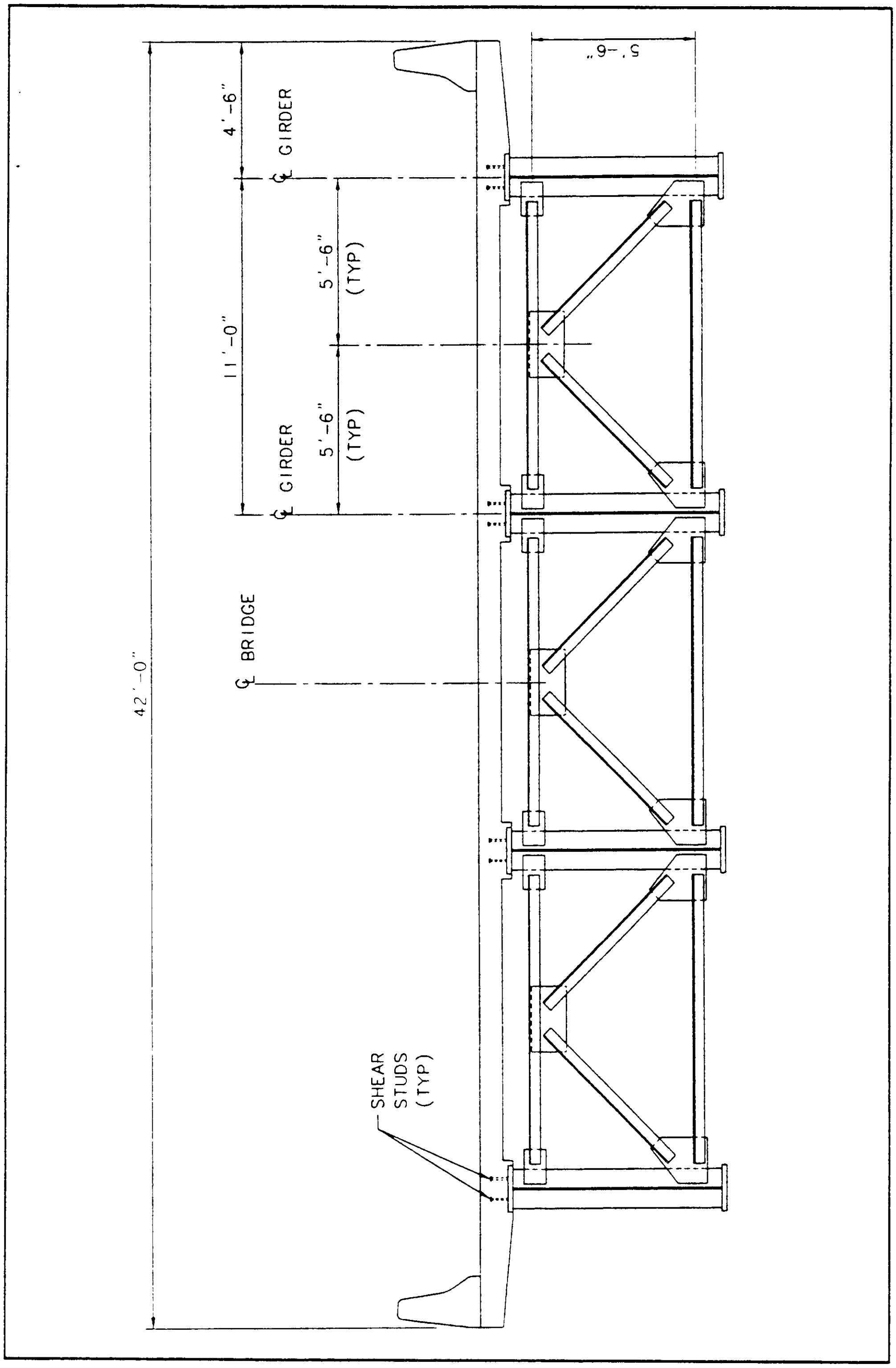


Figure 33 — Cross Frame at Pier

Design Step 10.2 (continued)

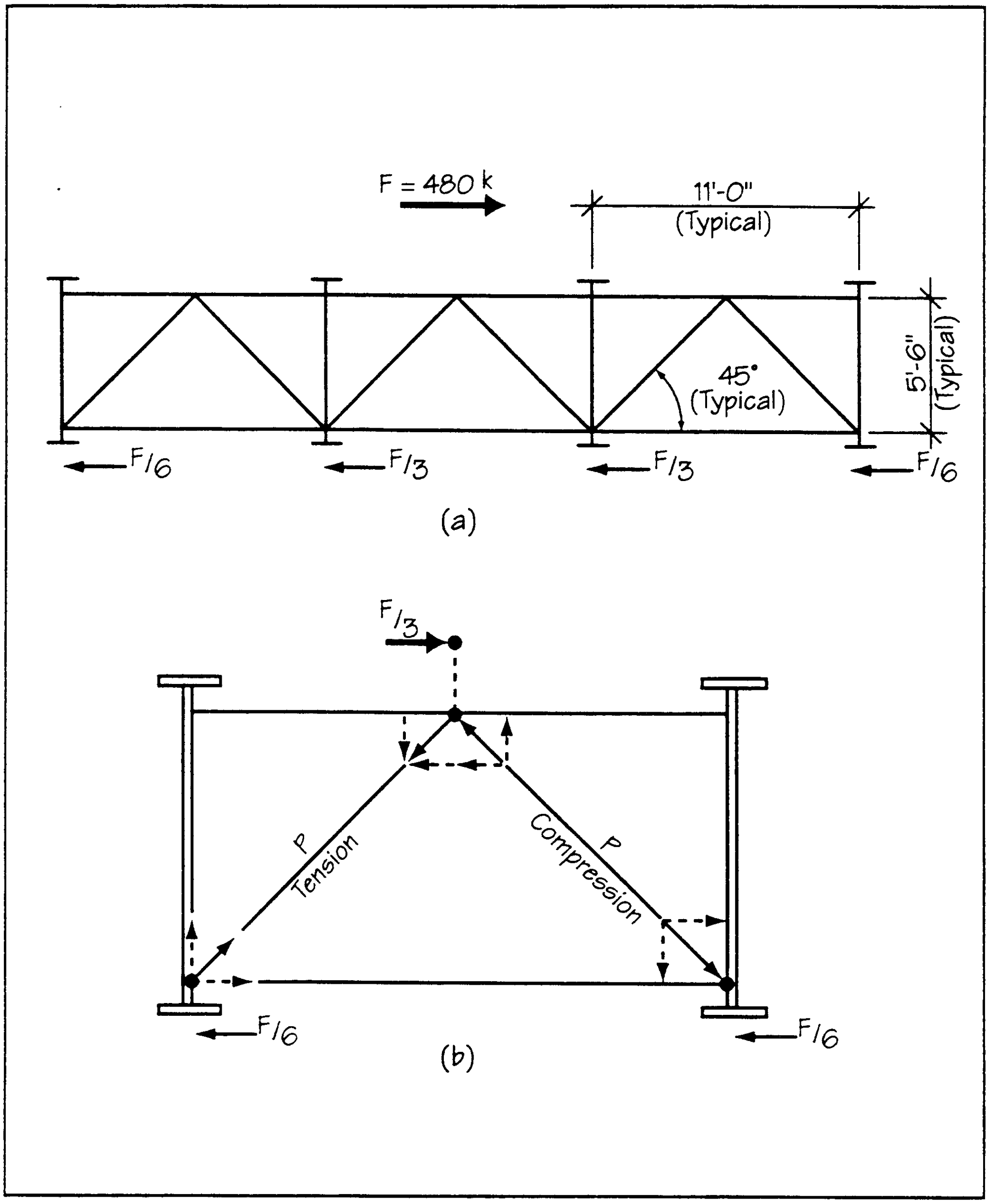


Figure 34 — Cross Frame

Design Step 10.2 (continued)

$F := V_{u2trans}$

$F = 420 \cdot kip$ Transverse force applied to entire crossframe

Assuming the force applied to the frame is resisted equally by each of three k-braces, the diagonal member forces are computed and shown in Figure 34(b). Axial tension and compression forces, P, are calculated as

$$P := \sqrt{2} \cdot \left(\frac{F}{6}\right)$$

$P = 99 \cdot kip$ Axial tension or compression force in diagonal member of k brace from LC2 (member not designed in this example)

Design Step 10.3

Connection Design

Design Step 10.3.1

Longitudinal Linkage

Not required for SPC B. Restrainers are not required because it has been determined in Design Step 9.1 [Division I-A, Article 6.3.1] that there is adequate seat width.

Design Step 10.3.2

Hold Downs

Not required for SPC B.

Design Step 10.3.3

Connection of Superstructure to Substructure

The connection of the superstructure to the substructure is provided by anchor bolts at the bearing locations. For this bridge, there are several different connection conditions to be designed because there are several different types of bearings.

- Connections at the pinned piers (Pier Nos. 1, 2, 3, 6, and 7)
- Connections at the sliding piers (Pier Nos. 5 and 8)
- Connection at the expansion locations (Pier No. 4 and abutments)

In this example, the connection of the superstructure via the pinned bearing anchor bolts at Pier No. 7 is designed. As discussed in Design Step 10.2, lateral forces from the acceleration of the mass of the superstructure are transferred to the substructure through bearing anchor bolts at the top of the pier. Design forces to the bearing anchor bolts are determined from the forces acting at the top of Pier No. 7.

From Design Step 10.2, in the longitudinal direction, the shear force is transferred directly through the bearing pin. Longitudinal shear forces at the top of the pier are

$V_{u1long} = 868 \cdot kip$ From LC1

$V_{u2long} = 365 \cdot kip$ From LC2

The transverse shear forces at the top of the pier are

$V_{u1trans} = 211 \cdot kip$ From LC1

$V_{u2trans} = 420 \cdot kip$ From LC2

Design transverse shears at the pin level of the bearings are F/3 for interior girders and F/6 for exterior girders as shown in Figure 34(a). Check an interior girder for the forces to the anchor bolts. Include the longitudinal shear for LC2 that is assumed to be distributed equally to all bearing locations. The bearing pin is taken as 1 foot above the level of the anchor bolts as shown in the bearing details of Figure 35.

$d := 1.0 \cdot ft$ Moment arm for shears applied to bearing anchor bolts

Design Step 10.3.3 (continued)

Six A449 anchor bolts are shown in Figure 35(b). The resultant forces for the bolt group for LC2 may be calculated as shown. This is conservative since the effect of any vertical dead load acting to reduce the bolt tension is neglected.

$b_{trans} := 28 \cdot in$ — Transverse dimension between outermost bolts of group

$b_{long} := 15 \cdot in$ — Longitudinal dimension between bolts of group

$N_{brg} := 4$ — Number of bearings

a) Compute the Anchor Bolt Forces for LC2

Moment applied to bolt group from LC2 transverse shear.

$$F := V_{u2trans}$$

$$M_{2trans} := \frac{F}{3} \cdot d$$

$$M_{2trans} = 1680 \cdot kip \cdot in$$

Bolt tension from M_{2trans} $\quad N_{bolts} := 2$

$$P_{2trans} := \left(\frac{M_{2trans}}{b_{trans}}\right) \cdot \frac{1}{N_{bolts}}$$

$$P_{2trans} = 30 \cdot kip$$

Moment applied to bolt group from LC2 longitudinal shear.

$$M_{2long} := \left(\frac{V_{u2long}}{N_{brg}}\right) \cdot d$$

$$M_{2long} = 1095 \cdot kip \cdot in$$

Design Step 10.3.3 (continued)

Bolt tension from M_{2long} $N_{bolts} := 3$

$$P_{2long} := \left(\frac{M_{2long}}{b_{long}}\right)\cdot\frac{1}{N_{bolts}}$$

$$P_{2long} = 24.3\cdot kip$$

Maximum bolt tension for LC2

$$P_{2tot} := P_{2trans} + P_{2long}$$

$$P_{2tot} = 54.3\cdot kip$$

Bolt tension at extreme corner

Check shear per bolt for LC2 $N_{bolts} := 6$

$$V_{2bolt} := \frac{\sqrt{\left(\frac{V_{u2long}}{N_{brg}}\right)^2 + \left(\frac{F}{3}\right)^2}}{N_{bolts}}$$

$$V_{2bolt} = 27.9\cdot kip$$

Shear per bolt neglecting friction from vertical dead load

b) Compute the Anchor Bolt Forces for LC1

Moment applied to bolt group from LC1 transverse shear.

$$F := V_{u1trans}$$

$$M_{1trans} := \frac{F}{3}\cdot d$$

$$M_{1trans} = 844\cdot kip\cdot in$$

Design Step 10.3.3 (continued)

Bolt tension from M_{1trans} $N_{bolts} := 2$

$$P_{1trans} := \left(\frac{M_{1trans}}{b_{trans}}\right) \cdot \frac{1}{N_{bolts}}$$

$$P_{1trans} = 15.1 \cdot kip$$

Moment applied to bolt group from LC1 longitudinal shear.

$$M_{1long} := \left(\frac{V_{u1long}}{N_{brg}}\right) \cdot d$$

$$M_{1long} = 2604 \cdot kip \cdot in$$

Bolt tension from M_{long} $N_{bolts} := 3$

$$P_{1long} := \left(\frac{M_{1long}}{b_{long}}\right) \cdot \frac{1}{N_{bolts}}$$

$$P_{1long} = 57.9 \cdot kip$$

Maximum bolt tension for LC1

$$P_{1tot} := P_{1trans} + P_{1long}$$

$P_{1tot} = 72.9 \cdot kip$ Bolt tension at extreme corner

Design Step 10.3.3 (continued)

Check shear per bolt for LC1 $\qquad N_{bolts} := 6$

$$V_{1bolt} := \frac{\sqrt{\left(\frac{V_{u1long}}{N_{brg}}\right)^2 + \left(\frac{F}{3}\right)^2}}{N_{bolts}}$$

$V_{1bolt} = 38 \cdot kip$ — Shear per bolt neglecting friction from vertical dead load

c) Summary of Anchor Bolt Forces

	Bolt tension	Bolt shear
For LC2,	$P_{2tot} = 54.3 \cdot kip$	$V_{2bolt} = 27.9 \cdot kip$
For LC1,	$P_{1tot} = 72.9 \cdot kip$	$V_{1bolt} = 38 \cdot kip$

d) Design Anchor Bolts

For high-strength bolts (A325 or A449), assuming they are adequately anchored for full shear and tension yield strength in the bolt itself, the design strength ϕF for shear and applied static tension is taken from Division I, Table 10.56A.

$\phi F_v := 36 \cdot ksi$ — Shear strength

$\phi F_t := 67 \cdot ksi$ — Tension strength

$A_b := 1.485 \cdot in^2$ — Area of bolt corresponding to nominal 1-3/8-inch diameter

Shear strength

$$\phi R_v := \phi F_v \cdot A_b$$

$\phi R_v = 53.5 \cdot kip$ — Greater than maximum shear from LC1, (= 38.0 kip) say okay with 1-3/8-inch bolts

Design Step 10.3.3 (continued)

Tensile strength

$\phi R_t := \left(\phi F_t \cdot A_b\right) \cdot 0.875$ — Reduce Division I, Table 10.56A tensile strength for diameters greater than 1 inch (multiply by 0.875)

$\phi R_t = 87.1 \cdot kip$ — Greater than maximum tension from LC1, (= 72.9 kip) say okay with 1-3/8-inch bolts

Anchor bolts should be anchored into the pier concrete to sufficiently develop the full yield strength of the bolt in tension. To accurately place the anchor bolts for the bearing assembly, the bolts are commonly grouted in standard pipe sections capped with a plate at the bolt head as shown in Figure 35(c). Alternately, the anchor bolts may be threaded at both ends and have a plate with a welded nut tack in lieu of a headed bolt.

Design Step
10.3.3
(continued)

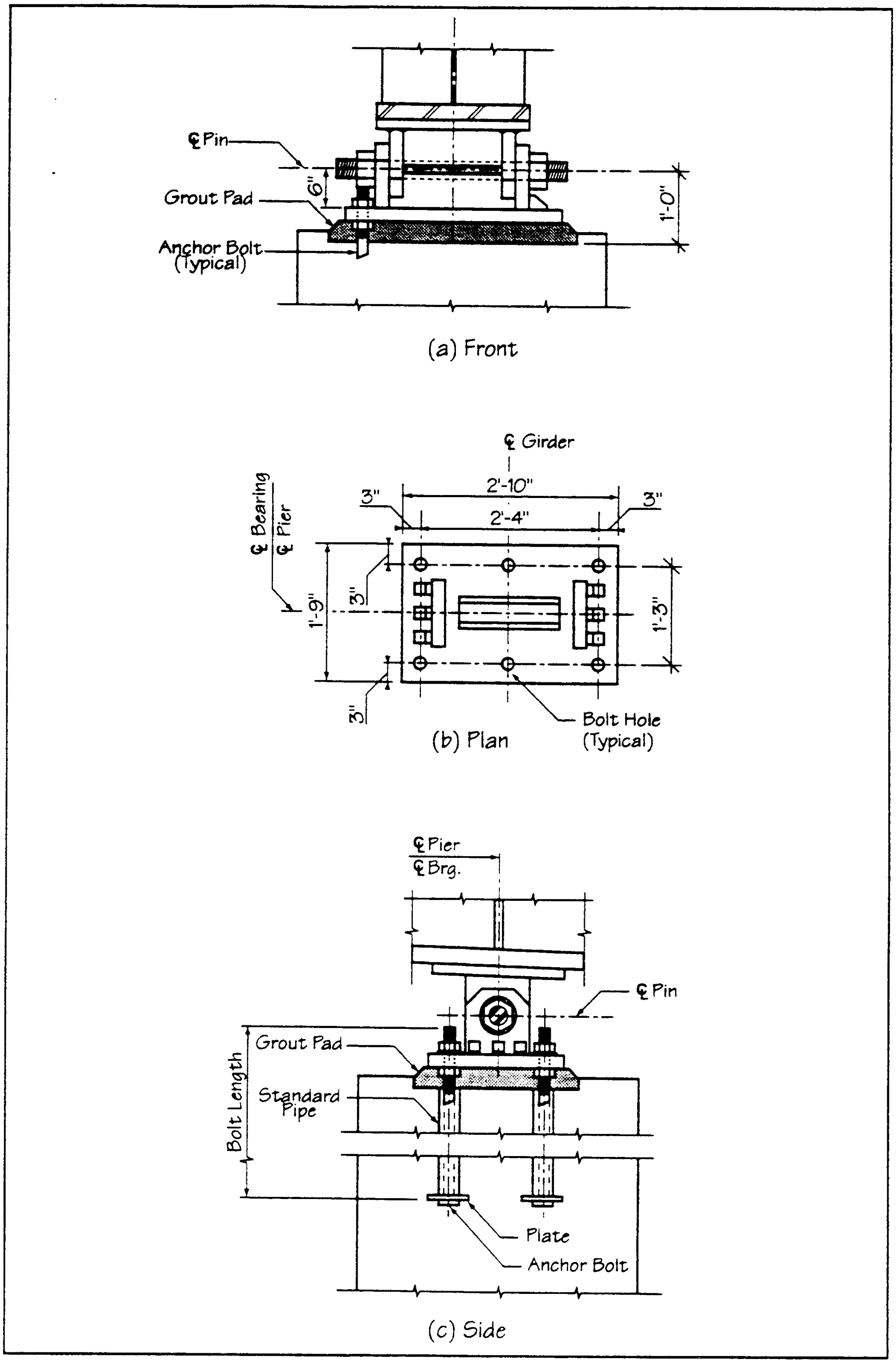


Figure 35 — Bearing Details

Design Step 10.3.4

Connection of Column to Pile Cap

See Figure 32 for detail at the base of the column. Check the development length for the #11 main reinforcement with 90 degree hooks as shown per Division I, Article 8.29.

Basic data to check development

$f_c := 4000$ Concrete compressive strength (psi)

$A_{b11} := 1.56 \cdot in^2$ Area of #11 bar

$d_b := 1.410 \cdot in$ Diameter of #11 bar

$f_y := 60 \cdot ksi$ Yield strength of reinforcing

Basic development for standard hooks

$$L_{hb} := 1200 \cdot \frac{d_b}{\sqrt{f_c}} \qquad L_{hb} = 26.8 \cdot in$$

This basic length (above) could be modified (multiplied by a factor of 0.7) per Division I, Article 8.29.3.2 for bar cover greater than 2.5 inches, however, the requirements of Division I-A, Article 6.6.2(B) to extend the transverse confinement reinforcing, as shown in Figure 32, will control the length of the #11 hook embedment.

To ensure proper force transfer ability and to simplify the construction, the #11 hooks should be extended to the bottom mat of reinforcing, just above the level of pile embedment.

DESIGN STEP 11

DESIGN FOUNDATIONS

[Division I-A, Article 6.4.2]

In this design example, only the foundation at Pier No. 7 will be designed.

Design Step 11.1

Design Pile Group

The pile foundation consists of driven plumb steel H-piles (HP 12 x 84) AASHTO M 270, Grade 36, with a cast-in-place concrete pile cap. The following ultimate pile capacities are assumed to apply to this bridge site. Actual soil properties and pile capacities would be specified in the geotechnical report. The pile ultimate compression capacity under seismic conditions may be determined by a number of different methods and is usually a combination of frictional and tip resistance. The magnitude of the ultimate compression capacity often exceeds the pile capacity value based upon allowable stresses from Division I, Article 4.5.7.3.

If buoyancy effects should be included, a design water surface elevation would be assumed. This would likely come from a hydrological report or from the geotechnical report. For this example, geotechnical information is provided in Appendix A.

$Pile_{ultc} := 370 \cdot kip$ — Ultimate pile capacity in compression (per pile)

$Pile_{ultt} := -100 \cdot kip$ — Ultimate pile capacity in tension (per pile)

Design Step 11.1.1

Determine Axial Forces in Piles

The forces for design of the pile group are taken from Design Step 7.3.2, Table 11. Initially, the pile layout will be assumed the same as used for development of the foundation spring stiffness in Design Step 6.2. This pile layout was based upon preliminary pile axial capacities, although the steps are not presented here. The layout is shown on Figure 36.

Design Step 11.1.1 (continued)

Foundation design is controlled by LC1. The following forces are applied at the bottom of the pile cap and do not include earth pressure, stream flow pressure, and buoyancy effects. (R = 1.5)

$P_u := 3378 \cdot kip$

$M_{ulong} := 36612 \cdot kip \cdot ft \qquad V_{ulong} := 664 \cdot kip$

$M_{utrans} := 12366 \cdot kip \cdot ft \qquad V_{utrans} := 225 \cdot kip$

For consideration of overturning on the pile group, the minimum axial load will control. Because the effects of buoyancy reduce the axial load, buoyancy should be included. To include the buoyancy, subtract the weight of water displaced by the pile cap, pier column, and overlying soil for the appropriate depth of water. In this example, the design water surface elevation at Pier No. 7 does not produce buoyancy effects for design of the foundation.

The method used here for computing the axial loads to the pile group from the applied moments is similar to the method used in Design Step 6.2 to determine the rocking rotational resistance of the pile group. This method is a simple static analysis described in a number of foundation engineering references such as Peck, Hanson, and Thornburn (1974). Each pile is assumed to take an equal share of the applied vertical load (P/A) and the vertical load in the pile caused by the applied moment is proportional to the distance from the center of the resisting group (Mc/I) as shown in Figure 37. This is the same basic formula for pressure beneath a soil supported footing subjected simultaneously to direct load and moment. A basic assumption for this approach is that the pile cap is considered rigid.

a) Compute Pile Axial Loads

The assumed pile layout from Design Step 6.2 is shown in Figure 36. Calculate the axial forces in the piles for the axial load and moments in each direction and then superimpose for final pile loads.

Design Step
11.1.1
(continued)

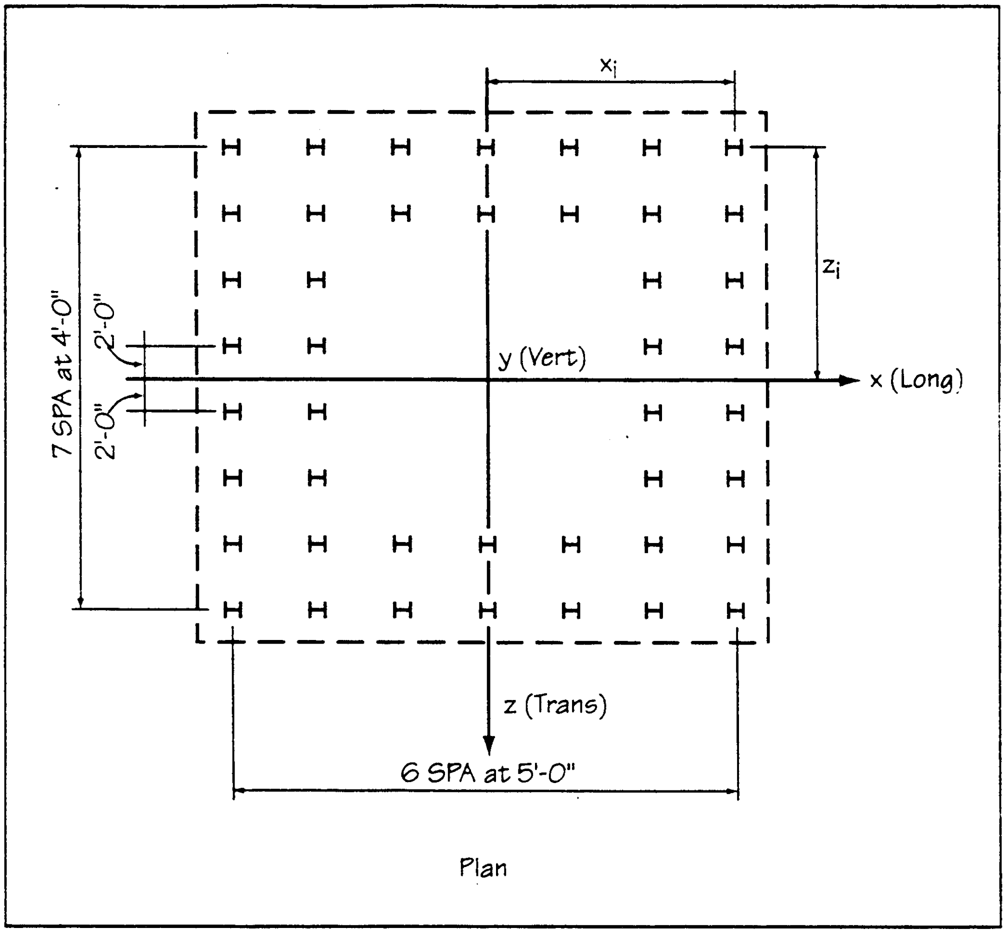


Figure 36 — Pile Layout Plan

Design Step
11.1.1
(continued)

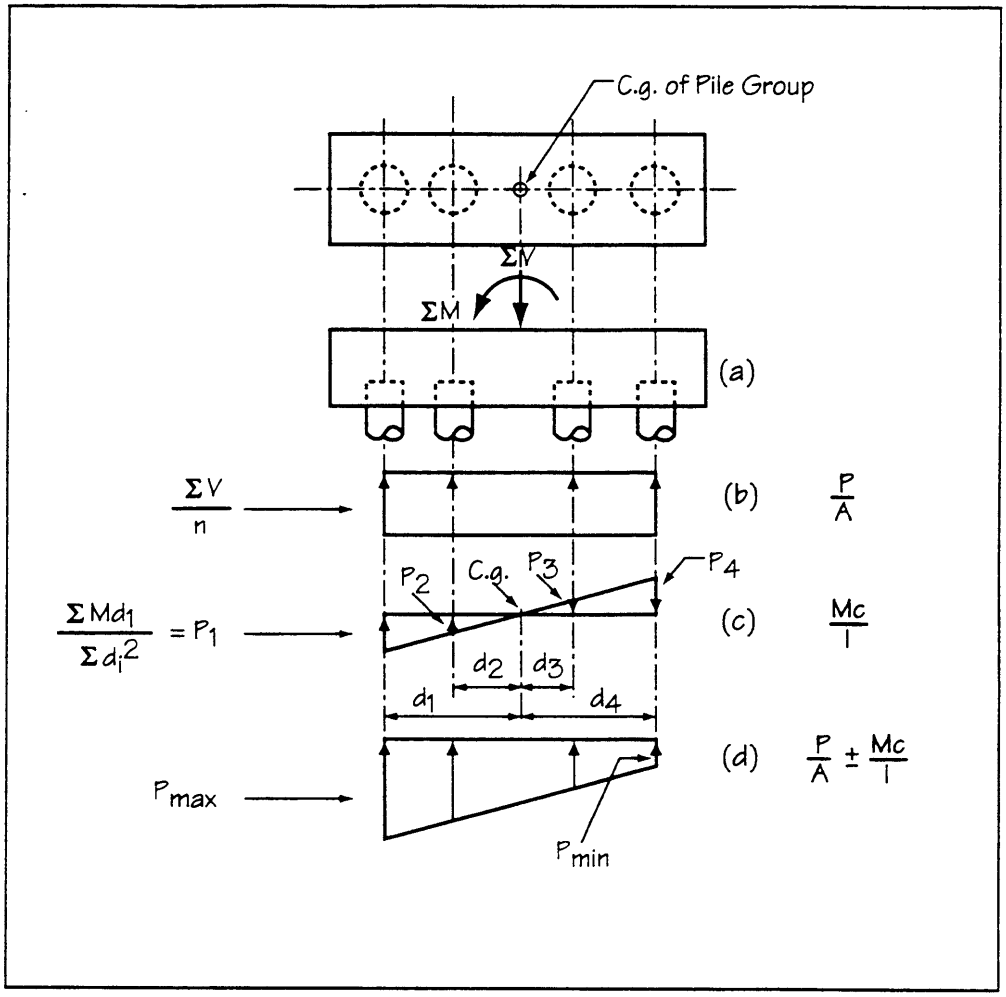


Figure 37 — Computation of Pile Loads

Design Step 11.1.1 (continued)

Assume that all of the piles in the group are loaded equally from the axial force. (The pile cap is assumed rigid.)

$P_u = 3378 \cdot kip$

$N_p := 44$ Number of piles in group

$$P_v := \frac{P_u}{N_p}$$

$P_v = 77 \cdot kip$ Axial load contribution, per pile

Consider the forces in the longitudinal direction first. Compute the pile loads from the applied axial load and longitudinal moment.

$M_{ulong} = 36612 \cdot kip \cdot ft$

Distance to row from group center	Number of piles per row
$x_1 := 15 \cdot ft$	$N_{x1} := 8$
$x_2 := 10 \cdot ft$	$N_{x2} := 8$
$x_3 := 5 \cdot ft$	$N_{x3} := 4$
$x_4 := 0 \cdot ft$	$N_{x4} := 4$
$x_5 := -5 \cdot ft$	$N_{x5} := 4$
$x_6 := -10 \cdot ft$	$N_{x6} := 8$
$x_7 := -15 \cdot ft$	$N_{x7} := 8$

$$x_{sum} := \sum_{i=1}^{7} (x_i)^2 \qquad x_{sum} = 700 \cdot ft^2$$

Design Step 11.1.1 (continued)

Load per pile without transverse moment (negative loads are tension)

$$P_{x1} := \left(\frac{M_{ulong} \cdot x_1}{x_{sum} \cdot N_{x1}}\right) + P_v \qquad P_{x1} = 175 \cdot kip \qquad \text{Row 1}$$

$$P_{x2} := \left(\frac{M_{ulong} \cdot x_2}{x_{sum} \cdot N_{x2}}\right) + P_v \qquad P_{x2} = 142 \cdot kip \qquad \text{Row 2}$$

$$P_{x3} := \left(\frac{M_{ulong} \cdot x_3}{x_{sum} \cdot N_{x3}}\right) + P_v \qquad P_{x3} = 142 \cdot kip \qquad \text{Row 3}$$

$$P_{x4} := \left(\frac{M_{ulong} \cdot x_4}{x_{sum} \cdot N_{x4}}\right) + P_v \qquad P_{x4} = 77 \cdot kip \qquad \text{Row 4}$$

$$P_{x5} := \left(\frac{M_{ulong} \cdot x_5}{x_{sum} \cdot N_{x5}}\right) + P_v \qquad P_{x5} = 11 \cdot kip \qquad \text{Row 5}$$

$$P_{x6} := \left(\frac{M_{ulong} \cdot x_6}{x_{sum} \cdot N_{x6}}\right) + P_v \qquad P_{x6} = 11 \cdot kip \qquad \text{Row 6}$$

$$P_{x7} := \left(\frac{M_{ulong} \cdot x_7}{x_{sum} \cdot N_{x7}}\right) + P_v \qquad P_{x7} = -21 \cdot kip \qquad \text{Row 7}$$

Now compute the pile loads from the applied transverse moment, and add these to the previously calculated loads.

$$M_{utrans} = 12366 \cdot kip \cdot ft$$

Distance to row <u>from group center</u>	Number of piles <u>per row</u>
$z_1 := 14 \cdot ft$	$N_{z1} := 7$
$z_2 := 10 \cdot ft$	$N_{z2} := 7$

Design Step 11.1.1 (continued)

$z_3 := 6 \cdot ft$ $\qquad N_{z3} := 4$

$z_4 := 2 \cdot ft$ $\qquad N_{z4} := 4$

$z_5 := -2 \cdot ft$ $\qquad N_{z5} := 4$

$z_6 := -6 \cdot ft$ $\qquad N_{z6} := 4$

$z_7 := -10 \cdot ft$ $\qquad N_{z7} := 7$

$z_8 := -14 \cdot ft$ $\qquad N_{z8} := 7$

$$z_{sum} := \sum_{i=1}^{8} \left(z_i\right)^2 \qquad z_{sum} = 672 \cdot ft^2$$

Loads from transverse moment only (negative loads are tension)

$$P_{z1} := \left(\frac{M_{utrans} \cdot z_1}{z_{sum} \cdot N_{z1}}\right) \qquad P_{z1} = 37 \cdot kip \qquad \text{Row 1}$$

$$P_{z2} := \left(\frac{M_{utrans} \cdot z_2}{z_{sum} \cdot N_{z2}}\right) \qquad P_{z2} = 26 \cdot kip \qquad \text{Row 2}$$

$$P_{z3} := \left(\frac{M_{utrans} \cdot z_3}{z_{sum} \cdot N_{z3}}\right) \qquad P_{z3} = 28 \cdot kip \qquad \text{Row 3}$$

$$P_{z4} := \left(\frac{M_{utrans} \cdot z_4}{z_{sum} \cdot N_{z4}}\right) \qquad P_{z4} = 9 \cdot kip \qquad \text{Row 4}$$

Design Step 11.1.1 (continued)

$$P_{z5} := \left(\frac{M_{utrans} \cdot z_5}{z_{sum} \cdot N_{z5}}\right) \qquad P_{z5} = -9 \cdot kip \qquad \text{Row 5}$$

$$P_{z6} := \left(\frac{M_{utrans} \cdot z_6}{z_{sum} \cdot N_{z6}}\right) \qquad P_{z6} = -28 \cdot kip \qquad \text{Row 6}$$

$$P_{z7} := \left(\frac{M_{utrans} \cdot z_7}{z_{sum} \cdot N_{z7}}\right) \qquad P_{z7} = -26 \cdot kip \qquad \text{Row 7}$$

$$P_{z8} := \left(\frac{M_{utrans} \cdot z_8}{z_{sum} \cdot N_{z8}}\right) \qquad P_{z8} = -37 \cdot kip \qquad \text{Row 8}$$

Check the outermost corner piles for maximum compression and tension cases.

$$P_{max} := P_{x1} + P_{z1}$$

$P_{max} = 212 \cdot kip$ Less than $Pile_{ultc}$ (= 370 kips) okay

$$P_{min} := P_{x7} + P_{z8}$$

$P_{min} = -58 \cdot kip$ Less than $Pile_{ultt}$ (= - 100 kips) okay

Resulting axial loads for all other piles in the group may be similarly calculated. The number of piles subjected to a net tension (uplift) is limited to one-half of the pile group per Division I-A, Article 6.4.2(B). Calculation of all of the pile loads results in 10 piles of the 44-pile group having tension for this load case is shown in Figure 38. (Negative loads are tension.) The design foundation forces associated with these pile loads were computed with an effective R Factor of 1.5.

The computed pile axial forces in Figure 38 show that there is significant reserve capacity for the ultimate compression limit and some uplift reserve capacity for the pile arrangement shown. Also, the number of tension piles

Design Step
11.1.1
(continued)

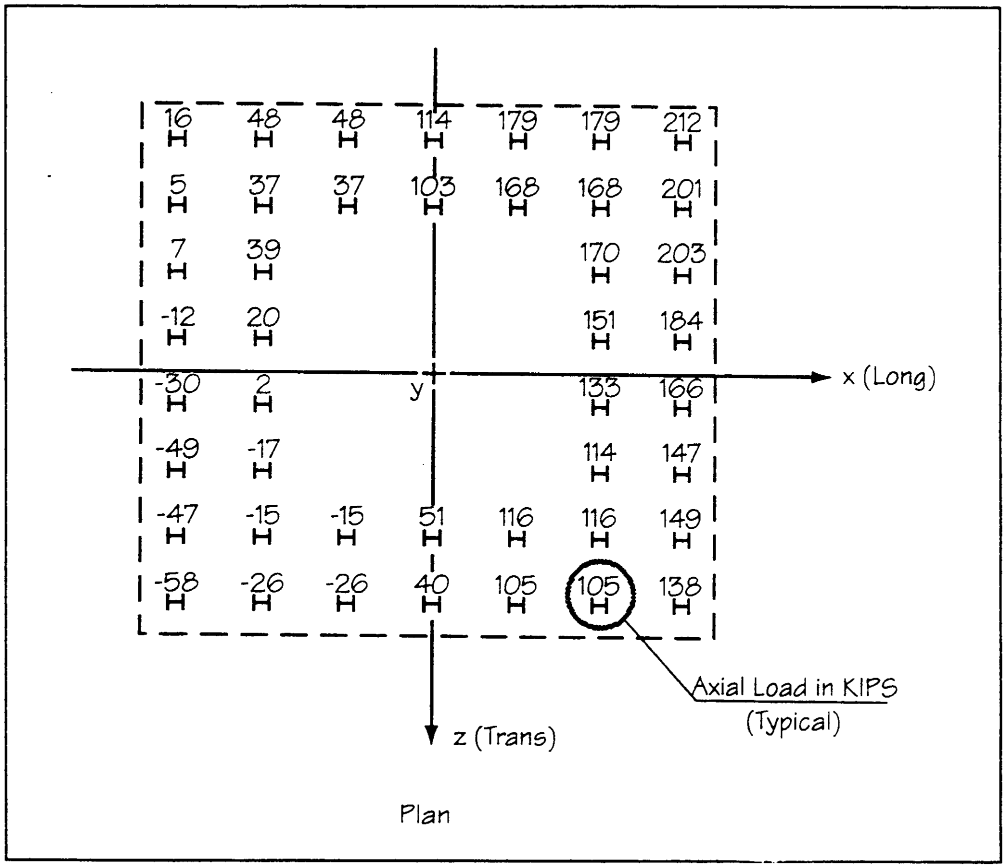


Figure 38 — Pile Axial Loads for R = 1.5

is less than allowed. Thus, it is possible to rearrange the piles of the group to take advantage of these limits. Rearrangement could allow the pile grid to become smaller and reduce the size of the pile cap, and/or reduce the number of piles. However, if pile spacings are reduced, the pile group effect will become more significant since the efficiency factor will be reduced.

If the pile group (arrangement and/or number of piles) is changed significantly from the group used to determine the spring stiffnesses for the analysis, the designer may wish to iterate the spring stiffness calculation with the new pile group, reanalyze the structure, and compare design forces.

Design Step
11.1.1
(continued)

For this example, the foundation design forces will be re-evaluated before consideration of any pile rearrangement. The pile forces shown so far for Design Step 11.1 use an effective R Factor of 1.5. As discussed in the Introduction to Design Step 7, the forces for design of the foundation should be determined to prevent inelastic behavior from occurring in the foundation. In Design Step 10.1.1(c), it was shown that the column is capable of behaving elastically for the full elastic seismic force combinations and so could carry full elastic seismic forces to the foundation.

Therefore, for this example, the designer should consider using an R Factor of 1.0 for the foundation design forces instead of an effective R Factor equal to half of the R Factor of the column (3/2 = 1.5) as currently permitted by Division I-A.

It should be noted that for multiple column bents where R = 5, the difference between the foundation design forces calculated using R/2, and those obtained by comparing the full elastic seismic forces with the plastic hinging forces, could be more significant.

The effect of increasing the foundation design forces (to R = 1.0) for the pile group is checked for this example.

b) Recompute Pile Axial Loads for R = 1.0

The pile axial loads for the group as shown in Figure 36 were recomputed using the foundation design forces from Design Step 7.3.2, Table 12, which use an R Factor of 1.0. The resulting pile axial loads are shown in Figure 39. (Negative loads are tension.)

The number of piles with a computed tension load is 15, still less than one-half of the pile group, though three piles have tension loads in excess of the 100-kip capacity. If these piles slip (fail in tension), the center of gravity of the group will shift and the compression force on the piles in the corner opposite the failed tension piles will increase. Since there is still significant compression capacity reserve in those piles, the increased compression could be tolerated without exceeding the compression capacity (this calculation is not shown here). Therefore, the foundation is able to resist the higher levels of force associated with R = 1.0 without failure in the pile group.

Use the pile group as shown and continue the foundation design using the design forces from Table 12 with R = 1.0.

Design Step
11.1.1
(continued)

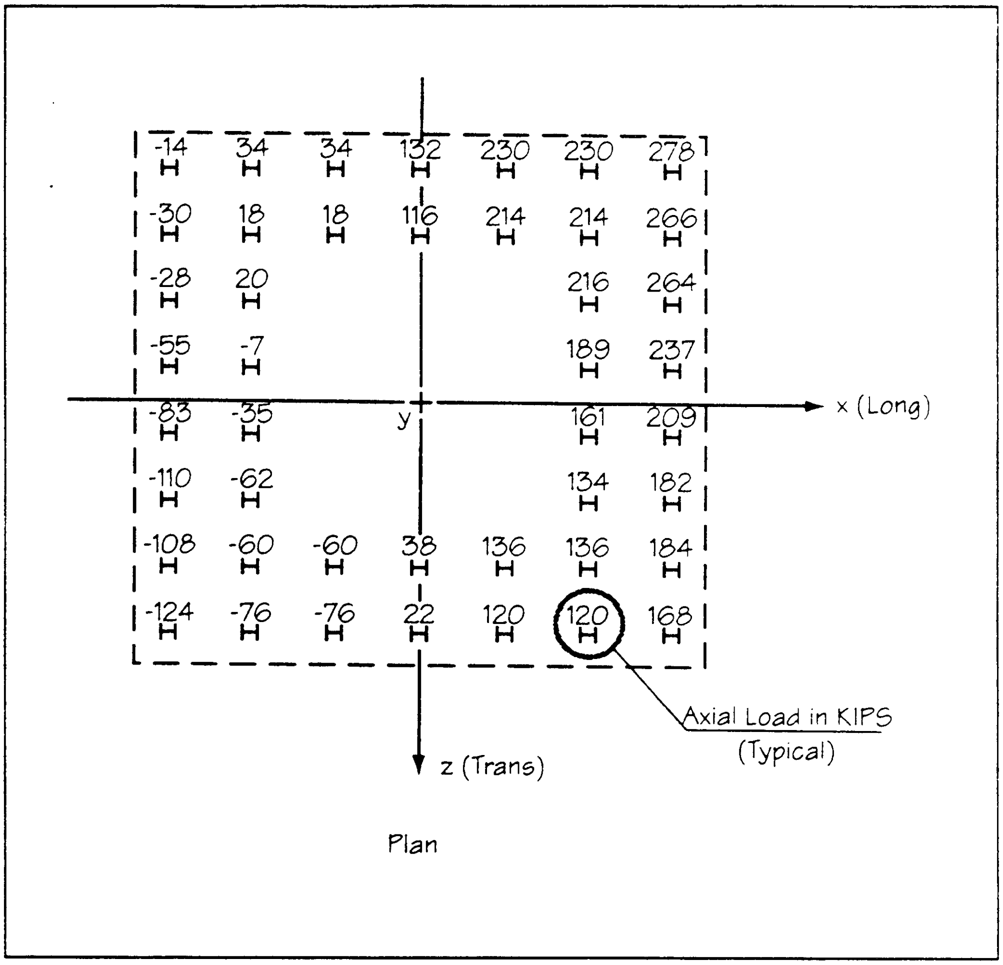


Figure 39 — Pile Axial Loads w/R = 1.0

Design Step 11.1.2

Determine Transverse Pile Forces

The transverse loading on the pile group produces shears and moments in the piles. A check can be made to see if the piles are likely to yield under this loading. In this example, influence charts from NAVFAC *Design Manual 7.02* (1986) are used to obtain pile shears and moments from lateral loading. A pinned pile head is assumed, consistent with the conditions used for development of the pile lateral spring stiffnesses.

In Caltrans' *Bridge Design Specifications* (1995), for piles embedded in material having a Standard Penetration test resistance value (N) equal to 10 or greater, the piles and footing are assumed to be capable of resisting all sustained lateral forces.

a) Determine Maximum Pile Shears and Moments

Look at transverse loading on the pile group. All of the piles are assumed to equally resist the total base shear. The resultant total shear is calculated from the shears in each direction. As previously discussed, the design forces are from Table 12 using R = 1.0.

$V_{ulong} := 992 \cdot kip$

$V_{utrans} := 338 \cdot kip$

$N_p = 44$ Total number of piles

Shear applied per pile in the strong direction

$P_{lats} := \dfrac{V_{ulong}}{N_p}$ $\qquad P_{lats} = 22.5 \cdot kip$

Shear applied per pile in the weak direction

$P_{latw} := \dfrac{V_{utrans}}{N_p}$ $\qquad P_{latw} = 7.7 \cdot kip$

Determine the pile design shears and moments using the relative stiffness factor (T), which was calculated in Design Step 6.2 for determination of the

Design Step 11.1.2 (continued)

individual pile lateral translational spring stiffnesses. Use Figures 40 and 41 (from NAVFAC *Design Manual 7.02*) to determine the shear coefficient F_V and the moment coefficient F_M. From Design Step 6.2, the L/T ratios for both axes of the piles are between 5 and 10.

$T_{ps} := 55 \cdot in$ — Relative stiffness factor for the strong direction of the pile (from Step 6.2)

$T_{pw} := 44 \cdot in$ — Relative stiffness factor for the weak direction of the pile (from Step 6.2)

From Figure 40

$F_V := 1.0$ — Maximum shear coefficient (at Z = 0)

From Figure 41

$F_M := -0.77$ — Maximum moment coefficient (at approximately Z = 1.25 L/T)

Compute maximum shears in the pile.

$$V_{ps} := F_V \cdot P_{lats}$$

$V_{ps} = 22.5 \cdot kip$ — Shear for the strong axis

$$V_{pw} := F_V \cdot P_{latw}$$

$V_{pw} = 7.7 \cdot kip$ — Shear for the weak axis

$$V_R := \sqrt{V_{ps}^2 + V_{pw}^2}$$

$V_R = 23.8 \cdot kip$ — Resultant pile shear

Design Step 11.1.2 (continued)

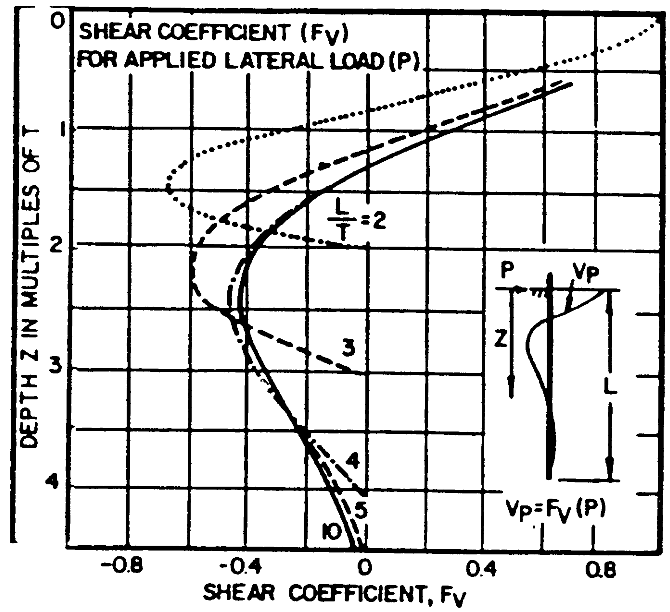


Figure 40 — Shear Coefficient, F_V

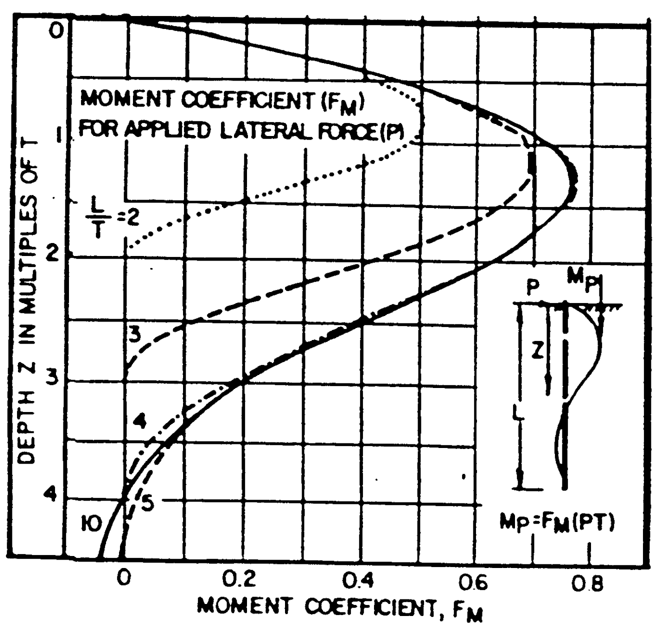


Figure 41 — Moment Coefficient, F_M

Design Step 11.1.2 (continued)

Compute maximum moments in the pile.

$M_{ps} := F_M \cdot P_{lats} \cdot T_{ps}$

$M_{ps} = -955 \cdot kip \cdot in$ Maximim moment for the strong axis

$M_{pw} := F_M \cdot P_{latw} \cdot T_{pw}$

$M_{pw} = -260 \cdot kip \cdot in$ Maximum moment for the weak axis

b) Determine Pile Shear and Moment Capacities

For the HP 12 x 84 section, check the pile capacities using the following properties and the provisions of Division I, Articles 10.42 to 10.60 for Load Factor Design.

$F_y := 36 \cdot ksi$ Minimum yield strength of pile

$d := 12.28 \cdot in$ Depth of web

$t_w := 0.685 \cdot in$ Thickness of web

$b_f := 12.295 \cdot in$ Width of flange

$t_f := 0.685 \cdot in$ Thickness of flange

$A := 24.6 \cdot in^2$ Pile cross-sectional area

Check shear capacity per Division I, Article 10.48.8.

$D := d - 2 \cdot t_f$

$D = 10.91 \cdot in$ Clear, unsupported distance between flanges

$k := 5$ Buckling coefficient for unstiffened members

Design Step 11.1.2 (continued)

Determine the constant C.

Determine the constant C

$Ratio := \frac{D}{t_w}$ Less than $Limit := \frac{6000 \cdot \sqrt{k}}{\sqrt{F_y}}$ for C = 1.0

$Ratio = 15.9$ Less than $Limit := 70.7$

Therefore, $C := 1.0$

$V_p := 0.58 \cdot F_y \cdot D \cdot t_w$ — Plastic shear force from Division 1, Equation (10-114)

$V_u := C \cdot V_p$ — Division I, Equation (10-112)

$V_u = 156 \cdot kip$ — Shear capacity of pile. This is much greater than the resultant pile shear, V_R = 23.8 kip, computed previously.

Check flexural capacity per Division I, Article 10.48.1.

For the pile strong direction

$Z_s := 120 \cdot in^3$ — Plastic section modulus for the strong direction

$M_{us} := F_y \cdot Z_s$ — Division I, Equation (10-91)

$M_{us} = 4320 \cdot kip \cdot in$ — Maximum flexural strength for the strong direction. This is much greater than M_{ps} = 955 kip*in

For the pile weak direction

$Z_w := 53.2 \cdot in^3$ — Plastic section modulus for the weak direction

Design Step
11.1.2
(continued)

$M_{uw} := F_y \cdot Z_w$ Division I, Equation (10-91)

$M_{uw} = 1915 \cdot kip \cdot in$ Maximum flexural strength for the weak direction (This is much greater than $M_{pw} = 260$ kip*in)

There is significant reserve shear and flexural capacity in the pile section, and yielding of the pile is not likely to occur for the computed levels of force using R = 1.0.

Design Step 11.1.3

Lateral Displacement of Pile Group

An important check for the base shear on the pile group is the amount of lateral displacement expected from the applied shear and whether this displacement can be tolerated. Displacements from the multimode analysis are reported in Design Step 6.3, Table 5. The values for the foundation level of Pier No. 7 are shown below. The R Factor for displacements is effectively 1.0 in any case.

Check the displacements for the foundation level at Pier No. 7. (Global X and Z directions are shown in Design Step 7, Figure 25.)

For the longitudinal earthquake, Eq_{long}

$x_{long} := 0.0364 \cdot ft$

$z_{long} := 0.0097 \cdot ft$

For the transverse earthquake, Eq_{trans}

$x_{trans} := 0.0053 \cdot ft$

$z_{trans} := 0.0369 \cdot ft$

For LC1, perform the combination of orthogonal seismic displacements

$x := 1.0 \cdot x_{long} + 0.3 \cdot x_{trans}$

$x = 0.038 \cdot ft$ $\qquad$ $x = 0.456 \cdot in$

$z := 1.0 \cdot z_{long} + 0.3 \cdot z_{trans}$

$z = 0.0208 \cdot ft$ $\qquad$ $z = 0.249 \cdot in$

The net lateral displacement at the foundation level for LC1 is

$d_{net} := \sqrt{x^2 + z^2}$

$d_{net} = 0.52 \cdot in$ $\qquad$ Displacement is small and can be tolerated without difficulty

Design Step
11.1.3
(continued)

For LC2, perform the combination of orthogonal seismic displacements.

$$x := 0.3 \cdot x_{long} + 1.0 \cdot x_{trans}$$

$x = 0.0162 \cdot ft$ $\quad$ $x = 0.19 \cdot in$

$$z := 0.3 \cdot z_{long} + 1.0 \cdot z_{trans}$$

$z = 0.0398 \cdot ft$ $\quad$ $z = 0.478 \cdot in$

The net lateral displacement at the foundation level for LC2 is

$$d_{net} := \sqrt{x^2 + z^2}$$

$d_{net} = 0.52 \cdot in$ $\quad$ Displacement is small and can be tolerated without difficulty

Criteria for tolerable lateral displacement for seismic design is not specifically quantified in AASHTO. Division I, Article 4.5.12 says that the structural engineer shall develop criteria that are "consistent with the function and type of structure, fixity of bearings, anticipated service life, and consequences of unacceptable displacements on the structural performance."

Division I, Article 4.4.7.2.5 states that "where combined horizontal and vertical movements are possible, horizontal movements should be limited to 1 inch or less."

However, these provisions do not directly address the seismic design purpose and philosophy as put forward in Division I-A. Tolerable structure displacements at the foundation level for seismic design should be an important item of discussion between the structural and geotechnical engineers.

Design Step 11.2 — Design Pile Cap

The design of the pile cap would be the same as for any conventional reinforced concrete pile cap and is not performed for this example.

Design Step 11.3

Design Pile Anchorage
[Division I-A, Article 6.4.2(C)]

All piles are to be adequately anchored to the pile cap. For steel piles, anchoring devices shall be provided to develop all uplift forces but not less than 10 percent of the allowable pile load. For seismic design, ultimate capacities of the piles should be used.

Take the design uplift force to be equal to the maximum pile tension load computed for R = 1.0 forces from Figure 39. For pile anchorage, use reinforcing bars inclined at 60 degrees through holes in the pile flanges as shown in Figure 42.

$P_u := 124 \cdot kip$ — Design uplift force

$f_y := 60 \cdot ksi$ — Yield strength of reinforcing

$\phi := 0.85$ — Strength reduction factor

Calculate the area of reinforcing steel required per pile.

$$A_s := \frac{P_u}{f_y \cdot \phi}$$ Steel area for direct tension

$A_s = 2.43 \cdot in^2$ — Provide 4 # 6 bars (two legs each) inclined at 60° ($A_s = 3.05\ in^2$)

Check the development length of the reinforcing per Division I, Article 8.25.

$f_c := 4000 \cdot psi$ — Concrete compressive strength

$A_b := 0.44 \cdot in^2$ — Area of #6 bar

$d_b := 0.75 \cdot in$ — Diameter of #6 bar

Design Step 11.3 (continued)

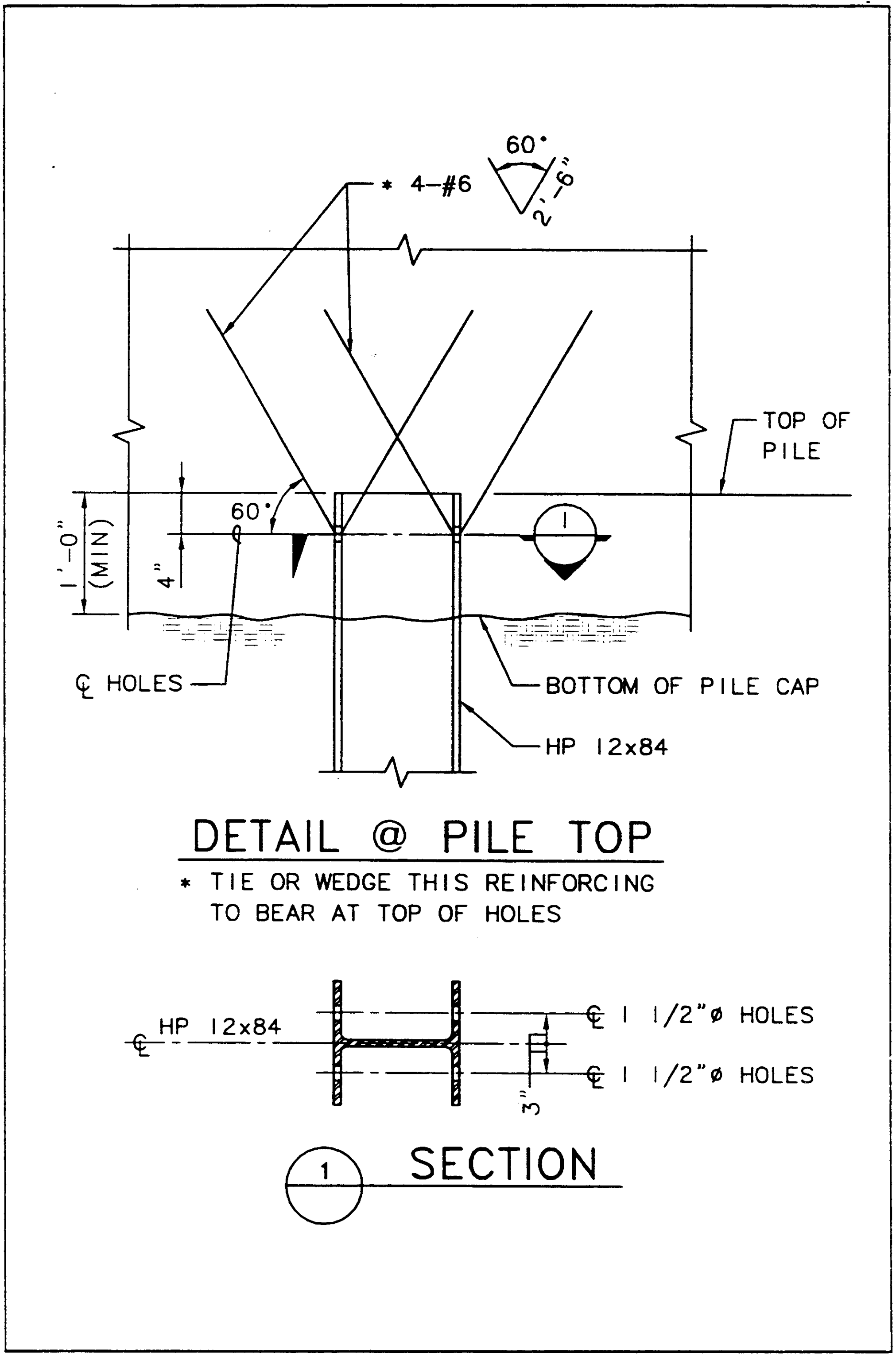


Figure 42 — Pile Anchorage Detail

Design Step 11.3 (continued)

Basic development length equation

$$L_{db} := 0.04 \cdot A_b \cdot \frac{f_y}{\sqrt{f_c}} \qquad L_{db} := 16.7 \cdot in$$

But not less than

$$L_{db} := 0.0004 \cdot d_b \cdot f_y \qquad L_{db} := 18 \cdot in \qquad \text{Controls}$$

The pile anchorage, as detailed in Figure 42, is sufficient to develop the full-uplift capacity of the pile.

There are a number of ways to successfully anchor steel piles into a concrete pile cap. The use of reinforcing bars as anchorage devices is recommended because the deformed bars provide excellent bond with concrete. Attachment of the reinforcing bars to the pile may be accomplished by mechanical means or by welding. From a constructability view, anchorage devices should provide no interference with handling or driving of the pile and should be easy to install. The use of reinforcing bars through holes burned in the pile section is a relatively easy and inexpensive way to provide good anchorage. The diameter of the hole should be limited to two times the bar diameter, and the center of the hole should be located at sufficient distance from the end of the pile. Care should be taken to tie or wedge the reinforcing bars tightly against the top of the hole to reduce the possibility of slip between the rebar anchor and the pile. Holes may be located in either the flanges or the web for an H-pile section.

DESIGN STEP 12 | **DESIGN ABUTMENTS**

For this example, the abutments are not designed.

DESIGN STEP 13 | **DESIGN SETTLEMENT SLABS**

Not applicable.

DESIGN STEP 14 | **REVISE STRUCTURE**

Not required.

DESIGN STEP 15 — SEISMIC DETAILS

DETAILS SUMMARY

Special details for resistance to seismic forces shown in this example are limited primarily to connections per requirements of Division I-A for SPC B. As longitudinal linkage and hold down connections are not required for SPC B, the important connection details for seismic design are the following. Details discussed in previous sections are repeated here as a summary.

Connection of Superstructure to Pier

Connection of the superstructure to an intermediate pier through bearing anchor bolts is shown in Figure 43.

Connection of Column to Pile Cap (including Confinement)

Figures 44 and 45 show special transverse reinforcing requirements and extent for confinement at the column base and connection of the column to the pile cap.

Connection of Pile to Pile Cap (Pile Anchorage)

One method of steel pile anchorage is shown in Figure 46. An alternate method of anchorage is detailed in Figure 47. Figure 47 also uses reinforcing bars as the anchoring devices, but provides the connection of the reinforcing to the pile by welding. This may be a preferred connection if there is concern about slippage between the reinforcing and the pile at holes under a seismic tension load. The designer should keep in mind that the welded connections of Figure 47 will be performed in the field and will require preheating for standard AASHTO M31, Grade 60 reinforcing steel.

DETAILS SUMMARY
(continued)

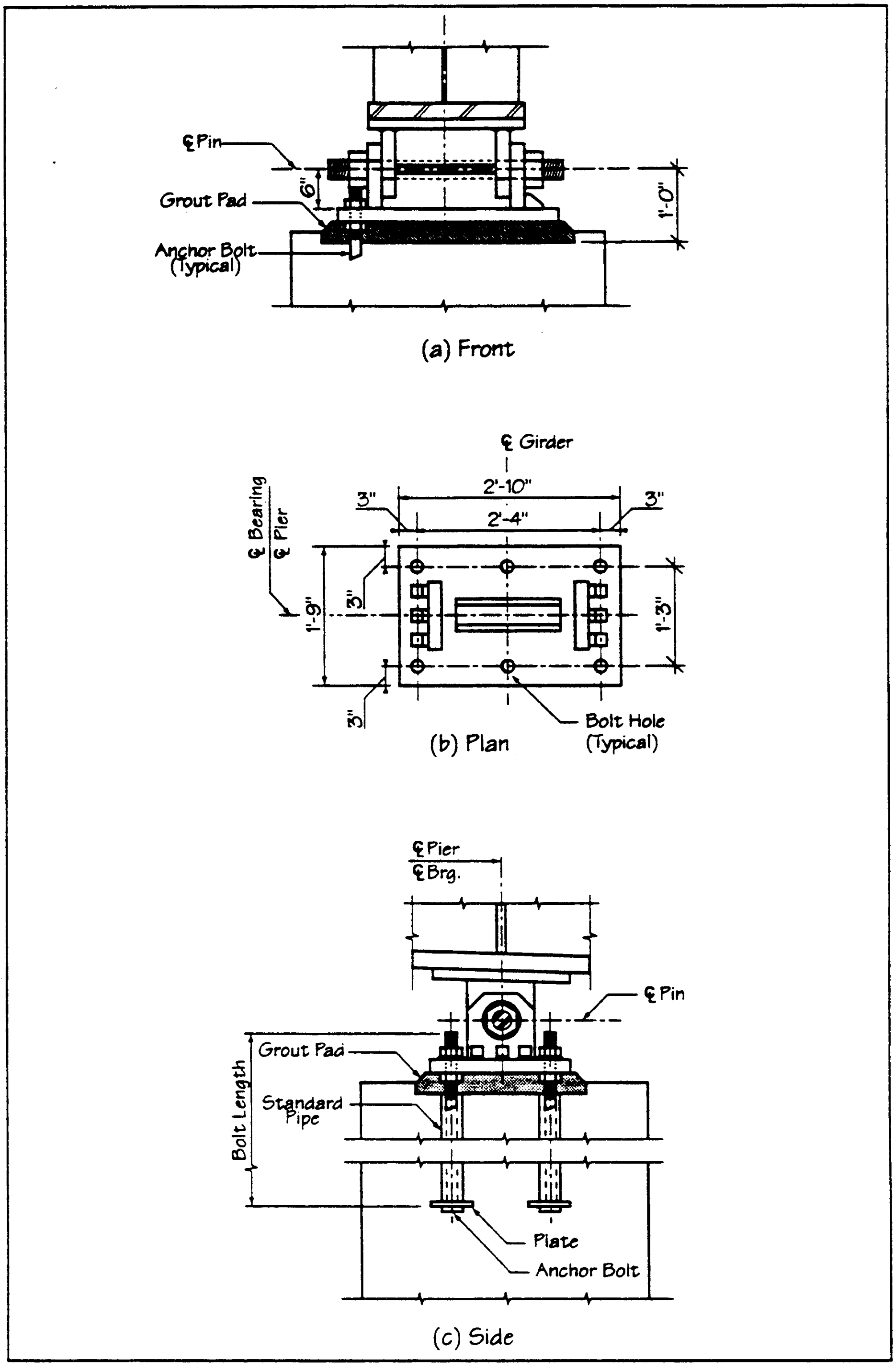


Figure 43 — Bearing Details

DETAILS SUMMARY
(continued)

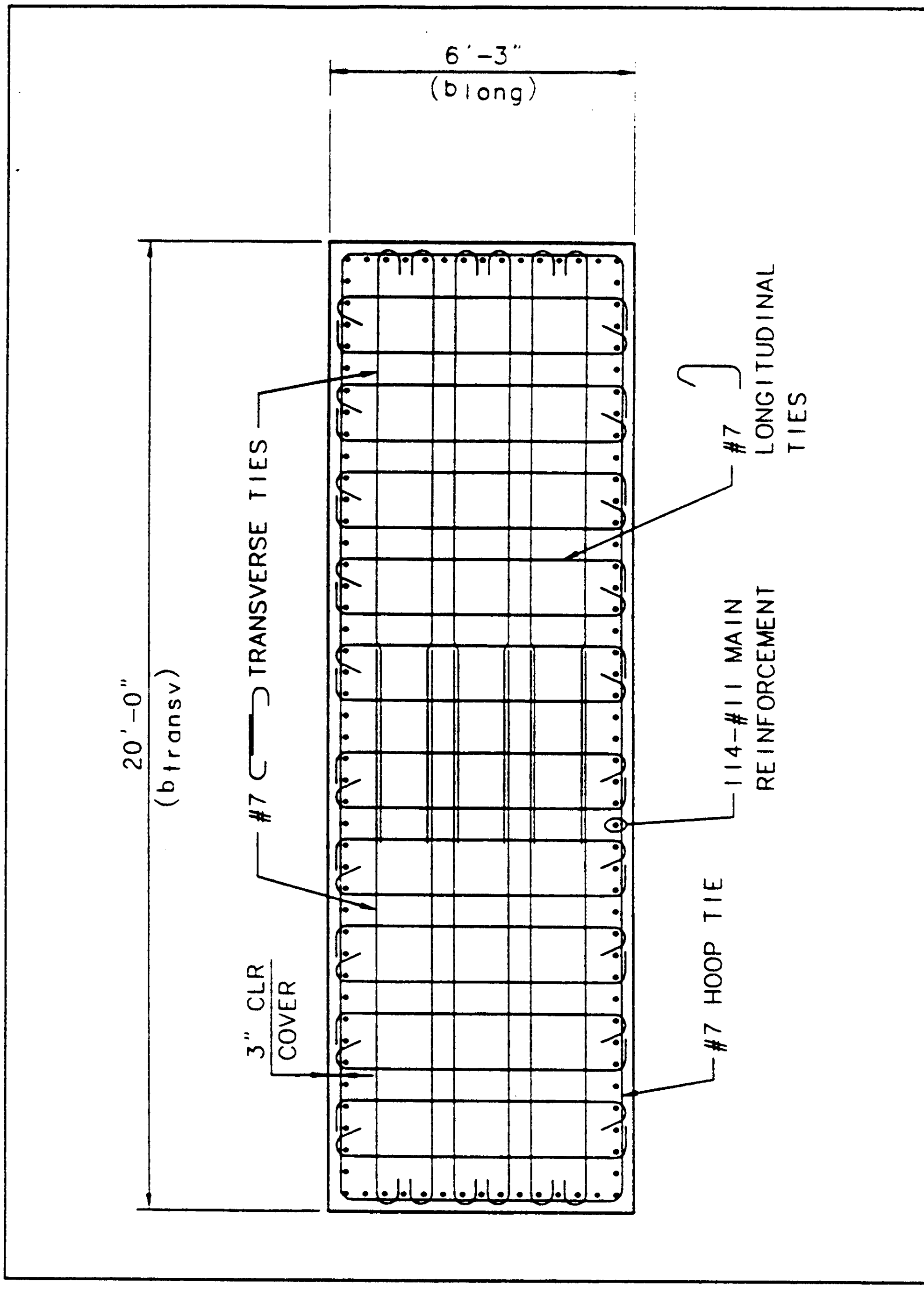


Figure 44 — Column Cross Section at Base

DETAILS SUMMARY
(continued)

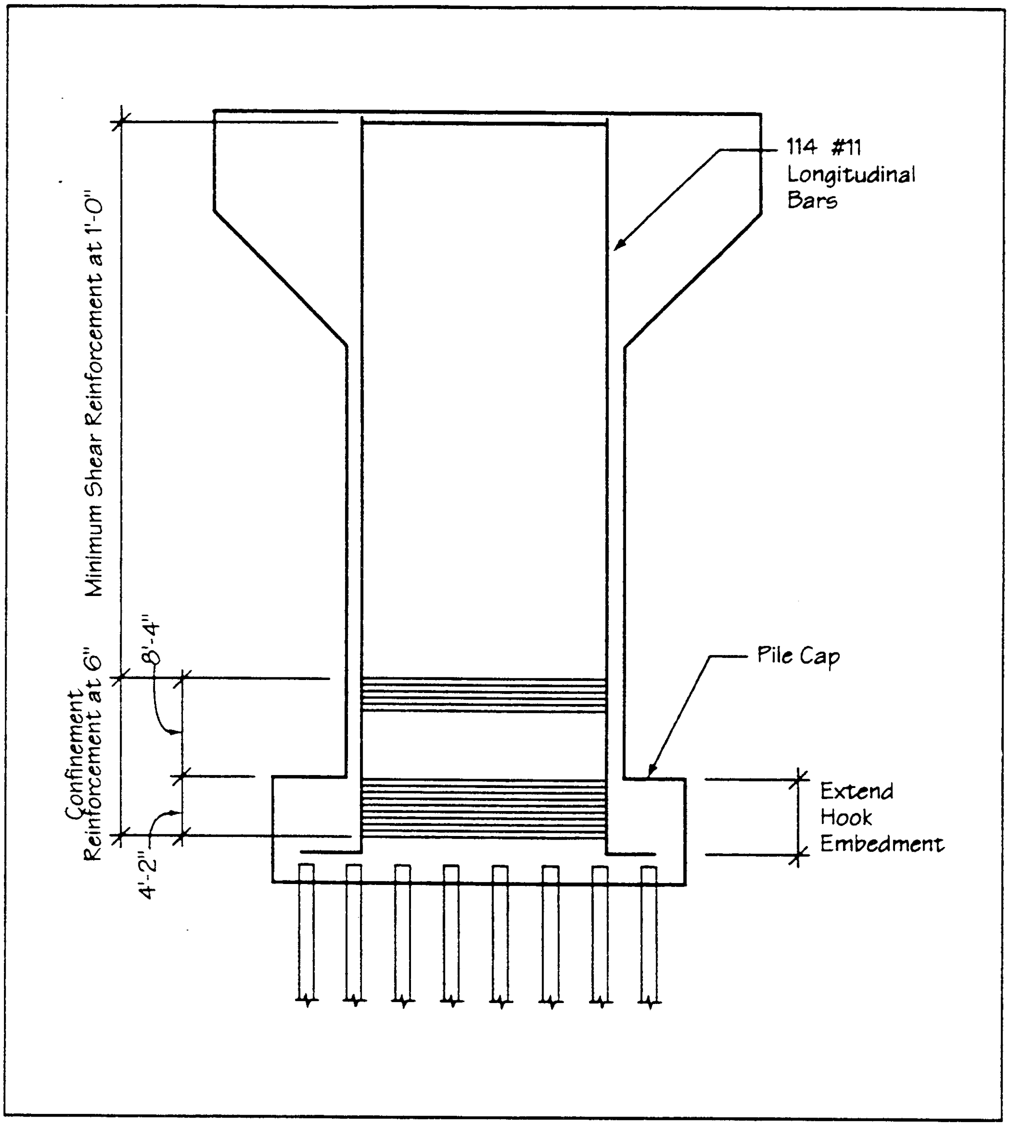


Figure 45 — Column Reinforcement Details

DETAILS SUMMARY
(continued)

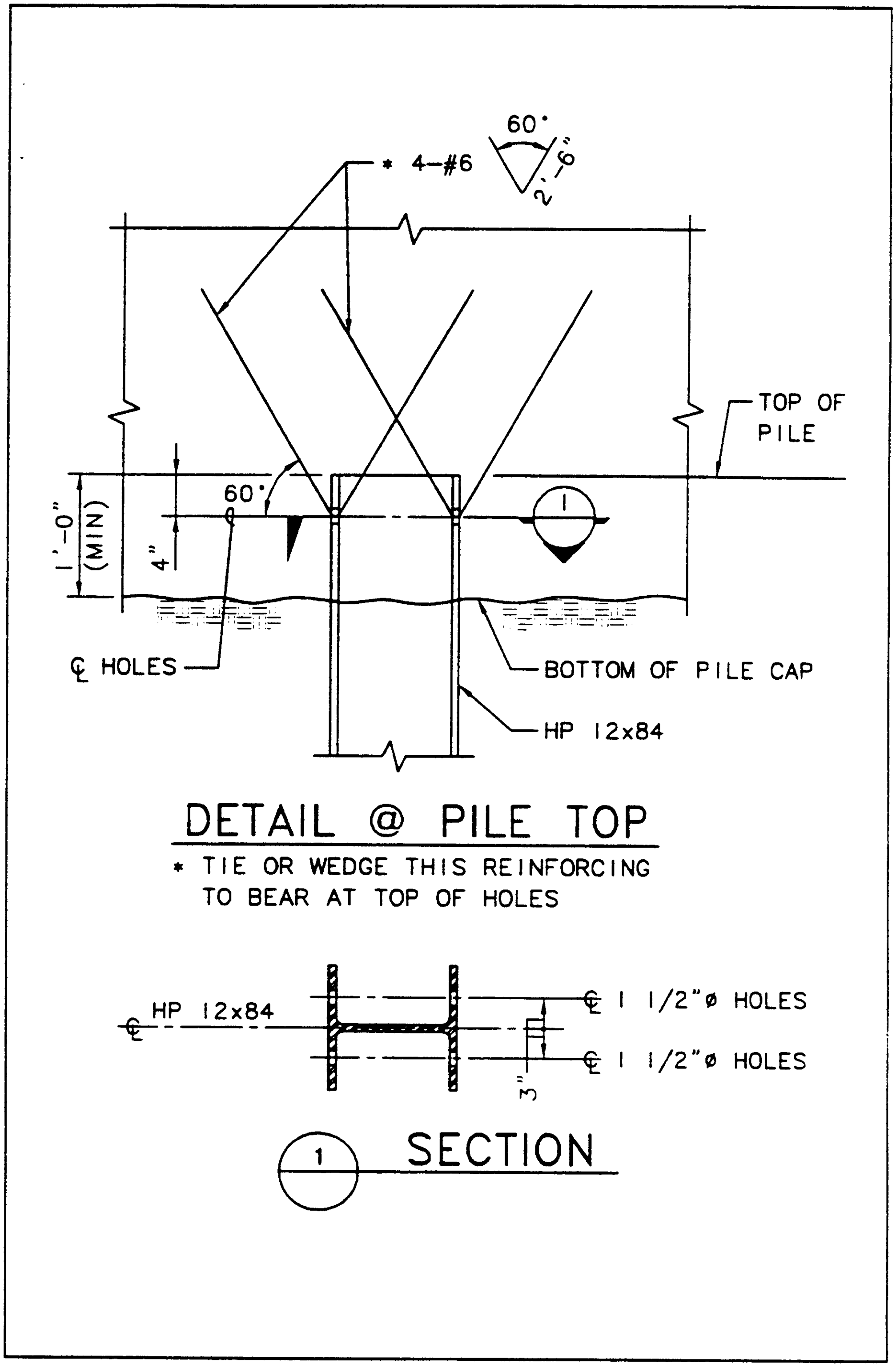


Figure 46 — Pile Anchorage Detail

DETAILS SUMMARY
(continued)

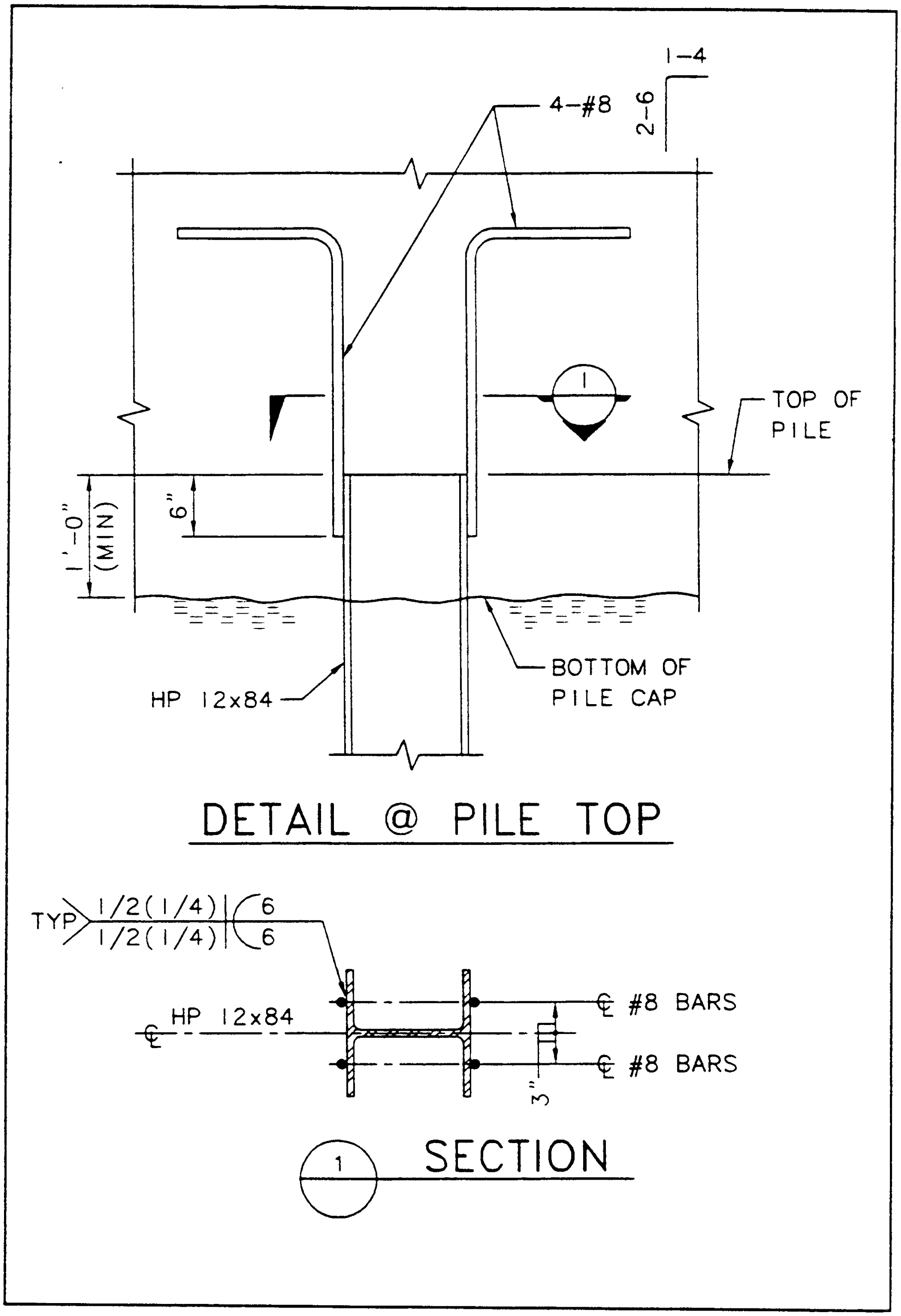


Figure 47 — Alternate Pile Anchorage

Section IV
Closing Statements

SECTION IV — CLOSING STATEMENTS

SEISMIC DESIGN

In preliminary seismic design, fundamental periods were calculated for comparison with multimode results, and column shear forces were computed. Although not discussed for this example, the preliminary seismic forces were used to determine foundation sizes for modeling and dynamic analysis.

Modeling of the structure for multimode analysis required some special attention to connection details between the superstructure and the piers owing to the multiple unit behavior and use of different bearing conditions. Because of the massive pier elements and stiffness of the foundations, the number of modes required to obtain sufficient participating mass for the multimode spectral analysis was more than specified in Division I-A.

The determination of "seismic design forces specified for bridges classified as SPC B (is) intended to be relatively simple but consistent with the overall design concepts and methodology," according to the Commentary of AASHTO Division I-A. However, it was shown in this example that there is the potential for calculating seismic design forces that are consistent with the provisions of Division I-A but result in potential foundation understrength and column shear understrength. Provisions of Division I-A for SPC B allow the designer to determine final seismic design forces without evaluating whether the structure will remain elastic or will form plastic hinges during the design shaking. The application of the Response Modification Factor (R) becomes an important consideration because any reduction in design forces must be consistent with the structure's ability to develop plastic mechanisms. This is not clearly defined in Division I-A for SPC B.

Pier or column strength capacities should be investigated to determine the forces that are capable of being transferred to the foundation. In this example, it was shown that the pier columns were likely to transmit full elastic seismic forces to the foundation. Therefore, an R Factor of 1.0 was used for the final foundation design forces, versus an R of 1.5 permitted by Division I-A. This avoids the possibility of inelastic behavior occurring in the foundation where it may be impossible to detect. Due to their large size and shear capacity, the pier columns were not critical for design shear forces even when they were computed with an R Factor of 1.0, versus R equal to 3.0 as allowed for SPC B.

SEISMIC DESIGN (continued)

In this example, design of more than one pier foundation and design of the abutments were not addressed. The level of seismic design forces are different for each of the piers, especially for Pier Nos. 5 and 8 with sliding bearings and Pier No. 4 with expansion bearings. Foundations for these piers would be significantly smaller than for Pier No. 7 as designed in the example. It would be prudent to adjust foundation spring stiffnesses for the final multimode analysis to account for the actual differences in the expected sizes of the foundations based upon forces from either preliminary design or a first round multimode analysis.

Section V
References

SECTION V REFERENCES

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Appendix A
Geotechnical Data

APPENDIX A GEOTECHNICAL DATA

SUBSURFACE CONDITIONS

Subsurface conditions were derived from seven borings drilled along the bridge alignment. As shown in Figure A1, the subsurface conditions consist of coarse alluvial flood deposits overlying volcaniclastic sediments (tuff). The tuff increases in strength with depth. The water table, which is controlled by the river, is located at or near the ground surface.

SOIL PROPERTIES

Soil properties for the subsurface materials are shown on Figure A1. These properties were estimated from empirical correlations to the standard penetration test resistance values in the borings. Laboratory tests may provide more detailed design values.

SOIL PROFILE TYPE

Type I — Stable deposits of sands and gravels where the soil depth is less than 200 feet.

SITE ACCELERATION

0.15g — Taken from AASHTO seismicity map.

FOUNDATION DESIGN

HP 12 x 84 — Pile foundations chosen for design.

Axial capacity based on U.S. Army Corps of Engineers (1991).

Tension

Critical depth = 30 feet (assumed for very dense sand and gravel)

$$Q_{T\,ult} = f_s pL = K_t \sigma_{v\,avg} (\tan \delta)\, pL$$

Where:

$Q_{T\,ult}$	ultimate tension capacity of single pile (kips)
K_t	coefficient of lateral earth pressure (assumed as 0.65 for tension)
$\sigma_{v\,avg}$	average effective vertical stress over the length of the pile; effective stress increases linearly to the critical depth of 30 feet and is constant below this depth (ksf)
δ	average of the angle of friction between soil and steel pile (0.75ϕ) and soil (ϕ) = 0.875ϕ

FOUNDATION DESIGN (continued)

p perimeter block around pile (4.1 feet)

L length of embedment of pile below ground surface (feet)

$Q_{T\,ult}$ = 0.65 (30' x 0.0676kcf x 1/2) (tan 0.875 x 42) (4.1')(30')
+ 0.65 (30' x 0.0676kcf)(tan 0.875 x 42)(4.1')(10')
= 100 kips

Compression

$$Q_{C\,ult} = A_t q + K_c \sigma_{v\,avg} (\tan \delta)\, pL$$

Where:

$Q_{C\,ult}$ ultimate compression capacity of single pile (kips)

K_c coefficient of lateral earth pressure (assumed as 1.0 for compression)

q tip resistance (ksf)
$= \sigma_v\ N_q$, where $N_q = 105$ for $\phi = 42°$

A_t area of tip of pile; for H-piles use block area of 1 ft^2

$Q_{C\,ult}$ = (1 ft^2)(30' x 0.0676kcf)(105)
+ (1.0)(30' x 0.0676kcf x 1/2)(tan 0.875 x 42)(4.1')(30')
+ (1.0) (30' x 0.676kcf)(tan 0.875 x 42)(4.1')(10')
= 370 kips

Lateral Resistance

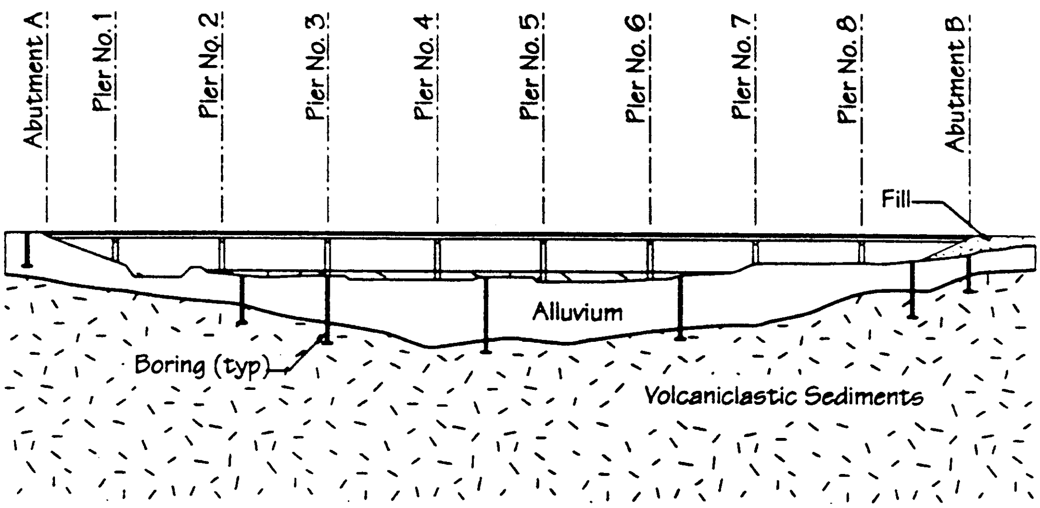
Pile stiffness values may be computed from simplified procedures (NAVFAC *Design Manual 7.02*, 1986) as indicated in Design Step 6, or from computer programs, such as COM624 or LPILE^{plus} (Reese and Wang, 1993), that are widely used by various Departments of Transportation and design consultants.

The base friction acting on the pier and any lateral spring derived from the base contact area should be neglected because of the possibility of loss of intimate contact between the base of the pile cap and the underlying soils due to settlement or scour.

OTHER CONCERNS

A detailed scour analysis should be completed to determine the depth of the pile cap and the need for protection.

Liquefaction is not likely to occur at this site because of the presence of very dense soil deposits. Similarly, the abutment slopes should be stable during earthquake shaking because of the presence of these deposits.



Elevation

SOIL PROPERTIES

Stratum	Depth (ft)	Soil Description	N (bpf)	γ (pcf)	φ (deg)	c (pcf)	n_h (tcf)
Alluvium	0 to 50	Very dense, sand, gravel, and cobbles	75	130	42	0	50
Volcaniclastic sediments	> 50	Very dense/hard silt (very soft tuff)	50	110	30	500	----

Where:

N standard penetration resistance (blows per foot)
γ total unit weight (pounds per cubic foot)
φ internal angle of friction (degrees)
c cohesion (pounds per square foot)
n_h constant of horizontal subgrade reaction (tons per cubic foot)

Figure A1 — Subsurface Conditions

Appendix B
SAP90 V6.0 Beta Input

```
FHWA BRIDGE NO 5 / SAP90 (BETA VERSION) INPUT FILE

SYSTEM

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COORDINATE

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             X=0    Y=0  Z=1
             X=1    Y=0  Z=1

  NAME=PIER1 X=0    Y=0  Z=0
             X=0    Y=0  Z=1
             X=1    Y=0  Z=1

  NAME=PIER2 X=0    Y=0  Z=0
             X=0    Y=0  Z=1
             X=1    Y=0  Z=1

  NAME=PIER3 X=0    Y=0  Z=0
             X=0    Y=0  Z=1
             X=1    Y=0  Z=1

  NAME=PIER4 X=620.000 Y=0  Z=0
             X=620.000 Y=0  Z=1300
             X=621.000 Y=0  Z=1300

  NAME=PIER5 X=792.490 Y=0  Z=11.494
             X=620.000 Y=0  Z=1300
             X=621.000 Y=0  Z=1300

  NAME=PIER6 X=961.929 Y=0  Z=45.773
             X=620.000 Y=0  Z=1300
             X=621.000 Y=0  Z=1300

  NAME=PIER7 X=1125.323 Y=0  Z=102.232
             X= 620.000 Y=0  Z=1300
             X= 621.000 Y=0  Z=1300

  NAME=PIER8 X=1279.780 Y=0  Z=179.870
             X= 620.000 Y=0  Z=1300
             X= 621.000 Y=0  Z=1300

  NAME=ABUTB X=1422.570 Y=0  Z=277.317
             X= 620.000 Y=0  Z=1300
             X= 621.000 Y=0  Z=1300

  NAME=EQK   X=0        Y=0  Z=0
             X=1422.570 Y=0  Z=277.317
             X=1        Y=0  Z=-1

JOINT
701  X= 0.000    Y=  0.0 Z=  0.000   ; L =   105
702  X= 26.250   Y=  0.0 Z=  0.000
703  X= 52.500   Y=  0.0 Z=  0.000
704  X= 78.750   Y=  0.0 Z=  0.000
711  X= 105.000  Y=  0.0 Z=  0.000   ; L =   185
712  X= 151.250  Y=  0.0 Z=  0.000
```

```
713   X= 197.500  Y=  0.0 Z=  0.000
714   X= 243.750  Y=  0.0 Z=  0.000
721   X= 290.000  Y=  0.0 Z=  0.000   ; L =   160
722   X= 330.000  Y=  0.0 Z=  0.000
723   X= 370.000  Y=  0.0 Z=  0.000
724   X= 410.000  Y=  0.0 Z=  0.000
731   X= 450.000  Y=  0.0 Z=  0.000   ; L =   170  ALFA = 0.133077 Radians
732   X= 492.500  Y=  0.0 Z=  0.000
733   X= 535.000  Y=  0.0 Z=  0.000
734   X= 577.500  Y=  0.0 Z=  0.000
741A  X= 620.000  Y=  0.0 Z=  0.000
741   X= 620.000  Y=  0.0 Z=  0.000   ; ALFA = 0.00000 Radians X = 620+1300*sin(ALFA)
741B  X= 620.000  Y=  0.0 Z=  0.000
742   X= 663.242  Y=  0.0 Z=  0.719   ; ALFA = 0.03327 Radians Y= 1300-1300*cos(ALFA)
743   X= 706.436  Y=  0.0 Z=  2.877   ; ALFA = 0.06654 Radians
744   X= 749.535  Y=  0.0 Z=  6.470   ; ALFA = 0.09981 Radians
751   X= 792.490  Y=  0.0 Z=  11.494  ; ALFA = 0.13308 Radians
752   X= 835.254  Y=  0.0 Z=  17.945  ; ALFA = 0.16635 Radians
753   X= 877.780  Y=  0.0 Z=  25.814  ; ALFA = 0.19962 Radians
754   X= 920.021  Y=  0.0 Z=  35.094  ; ALFA = 0.23288 Radians
761   X= 961.929  Y=  0.0 Z=  45.773  ; ALFA = 0.26615 Radians
762   X= 1003.460 Y=  0.0 Z=  57.841  ; ALFA = 0.29942 Radians
763   X= 1044.566 Y=  0.0 Z=  71.284  ; ALFA = 0.33269 Radians
764   X= 1085.202 Y=  0.0 Z=  86.086  ; ALFA = 0.36596 Radians
771   X= 1125.323 Y=  0.0 Z=  102.232 ; ALFA = 0.39923 Radians
772   X= 1164.884 Y=  0.0 Z=  119.703 ; ALFA = 0.43250 Radians
773   X= 1203.843 Y=  0.0 Z=  138.481 ; ALFA = 0.46577 Radians
774   X= 1242.156 Y=  0.0 Z=  158.544 ; ALFA = 0.49904 Radians
781   X= 1279.780 Y=  0.0 Z=  179.870 ; ALFA = 0.53231 Radians
782   X= 1316.674 Y=  0.0 Z=  202.437 ; ALFA = 0.56558 Radians
783   X= 1352.797 Y=  0.0 Z=  226.217 ; ALFA = 0.59885 Radians
784   X= 1388.109 Y=  0.0 Z=  251.187 ; ALFA = 0.63212 Radians
791   X= 1422.570 Y=  0.0 Z=  277.317 ; ALFA = 0.66538 Radians

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411   X= 105.000  Y=      -25.0    Z=     0.000
311   X= 105.000  Y=      -58.0    Z=     0.000
211   X= 105.000  Y=      -64.5    Z=     0.000

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321   X= 290.000  Y=      -78.0    Z=     0.000
221   X= 290.000  Y=      -84.5    Z=     0.000

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531   X= 450.000  Y=      -15.0    Z=     0.000
431   X= 450.000  Y=      -25.0    Z=     0.000
331   X= 450.000  Y=      -78.0    Z=     0.000
231   X= 450.000  Y=      -84.5    Z=     0.000

641   X= 620.000  Y=      -6.5     Z=     0.000
541   X= 620.000  Y=      -15.0    Z=     0.000
441   X= 620.000  Y=      -25.0    Z=     0.000
341   X= 620.000  Y=      -78.0    Z=     0.000
241   X= 620.000  Y=      -84.5    Z=     0.000
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551   X= 792.490   Y=      -15.0     Z=      11.494
451   X= 792.490   Y=      -25.0     Z=      11.494
351   X= 792.490   Y=      -78.0     Z=      11.494
251   X= 792.490   Y=      -84.5     Z=      11.494

661   X= 961.929   Y=       -6.5     Z=      45.773
561   X= 961.929   Y=      -15.0     Z=      45.773
461   X= 961.929   Y=      -25.0     Z=      45.773
361   X= 961.929   Y=      -78.0     Z=      45.773
261   X= 961.929   Y=      -84.5     Z=      45.773

671   X= 1125.323 Y=  -6.5     Z=      102.232
571   X= 1125.323 Y=  -15.0    Z=      102.232
471   X= 1125.323 Y=  -25.0    Z=      102.232
371   X= 1125.323 Y=  -58.0    Z=      102.232
271   X= 1125.323 Y=  -64.5    Z=      102.232

681   X= 1279.780 Y=  -6.5     Z=      179.870
581   X= 1279.780 Y=  -15.0    Z=      179.870
481   X= 1279.780 Y=  -25.0    Z=      179.870
381   X= 1279.780 Y=  -58.0    Z=      179.870
281   X= 1279.780 Y=  -64.5    Z=      179.870


LOCAL
  ADD=701 CSYS=ABUTA
  ADD=211 CSYS=PIER1
  ADD=221 CSYS=PIER2
  ADD=231 CSYS=PIER3
  ADD=241 CSYS=PIER4
  ADD=251 CSYS=PIER5
  ADD=261 CSYS=PIER6
  ADD=271 CSYS=PIER7
  ADD=281 CSYS=PIER8
  ADD=791 CSYS=ABUTB

; Abutment is restrained in Vertical and for Rotational Stiffness around the
longitudinal axis of the bridge.

RESTRAINT
  ADD=701 DOF=U2,R1
  ADD=791 DOF=U2,R1

CONSTRAINTS
  NAME=EXP TYPE=EQUAL DOF=UY,UZ,RX
  ADD=741
  ADD=741A
  ADD=741B

; Abutment is released for longitudinal movement (U1), rotation around vertical axis
; (R2) and rotation around transverse axis (R3).   Stiffness in the transverse
; direction is: U3 = 4.663 k/ft
```

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SPRING
  CSYS=ABUTA
    ADD=701 U1=0                U3=4.66E3             R2=0      R3=0
  CSYS=PIER1
    ADD=211 U1=2.67E4 U2=7.85E5 U3=1.71E4 R1=7.96E7 R2=4.80E6 R3=9.63E7
  CSYS=PIER2
    ADD=221 U1=2.67E4 U2=7.85E5 U3=1.71E4 R1=7.96E7 R2=4.80E6 R3=9.63E7
  CSYS=PIER3
    ADD=231 U1=2.67E4 U2=7.85E5 U3=1.71E4 R1=7.96E7 R2=4.80E6 R3=9.63E7
  CSYS=PIER4
    ADD=241 U1=2.67E4 U2=7.85E5 U3=1.71E4 R1=7.96E7 R2=4.80E6 R3=9.63E7
  CSYS=PIER5
    ADD=251 U1=2.67E4 U2=7.85E5 U3=1.71E4 R1=7.96E7 R2=4.80E6 R3=9.63E7
  CSYS=PIER6
    ADD=261 U1=2.67E4 U2=7.85E5 U3=1.71E4 R1=7.96E7 R2=4.80E6 R3=9.63E7
  CSYS=PIER7
    ADD=271 U1=2.67E4 U2=7.85E5 U3=1.71E4 R1=7.96E7 R2=4.80E6 R3=9.63E7
  CSYS=PIER8
    ADD=281 U1=2.67E4 U2=7.85E5 U3=1.71E4 R1=7.96E7 R2=4.80E6 R3=9.63E7
  CSYS=ABUTB
    ADD=791 U1=0                U3=4.66E3             R2=0      R3=0


MATERIAL

  NAME=SUPER TYPE=ISO M=0.152/32.2  W=0.152  IDES=C ;INCLUDES WEIGHT OF X-FRAMES,
OVERLAY, BARRIERS & ETC
    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
  NAME=RIGID TYPE=ISO M=0 W=0 IDES=C
    E=519000 U=0.18   A=6.0E-06

SECTION

  NAME=SUPER MAT=SUPER A=60.39 I=518,9003 J=5.8
  NAME=M6 MAT=RIGID SH=R T=625.,4000.
  NAME=M5 MAT=SUB   SH=R T=6.25,40.0
  NAME=M3 MAT=SUB   SH=R T=6.25,20.0
  NAME=M2 MAT=SUB   SH=R T=25.0,25.0


FRAME
  CSYS=0
  701 J=701,702 SEC=SUPER PLANE13=+Z
  702 J=702,703 SEC=SUPER PLANE13=+Z
  703 J=703,704 SEC=SUPER PLANE13=+Z
  704 J=704,711 SEC=SUPER PLANE13=+Z
  711 J=711,712 SEC=SUPER PLANE13=+Z
  712 J=712,713 SEC=SUPER PLANE13=+Z
  713 J=713,714 SEC=SUPER PLANE13=+Z
  714 J=714,721 SEC=SUPER PLANE13=+Z
  721 J=721,722 SEC=SUPER PLANE13=+Z
  722 J=722,723 SEC=SUPER PLANE13=+Z
  723 J=723,724 SEC=SUPER PLANE13=+Z
  724 J=724,731 SEC=SUPER PLANE13=+Z
  731 J=731,732 SEC=SUPER PLANE13=+Z
  732 J=732,733 SEC=SUPER PLANE13=+Z
  733 J=733,734 SEC=SUPER PLANE13=+Z
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  734 J=734,741A SEC=SUPER PLANE13=+Z
  741 J=741B,742 SEC=SUPER PLANE13=+Z
  742 J=742,743 SEC=SUPER PLANE13=+Z
  743 J=743,744 SEC=SUPER PLANE13=+Z
  744 J=744,751 SEC=SUPER PLANE13=+Z
  751 J=751,752 SEC=SUPER PLANE13=+Z
  752 J=752,753 SEC=SUPER PLANE13=+Z
  753 J=753,754 SEC=SUPER PLANE13=+Z
  754 J=754,761 SEC=SUPER PLANE13=+Z
  761 J=761,762 SEC=SUPER PLANE13=+Z
  762 J=762,763 SEC=SUPER PLANE13=+Z
  763 J=763,764 SEC=SUPER PLANE13=+Z
  764 J=764,771 SEC=SUPER PLANE13=+Z
  771 J=771,772 SEC=SUPER PLANE13=+Z
  772 J=772,773 SEC=SUPER PLANE13=+Z
  773 J=773,774 SEC=SUPER PLANE13=+Z
  774 J=774,781 SEC=SUPER PLANE13=+Z
  781 J=781,782 SEC=SUPER PLANE13=+Z
  782 J=782,783 SEC=SUPER PLANE13=+Z
  783 J=783,784 SEC=SUPER PLANE13=+Z
  784 J=784,791 SEC=SUPER PLANE13=+Z

  211 J=211,311 SEC=M2               PLANE13=+Z
  311 J=311,411 SEC=M3               PLANE13=+Z
  411 J=411,511 SEC=M3,M5 EIVAR=1,3 PLANE13=+Z
  511 J=511,611 SEC=M5               PLANE13=+Z

  GEN=211,241,10 IINC=10 JINC=10
  GEN=311,341,10 IINC=10 JINC=10
  GEN=411,441,10 IINC=10 JINC=10
  GEN=511,541,10 IINC=10 JINC=10

  611 J=611,711 SEC=M6 IREL=R3      PLANE13=+Z
  621 J=621,721 SEC=M6 IREL=R3      PLANE13=+Z
  631 J=631,731 SEC=M6 IREL=R3      PLANE13=+Z
  641 J=641,741 SEC=M6               PLANE13=+Z

CSYS=PIER5
  251 J=251,351 SEC=M2               PLANE13=+Z
  351 J=351,451 SEC=M3               PLANE13=+Z
  451 J=451,551 SEC=M3,M5 EIVAR=1,3 PLANE13=+Z
  551 J=551,651 SEC=M5               PLANE13=+Z
  651 J=651,751 SEC=M6 IREL=U2,R3   PLANE13=+Z

CSYS=PIER6
  261 J=261,361 SEC=M2               PLANE13=+Z
  361 J=361,461 SEC=M3               PLANE13=+Z
  461 J=461,561 SEC=M3,M5 EIVAR=1,3 PLANE13=+Z
  561 J=561,661 SEC=M5               PLANE13=+Z
  661 J=661,761 SEC=M6 IREL=R3      PLANE13=+Z

CSYS=PIER7
  271 J=271,371 SEC=M2               PLANE13=+Z
  371 J=371,471 SEC=M3               PLANE13=+Z
  471 J=471,571 SEC=M3,M5 EIVAR=1,3 PLANE13=+Z
  571 J=571,671 SEC=M5               PLANE13=+Z
  671 J=671,771 SEC=M6 IREL=R3      PLANE13=+Z
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CSYS=PIER8
  281 J=281,381 SEC=M2                PLANE13=+Z
  381 J=381,481 SEC=M3                PLANE13=+Z
  481 J=481,581 SEC=M3,M5 EIVAR=1,3 PLANE13=+Z
  581 J=581,681 SEC=M5                PLANE13=+Z
  681 J=681,781 SEC=M6  IREL=U2,R3    PLANE13=+Z


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

  NAME=TL
    TYPE=TEMPERATURE ELEM=FRAME
      ADD=701,704,1,781,10    T=10


MODES

  TYPE=EIGEN  N=36 ; 9 SPANS AND 4 MODES PER SPAN


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