

Appendix A

WORKED EXAMPLE USING THE GUIDELINES

This example is intended to illustrate the application of the guidelines. Each section of the Guidelines that applies is identified by a corresponding number in the example which is prefixed by the letter "A" for easy reference and identification. Comments are also included to assist in interpretation of the Guidelines and to clarify assumptions made relative to the structure idealization.

The state-of-the-art in seismic design of bridges has not yet progressed to the point where exact solutions are available. The number of significant figures used in the following example should not be interpreted as an exact theoretical answer or infer that the same number of significant figures be used in design. They are used to avoid confusion in the use of the Guidelines and also to provide comparative numbers for designers who choose to compare results using analytical techniques and algorithms available to them.

The bridge selected for the example is a three-span continuous box girder structure with dimensions and member properties as shown in Fig. A-1. Coordinate systems chosen for the overall structure and the columns are also shown in the figure. The coordinate axes for the individual superstructure members have directions corresponding to the overall structure coordinate system. The modulus of elasticity is assumed to be 3,000,000 psi. The bridge is assumed to be located in the highest seismic map area with an Acceleration Coefficient (A) of 0.40. Other assumptions pertinent to the example are identified in each of the appropriate sections.

A3.1 APPLICABILITY OF GUIDELINES

The three-span continuous box girder bridge having the alignment, dimensions, and member articulations shown in Fig. A-1, is within the range of applicability intended by the Guidelines.

A3.2 ACCELERATION COEFFICIENT

The bridge is assumed to be located within the 0.40 contour in Fig. 3 and for the purposes of this example will have an Acceleration Coefficient (A) equal to 0.40.

A3.3 IMPORTANCE CLASSIFICATION

Assume for the purposes of this example that the bridge is essential in terms of Social/Survival and Security/Defense requirements and is therefore assigned an Importance Classification (IC) of I.

A3.4 SEISMIC PERFORMANCE CATEGORY

For $A > 0.29$ and an IC equal to I, the Seismic Performance Category (SPC) is D as shown in Table 1 of the Guidelines.

A3.5 SITE EFFECTS

A Soil Profile Type II is assumed for the site which yields a Soil Profile Coefficient (S) of 1.2 as obtained from Table 2. Note that this Soil Profile is also used if information is not available on the soil properties and profile.

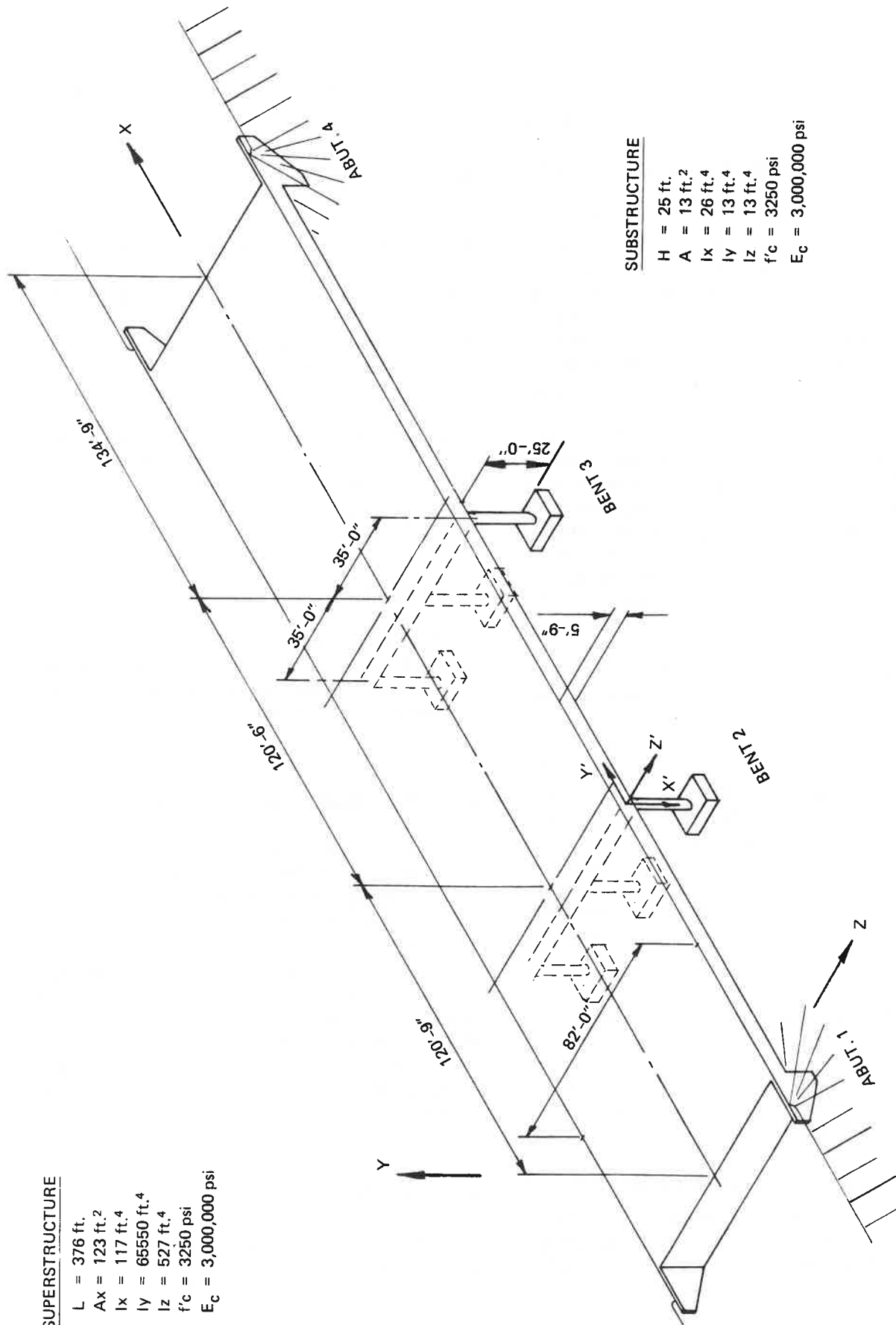
A3.6 RESPONSE MODIFICATION FACTOR

Substructure—The multiple column bent has a Response Modification Factor (R) of 5 for both orthogonal axes of the columns as shown in Table 3.

Connections—From Table 3 the R-Factor for the superstructure to abutment connection is 0.8. An R-Factor of 1.0 for the connection at the column to bent cap (i.e., at the superstructure soffit) and at the column to foundation are also given. For bridges classified as SPC D, however, the recommended design forces for connections are those corresponding to the maximum force capable of being developed by column hinging as described in Sec. 4.8.5(C), and these R-Factors are not used.

A4.1 GENERAL

The requirements of Chapter 4 shall control the selection of the seismic analysis and design procedures.



Note: Global coordinate axes [X, Y, Z] for the structure do not have to coincide with the local coordinate axes [X', Y', Z'] for the bents.

Figure A-1. Dimensions of example bridge.

A4.2 ANALYSIS PROCEDURE

The structure geometry and related stiffness variation falls within the range defined for a "regular bridge". As shown in Table 4 for a regular bridge with 2 or more spans classified as SPC D, Method 1 (Single Mode Spectral Method) is specified as the minimum required analysis procedure.

A4.3 DETERMINATION OF ELASTIC FORCES AND DISPLACEMENTS

Earthquake motions shall be directed along the longitudinal and transverse axes of the bridge. These are the global X and Z axes respectively, shown in Fig. A-1. Note that for other bridges the local Y' and Z' axes of the columns are not necessarily required to coincide with the longitudinal and transverse axes of the bridge. For a straight bridge with no skewed columns, piers or abutments, it is recommended, for simplicity of calculations, that the local Y' axis of the column or pier coincide with the longitudinal axis of the bridge as shown in this example.

Calculation of seismic forces resulting from the two earthquake motions is given in Sec. A5.3.

A4.4 COMBINATION OF ORTHOGONAL SEISMIC FORCES

Load Case 1 consists of 100% of forces from the longitudinal motion plus 30% of forces from the transverse motion.

Load Case 2 has 100% of forces from the transverse motion and 30% of forces from the longitudinal motion.

See Table A-4 for the combined forces and Sec. C4.4 of the Commentary for a more detailed description.

A5.3 SINGLE MODE SPECTRAL ANALYSIS METHOD—PROCEDURE 1

Longitudinal Earthquake Loading

STEP 1: Neglecting axial deformation in the deck and assuming that the deck behaves as a rigid member, the bridge may be idealized as shown in Fig. A-2. Note that the bridge is idealized so that the abutment does not contribute to the longitudinal stiffness. This was done for purposes of simplicity and in this case the resulting forces on the substructure are more conservative. To include the abutment stiffness see Sec. C5.4.2. Applying the assumed uniform longitudinal loading

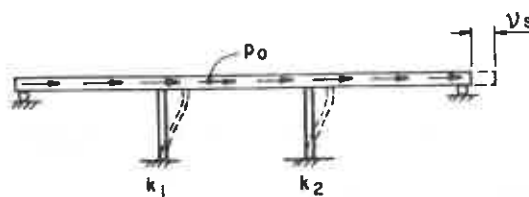


Figure A-2. Structural idealization and application of assumed uniform loading for longitudinal mode of vibration.

yields a constant displacement (i.e., $v_s(x) = v_s$) along the bridge. Assuming that the columns alone resist the longitudinal motion, the displacement is obtained by using a column stiffness of $12 EI/H^3$ in the longitudinal direction. Using the column properties included in Fig. A-1, the stiffness for Bents 2 and 3, denoted in Fig. A-2 as k_1 and k_2 , respectively, are calculated as:

$$k_1 = k_2 = 3 \frac{(12EI)}{H^3} = 3 \times \frac{3 \times 10^3 \times 44}{25^3} \times 13 = 12940 \text{ kips/ft}$$

which yields a displacement of:

$$v_s = \frac{P_0 L}{k_1 + k_2} = \frac{1 \times 376}{2 \times 12940} = 0.0145 \text{ ft}$$

STEP 2: Assuming a weight density of the superstructure at 165 lb/ft^3 , yields a dead weight per unit length of superstructure of: $w(x) = 0.165 Ax = 0.165(123) = 20.3 \text{ kips/ft}$. Note that this weight density is higher than plain concrete to include the weight of the upper half of the columns, the embedded column cap and intermediate diaphragms. The α , β , and γ factors are then calculated by evaluating the integrals in Eqs. 5-5, 5-6 and 5-7. For this case, both the dead weight per unit length of the superstructure, $w(x)$, and the displacement, $v_s(x)$, are constant thus simplifying the integration and yielding:

$$\alpha = \int_{\text{Abut. 1}}^{\text{Abut. 4}} v_s(x) dx = v_s L = 0.0145 \times 376 = 5.46 \text{ ft}^2$$

(See Fig. A-1 for location of abutments.)

$$\begin{aligned} \beta &= \int_{\text{Abut. 1}}^{\text{Abut. 4}} w(x) v_s(x) dx = w v_s L \\ &= 20.3 \times 0.0145 \times 376 = 110.9 \text{ kip-ft} \end{aligned}$$

$$\gamma = \int_{\text{Abut. 1}}^{\text{Abut. 4}} w(x)v_s(x)^2 dx = wv_s^2 L$$

$$= 20.3 \times (.0145)^2 \times 376 = 1.61 \text{ kip-ft}^2$$

STEP 3: Calculate the period, T , using Eq. 5-8.

$$T = 2\pi \sqrt{\frac{\gamma}{p_0 g \alpha}} = 2\pi \left[\frac{1.61}{1.0 \times 32.2 \times 5.46} \right]^{1/2}$$

$$= 0.60 \text{ sec.}$$

STEP 4: The elastic seismic response coefficient, C_s , is obtained from Eq. 5-1. Substituting for A , S and T yields:

$$C_s = \frac{1.2AS}{T^{2/3}} = \frac{1.2 \times 0.4 \times 1.2}{(0.60)^{2/3}} = 0.81$$

Since the seismic response coefficient does not exceed $2.5A$ ($2.5 \times 0.4 = 1.0$), use $C_s = 0.81$. The intensity of the seismic loading expressed by Eq. 5-9 is therefore:

$$P_e(x) = \frac{\beta C_s w(x) v_s(x)}{\gamma}$$

$$= \frac{110.9 \times 0.81 \times 20.3 \times 0.0145}{1.61}$$

$$= 16.45 \text{ kips/ft}$$

STEP 5: Apply the equivalent static loading as shown in Fig. A-3. The displacement of 0.239 ft and member forces for the longitudinal earthquake loading which are tabulated in Table A-1 are obtained as follows:

$$v_s = \frac{P_e(x) \cdot L}{k_1 + k_2} = \frac{16.45 \times 376}{2 \times 12960} = 0.239 \text{ ft}$$

$$V_{Y'}\text{-Shear per Column} = \frac{16.45 \times 376}{6} = 1030 \text{ kips}$$

$$M_{Z'}\text{-Moment per Column} = 1030 \times 12.5$$

$$= 12900 \text{ kip-ft}$$

Note that for this bridge $V_{Z'}$ and $M_{Y'Y'}$ are zero for the longitudinal earthquake motion.

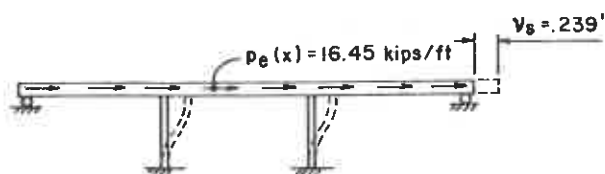
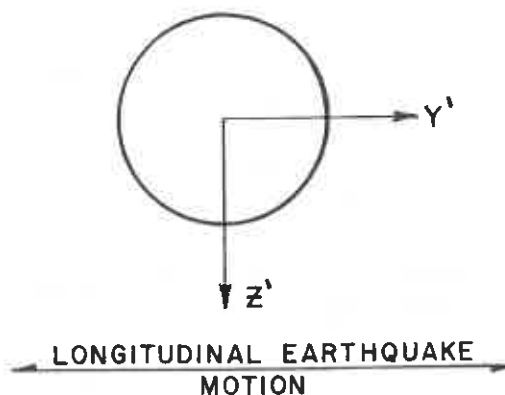


Figure A-3. Displacements and seismic loading intensity for longitudinal loading.

Transverse Earthquake Loading

STEP 1: Apply an assumed uniform transverse loading of 1 kip/ft to the bridge as shown in Fig. A-4. The resulting transverse displacements, $v_s(x)$, are tab-

TABLE A-1 Elastic and modified forces due to longitudinal earthquake motion



Location	$V_{Y'}$ Longit. Shear (kips)	$M_{Z'Z'}$ Longit. Moment (kip-ft)	$V_{Z'}$ Trans. Shear (kips)	$M_{Y'Y'}$ Trans. Moment (kip-ft)	$P_{X'}$ Axial Force (kips)
Abutment 1	0	0	0	0	106 ⁽³⁾
Bent 2 (per column)	1030	12900 (2580) ⁽²⁾	0	0	110
Bent 3 (per column)	1030	12900 (2580)	0	0	115
Abutment 4	0	0	0	0	92

(1) The local Y' and Z' axes of a column or pier do not necessarily have to coincide with the longitudinal and transverse axes of the bridge. However for a straight bridge with no skewed piers, columns or abutments it is recommended, for simplicity of calculations, that the local Y' axis of the column or pier does coincide with the longitudinal axis of the bridge as shown in this example.

(2) Reduced design earthquake forces as described in Sec. 4.8.1, for an R -Factor of 5. Note that shear and axial forces are excluded from reduction.

(3) The elastic axial forces at the abutments and bents are determined for the loading condition shown in Fig. A-3 using the moment distribution method and considering the flexibility of the superstructure.

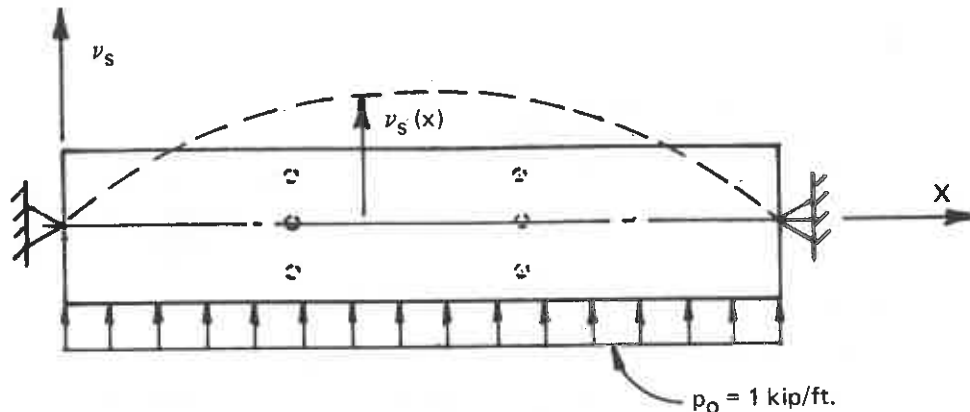


Figure A-4. Plan view of three span bridge subjected to assumed transverse loading.

ulated at the span $1/4$ points and shown in Table A-2. A computer program with space frame analysis capability was used for this portion of the example problem. Conventional methods of analyses can be used if desired. The transverse abutment stiffness may be included by using the approach outlined in Sec. C5.4.2.

STEP 2: Calculate the α , β , and γ factors by evaluating the integrals numerically in Eqs. 5-5, 5-6 and 5-7, respectively.

$$\alpha = \int_{\text{Abut. 1}}^{\text{Abut. 4}} v_s(x) dx = 1.21 \text{ ft}^2$$

$$\beta = \int_{\text{Abut. 1}}^{\text{Abut. 4}} w(x) v_s(x) dx = 24.5 \text{ kip-ft}$$

$$\gamma = \int_{\text{Abut. 1}}^{\text{Abut. 4}} w(x) v_s(x)^2 dx = 0.096 \text{ kip-ft}^2$$

STEP 3: Calculate the period, T , using Eq. 5-8.

$$T = 2\pi \sqrt{\frac{\gamma}{p_0 g \alpha}} = 2\pi \left[\frac{0.096}{1.0(32.2)(1.21)} \right]^{1/2} = 0.314 \text{ sec.}$$

STEP 4: The elastic response coefficient, C_s , is obtained from Eq. 5-1. Substituting for A , S and T yields:

$$C_s = 1.2 \frac{AS}{T^{2/3}} = \frac{1.2 \times 0.4 \times 1.2}{(0.314)^{2/3}} = 1.24$$

This is greater than $2.5A$, therefore use $C_s = 1.0$ as described in Sec. 5.2.1. The intensity of seismic loading, $p_e(x)$, is calculated using Eq. 5-9. Substituting for β , C_s , $w(x)$ and γ yields:

$$\begin{aligned} p_e(x) &= \frac{\beta C_s w(x) v_s(x)}{\gamma} \\ &= \frac{24.5 \times 1.0 \times 20.3}{0.096} v_s(x) \\ &= 5157 v_s(x) \text{ kips/ft}^2 \end{aligned}$$

Using this expression, the load intensity at the span $1/4$ points is computed and tabulated as shown in Table A-2.

STEP 5: Applying the equivalent static loading as shown in Fig. A-5 yields the member end forces due to the transverse earthquake loading shown in Table A-3. The member forces and displacements in this example were obtained using a computer program with space frame analysis capabilities. Conventional methods of analyses can be used if desired. Note that longitudinal moments and shears, ($M_{Z'Z'}$ and $V_{Y'}$), were generated by the transverse earthquake because of the eccentricity of the outer columns with respect to the longitudinal axis of the superstructure.

The transverse deck displacements are:

Bent 2	0.086 ft
Center Span 2	0.102 ft
Bent 3	0.092 ft

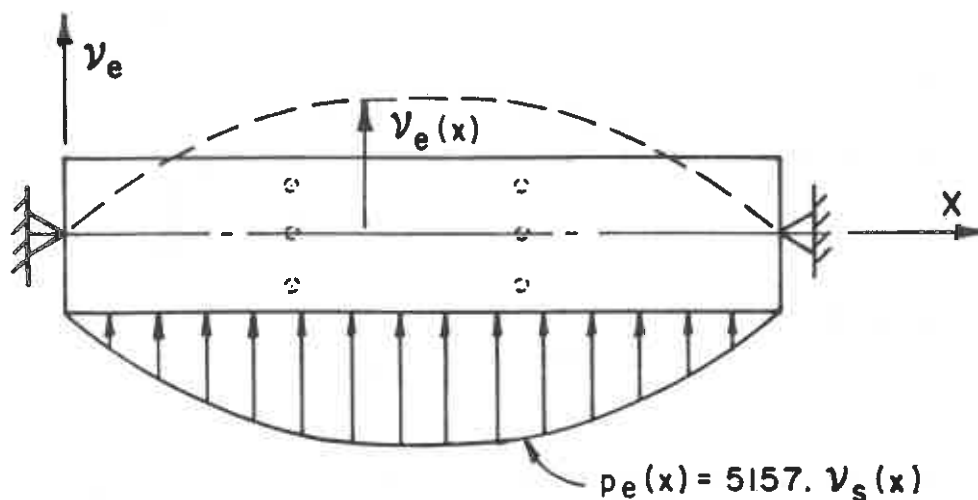


Figure A-5. Plan view of three span bridge subjected to equivalent static seismic loading.

TABLE A-2 Displacements and seismic loading intensity for transverse loading

Location	Displacements Due to Uniform Transverse Loading $v_s(x)$ (ft)	Seismic Loading Intensity $p_e(x)$ (kips/ft)
Abutment 1	0.0	0.0
Span 1 1/4	0.00129	6.66
Span 1 1/2	0.00248	12.77
Span 1 3/4	0.00348	17.94
Bent 2	0.00425	21.91
Span 2 1/4	0.00476	24.54
Span 2 1/2	0.00498	25.69
Span 2 3/4	0.00490	25.28
Bent 3	0.00453	23.37
Span 3 1/4	0.0038	19.58
Span 3 1/2	0.00275	14.18
Span 3 3/4	0.00145	7.47
Abutment 4	0.0	0.0

$$\alpha = \int v_s(x) dx = 1.21 \text{ ft}^2$$

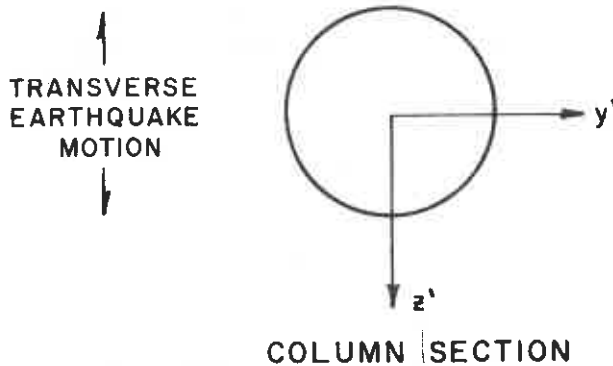
$$\beta = \int w(x) v_s(x) dx = 24.5 \text{ kip-ft}$$

$$\gamma = \int w(x) v_s(x)^2 dx = 0.0965 \text{ kip-ft}^2$$

$$T = 0.314 \text{ sec.}$$

$$p_e(x) = 5157 v_s(x) \text{ kips/ft}$$

TABLE A-3 Elastic and modified forces due to transverse earthquake motion



Location	$V_{Y'}$ Longit. Shear (kips)	$M_{Z'Z'}$ Longit. Moment (kip-ft)	$V_{Z'}$ Trans. Shear (kips)	$M_{Y'Y'}$ Trans. Moment (kip-ft)	$P_{X'}$ Axial Force (kips)
Abutment 1*	0	0	1826 (2283)**	0	0
Bent 2 (per column)	74	887 (177)	396	4757 (951)	205
Bent 3* (per column)	59	707 (141)	424	5089 (1018)	219
Abutment 4	0	0	1892 (2365)	0	0

*Use larger forces at Abutment 1 and Bent 3 for design.

**Reduced design earthquake forces described in Sec. 4.8.1, for an R-Factor of 0.8 at the abutment and 5 for the columns. Note that the column shear and axial forces are not reduced.

A4.8 DESIGN FORCES FOR SEISMIC PERFORMANCE CATEGORIES C AND D

There are two sets of forces to be determined for ductile members capable of forming plastic hinges. The first set determined for the preliminary design of the columns is described in Sec. 4.8.1 and entitled "Modified Design Forces". The second set is used to refine further the design of the column and the various components connected to the columns as described in Sec. 4.8.2 entitled "Forces Resulting from Plastic Hinging in the Columns, Piers or Bents".

A4.8.1 Modified Design Forces

These forces shall be determined in the same way as for Seismic Performance Category B with the exception of the treatment of axial forces, thus reference is made to Sec. 4.8.1.

A4.7.1 Design Forces for Structural Members and Connections

The structural members and connections specified in Sec. 4.8.1 which are applicable to this example are the column members and the abutment shear keys. For design purposes the largest shear and bending forces, which occur at Abutment 1 and Bent 3 as determined from the analyses, were used for each of the load cases tabulated in Table A-4. Member dead load forces are shown in Table A-5 for the critical column in Bent 3 and Abutment 1.

Assume that the earth pressure, buoyancy and stream flow are equal to zero. Using Eq. 4-1, the dead load forces tabulated in Table A-5, and the maximum

TABLE A-4 Maximum seismic forces and moments for load cases 1 and 2

Component	Load Case 1 (1.0 Long. + 0.3 Trans.)	Load Case 2 (1.0 Trans. + 0.3 Long.)
Abutments		
$V_{Z'Z'}$ -Shear	685 kips	2283 kips
$P_{X'X'}$ -Axial Force	± 106 kips*	± 32 kips
Bents		
$V_{Y'Y'}$ -Shear	$(1030 + 18)$ $= 1048$ kips	$(59 + 309) = 368$ kips
$M_{Z'Z'}$ -Moment	$(2580 + 42)$ $= 2622$ kip-ft	$(141 + 774)$ $= 915$ kip-ft
$P_{X'X'}$ -Axial Force	$\pm(115 + 66)$ $= \pm 181$ kips	$\pm(219 + 35)$ $= \pm 254$ kips
$V_{Z'Z'}$ -Shear	$(0 + 127) = 127$ kips	$(424 + 0) = 424$ kips
$M_{Y'Y'}$ -Moment	$(0 + 305) = 305$ kip-ft	$(1018 + 0) = 1018$ kip-ft

*The axial (i.e., vertical) forces shown are for Abutment 1 and Bent 3 and were determined using the moment distribution method as previously stated.

TABLE A-5 Dead load forces

Component	Column (Bent 3)	Abutment 1
$V_{Y'Y'}$ -Shear	69 kips	0
$M_{Z'Z'}$ -Moment	1170 kip-ft	0
$P_{X'X'}$ -Axial Force	960 kips	624
$V_{Z'Z'}$ -Shear	0	0
$M_{Y'Y'}$ -Moment	0	0

seismic forces, the modified design forces are computed as follows:

Modified Design Forces—Column

By inspection Load Case 1 controls:

$$\begin{aligned} V_{Y'Y'}\text{-Shear} &= 1.0(D + B + SF + E + EQM) \\ &= 1.0(69 + 1048) = 1117 \text{ kips} \end{aligned}$$

$$M_{Z'Z'}\text{-Moment} = 1.0(1170 + 2622) = 3792 \text{ kip-ft}$$

$$\begin{aligned} P_{X'X'}\text{-Axial} &= 1.0(960 \pm 181) \\ &= 779 \text{ or } 1141 \text{ kips} \end{aligned}$$

$$V_{Z'Z'}\text{-Shear} = 1.0(127 + 0) = 127 \text{ kips}$$

$$M_{Y'Y'}\text{-Moment} = 1.0(305 + 0) = 305 \text{ kips}$$

Thus for a circular column, the modified design moment is:

$$M = \sqrt{M_{Z'Z'}^2 + M_{Y'Y'}^2} = 3804 \text{ kip-ft.}$$

Modified Design Forces—Abutment

By inspection Load Case 2 controls:

$$\begin{aligned} V_{Z'Z'}\text{-Shear} &= 1.0(D + B + SF + E + EQM) \\ &= 1.0(0 + 2283) = 2283 \text{ kips.} \end{aligned}$$

Thus the shear keys at the abutment must resist a modified design transverse shear force of 2283 kips. After the modified design forces are calculated the preliminary design of the column, as described in Chapter 8 of the Guidelines, can proceed.

A8.4 SEISMIC PERFORMANCE CATEGORIES C AND D

A8.4.1 Column Requirements

A column is defined by a ratio of the clear height to maximum plan dimension equal to or greater than 2-1/2. For this example, the vertical support has a clear height of approximately 22 ft and a width of 4.0 ft yielding a ratio of 5.5 and thus is classified as a column.

A8.4.1(A) Vertical Reinforcement

The vertical reinforcement shall not be less than 0.01 or more than 0.06 times the gross area. A ratio not exceeding 0.04 is recommended to minimize placing and congestion problems at splices.

A8.4.1(B) Flexural Strength

The modified design forces determined in Sec. 4.8.1 are used for the preliminary column design. Considering both the minimum and maximum axial loads the design loads are:

$$P = 779 \text{ kips}, M = 3804 \text{ kip-ft}$$

$$P = 1141 \text{ kips}, M = 3804 \text{ kip-ft}$$

The magnification of moment due to slenderness effects is specified in AASHTO Art. 8.16.5.2 for compression members not braced against sidesway. As

specified the effects of slenderness may be neglected when kl_u/r is less than 22. For these columns, kl_u/r is slightly greater than 22 and thus slenderness should theoretically be considered. For the purpose of simplicity, however, it has been ignored in this example problem.

Using the appropriate strength reduction factors and the design loads given above, the column design requires 50 #11 bars of reinforcing steel. This yields a reinforcement ratio of 0.043 for the longitudinal reinforcement which is within the specified limits. A column ultimate capacity interaction diagram along with the reduced design capacity curve is shown in Fig. A-6. The controlling design moment of 3804 kip-ft and axial load are also shown plotted in the figure. The darkened vertical bar indicates the range of axial loads.

A4.8.2 Forces Resulting from Plastic Hinges in Columns, Piers or Bents

Using the preliminary design of the column, the forces resulting from plastic hinging may be calculated.

Bents With Two or More Columns

The forces resulting from plastic hinging in the plane of the bent are calculated using the procedures outlined in Table A-6. The column overstrength plastic moment capacity is included on the interaction diagram shown in Fig. A-6.

TABLE A-6. Calculation of forces resulting from plastic hinging in columns (4.8.2B)

Diagram of a bent with three columns. The left column is 38' high, the center column is 22' high, and the right column is 25' high. The center of mass is indicated at the top. The diagram shows plastic hinges at the base of each column with moments $1.3 M_p$ and shear forces V_L , V_C , and V_R . The axial forces are $P - \Delta p$ and $P + \Delta p$.

	1.3 × Mp			Column Shear Forces					Column Axial Forces			
Step	Left	(kip-ft) Center	Right	Left	Center	Right	Total	P	Left	(kips) Center	Right	% Difference*
1	7800	7800	7800						960	960	960	
2				709	709	709	2127		960	960	960	
3								425	535	960	1385	
4	7600	7800	7900	691	709	718	2118		536	960	1384	
5								424	536	960	1384	0.2

*Maximum shear force for the bent must be within 10% of previous value as described in Section 4.8.2.

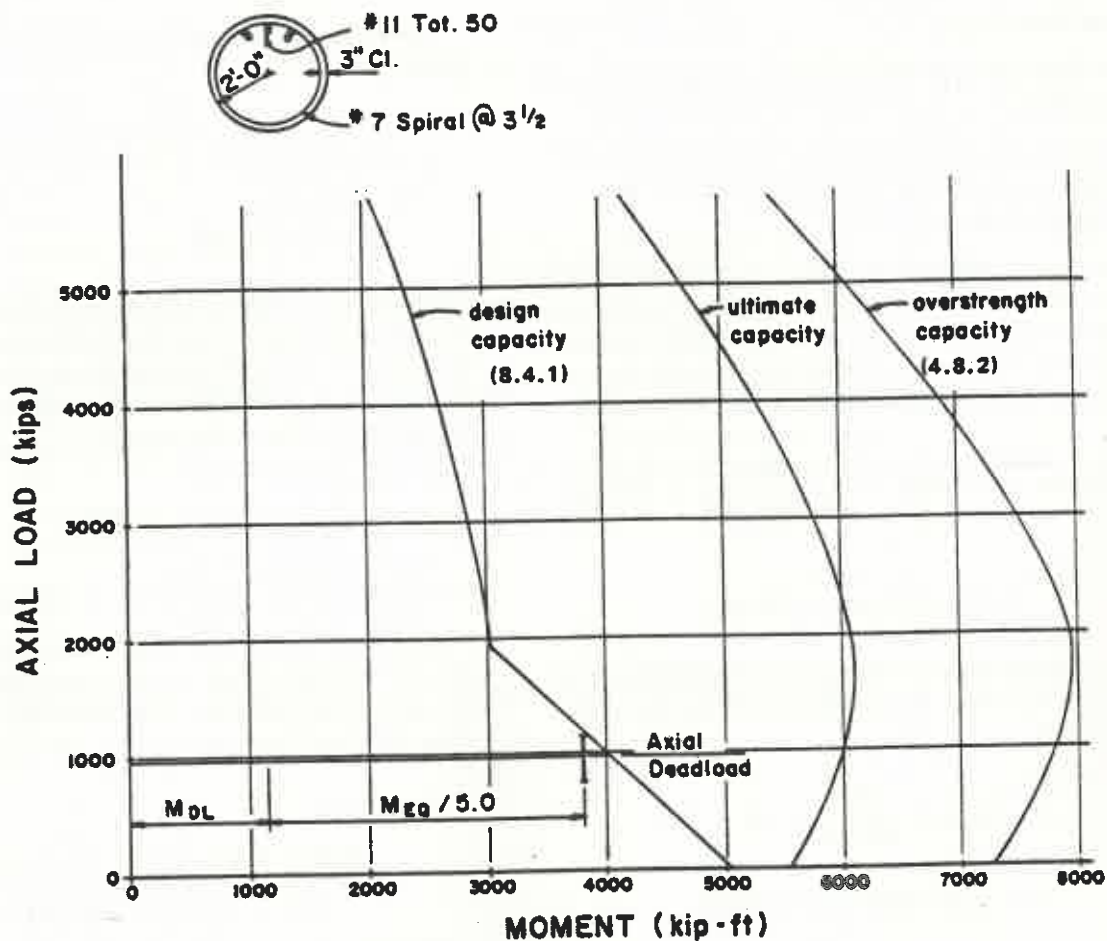


Figure A-6. Column interaction diagram.

A4.8.3 Column and Pile Bent Design Forces

Moment: 3804 kip ft

Axial Force:

Elastic 960 ± 181 kips

Plastic Hinging 960 ± 424 kips

Shear:

Elastic $\sqrt{1048^2 + 127^2} = 1056$ kips

Plastic Hinging 718 kips

A8.4.1(C) Column Shear and Transverse Reinforcement

The factored (i.e., plastic hinging) design shear force, V_u , obtained in Sec. A4.8.3 is 718 kips. Using the strength reduction factor for shear specified in Sec. 8.16 of AASHTO and Eq. 8-46 of Art. 8.16.6 the factored shear stress for a circular column is

$$v_u = \frac{V_u}{\phi b_w d} = \frac{718}{0.85 \times 48 \times 43} = 409 \text{ psi.}$$

The shear stress carried by the concrete outside the column end regions is (see AASHTO Art. 8.16.6.2).

$$v_c = 2\sqrt{f'_c} = 114 \text{ psi.}$$

Using Eq. 8-50 of AASHTO Art. 8.16.6.3 and the values calculated above for the factored shear stress and the shear stress carried by concrete, the total shear reinforcement A_v is

$$A_v = \frac{(v_u - v_c)}{f'_y} b_w s = \frac{(409 - 114)}{60,000} 48 \times 3.5$$

$$= 0.83 \text{ in}^2 \text{ total area required}$$

$$\text{or } \frac{0.83}{2} \text{ in}^2 = 0.41 \text{ in}^2 \text{ per leg}$$

Therefore, a #6 spiral at 3 1/2 in. pitch should be used outside the column end region.

Column End Region

The dimensions of the column end region are given by the larger of:

1. Maximum cross-section dimension, $d = 4.0$ ft
2. One-sixth of clear height, $22/6 = 3.67$ ft
3. Eighteen inches

The column cross-section dimension of 4.0 ft is the largest and should be used as the length of the top and bottom end regions. If the minimum axial compression stress is less than $0.1f'_c$ then the concrete shear resistance in the end regions shall be neglected. Since

$$\text{Minimum axial stress} = \frac{536}{12.57 \times 144} = 296 \text{ psi}$$

and

$$0.1f'_c = 325 \text{ psi} > 296 \text{ psi},$$

the shear stress taken by the concrete is assumed to be zero. This will yield shear reinforcement, A_v , in the end areas of:

$$A_v = \frac{v_u b_w S}{f_y} = \frac{394}{60,000} \times 48 \times 3.5$$

$$= 1.1 \text{ in.}^2 \text{ total area required.}$$

$$\text{or } \frac{1.1}{2} \text{ in.}^2 = 0.55 \text{ in.}^2 \text{ per leg}$$

Thus, a #7 spiral with a $3\frac{1}{2}$ in. pitch in the 4 ft-0 in. end regions at top and bottom of columns should be used.

A.8.4.1(D) Transverse Reinforcement for Confinement at Plastic Hinges

a) The volumetric ratio of spiral reinforcement is the greater value given by Eq. 8-1 or Eq. 8-2. Therefore,

$$\begin{aligned} \rho_s &= 0.45 \left(\frac{A_1}{A_c} - 1 \right) \frac{f'_c}{f_{yh}} \\ &= 0.45 \left(\frac{12.57}{9.62} - 1 \right) \frac{3250}{60,000} = 0.0075 \end{aligned}$$

or

$$\rho_s = 0.12 \frac{f'_c}{f_{yh}} = \frac{0.12 \times 3250}{60,000} = 0.0065$$

The cross-sectional area of a spiral at $3\frac{1}{2}$ in. pitch is given by:

$$\begin{aligned} A_{sp} &= \frac{\rho_s s d_s}{4} = \frac{0.0075 \times 3.5 \times 41.25}{4} \\ &= 0.270 \text{ in.}^2 \end{aligned}$$

Since this is less than the shear reinforcement, there is no additional requirement for confinement at the plastic hinges; thus use #7 spiral at $3\frac{1}{2}$ in. the 4 ft-0 in. end regions and #6 spiral at $3\frac{1}{2}$ in. throughout the remaining center portion of the column.

A4.8.5 Connection Design Forces**A4.8.5(B) Hold-Down Forces at Abutments**

Hold-down devices are required if the upward reaction due to longitudinal seismic forces exceeds 50% of the dead load reaction. The following calculations show that hold-down devices are not required.

Abutment 1

$$0.5DL = 0.5 \times 624 = 312 \text{ kips}$$

$$312 > 106 \text{ None Required}$$

Abutment 4

$$0.5 \times 701 = 350 \text{ kips}$$

$$350 > 92 \text{ None Required}$$

A4.8.5(C) Column and Pier Connection Design Forces

The following design forces which result from plastic hinging shall be used to design the column connections at the bent cap and the column footings.

Min	Axial	536 kips
	Shear	691 kips
	Moment	7600 kip-ft
Max	Axial	1384 kips
	Shear	718 kips
	Moment	7900 kip-ft

A4.8.6 Foundation Design Forces

The following design forces which result from plastic hinging shall be used to design the foundations. Foundation dead load should be added to these forces.

Min	Axial*	536 kips
	Shear	691 kips
	Moment	7600 kip-ft
Max	Axial*	1384 kips
	Shear	718 kips
	Moment	7900 kip-ft

A4.8.7 Abutment and Retaining Wall Design Force

The design forces at the abutment are:

$$\begin{aligned} \text{Axial-bearings } 701 + 92 &= 793 \text{ kips} \\ \text{Shear-keys} &= 2283 \text{ kips} \end{aligned}$$

*The foundation dead load should be added to these forces.

A4.9 DESIGN DISPLACEMENTS**A4.9.3 Seismic Performance Categories C and D**

The longitudinal displacement at the abutment due to the longitudinal earthquake loading was calculated in Step 5 of Sec. A.5.3 and is

$$N = 0.239 \text{ ft} = 2.9 \text{ in.}$$

The minimum support length at the abutment bearing seat is calculated from Eq. 4-4 as follows:

$$\begin{aligned} N &= 12 + 0.03L + 0.12H \\ &= 12 + 0.03 \times 376 + 0.12 \times 25 \\ &= 26 \text{ in.} \end{aligned}$$

Thus the support length at the abutments shall be 26 in.

