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Seismic Design Guidelines for Highway Bridges

Applied Technology Council
Berkeley, CA

Prepared for

Federal Highway Administration
Washington, DC

Oct 81
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SEISMIC DESIGN GUIDELINES FOR HIGHWAY BRIDGES

October 1981
Final Report

Earthquake Protection
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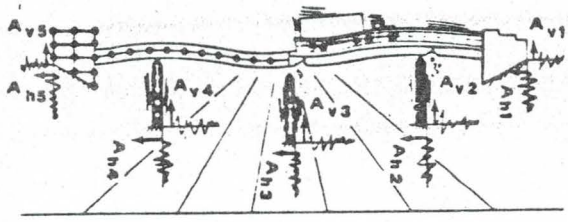

Transportation Structures

Prepared for

U.S. Department of Transportation
Federal Highway Administration
Offices of Research & Development
Structures and Applied Mechanics Division
Washington, D.C. 20590

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Presented in this report are guidelines for use in the seismic design of typical highway bridges which will be of interest to highway bridge engineers, structural engineers, and researchers. This document was prepared by the Applied Technology Council, Berkeley, California, for the Federal Highway Administration (FHWA), Office of Research, under contract DOT-FH-11-9295. Earthquake engineering research has been included in the FHWA Federally Coordinated Program of Highway Research and Development as Task 1 of Project 5A, "Improved Protection Against Natural Hazards of Earthquake and Wind."

The guidelines are formulated and based on both the observed performance of bridges during past earthquakes and on recent research conducted in the United States and abroad. They are the result of input provided by a distinguished group of academicians, designers, and highway bridge engineers and are applicable for use in all parts of the country.

Sufficient copies of the report are being distributed to provide a minimum of five copies to each FHWA regional office, division office, and State highway agency. Direct distribution is being made to the division offices.

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

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16. Abstract

This document contains Guidelines for the seismic design of highway bridges. The Guidelines are the recommendations of a team of nationally recognized experts which included consulting engineers, academicians, State highway, and Federal agency representatives from throughout the United States.

The Guidelines are comprehensive in nature and they embody several new concepts which are significant departures from existing design provisions. An extensive commentary documenting the basis for the Guidelines and an example demonstrating their use are included. A draft of the Guidelines was used to seismically redesign twenty-one bridges. Significant changes resulted from the redesigns and are reflected in this report. A summary of the redesigns is included.

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PREFACE

This document, prepared by Applied Technology Council, contains guidelines for the seismic design of highway bridges. The guidelines are the recommendations of a team of nationally recognized experts, composed of consulting engineers, academicians, state highway engineers and federal agency representatives from throughout the United States.

The guidelines are comprehensive in nature and embody several new concepts which are significant departures from existing design provisions. An extensive commentary documenting the basis for the guidelines and an example illustrating their use are included. The third draft of the Guidelines was used for the seismic redesign of twenty-one bridges in order to assess the practicability and cost impact of the Guidelines. The redesigns were performed by four consulting firms and five state highway departments. A number of significant changes resulted from the redesigns. A discussion of the changes and a summary of the redesign results are included as an appendix to the Guidelines.

The document represents a consensus of the project participants. It will be submitted to the American Association of State Highway and Transportation Officials in 1981 for possible adoption in their Standard Specifications for Highway Bridges.

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CHAPTER 1

INTRODUCTION

1.1 PURPOSE

These Guidelines establish design and construction provisions for bridges to minimize their susceptibility to damage from earthquakes.

The design earthquake motions and forces specified in these provisions are based on a low probability of their being exceeded during the normal life expectancy of a bridge.[1] Bridges and their components that are designed to resist these forces and that are constructed in accordance with the design details contained in the provisions may suffer damage, but should have low probability of collapse due to seismically induced ground shaking.

The principles used for the development of the provisions are:

1. Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage.

2. Realistic seismic ground motion intensities and forces are used in the design procedures.

3. Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge. Where possible, damage that does occur should be readily detectable and accessible for inspection and repair.

A basic premise in developing these seismic design guidelines was that they be applicable to all parts of the United States. The seismic risk varies from very small to high across the country. Therefore, for purposes of design, four Seismic Performance Categories (SPC) are defined on the basis of an Acceleration Coefficient (A) for the site, determined from the map provided, and the Importance Classification (IC). Different degrees of complexity and sophistication of seismic analysis and design are specified for each of the four Seismic Performance Categories.

An essential bridge must be designed to function during and after an earthquake. In areas with an Acceleration Coefficient greater than 0.29

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[1] The probability of the elastic design force levels not being exceeded in 50 years is in the range of 80 to 95%. However, the design earthquake force level by itself does not determine risk; the risk is also affected by the design rules and analysis procedures used in connection with the design ground motion. See Commentary for a more detailed discussion.

essential bridges must meet additional requirements. A bridge is designated essential on the basis of Social/Survival and Security/Defense classifications presented in the Commentary.

1.2 BACKGROUND

The 1971 San Fernando earthquake was a major turning point in the development of seismic design criteria for bridges in the United States. Prior to 1971 the American Association of State Highway and Transportation Officials (AASHTO) specifications for the seismic design of bridges were based in part on the lateral force requirements for buildings developed by the Structural Engineers Association of California. In 1973 the California Department of Transportation introduced new seismic design criteria for bridges, which included the relationship of the site to active faults, the seismic response of the soils at the site and the dynamic response characteristics of the bridge. In 1975 AASHTO adopted Interim Specifications which were a slightly modified version of the 1973 CalTrans provisions, and made them applicable to all regions of the United States. In addition to these code changes the 1971 San Fernando earthquake stimulated research activity on seismic problems related to bridges. In light of new research findings the Federal Highway Administration awarded a contract to Applied Technology Council (ATC) to:

- Evaluate current criteria used for seismic design of highway bridges.
- Review recent seismic research findings for design applicability and use in new specifications.
- Develop new and improved seismic design guidelines for highway bridges applicable to all regions of the United States.
- Evaluate the impact of these guidelines and modify them as appropriate.

In order to facilitate these objectives a "Workshop on Earthquake Resistance of Highway Bridges" was conducted by ATC in San Diego in January 1979. The workshop considered current state-of-the-art and practice, problem areas in seismic design and current research efforts and findings. The findings and recommendations of the workshop were considered in the development of these seismic design Guidelines.

1.3 BASIC CONCEPTS

Development of the Guidelines has been predicated on the following basic concepts.

- Hazard to life be minimized.

- Bridges may suffer damage but have low probability of collapse due to earthquake motions.

- Function of essential bridges be maintained.

- Design ground motions have low probability of being exceeded during normal lifetime of bridge.

- Provisions be applicable to all of the United States.

- Ingenuity of design not be restricted.

1.4 PROJECT ORGANIZATION

It was considered essential in the development of these Guidelines that representative segments of the bridge design and construction profession be involved. To ensure representative input and adequate consideration of the many factors involved, a Project Engineering Panel (PEP) comprised of the following was assembled:

- Four AASHTO representatives—California (Mr. James Gates), Idaho (Mr. Robert Jarvis), New York (Mr. Edward Hourigan), and Oklahoma (Mr. Veldo Goins).

- Four private design firm representatives—California (Mr. James Libby and Dr. Geoffrey Martin), New York (Mr. Gerard Fox), and Pennsylvania (Mr. Robert Kealey).

- Three university researchers--California (Professor Joseph Penzien), Illinois (Professor William Hall), and New Jersey (Professor Robert Scanlan).

- Two FHWA representatives (Mr. James Cooper and Dr. Walter Podolny).

- An ATC Board technical representative (Mr. Joseph Nicoletti).

- Two ATC staff--Project Manager (Mr. Roland Sharpe) and Project Technical Director (Dr. Ronald Mayes).

- Subcontractors (Mr. Roy Imbsen and Dr. David Elms).

See Appendix C for a list of the PEP, Subcontractors and their affiliations.

The work was conducted in several phases:

- Review of 1975 AASHTO Interim Specifications, current specifications in other countries, and current research findings.

- Development of draft seismic design guidelines.

- Redesign studies of bridges using draft guidelines.
- Assessment and evaluation of the redesign studies.
- Modification of design guidelines as appropriate.

1.5 QUALITY ASSURANCE REQUIREMENTS

Structural failures which have occurred during earthquakes and are directly traceable to poor quality control during construction are innumerable. The literature is replete with reports pointing out that collapse may have been prevented had proper inspection been exercised. To provide adequate seismic quality assurance requirements the engineer specifies the quality assurance requirements, the contractor exercises the control to achieve the desired quality and the owner monitors the construction process through special inspection. It is essential that each party recognizes its responsibilities, understands the procedures and has the capability to carry them out. Because the contractor does the work and exercises quality control it is essential that the inspection be performed by someone approved by the owner and not the contractor's direct employee.

In recognition of the fact that responsibility must be coordinated during construction, the PEP examined the responsibility of each party in the current AASHTO specifications. The PEP found the quality assurance requirements of the AASHTO specifications adequate to cover seismic as well as other design requirements. Therefore no special quality assurance requirements are included in these Guidelines.

1.6 FLOW CHARTS AND AN EXAMPLE FOR USE OF GUIDELINES

Flow charts outlining the steps in the seismic design procedures are given in Figures 1 and 2. The Commentary provides background information to assist the user in understanding the intent of the Guidelines; an example is given in Appendix A.

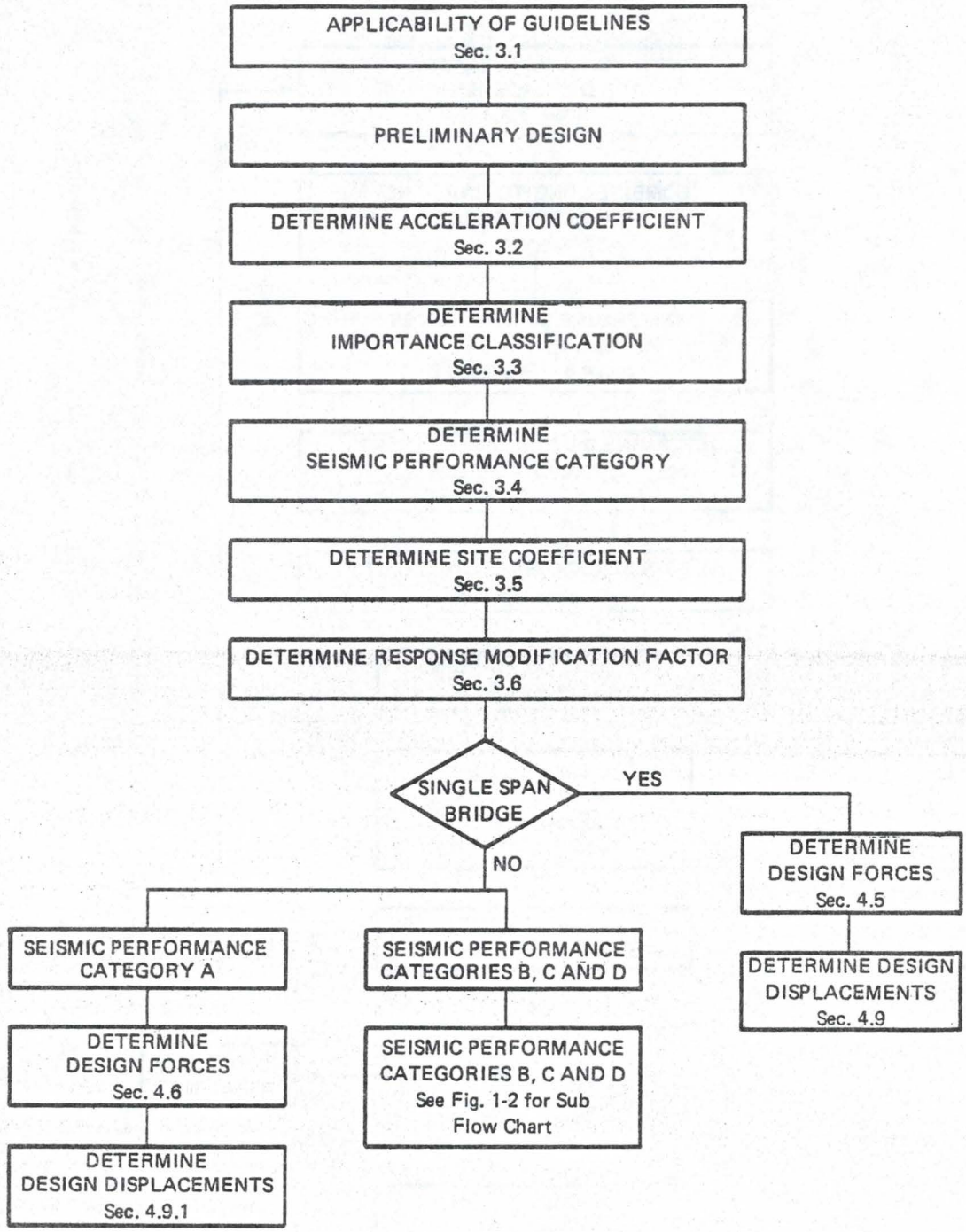

FIGURE 1. DESIGN PROCEDURE FLOW CHART

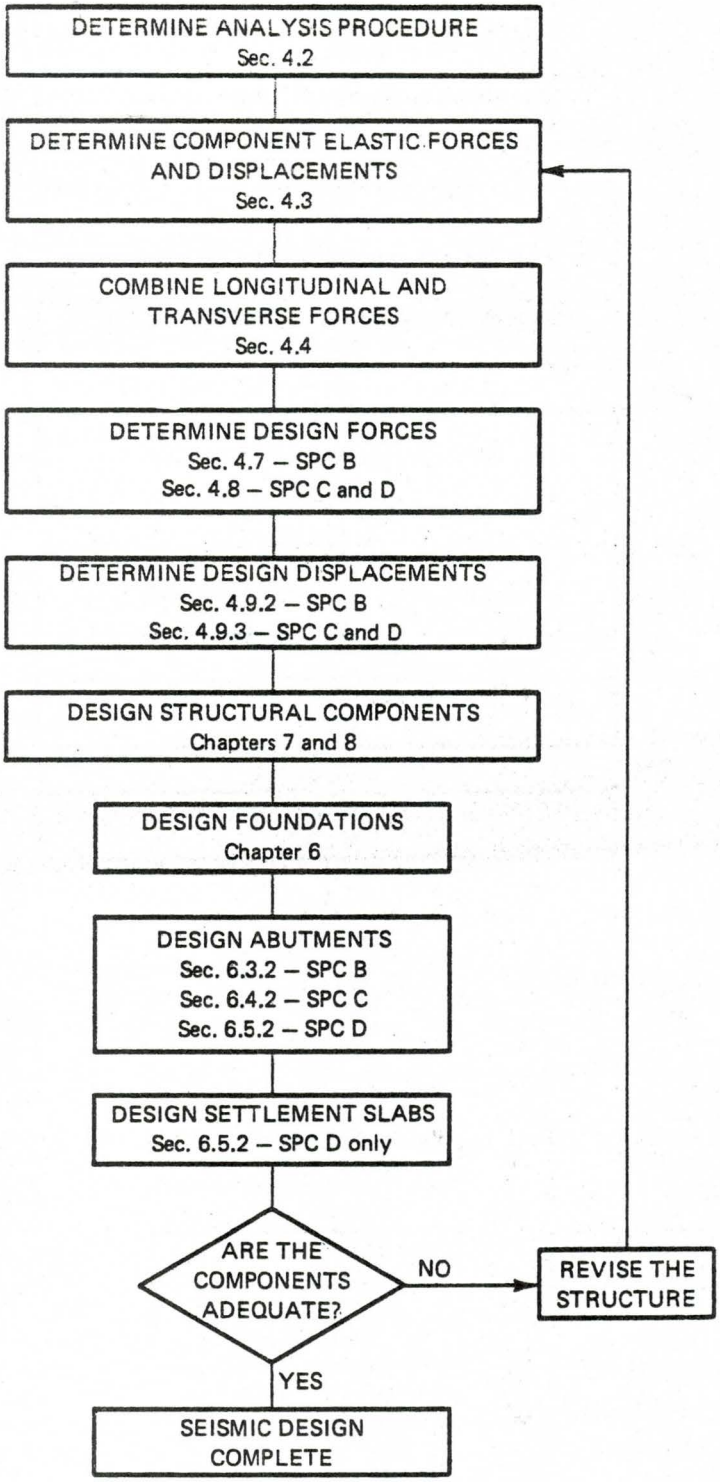

FIGURE 2. SUB-FLOW CHART FOR SEISMIC PERFORMANCE CATEGORIES B, C, AND D

CHAPTER 2

SYMBOLS AND DEFINITIONS

The following symbols and definitions apply to these Guidelines:

a = Vertical spacing of transverse reinforcement (hoops or stirrups) in rectangular reinforced concrete columns (in. or mm)

A = Acceleration coefficient determined in Sec. 3.2 (dimensionless)

A_c = Area of reinforced concrete column core (in.^{2} or mm^2)

A_g = Gross area of reinforced concrete column (in.^{2} or mm^2)

A_s = Area of longitudinal reinforcement in a concrete pile (in.^{2} or mm^2)

A_{sh} = Total cross-sectional area of transverse reinforcement (hoops or strirrups) used in rectangular reinforced concrete columns (in.^{2} or mm^2)

A_{vf} = Total amount of reinforcement normal to a construction joint (in.^{2} or mm^2)

B = Loads resulting from buoyancy forces and used in the group load combinations of Eqs. 4-1 and 4-2

C_m = Coefficient used in steel design to account for boundary conditions (dimensionless)

C_s = Elastic seismic response coefficient defined in Sec. 5.2.1 (dimensionless)

C_{sm} = Elastic seismic response spectrum defined in Sec. 5.2.2 (dimensionless)

d = Diameter of a reinforced concrete column (in.^{2} or mm^2)

D = Loads resulting from dead load and used in the group load combinations of Eqs. 4-1 and 4-2

E = Loads resulting from earth pressure and used in the group load combinations of Eqs. 4-1 and 4-2

EQF = Modified foundation seismic forces used in the group load combination of Eq. 4-2 and defined in Sec. 4.7.2

EQM = Modified seismic forces used in the group load combination of Eq. 4-1 and defined in Sec. 4.7.1

SYMBOLS AND DEFINITIONS (Continued)

f'_c = Specified compressive strength of reinforced concrete (psi or MPa)

f_y = Yield strength of reinforcement in reinforced concrete members (psi or MPa)

f_{yh} = Yield strength of transverse reinforcement (psi or MPa)

F_a = Axial stress in steel design that would be permitted if axial force alone existed (psi or Mpa)

F_{cr} = Buckling stress for load factor steel design (psi or Mpa)

F_e = Euler buckling stress in the plane of bending (psi or Mpa)

F'_e = Euler buckling stress for service load steel design (psi or Mpa)

F_y = Yield strength of structural steel (psi or MPa)

g = Acceleration of gravity (in./sec^2 or cm/sec^2)

h_c = Core dimension of a rectangular reinforced concrete column (in. or mm)

H = Height of a column or pier defined in Sec. 4.9 (ft or m)

IC = Importance Classification given in Sec. 3.3 (dimensionless)

K = Effective length factor used in steel design and given in Sec. 7.3 (dimensionless)

k_h = Seismic coefficient used to calculate lateral earth pressures and defined in Sec. 6.3.2 (dimensionless)

L = Length of bridge deck defined in Sec. 4.9 (ft or m)

N = Minimum support length for girders specified in Sec. 4.9 (in. or mm)

$p_e(x)$ = Intensity of the equivalent static seismic loading applied to represent the primary mode of vibration in Sec 5.3 (force/unit length)

P_n = Minimum axial load specified in Sec. 4.8.3 for columns and 4.8.4 for piers (lb or N)

p_o = Assumed uniform loading used to calculate the period in Sec. 5.3 (force/unit length)

P_u = Maximum strength of concentrically loaded steel columns (lb or N)

SYMBOLS AND DEFINITIONS (Continued)

R = Response modification factor specified in Sec. 3.6 (dimensionless)

s = Spacing of spiral reinforcement in reinforced concrete columns (in. or mm)

S = Site coefficient specified in Sec. 3.5.1 (dimensionless)

SF = Loads resulting from stream flow forces and used in the group load combinations of Eqs. 4-1 and 4-2

SPC = Seismic Performance Category specified in Sec. 3.4 (dimensionless)

T = Fundamental period of the bridge determined in Sec. 5.3 (sec.)

T_m = Period of the mth mode of vibration of a bridge (sec.)

V_j = Limiting shear force across a construction joint (lb or N)

v_u = Shear stress (psi or MPa)

V_u = Shear force (lb or N)

$v_s(x)$, $v_e(x)$ = Static displacement profiles resulting from applied loads p_o and p_e, respectively, and used in Sec. 5.3 (in. or mm)

$w(x)$ = Dead weight of the bridge superstructure and tributary substructure per unit length (force/unit length)

ρ_h = The ratio of horizontal shear reinforcement area to gross concrete area of a vertical section - Sec. 8.4.2 (dimensionless)

ρ_n = The ratio of vertical shear reinforcement area to the gross concrete area of a horizontal section - Sec. 8.4.2 (dimensionless)

ρ_s = Volumetric ratio of spiral reinforcement for a circular column (dimensionless)

ϕ = Strength reduction factor (dimensionless)

α = Coefficient used to calculate the period of the bridge in Sec. 5.3 (length^2)

β = Coefficient used to calculate the period of the bridge in Sec. 5.3 (force·length)

γ = Coefficient used to calculate the period of the bridge in Sec. 5.3 (force·length^2)

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CHAPTER 3

GENERAL REQUIREMENTS

3.1 APPLICABILITY OF GUIDELINES

These Guidelines are for the design and construction of new bridges to resist the effect of earthquake motions. The provisions apply to bridges of conventional steel and concrete girder and box girder construction with spans not exceeding 500 ft (152.4 m). Suspension bridges, cable-stayed bridges, arch type and movable bridges are not covered by these Guidelines but general considerations for designing such bridges are presented in the Commentary. Seismic design is usually not required for buried type (culvert) bridges.

The provisions specified in these Guidelines are minimum requirements. The Acceleration Coefficient (A) for a bridge site located within a 0.40 contour of Figs. 3 and 4 has a minimum value of 0.40. It may, however, be higher especially if the site is located near an active fault. It is recommended that a qualified professional be consulted to determine an appropriate value for the Acceleration Coefficient for sites located within the 0.40 contours of Figs. 3 and 4.

No detailed seismic analysis is required for any single span bridge or for any bridge in Seismic Performance Category A. For both single span bridges (Sec. 4.5) and bridges classified as SPC A (Sec. 4.6) the connections must be designed for specified forces and must also meet minimum support length requirements (Sec. 4.9).

3.2 ACCELERATION COEFFICIENT

The coefficient A to be used in the application of these provisions shall be determined from the contour maps of Figs. 3 and 4. (Note: An enlarged version of Fig. 3 is given in the back cover of this report.)

3.3 IMPORTANCE CLASSIFICATION

An Importance Classification (IC) shall be assigned for all bridges with an Acceleration Coefficient greater than 0.29 for the purpose of determining the Seismic Performance Category (SPC) in Sec 3.4 as follows:

1. Essential bridges - IC = I

2. Other bridges - IC = II

Bridges shall be classified on the basis of Social/Survival and Security/Defense requirements, guidelines for which are given in the Commentary.

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3.4 SEISMIC PERFORMANCE CATEGORIES

Each bridge shall be assigned to one of four Seismic Performance Categories (SPC), A through D, based on the Acceleration Coefficient (A) and the Importance Classification (IC), as shown in Table 1. Minimum analysis and design requirements are governed by the SPC.

TABLE 1: SEISMIC PERFORMANCE CATEGORY (SPC)

Acceleration Coefficient	Importance Classification (IC)	
A	I	II
$A \leq 0.09$	A	A
$0.09 < A \leq 0.19$	B	B
$0.19 < A \leq 0.29$	C	C
$0.29 < A$	D	C

3.5 SITE EFFECTS

The effects of site conditions on bridge response shall be determined from a site coefficient (S) based on soil profile types defined as follows:

SOIL PROFILE TYPE I is a profile with either

1. Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2,500 ft/sec (762 m/sec), or by other appropriate means of classification); or

2. Stiff soil conditions where the soil depth is less than 200 ft (61 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

SOIL PROFILE TYPE II is a profile with stiff clay or deep cohesionless conditions where the soil depth exceeds 200 ft (61 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

SOIL PROFILE TYPE III is a profile with soft to medium-stiff clays and sands, characterized by 30 ft (9 m) or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.

In locations where the soil properties are not known in sufficient detail to determine the soil profile type or where the profile does not fit any of the three types, the site coefficient for Soil Profile Type II shall

be used. The soil profile coefficients apply to all foundation types including pile supported and spread footings.

3.5.1 SITE COEFFICIENT

The site coefficient (S) approximates the effects of the site conditions on the elastic response coefficient or spectrum of Sec. 5.2 and is given in Table 2.

TABLE 2: SITE COEFFICIENT (S)

	Soil Profile Type		
	I	II	III
S	1.0	1.2	1.5

3.6 RESPONSE MODIFICATION FACTORS

Seismic design forces for individual members and connections of bridges classified as SPC B, C and D are determined by dividing the elastic forces by the appropriate Response Modification Factor (R) as specified in Sec. 4.7 and 4.8. The Response Modification Factors for the various components are given in Table 3.

TABLE 3: RESPONSE MODIFICATION FACTOR (R)

Substructure[1]	R	Connections	R
Wall-Type Pier[2]	2	Superstructure to Abutment	0.8
Reinforced Concrete Pile Bents		Expansion Joints within	
a. Vertical Piles Only	3	a Span of the Superstructure	0.8
b. One or more Batter Piles	2		
		Columns, Piers or Pile Bents	
Single Columns	3	to Cap Beam or Superstructure[3]	1.0
Steel or Composite Steel		Columns or Piers	
and Concrete Pile Bents		to Foundations[3]	1.0
a. Vertical Piles Only	5		
b. One or more Batter Piles	3		
Multiple Column Bent	5		

[1]The R-Factor is to be used for both orthogonal axes of the substructure.

[2]A wall-type pier may be designed as a column in the weak direction of the pier provided all the provisions for columns in Chapter 8 are followed. The R-Factor for a single column can then be used.

[3]For bridges classified as SPC C and D it is recommended that the connections be designed for the maximum forces capable of being developed by plastic hinging of the column or column bent as specified in Sec. 4.8.5. These forces will often be significantly less than those obtained using an R-Factor of 1.

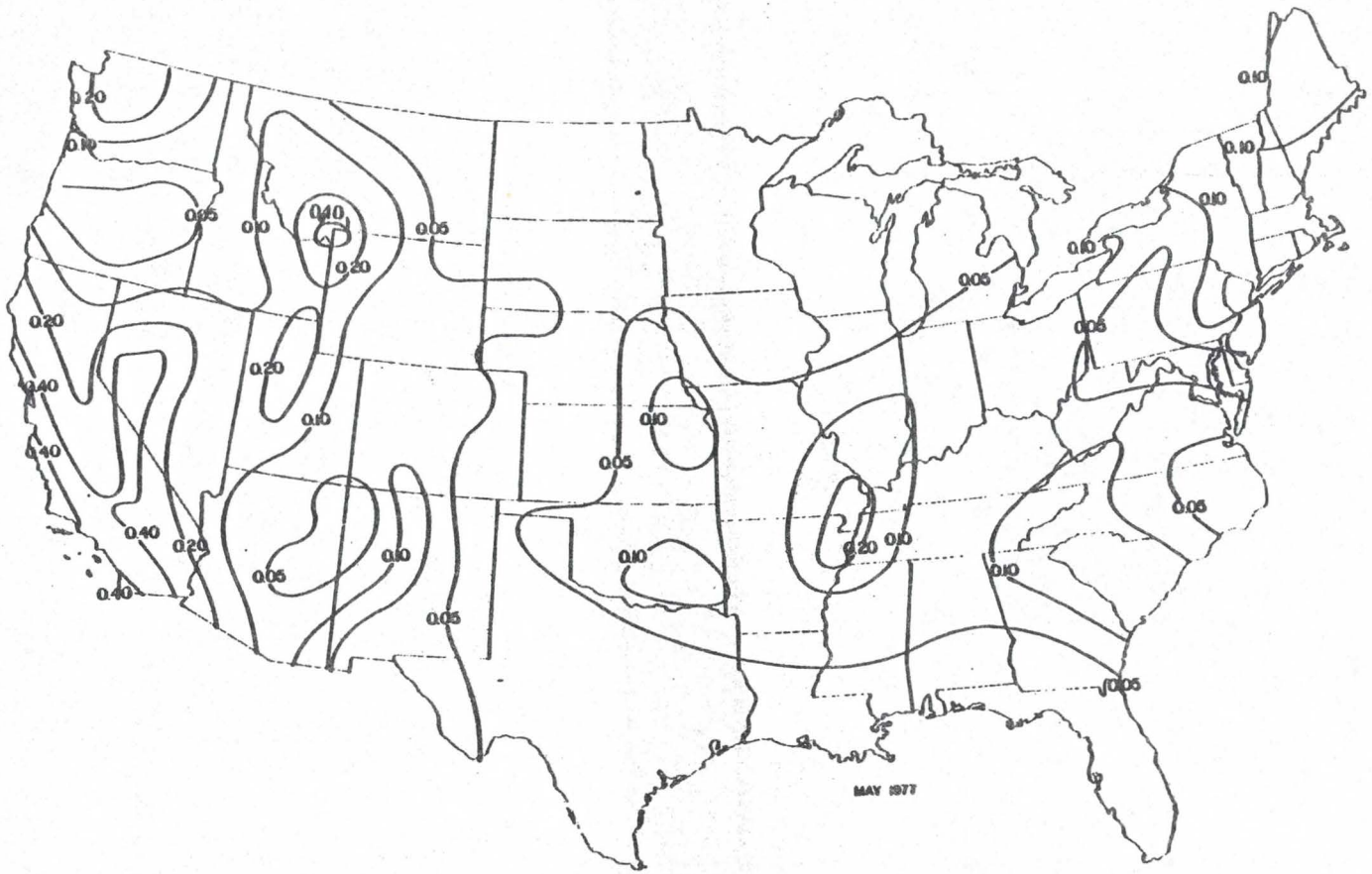

FIGURE 3. ACCELERATION COEFFICIENT – CONTINENTAL UNITED STATES
(An enlarged version of this map, including counties, is given in the back cover of the report).

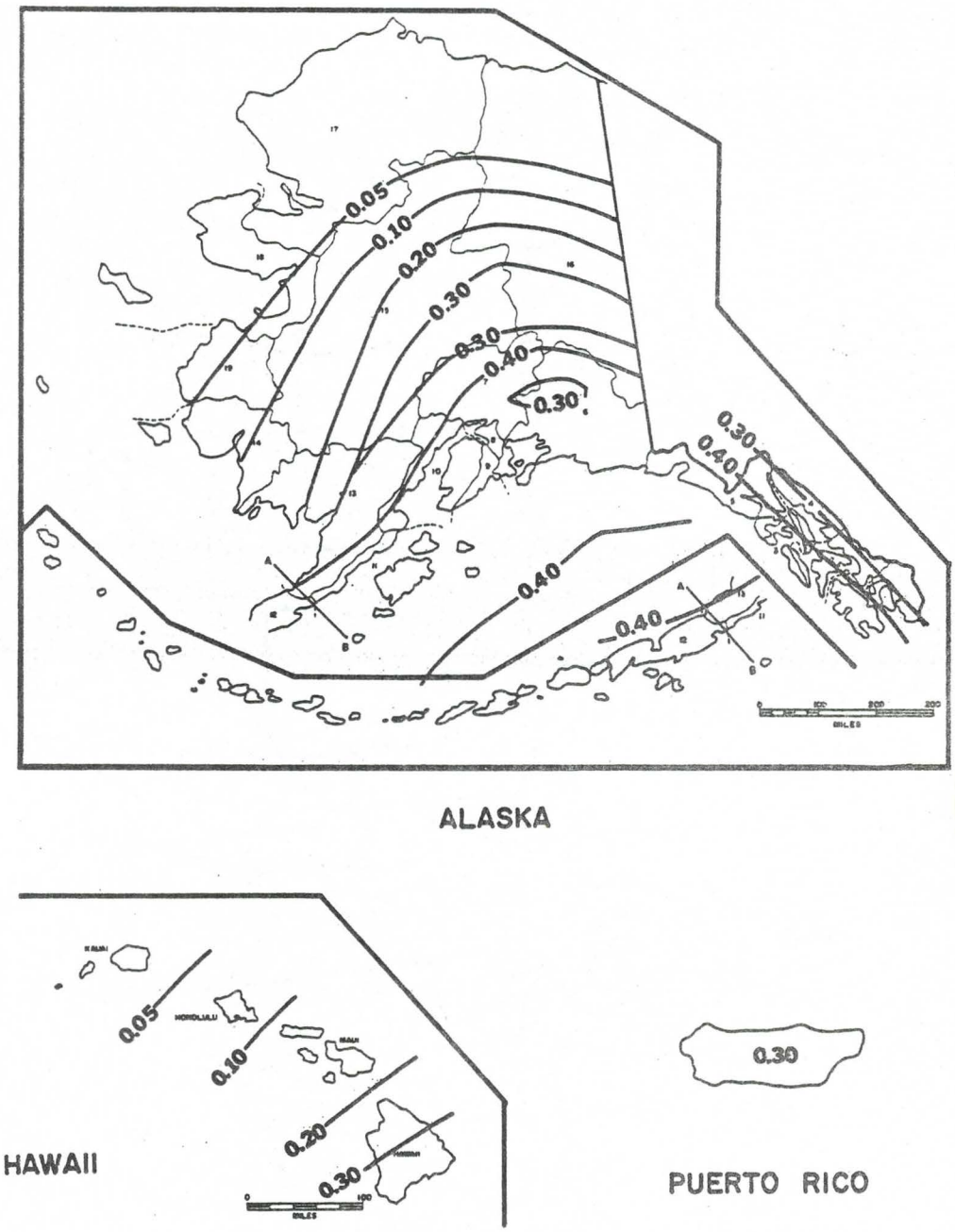

FIGURE 4. ACCELERATION COEFFICIENT – ALASKA, HAWAII, AND PUERTO RICO

CHAPTER 4

ANALYSIS AND DESIGN REQUIREMENTS

4.1 GENERAL

The provisions of this chapter shall control the selection of the analysis procedure and seismic design forces and displacements. The elastic seismic forces shall be determined in accordance with the procedures of Chapter 5. Material and foundation design requirements are given in Chapters 6, 7, and 8.

EXCEPTION:
Seismic design requirements for single span bridges are given in Sec. 4.5 and 4.9 and design requirements for bridges classified as SPC A are given in Sec. 4.6 and 4.9.1.

4.2 ANALYSIS PROCEDURE

Two minimum analysis procedures are defined and the applicable procedure for a given type of bridge, which depends on the number of spans, the geometrical complexity and the Seismic Performance Category, is given in Table 4. A more rigorous, generally accepted procedure may be used in lieu of the recommended minimum.

TABLE 4: ANALYSIS PROCEDURE

Seismic Performance Category	Regular[1] Bridges with 2 or More Spans	Irregular[2] Bridges with 2 or More Spans
A	-	-
B	1	1
C	1	2
D	1	2

[1]A "regular" bridge has no abrupt or unusual changes in mass, stiffness or geometry along its span and has no large differences in these parameters between adjacent supports (abutments excluded). For example a bridge may be considered regular if it is straight or describes a sector of an arc not exceeding 90° and has adjacent columns, or piers, that do not differ in stiffness by more than 25%. (Percentage difference is to be based on the lesser of two adjacent quantities as the reference.)

[2]An "irregular" bridge is any bridge that does not satisfy the definition of a regular bridge.

The two analysis procedures to be used are as follows:

PROCEDURE 1: Single-Mode Spectral Method

PROCEDURE 2: Multimode Spectral Method

Details of these procedures are given in Chapter 5.

EXCEPTION:
Detailed seismic analysis is not required for a single-span bridge or for bridges classified as SPC A.

4.3 DETERMINATION OF ELASTIC FORCES AND DISPLACEMENTS

For bridges classified as SPC B, C or D the elastic forces and displacements shall be determined independently along two perpendicular axes by use of the analysis procedure specified in Sec. 4.2. The resulting forces shall then be combined as specified in Sec. 4.4. Typically the perpendicular axes are the longitudinal and transverse axes of the bridge but the choice is open to the designer. The longitudinal axis of a curved bridge may be a chord connecting the two abutments.

4.4 COMBINATION OF ORTHOGONAL SEISMIC FORCES

A combination of orthogonal seismic forces is used to account for the directional uncertainty of earthquake motions and the simultaneous occurrences of earthquake forces in two perpendicular horizontal directions. The elastic seismic forces and moments resulting from analyses in the two perpendicular directions of Sec. 4.3 shall be combined to form two load cases as follows:

LOAD CASE 1: Seismic forces and moments on each of the principal axes of a member shall be obtained by adding 100% of the absolute value of the member elastic seismic forces and moments resulting from the analysis in one of the perpendicular (longitudinal) directions to 30% of the absolute value of the corresponding member elastic seismic forces and moments resulting from the analysis in the second perpendicular direction (transverse). (NOTE: The absolute values are used because a seismic force can be positive or negative.)

LOAD CASE 2: Seismic forces and moments on each of the principal axes of a member shall be obtained by adding 100% of the absolute value of the member elastic seismic forces and moments resulting from the analysis in the second perpendicular direction (transverse) to 30% of the absolute value of the corresponding member elastic seismic forces and moments resulting from the analysis in the first perpendicular direction (longitudinal).

EXCEPTION:
For SPC C and D when foundation and/or column connection forces are determined from plastic hinging of the columns (Sec. 4.8.2)

the resulting forces need not be combined as specified in this section. If a pier is designed as a column per Sec. 4.8.4 this exception only applies for the weak direction of the pier when forces resulting from plastic hinging are used. The combination specified must be used for the strong direction of the pier.

4.5 DESIGN REQUIREMENTS FOR SINGLE SPAN BRIDGES

A detailed seismic analysis is not required for single span bridges. However, the connections between the bridge span and the abutments shall be designed both longitudinally and transversely to resist the gravity reaction force at the abutment multiplied by the Acceleration Coefficient of the site. The minimum support lengths shall be as specified in Sec. 4.9.

4.6 DESIGN FORCES FOR SEISMIC PERFORMANCE CATEGORY A

The connection of the superstructure to the substructure shall be designed to resist a horizontal seismic force equal to 0.20 times the dead load reaction force in the restrained directions.

4.7 DESIGN FORCES FOR SEISMIC PERFORMANCE CATEGORY B

4.7.1 DESIGN FORCES FOR STRUCTURAL MEMBERS AND CONNECTIONS

Seismic design forces specified in this subsection shall apply to:

(a) The superstructure, its expansion joints and the connections between the superstructure and the supporting substructure.

(b) The supporting substructure down to the base of the columns and piers but not including the footing, pile cap or piles.

(c) Components connecting the superstructure to the abutment.

Seismic design forces for the above components shall be determined by dividing the elastic seismic forces obtained from Load Case 1 and Load Case 2 of Sec. 4.4 by the appropriate Response Modification Factor of Sec. 3.6. The modified seismic forces resulting from the two load cases shall then be combined independently with forces from other loads as specified in the following group loading combination for the components. Note that the seismic forces are reversible (positive and negative) and the maximum loading for each component shall be calculated as follows:

$$\text{Group Load} = 1.0\ (D + B + SF + E + EQM) \qquad (4\text{-}1)$$

where D = dead load; SF = stream-flow pressure
B = buoyancy; E = earth pressure
EQM = elastic seismic force for either Load Case 1 or Load Case 2 of Sec. 4.4 modified by dividing by the appropriate R-Factor.

Each component of the structure shall be designed to withstand the forces resulting from each load combination according to the Standard AASHTO Specifications for Highway Bridges, 12th Edition adopted by AASHTO and the additional requirements of Chapters 6, 7 and 8 of these Guidelines. Note that Eq. 4-1 shall be used in lieu of AASHTO Group VII group loading combination and that the and factors equal 1. For Service Load Design a 50% increase is permitted in the allowable stresses for structural steel and a 33-1/3% increase for reinforced concrete.

4.7.2 DESIGN FORCES FOR FOUNDATIONS

Seismic design forces for foundations, including footings, pile caps, and piles shall be the elastic seismic forces obtained from Load Case 1 and Load Case 2 of Sec. 4.4 divided by the Response Modification Factor (R) specified below. These modified seismic forces shall then be combined independently with forces from other loads as specified in the following group loading combination to determine two alternate load combinations for the foundations.

$$\text{Group Load} = 1.0\ (D + B + SF + E + EQF) \qquad (4\text{-}2)$$

where D, B, E and SF are as defined in Sec. 4.7.1 and

EQF = the elastic seismic force for either Load Case 1 or Load Case 2 of Sec. 4.4 divided by half the R-Factor for the substructure (column or pier) to which it is attached.

EXCEPTION:
For pile bents the R-Factor shall not be divided by 2.

Each component of the foundation shall be designed to resist the forces resulting from each load combination according to the requirements of Chapter 6 and the Standard Specifications for Highway Bridges, 12th Edition, adopted by AASHTO.

4.7.3 ABUTMENTS AND RETAINING WALLS

The components (bearings, shear keys) connecting the superstructure to an abutment shall be designed to resist the forces specified in Sec. 4.7.1.

Design requirements for abutments are given in Sec. 6.3.2.

4.8 DESIGN FORCES FOR SEISMIC PERFORMANCE CATEGORIES C AND D

Two sets of design forces are specified in Sec. 4.8.1 and 4.8.2 for bridges classified as Category C or D. The design forces for the various components are specified in Sec. 4.8.3 through 4.8.7.

4.8.1 MODIFIED DESIGN FORCES

These shall be determined as given in Sec. 4.7.1 except that for columns a maximum and minimum axial force shall be calculated for each load case by taking the seismic axial force as positive and negative.

4.8.2 FORCES RESULTING FROM PLASTIC HINGING IN THE COLUMNS, PIERS OR BENTS

The force resulting from plastic hinging at the top and/or bottom of the column shall be calculated after the preliminary design of the columns is complete. The forces resulting from plastic hinging are recommended for determining design forces for most components as specified in Sec. 4.8.3 through 4.8.6. Alternate conservative design forces are specified if forces resulting from plastic hinging are not calculated. The procedures for calculating these forces for single column and pier supports and bents with two or more columns are given in the following subsections.

4.8.2(A) Single Columns and Piers

The forces shall be calculated for the two principal axes of a column and in the weak direction of a pier or bent as follows:

Step 1. Determine the column overstrength plastic moment capacities. For reinforced concrete columns, use a strength reduction factor (ϕ) of 1.3 and for structural steel columns use 1.25 times the nominal yield strength. (NOTE: This corresponds to the normal use of a strength reduction factor for reinforced concrete. In this case it provides an increase in the ultimate strength.) For both materials use the maximum elastic column axial load from Sec. 4.4 added to the column dead load.

Step 2. Using the column overstrength plastic moments, calculate the corresponding column shear force. For flared columns this calculation shall be performed using the overstrength plastic moments at both the top and bottom of the flare with the appropriate column height. If the foundation of a column is significantly below ground level, consideration should be given to the possibility of the plastic hinge forming above the foundation. If this can occur the column length between plastic hinges shall be used to calculate the column shear force.

The forces corresponding to a single column hinging are:

1. Axial Forces - unreduced maximum and minimum seismic axial load of Sec. 4.4 plus the dead load.

2. Moments - those calculated in Step 1.

3. Shear Force - that calculated in Step 2.

4.8.2(B) Bents with Two or More Columns

The forces for bents with two or more columns shall be calculated both in the plane of the bent and perpendicular to the plane of the bent. Perpendicular to the plane of the bent the forces shall be calculated as for single columns in Sec. 4.8.2(A). In the plane of the bent the forces shall be calculated as follows:

Step 1. Determine the column overstrength plastic moment capacities. For reinforced concrete use a strength reduction factor (ϕ) of 1.3 and for structural steel use 1.25 times the nominal yield strength. (NOTE: This corresponds to the normal use of a strength reduction factor for reinforced concrete. In this case it provides an increase in the ultimate strength.) For both materials use the axial load corresponding to the dead load.

Step 2. Using the column overstrength plastic moments calculate the corresponding column shear forces. Sum the column shears of the bent to determine the maximum shear force for the bent. Note that, if a partial-height wall exists between the columns, the effective column height is taken from the top of the wall. For flared columns and foundations below ground level see Sec. 4.8.2(A) Step 2. For pile bents the length of pile above the mud line shall be used to calculate the shear force.

Step 3. Apply the bent shear force to the top of the bent (center of mass of the superstructure above the bent) and determine the axial forces in the columns due to overturning when the column overstrength plastic moments are developed.

Step 4. Using these column axial forces combined with the dead load axial forces, determine revised column overstrength plastic moments. With the revised overstrength plastic moments calculate the column shear forces and the maximum shear force for the bent. If the maximum shear force for the bent is not within 10% of the value previously determined, use this maximum bent shear force and return to Step 3.

The forces in the individual columns in the plane of a bent corresponding to column hinging, are:

1. Axial Forces - the maximum and minimum axial load is the dead load plus, or minus, the axial load determined from the final iteration of Step 3.

2. Moments - the column overstrength plastic moments corresponding to the maximum compressive axial load specified in (1) with a strength reduction factor of 1.3 for reinforced concrete and 1.25 times the nominal yield strength for structural steel.

3. Shear Force - the shear force corresponding to the column overstrength moments in (2), noting the provisions in Step 2 above.

4.8.3 COLUMN AND PILE BENT DESIGN FORCES

Design forces for columns and pile bents shall be the following:

(a) Axial Forces - the minimum and maximum design force shall either be the elastic design values determined in Sec. 4.4 added to the dead load, or the values corresponding to plastic hinging of the column and determined in Sec. 4.8.2. Generally the values corresponding to column hinging will be smaller and then it is recommended that these smaller values be used.

(b) Moments - the modified design moments determined in Sec. 4.8.1.

(c) Shear Force - either the elastic design value determined from Sec. 4.8.1 using an R-Factor of 1 for the column or the value corresponding to plastic hinging of the column as determined in Sec. 4.8.2. Generally the value corresponding to column hinging will be significantly smaller and then it is recommended that this smaller value be used.

4.8.4 PIER DESIGN FORCES

The design forces shall be those determined in Sec. 4.8.1 except if the pier is designed as a column in its weak direction. If the pier is designed as a column the design forces in the weak direction shall be as specified in Sec. 4.8.3 and all the design requirements for columns of Chapter 8 shall apply. (NOTE: When the forces due to plastic hinging are used in the weak direction the combination of forces specified in Sec. 4.4 is not applicable.)

4.8.5 CONNECTION DESIGN FORCES

The design forces shall be those determined in Sec. 4.8.1 except that for superstructure connections to columns and column connections to cap beams or footings, the alternate forces specified in 4.8.5(C) below are recommended. Additional design forces at connections are as follows:

4.8.5(A) Longitudinal Linkage Forces

Positive horizontal linkage shall be provided between adjacent sections of the superstructure at supports and expansion joints within a span. The linkage shall be designed for a minimum force of the Acceleration Coefficient times the weight of the lighter of the two adjoining spans or parts of the structure. If the linkage is at a point where relative displacement of the sections of superstructure is designed to occur during seismic motions, sufficient slack must be allowed in the linkage so that the

linkage force does not start to act until the design displacement is exceeded. Where linkage is to be provided at columns or piers, the linkage of each span may be attached to the column or pier rather than between adjacent spans. Positive linkage shall be provided by ties, cables, dampers or an equivalent mechanism. Friction shall not be considered a positive linkage.

4.8.5(B) Hold-Down Devices

Hold-down devices shall be provided at all supports or hinges in continuous structures, where the vertical seismic force due to the longitudinal horizontal seismic load opposes and exceeds 50% but is less than 100% of the dead-load reaction. In this case the minimum net upward force for the hold-down device shall be 10% of the dead load downward force that would be exerted if the span were simply supported.

If the vertical seismic force (Q) due to the longitudinal horizontal seismic load opposes and exceeds 100% of the dead load reaction (DR), the net upwards force for the hold-down device shall be 1.20(Q-DR) but it shall not be less than that specified in the previous paragraph.

4.8.5(C) Column and Pier Connection Design Forces

The recommended connection design forces between the superstructure and columns, columns and cap beams, and columns and spread footings or pile caps are the forces developed at the top and bottom of the columns due to column hinging and determined in Sec. 4.8.2. The smaller of these or the values specified in Sec. 4.8.1 may be used. Note that these forces should be calculated after the column design is complete and the overstrength moment capacities have been obtained.

4.8.6 FOUNDATION DESIGN FORCES

The design forces for foundations including footings, pile caps and piles may be either those forces determined in Sec. 4.8.1 or the forces at the bottom of the columns corresponding to column plastic hinging and determined in Sec. 4.8.2. Generally the values corresponding to column hinging will be significantly smaller and then these smaller values are recommended for design.

When the columns of a bent have a common footing the final force distribution at the base of the columns in Step 4 of Sec. 4.8.2(B) may be used for the design of the footing in the plane of the bent. This force distribution produces lower shear forces and moments on the footing because one exterior column may be in tension and the other in compression due to the seismic overturning moment. This effectively increases the ultimate moments and shear forces on one column and reduces them on the other.

4.8.7 ABUTMENT AND RETAINING WALL DESIGN FORCES

The components (bearings, shear keys, etc.) connecting the superstructure to an abutment shall be designed to resist the forces specified in Sec. 4.8.1.

Design requirements for abutments are given in Sec. 6.4.2 for SPC C and Sec. 6.5.2 for SPC D.

4.9 DESIGN DISPLACEMENTS

Minimum bearing support lengths as determined in this section shall be provided for the expansion ends of all girders.

4.9.1 SEISMIC PERFORMANCE CATEGORY A

Bridges classified as SPC A shall meet the following requirement: Bearing seats supporting the expansion ends of girders, as shown in Fig. 5, shall be designed to provide a minimum support length N (in. or mm) measured normal to the face of an abutment or pier, not less than that specified below.

$$N = 8 + 0.02L + 0.08H \text{ (in.)} \tag{4-3A}$$

or

$$N = 203 + 1.67L + 6.66H \text{ (mm)} \tag{4-3B}$$

where L = length, in feet for Eq. 4-3A or meters for Eq. 4-3B, of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck. For hinges within a span, L shall be the sum of L_1 and L_2, the distances to either side of the hinge. For single span bridges L equals the length of the bridge deck. These lengths are shown in Fig. 5.

For abutments

H = average height, in feet for Eq. 4-3A or meters for Eq. 4-3B, of columns supporting the bridge deck to the next expansion joint. H = 0 for single span bridges.

For columns and/or piers

H = column or pier height in feet for Eq. 4-3A or meters for Eq. 4-3B.

For hinges within a span

H = average height of the adjacent two columns or piers in feet for Eq. 4-3A or meters for Eq. 4-3B.

4.9.2 SEISMIC PERFORMANCE CATEGORY B

The seismic design displacements shall be the maximum of those determined in accordance with Sec. 4.3 or those specified in Sec. 4.9.1.

4.9.3 SEISMIC PERFORMANCE CATEGORIES C AND D

The seismic design displacements shall be the maximum of those determined in accordance with Sec. 4.3 or those specified in Sec. 4.9.1 except that Eq. 4-3 shall be replaced by

$$N = 12 + 0.03L + 0.12H \text{ (in.)} \qquad (4\text{-}4A)$$

or

$$N = 305 + 2.5L + 10H \text{ (mm)} \qquad (4\text{-}4B)$$

where N, L and H are as specified in Sec. 4.9.1.

Positive horizontal linkage shall be provided at all superstructure gaps or expansion joints within a span as specified in Sec. 4.8.5.

Relative displacements between different segments of the bridge should be carefully considered in the evaluation of the results determined in accordance with Sec. 4.3. Relative displacements arise from effects that are not easily included in the analysis procedure but should be considered in determining the design displacements. They include the following:

(a) Torsional displacements of bridge decks on skewed supports.

(b) Rotation and/or lateral displacements of the foundations.

(c) Out-of-phase displacements of different segments of the bridge. This is especially important in determining seat widths at expansion joints.

(d) Out-of-phase rotation of abutments and columns induced by travelling seismic waves.

See the Commentary on this section for a more detailed discussion.

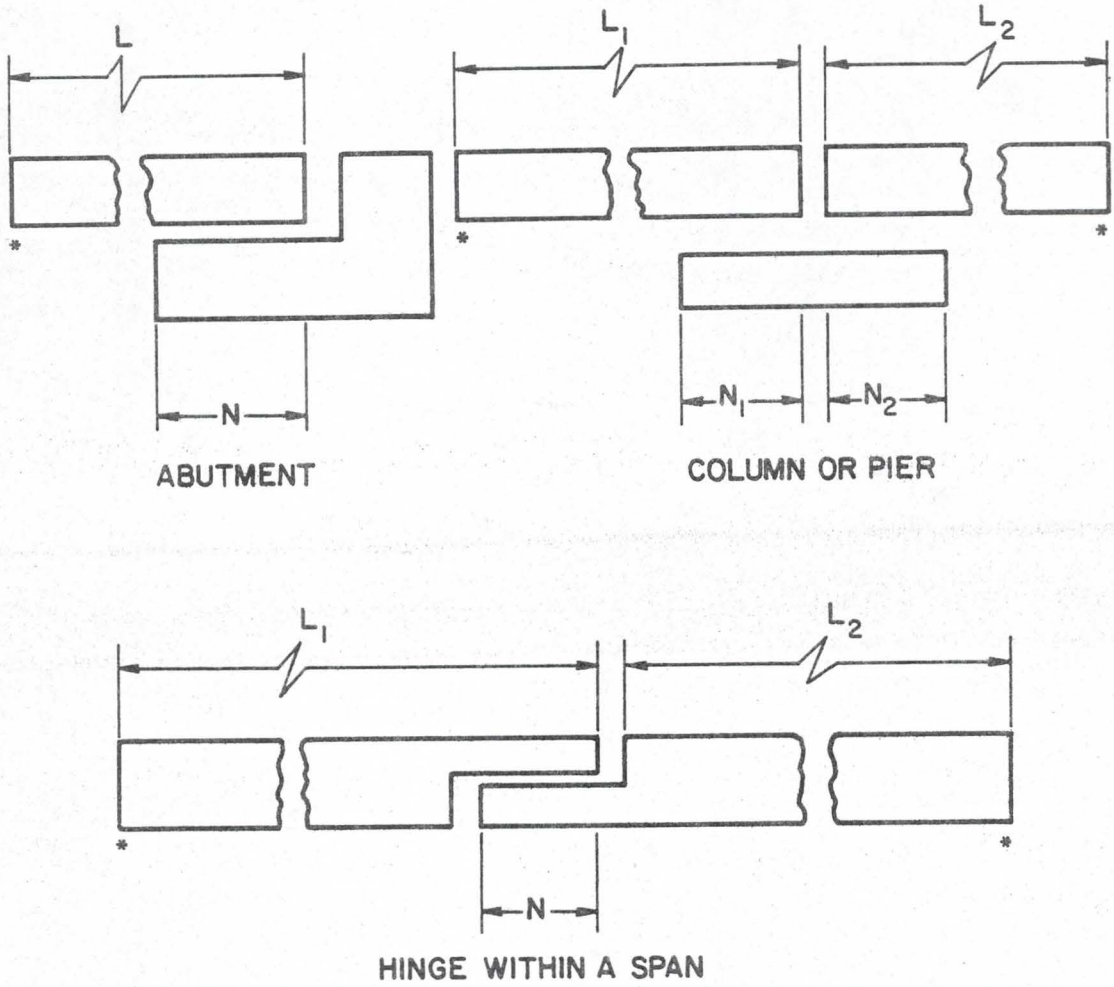

FIGURE 5. DIMENSIONS FOR MINIMUM SUPPORT LENGTH REQUIREMENTS

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CHAPTER 5

ANALYSIS METHODS

5.1 GENERAL

The requirements of this chapter shall control the seismic analysis of bridges prescribed in Sec. 4.2. Two analysis procedures are presented.

Procedure 1. Single-Mode Spectral Method
Procedure 2. Multimode Spectral Method

In both methods, all fixed column, pier or abutment supports are assumed to have the same ground motion at the same instant in time. At movable supports, displacements determined from the analysis prescribed in this chapter, which exceed the minimum requirements as specified in Sec. 4.9.2 and 4.9.3, shall be used in design without reduction.

5.2 ELASTIC SEISMIC RESPONSE COEFFICIENT AND SPECTRUM

5.2.1 ELASTIC SEISMIC RESPONSE COEFFICIENT: PROCEDURE 1

The elastic seismic response coefficient C_s used to determine the design forces is given by the dimensionless formula:

$$C_s = \frac{1.2AS}{T^{2/3}} \tag{5-1}$$

where A = the Acceleration Coefficient from Sec. 3.2,
S = the dimensionless coefficient for the soil profile characteristics of the site as given in Sec. 3.5,
T = the period of the bridge as determined in Sec. 5.3 or by other acceptable methods.

The value of C_s need not exceed 2.5A. For Soil Profile Type III soils in areas where $A \geq 0.30$, C_s need not exceed 2.0A.

5.2.2 ELASTIC SEISMIC RESPONSE SPECTRUM: PROCEDURE 2

The elastic seismic response coefficient for mode "m", C_{sm}, shall be determined in accordance with the following formula:

$$C_{sm} = \frac{1.2AS}{T_m^{2/3}} \tag{5-2}$$

where T_m = the period of the mth mode of vibration.

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The value of C_{sm} need not exceed 2.5A. For Type III soils in areas where the coefficient $A \geq 0.30$, C_{sm} need not exceed 2.0A.

EXCEPTIONS:

1. For Soil Profile Type III soils, C_{sm} for modes other than the fundamental mode, which have periods less than 0.3 sec. may be determined in accordance with the following formula:

$$C_{sm} = A(0.8 + 4.0T_m) \tag{5-3}$$

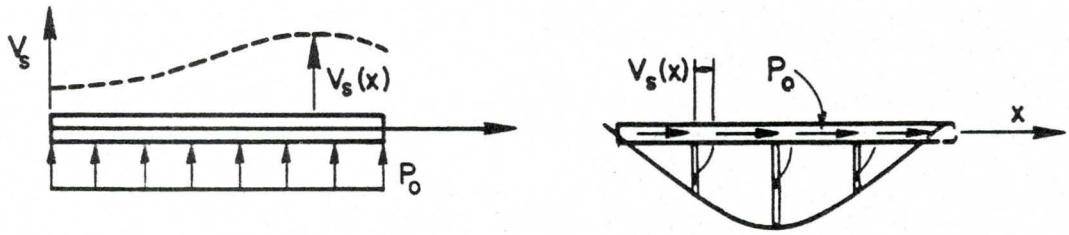
2. For structures in which any T_m exceeds 4.0 sec, the value of C_{sm} for that mode may be determined in accordance with the following formula:

$$C_{sm} = \frac{3AS}{T_m^{4/3}} \tag{5-4}$$

5.3 SINGLE MODE SPECTRAL ANALYSIS METHOD--PROCEDURE 1

The single mode spectral analysis method described in the following steps may be used for both transverse and longitudinal earthquake motions. Examples illustrating its application are given in the Commentary.

Step 1. Calculate the static displacements $v_s(x)$ due to an assumed uniform loading p_o as shown in Figure 6. Abutment stiffness, if desired, can be incorporated by the procedure outlined in Sec. C5.3 of the Commentary. The uniform loading p_o is applied over the length of the bridge; it has units of force/unit length and is arbitrarily set equal to 1. The static displacement $v_s(x)$ has units of length.



(a) Plan-Transverse Loading (b) Elevation-Longitudinal Loading

FIGURE 6. BRIDGE DECK SUBJECTED TO ASSUMED TRANSVERSE AND LONGITUDINAL LOADING

Step 2. Calculate factors α, β, and γ from the expressions

$$\alpha = \int v_s(x)dx \tag{5-5}$$

$$\beta = \int w(x)v_s(x)dx \tag{5-6}$$

$$\gamma = \int w(x)v_s(x)^2dx \tag{5-7}$$

where w(x) is the weight of the dead load of the bridge superstructure and tributary substructure (see Appendix A) (force/unit length). The computed factors, α, β, γ, have units of (length^2), (force·length), and (force·length^2), respectively.

The weight should take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns and footings. Other loads such as live loads may be included. (Generally, the inertia effects of live loads are not included in the analysis; however, the design of bridges having high live to dead load ratios located in metropolitan areas where traffic congestion is likely to occur should consider the probability of a large live load being on the bridge during an earthquake.)

Step 3. Calculate the period of the bridge using the expression:

$$T = 2\pi \sqrt{\frac{\gamma}{p_o g \alpha}} \tag{5-8}$$

where g = acceleration of gravity (length/time^2).

Step 4. Calculate the equivalent static earthquake loading $p_e(x)$ from the expression:

$$p_e(x) = \frac{\beta C_s}{\gamma} w(x)v_s(x) \tag{5-9}$$

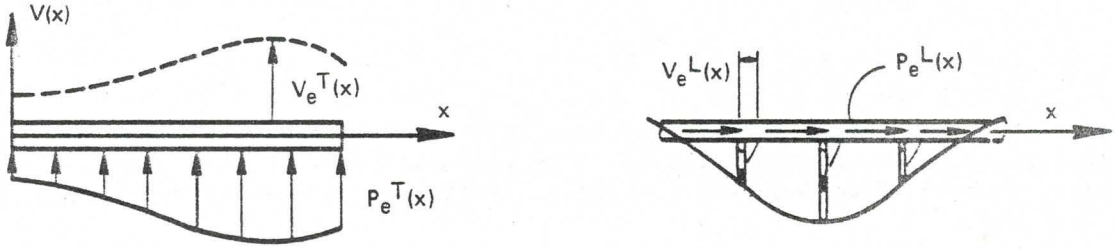
where C_s = the dimensionless elastic seismic response coefficient given by Eq. (5-1),

$p_e(x)$ = the intensity of the equivalent static seismic loading applied to represent the primary mode of vibration (force/unit length).

Step 5. Apply loading $p_e(x)$ to the structure as shown in Fig. 7 and determine the resulting member forces and displacements for design.

5.4 MULTIMODE SPECTRAL ANALYSIS METHOD--PROCEDURE 2

The multimode response spectrum analysis should be performed with a suitable space frame linear dynamic analysis computer program. Currently available computer programs are included in the Commentary.



(a) Plan -Transverse Loading

(b) Elevation-Longitudinal Loading

FIGURE 7. BRIDGE DECK SUBJECTED TO EQUIVALENT TRANSVERSE AND LONGITUDINAL SEISMIC LOADING

5.4.1 GENERAL

The multimode spectral analysis method applies to bridges with irregular geometry which induces coupling in the three coordinate directions within each mode of vibration. These coupling effects make it difficult to categorize the modes into simple longitudinal or transverse modes of vibration and, in addition, several modes of vibration will in general contribute to the total response of the structure. A computer program with space frame dynamic analysis capabilities should be used to determine coupling effects and multimodal contributions to the final response. Motions applied at the supports in any one of the two horizontal directions will produce forces along both principal axes of the individual members because of the coupling effects. For curved structures, the longitudinal motion shall be directed along a chord connecting the abutments and the transverse motion shall be applied normal to the chord. Forces due to longitudinal and transverse motions shall be combined as specified in Sec. 4.4.

5.4.2 MATHEMATICAL MODEL

The bridge should be modeled as a three-dimensional space frame with joints and nodes selected to realistically model the stiffness and inertia effects of the structure. Each joint or node should have six degrees of freedom, three translational and three rotational. The structural mass should be lumped with a minimum of three translational inertia terms.

The mass should take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns and footings. Other loads such as live loads may be included. (Generally, the inertia effects of live loads are not included in the analysis; however, the design of bridges having high live to dead load ratios located in metropolitan areas where traffic congestion is likely to occur should consider the probability of a large live load being on the bridge during an earthquake.)

5.4.2(A) Superstructure

The superstructure should, as a minimum, be modeled as a series of space frame members with nodes at such points as the span quarter points in addition to joints at the ends of each span. Discontinuities should be included in the superstructure at the expansion joints and abutments. Care should be taken to distribute properly the lumped mass inertia effects at these locations. The effect of earthquake restrainers at expansion joints may be approximated by superimposing one or more linearly elastic members having the stiffness properties of the engaged restrainer units.

5.4.2(B) Substructure

The intermediate columns or piers should also be modeled as space frame members. Generally, for short, stiff columns having lengths less than one-third of either of the adjacent span lengths, intermediate nodes are not necessary. Long, flexible columns should be modeled with intermediate nodes at the third points in addition to the joints at the ends of the columns. The model should consider the eccentricity of the columns with respect to the superstructure. Foundation conditions at the base of the columns and at the abutments may be modeled using equivalent linear spring coefficients.

5.4.3 MODE SHAPES AND PERIODS

The required periods and mode shapes of the bridge in the direction under consideration shall be calculated by established methods for the fixed base condition using the mass and elastic stiffness of the entire seismic resisting system.

5.4.4 MULTIMODE SPECTRAL ANALYSIS

The response should, as a minimum, include the effects of a number of modes equivalent to three times the number of spans up to a maximum of 25 modes.

5.4.5 MEMBER FORCES AND DISPLACEMENTS

The member forces and displacements can be estimated by combining the respective response quantities (e.g., force, displacement, or relative displacement) from the individual modes by the Square Root of the Sum of the Squares (SRSS) method. The member forces and displacements obtained using the SRSS method of combining modes is generally adequate for most bridge systems because they have well-separated modes of vibration characterized by significant differences in the natural periods of each of the modes. For bridges with closely spaced modes (within 10%), other more appropriate methods of combining or weighting the individual contributions should be considered to obtain the total final response (see Commentary).

CHAPTER 6

FOUNDATION AND ABUTMENT DESIGN REQUIREMENTS

6.1 GENERAL

This chapter includes only those foundation and abutment requirements that are specifically related to seismic resistant construction. It assumes compliance with all the basic requirements necessary to provide support for vertical loads and lateral loads other than those due to earthquake motions. These include, but are not limited to, provisions for the extent of foundation investigation, fills, slope stability, bearing and lateral soil pressures, drainage, settlement control, and pile requirements and capacities.

6.2 SEISMIC PERFORMANCE CATEGORY A

There are no special seismic design requirements for this category.

6.3 SEISMIC PERFORMANCE CATEGORY B

Bridges classified as SPC B shall conform to all the requirements of this section.

6.3.1 FOUNDATIONS

Foundation and abutment seismic design requirements for SPC B are given in the following subsections.

6.3.1(A) Investigation

In addition to the normal site investigation report, the Engineer may require the submission of a report which shall include the results of an investigation to determine potential hazards and seismic design requirements related to (1) slope instability, (2) liquefaction, (3) fill settlement, and (4) increases in lateral earth pressure, all as a result of earthquake motions. Seismically-induced slope instability in approach fills or cuts may displace abutments and lead to significant differential settlement and structural damage. Fill settlement and abutment displacements due to lateral pressure increases may lead to bridge access problems and structural damage. Liquefaction of saturated cohesionless fills or foundation soils may contribute to slope and abutment instability, and lead to a loss of foundation bearing capacity and lateral pile support. Liquefaction failures of the above types have led to many bridge failures during past earthquakes (a discussion on liquefaction potential assessment is provided in the Commentary).

6.3.1(B) Foundation Design

For the load combinations specified in Sec. 4.7.2, the soil strength capable of being mobilized by the foundations shall be established in the site investigation report. Because of the dynamic cyclic nature of seismic loading, the ultimate capacity of the foundation supporting medium should be used in conjunction with these load combinations. Due consideration shall be given to the magnitude of the seismically-induced foundation settlement that the bridge can withstand.

Transient foundation uplift or rocking involving separation from the subsoil of up to one-half of an end bearing foundation pile group or up to one-half of the contact area of foundation footings is permitted under seismic loading, provided that foundation soils are not susceptible to loss of strength under the imposed cyclic loading.

General comments on soil strength and stiffness mobilized during earthquakes, foundation uplift, lateral loading of piles, soil-structure interaction and foundation design in environments susceptible to liquefaction are provided in the Commentary.

6.3.1(C) Special Pile Requirements

The following special pile requirements are in addition to the requirements for piles in other applicable specifications.

Piles may be used to resist both axial and lateral loads. The minimum depth of embedment, together with the axial and lateral pile capacities, required to resist seismic loads shall be determined by means of the design criteria established in the site investigation report. Note that the ultimate capacity of the piles should be used in designing for seismic loads.

All piles shall be adequately anchored to the pile footing or cap. Concrete piles shall be anchored by embedment of sufficient length of pile reinforcement (unless special anchorage is provided) to develop uplift forces but in no case shall this length be less than the development length required for the reinforcement. Each concrete-filled pipe pile shall be anchored by at least 4 reinforcing steel dowels with a minimum steel ratio of 0.01 embedded sufficiently as required for concrete piles. Timber and steel piles, including unfilled pipe piles, shall be provided with anchoring devices to develop all uplift forces adequately but in no case shall these forces be less than 10% of the allowable pile load.

All concrete piles shall be reinforced to resist the design moments, shears, and axial loads. Minimum reinforcement shall be not less than the following:

1. Cast-in-Place Concrete Piles. Longitudinal reinforcing steel shall be provided for cast-in-place concrete piles

in the upper one-third (8 ft or 2.4 m minimum) of the pile length with a minimum steel ratio of 0.005 provided by at least 4 bars. Spiral reinforcement or equivalent ties of 1/4 in. (6.3 mm) diameter or larger shall be provided at 9 in. (229 mm) maximum pitch, except for the top 2 ft (610 mm) below the pile cap reinforcement where the pitch shall be 3 in. (76 mm) maximum.

2. Precast Piles. Longitudinal reinforcing steel shall be provided for each precast concrete pile with a minimum steel ratio of 0.01 provided by at least 4 bars. Spiral reinforcement or equivalent ties of No. 3 bars or larger shall be provided at 9 in. (229 mm) maximum pitch, except for the top 2 ft (610 mm) below the pile cap reinforcement where the pitch shall be 3 in. (76 mm) maximum.

3. Precast-Prestressed Piles. Ties in precast-prestressed piles shall conform to the requirements of precast piles.

6.3.2 ABUTMENTS

6.3.2(A) Free-Standing Abutments

For free-standing abutments or retaining walls which may displace horizontally without significant restraint (e.g., superstructure supported by sliding bearings), the pseudo-static Mononobe-Okabe method of analysis (see Commentary) is recommended for computing lateral active soil pressures during seismic loading. A seismic coefficient equal to one-half the acceleration coefficient (k_h=A/2) is recommended. The effects of vertical acceleration may be omitted. Abutments should be proportioned to slide rather than tilt, and provisions should be made to accommodate small horizontal seismically-induced abutment displacements when minimal damage is desired at abutment supports. Abutment displacements of up to 10A in. (254A mm) may be expected.

The seismic design of free-standing abutments should take into account forces arising from seismically-induced lateral earth pressures, additional forces arising from wall inertia effects and the transfer of seismic forces from the bridge deck through bearing supports which do not slide freely (e.g., elastomeric bearings).

For free-standing abutments which are restrained from horizontal displacement by anchors or batter piles, the magnitudes of seismically-induced lateral earth pressures are higher than those given by the Mononobe-Okabe method of analysis. As a first approximation, it is recommended that the maximum lateral earth pressure be computed by using a seismic coefficient k_h=1.5A in conjunction with the Mononobe-Okabe analysis method.

6.3.2(B) Monolithic Abutments

For monolithic abutments where the abutment forms an integral part of the bridge superstructure, maximum earth pressures acting on the abutment may be assumed equal to the maximum longitudinal earthquake force transferred from the superstructure to the abutment. To minimize abutment damage, the abutment should be designed to resist the passive pressure capable of being mobilized by the abutment backfill, which should be greater than the maximum estimated longitudinal earthquake force transferred to the abutment. It may be assumed that the lateral active earth pressure during seismic loading is less than the superstructure earthquake load.

When longitudinal seismic forces are also resisted by piers or columns, it is necessary to estimate abutment stiffness in the longitudinal direction in order to compute the proportion of earthquake load transferred to the abutment (see Commentary Sec. C5.4.2).

6.4 SEISMIC PERFORMANCE CATEGORY C

Bridges classified as SPC C shall conform to all of the requirements of Sec. 6.3 plus the additional requirements of this section.

6.4.1 FOUNDATIONS

Foundation design requirements for bridges classified as SPC C shall meet the requirements of Sec. 6.3.1 plus the additional requirements of this section.

6.4.1(A) Investigation

The Engineer may require the submission of a written report which shall include, in addition to the requirements of Sec. 6.3.1(A), a determination of the potential for surface rupture due to faulting or differential ground displacement (lurching), all as a result of earthquake motions.

6.4.1(B) Foundation Design

The design forces for the foundations shall be those specified in Sec. 4.8.6.

For saturated sand and soft clay foundation soils, due consideration shall be given to the potential for soil strength loss under the imposed cyclic loading in assessing the ultimate capacity of foundations.

6.4.1(C) Special Pile Requirements

The following special requirements for concrete piles shall apply:

1. Anchorage. The longitudinal reinforcement of all concrete piles shall be anchored to the pile footing or cap to develop a force of at least $1.25A_s f_y$ where A_s is the area of longitudinal reinforcement in the concrete pile and f_y is its nominal yield strength.

2. Confinement Length. The upper end of every pile shall be reinforced as a potential plastic hinge region, except where it can be established that there is no possibility of any significant lateral deflections in the pile resulting from deformation. The potential plastic hinge region shall, as a minimum, be considered to extend from the underside of the pile cap over a length of not less than two pile diameters or 24 in. (610 mm). If an analysis of the bridge and pile system indicates that a plastic hinge can form at a lower level, the transverse reinforcement requirements of (3) shall extend to that level. Note the special requirements for pile bents given in Sec. 8.4.1(C), (D) and (E).

3. Volumetric Ratio for Confinement. The volumetric ratio of transverse reinforcement to the distance specified in (2) shall be as required for columns in Sec. 8.4.1(D).

4. Cast-in-Place Concrete Piles. Longitudinal steel shall be provided for cast-in-place concrete piles for the full length of the pile. The upper two-thirds of the pile shall have a minimum longitudinal steel ratio of 0.0075 provided by at least 4 bars. Spiral reinforcement or equivalent ties of 1/4 in. (6.3 mm) diameter or larger shall be provided at 9 in. (229 mm) maximum pitch, except for the top 4 ft (1.2 m) where the pitch shall be 3 in. (76 mm) maximum, and where the volumetric ratio shall conform to 8.4.1(D).

5. Precast Concrete Piles. Spiral reinforcement ties in precast, including prestressed, concrete piles shall be No. 3 bars or larger and shall be provided at 9 in. (229 mm) maximum pitch except for the top 4 ft (1.2 m) where the pitch shall be 3 in. (76 mm) and the volumetric ratio shall conform to 8.4.1(D).

6.4.2 ABUTMENTS

In addition to the provisions outlined in Sec. 6.3.2, consideration should be given to the mechanism of transfer of superstructure transverse

inertial forces to the bridge abutments. Adequate resistance to lateral pressure should be provided by wing walls or abutment keys to minimize lateral abutment displacements.

6.5 SEISMIC PERFORMANCE CATEGORY D

Bridges classified as SPC D shall conform to all requirements of Sec. 6.3 and 6.4 and the additional requirements of this section.

6.5.1 FOUNDATIONS

Foundation design requirements for bridges classified as SPC D shall meet the requirements of Sec. 6.3.1 and 6.4.1 plus the additional requirements of this section.

6.5.1(A) Investigation

The Engineer may require the submission of a written report which shall include, in addition to the requirements of Sec. 6.3.1 and 6.4.1, a site-specific study to investigate the influence of cyclic loading on the deformation and strength characteristics of foundation soils. Potential progressive degradation in the stiffness and strength characteristics of saturated sands and soft clays should be given particular attention. More detailed analyses of slope and/or abutment settlement during earthquake loading should be undertaken.

6.5.1(B) Foundation Design

The design forces for foundations shall be those specified in Sec. 4.8.6.

6.5.2 ABUTMENTS

In addition to the requirements outlined in Sec. 6.3.2 and 6.4.2, consideration should be given to the mechanism of transfer of superstructure longitudinal and transverse inertia forces to the abutments, and also to abutment-soil interaction. To minimize potential loss of bridge access arising from abutment damage, monolithic or end diaphragm construction is strongly recommended for short span bridges.

Settlement or approach slabs providing structural support between approach fills and abutments shall be provided for all bridges classified as SPC D. Slabs shall be adequately linked to abutments using flexible ties.

CHAPTER 7

STRUCTURAL STEEL

7.1 GENERAL

Design and construction of structural steel columns and connections shall conform to the requirements of AASHTO Standard Specifications for Highway Bridges and to the additional requirements of this Chapter. Either Service Load or Load Factor design may be used. If Service Load design is used the allowable stresses are permitted to increase by 50%. It should be noted that when Service Load design is used for SPC C and D a conservative design may result because elastic design forces will be required for the design of most components unless the forces resulting from plastic hinging of the columns are used per Sec. 4.8.2.

7.2 SEISMIC PERFORMANCE CATEGORY A

No consideration of seismic forces is required for the design of structural components except for the design of the connection of the superstructure to the substructure as specified in Sec. 4.6.

7.3 SEISMIC PERFORMANCE CATEGORIES B, C AND D

Where axial and flexural stresses are determined by considering secondary bending resulting from the design P-delta effects (moments induced by the eccentricity resulting from the seismic displacements and the column axial force), all axially loaded members may be proportioned in accordance with AASHTO Sec. 1.7.45 or 1.7.69:
10.36 10.54

EXCEPTIONS:

1. The effective length factor, K, in the plane of bending may be assumed to be unity in the calculation of F_a, F_e', F_{cr}, or F_e.

2. The coefficient C_m is computed as for the cases where joint translation is prevented.

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CHAPTER 8

REINFORCED CONCRETE

8.1 GENERAL

Design and construction of cast-in-place monolithic reinforced concrete columns, pier footings and connections shall conform to the requirements of AASHTO Standard Specifications for Highway Bridges and to the additional requirements of this chapter. Either Service Load or Load Factor design may be used. If Service Load design is used the allowable stresses are permitted to increase by 33-1/3%. It should be noted that when Service Load design is used for SPC C and D a conservative design may result because elastic design forces will be required for the design of most components unless the forces resulting from plastic hinging of the columns are used per Sec. 4.8.2.

8.2 SEISMIC PERFORMANCE CATEGORY A

No consideration of seismic forces is required for the design of structural components except for the design of the connection of the superstructure to the substructure as specified in Sec. 4.6.

8.3 SEISMIC PERFORMANCE CATEGORY B

For bridges classified as SPC B the minimum transverse reinforcement requirements at the top and bottom of a column shall be as required in Sec. 8.4.1(D). The spacing of the transverse reinforcement shall be as required in Sec. 8.4.1(E) except that the maximum spacing is permitted to increase to 6 in. (152 mm).

8.4 SEISMIC PERFORMANCE CATEGORIES C AND D

Bridges classified as SPC C or D shall meet the requirements for SPC B and the requirements of Sec. 8.4.1 through 8.4.4.

8.4.1 COLUMN REQUIREMENTS

For the purpose of these provisions a vertical support is considered to be a column if the ratio of the clear height to the maximum plan dimensions of the support is equal to or greater than 2½. Note that the maximum plan dimension is taken at the minimum section of the flare for a flared column. For supports with a ratio less than 2½, the provisions for piers of Sec. 8.4.2 shall apply. For columns the provisions of this section are applicable. Note that a pier may be designed as a pier in its strong direction and a column in its weak direction.

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8.4.1(A) Vertical Reinforcement

The area of longitudinal reinforcement shall not be less than 0.01 or more than 0.06 times the gross cross-section area A_g.

EXCEPTION: 8.18.2.1
Article 1.5.11(A)(2) of AASHTO applies to columns where a larger cross-section is used for architectural reasons.

8.4.1(B) Flexural Strength

The biaxial strength of columns shall not be less than that required for the bending moments determined in Sec. 4.8.3. The design of the column shall be checked for both the minimum and maximum axial loads 8.16 specified in Sec. 4.8.3. The strength reduction factors of Sec. 1.5.30 of AASHTO shall be replaced for both spirally and tied reinforced columns by the value of 0.50 when the stress due to the maximum axial load for the column exceeds $0.20f_c'$. The value of ϕ may be increased linearly from 0.50 to the value for flexure (0.90) when the stress due to the maximum axial load is between $0.20f_c'$ and 0.

8.16.5 Moment magnification for slenderness effects (AASHTO Sec. 1.5.34) shall be considered in the design of the column.

8.4.1(C) Column Shear and Transverse Reinforcement

8.46

The factored design shear force V_u of Eq. 6-20 of AASHTO on each principal axis of each column and pile bent shall be the value determined in Sec. 4.8.3.

8.46

The factored shear stress v_u shall be computed by Eq. 6-20 of AASHTO using V_u specified above and the strength reduction factor for shear of Sec. 1.5.30 of AASHTO.

8.16

The amount of transverse reinforcement shall be at least that specified by Sec. 1.5.35 of AASHTO. In the end regions of the top and bottom of the column and pile bents the following provisions shall apply in addition to those of AASHTO:

1. In the end regions the quantity of shear stress taken by the concrete, v_c, shall be assumed to be zero unless the minimum design axial compression force produces an average stress in excess of $0.10f_c'$ over the gross concrete area.

2. When the average compression stress in the member exceeds $0.10f_c'$ the value of v_c shall be computed by Sec. 1.5.35 of AASHTO.
8.16.6

3. The end region shall be assumed to extend from the soffit of girders or cap beams at the top of columns, or the top of foundations at the bottom of columns, a distance not less than (a) the maximum cross-sectional dimension of the column, (b) one-sixth of the clear height of the column, (c) 18 in. (457 mm).

4. The end region of a pile bent shall be the same as specified for columns at the top of the pile bent, and three pile diameters below the calculated point of moment fixity to one pile diameter but not less than 18 in. above the mud line at the bottom of the pile bent.

8.4.1(D) <u>Transverse Reinforcement for Confinement at Plastic Hinges</u>

The cores of columns and pile bents shall be confined by transverse reinforcement in the expected plastic hinge regions, generally located at the top and bottom of columns and pile bents, as specified in this subsection. The largest of these requirements or those of Sec. 8.4.1(C) shall govern; these requirements are not in addition to those of Sec. 8.4.1(C). The transverse reinforcement for confinement shall have a yield strength not more than that of the longitudinal reinforcement and the spacing shall be as specified in Sec. 8.4.1(E).

The volumetric ratio of spiral reinforcement (ρ_s) for a circular column shall be either that required in AASHTO Sec. 1.5.11 or
8.18

$$\rho_s = 0.45 \left[\frac{A_g}{A_c} - 1 \right] \frac{f'_c}{f_{yh}} \tag{8-1}$$

or

$$\rho_s = 0.12 \frac{f'_c}{f_{yh}} \tag{8-2}$$

whichever is greater.

The total gross sectional area (A_{sh}) of rectangular hoop (stirrup) reinforcement for a rectangular column shall be either

$$A_{sh} = 0.30\ ah_c \frac{f'_c}{f_{yh}} \left[\frac{A_g}{A_c} - 1 \right] \tag{8-3}$$

or

$$A_{sh} = 0.12\ ah_c \frac{f'_c}{f_{yh}} \qquad (8\text{-}4)$$

whichever is greater, where:

a = vertical spacing of hoops (stirrups) in inches (millimeters) with a maximum of 4 in. (102 mm)

A_c = area of column core

A_g = gross area of column

A_{sh} = total cross sectional area in square inches (square millimeters) of hoop (stirrup) reinforcement including supplementary cross ties having a vertical spacing of a in. (mm) and crossing a section having a core dimension of h_c in. (mm). Note that this should be calculated for both principal axes of a rectangular column.

f'_c = specified compressive strength of concrete in psi (MPa)

f_{yh} = yield strength of hoop or spiral reinforcement in psi (MPa)

h_c = core dimension of tied column in inches (millimeters) in the direction under consideration

ρ_s = ratio of volume of spiral reinforcement to total volume of concrete core (out-to-out of spirals).

Transverse hoop reinforcement may be provided by single or overlapping hoops. Cross-ties having the same bar size as the hoop may be used. Each end of the cross-tie shall engage a peripheral longitudinal reinforcing bar. All ties shall have 135 degree hooks with extensions not less than the larger of ten tie diameters or 6 in. (152 mm).

8.4.1(E) <u>Spacing of Transverse Reinforcement for Confinement</u>

1. Transverse reinforcement for confinement shall be provided at the top and bottom of the column over a length equal to the maximum cross-sectional column dimension or one-sixth of the clear height of the column but not less than 18 in. Transverse reinforcement shall be extended into the top and bottom connections as specified in Sec. 8.4.3.

2. Transverse reinforcement for confinement shall be provided at the top of piles in pile bents over the same length as specified for columns. At the bottom of piles in pile bents, transverse reinforcement for confinement shall be

provided over a length extending from three pile diameters below the calculated point of moment fixity to one pile diameter but not less than 18 in. above the mud line.

3. The maximum spacing for reinforcement shall not exceed the smaller of one-quarter of the minimum member dimension or 4 in. (102 mm).

4. Lapping of spiral reinforcement in the transverse confinement regions specified in 1 and 2 shall not be permitted. Connections of spiral reinforcement in this region must be full strength lap welds.

8.4.1(F) Splices

Splices shall be in accordance with those specified in AASHTO Sec. 1.5.22 and the additional requirements of this section. Lap splices shall be permitted only within the center half of column height, and the splice length shall not be less than 16 in. (406 mm) or 60 bar diameters whichever is greater.

The maximum spacing of the transverse reinforcement over the length of the splice shall not exceed the smaller of 4 in. (102 mm) or one-quarter of the minimum member dimension.

Welded splices conforming to Sec. 12.15.3.3 of ACI 318-77 and approved mechanical splices conforming to Sec. 12.15.3.4 of ACI 318-77 may be used for splicing provided not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section and the distance between splices of adjacent bars is greater than 24 in. (610 mm) as measured along the longitudinal axis of the column.

8.4.2 PIER REQUIREMENTS

The provisions of this section are applicable to the design for the strong direction of a pier. The weak direction of a pier may be designed as a column and then the provisions of Sec. 8.4.1 are applicable with the Response Modification Factor for columns used to determine the design forces in Sec. 4.8.1. If the pier is not designed as a column in its weak direction then the limitations for shear stress in this section are applicable.

The minimum reinforcement ratio both horizontally, ρ_h, and vertically, ρ_n, in any pier shall not be less than 0.0025. Reinforcement spacing either horizontally or vertically shall not exceed 18 in. (457 mm). The reinforcement required for shear shall be continuous and shall be distributed uniformly.

ρ_h = the ratio of horizontal shear reinforcement area to gross concrete area of a vertical section

ρ_n = the ratio of vertical shear reinforcement area to the gross concrete area of a horizontal section.

The allowable shear stress, v_u, in the pier shall be determined in accordance with the following equation:

$$v_u = 2\sqrt{f'_c} + \rho_h f_y \qquad (8\text{-}5)$$

The allowable shear stress shall not exceed $8\sqrt{f'_c}$.

For lightweight aggregate concrete, the limiting shear stress, v_u, calculated from Eq. 8-5, shall be multiplied by 0.75. Two curtains of reinforcement shall be used and the reinforcement ratios ρ_n and ρ_h shall be equal. The reinforcement required by shear shall be distributed uniformly. Splices in horizontal pier reinforcement shall be staggered and splices in the two curtains shall not occur at the same location.

8.4.3 COLUMN CONNECTIONS

A column connection as referred to in this section is the vertical extension of the column area into the adjoining member.

The design force for the connection between the column and the cap beam superstructure, pile cap or spread footing shall be that specified in Sec. 4.8.5(C). The development length for all longitudinal steel shall be that required for a steel stress of $1.25f_y$ as given in Sec. 1.5.13 through 1.5.22 of AASHTO. 8.24

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Column transverse reinforcement required by Sec. 8.4.1(D) shall be continued for a distance equal to one-half the maximum column dimension but not less than 15 in. (381 mm) from the face of the column connection into the adjoining member.

The shear stress in the joint of a frame or bent, in the direction under consideration, shall not exceed $12\sqrt{f'_c}$ for normal-weight aggregate concrete or $9\sqrt{f'_c}$ for lightweight aggregate concrete.

8.4.4 CONSTRUCTION JOINTS IN PIERS AND COLUMNS

Construction joints in piers and columns resisting seismic forces shall be designed and constructed to resist the design forces at the joint.

Where shear is resisted at a construction joint solely by dowel action and friction on a roughened concrete surface, the total shear force across the joint shall not exceed V_j determined from the following formula:

$$V_j = \phi(A_{vf} f_y + 0.75\ P_n) \tag{8-6}$$

where A_{vf} is the total area of reinforcement (including flexural reinforcement) and P_n is the minimum axial load specified in Sec. 4.8.3 for columns and Sec. 4.8.4 for piers and ϕ is the strength reduction factor for shear of Sec. 1.5.30 of AASHTO.

The surfaces of all construction joints in components resisting lateral forces shall be thoroughly cleaned and roughened.

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COMMENTARY

CHAPTER 1 - INTRODUCTION

DESIGN PHILOSOPHY

Conceptually there are two seismic design approaches currently in use and both employ a "force design" concept. These are the current New Zealand and CalTrans criteria and are discussed in detail in references 1 and 2, respectively.

In the New Zealand Code, which accepts the philosophy that it is uneconomical to design a bridge to resist a large earthquake elastically, bridges are designed to resist small-to-moderate earthquakes in the elastic range. For large earthquakes the design philosophy is that bridges be ductile where possible. Flexural plastic hinging in the columns is acceptable but significant damage to the foundations and other joints is not. Consequently, as a second step in the design process, forces resulting from plastic hinging in all columns are determined and the capacities of connections to columns are checked to determine if they are able to resist these forces. Hence, critical elements in the bridge are designed to resist the maximum forces to which they will be subjected in a large earthquake.

In the CalTrans approach the member forces are determined from an elastic design response spectrum for a maximum credible earthquake. The design forces for each component of the bridge are then obtained by dividing the elastic forces by a reduction factor (Z). The Z-Factor is 1.0 and 0.8, respectively, for hinge restrainers and shear keys. These components are therefore designed for expected and greater-than-expected (in the case of shear keys) elastic forces resulting from a maximum credible earthquake. Well-confined ductile columns are designed for lower-than-expected forces from an elastic analysis as Z varies from 4 to 8. This assumes that the columns can deform plastically when the seismic forces exceed these lower design forces. The end result is similar to the New Zealand approach although the procedures are quite different.

In assessing bridge failures of past earthquakes in Alaska, California and Japan, many loss-of-span type failures are attributed in part to relative displacement effects. Relative displacements arise from out-of-phase motion of different parts of a bridge, from lateral displacement and/or rotation of the foundations and differential displacements of abutments. Therefore in developing the Guidelines the design displacements and forces were considered equally important. Thus minimum support lengths at abutments, columns and hinge seats are specified, and for bridges in areas of high seismic risk ties between noncontinuous segments of a bridge are specified. Special attention to the problem of relative displacements is required for bridges with high columns or piers.

The methodology used in the Guidelines is, in part, a combination of the CalTrans[2] and New Zealand[1] "force design" approaches but also

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addresses the relative displacement problem. The complexity of the methodology increases as the seismic intensity of an area increases. Four additional concepts are included in the Guidelines that are not included in either the CalTrans or New Zealand approach. First, minimum requirements are specified for support lengths of girders at abutments, columns and hinge seats to account for some of the important relative displacement effects that cannot be calculated by current state-of-the-art methods. A somewhat similar requirement is included in the latest Japanese[3] bridge criteria. Second, member design forces are calculated to account for the directional uncertainty of earthquake motions and the simultaneous occurrence of earthquake forces in two perpendicular horizontal directions. Third, design requirements and forces for foundations are intended to minimize foundation damage which is not readily detectable. Fourth, a basic premise in developing the Guidelines was that they be applicable to all parts of the United States. In order to provide flexibility in specifying design provisions associated with areas of different seismic risk, four Seismic Performance Categories (SPC) were defined. The four categories permit variation in the design requirements and analysis methods in accordance with the seismic risk associated with a particular bridge location. Bridges classified as SPC D are designed for the highest level of seismic performance and bridges classified as SPC A for the lowest level of seismic performance.

For bridges classified as SPC A, prevention of superstructure collapse is all that was deemed necessary for their level of seismic exposure. The requirements for these bridges are minimal and specify the support lengths for girders at abutments, columns and expansion joints, and that the design of the connections of the superstructure to the substructure be for 0.20 times the dead load reaction forces.

For bridges classified as SPC B the approach used is similar to that of CalTrans where elastic member forces are determined from a single-mode spectral method of analysis. Design forces for each component are obtained by dividing the elastic forces by a response modification factor (R). For connections at abutments, columns and expansion joints the R-Factor is either 1.0 or 0.8; therefore these components are designed for expected or greater-than-expected elastic forces. For columns and piers the R-Factor varies between 2 and 5 resulting in design forces lower than predicted by the elastic analysis. Therefore the columns are expected to yield when subjected to the forces of the design earthquake. This yielding in turn implies relative distortions of the structural system that must be considered in assessing the adequacy of the final bridge design. Design requirements to ensure reasonable ductility capacity of columns for bridges classified as SPC B are specified but they are not as stringent as those for bridges classified as SPC C and D. Foundations are designed for twice the seismic design forces of a column or pier.

For bridges classified as SPC C and D the general approach is similar to that for SPC B however several additional requirements are included. For columns, additional requirements are included to ensure that they are capable of developing reasonable ductility capacities. For connections and foundations, the recommended design forces are based on the maximum shears

and moments that can be developed by column yielding. Horizontal linkage and tie-down requirements at connections are also provided. For SPC D, approach slabs are required to ensure useability of the bridge after an earthquake.

SEISMIC GROUND MOTION ACCELERATIONS

Selection of the seismic ground motion to be used with the design provisions was carefully considered. Fortunately a comprehensive study entitled "Tentative Provisions for the Development of Seismic Regulations for Buildings" (ATC-3-06) had recently been completed in which seismic risk maps and an associated design spectrum were developed.[4] The ATC-3-06 maps are based on (1) a realistic appraisal of expected levels of ground motion shaking, (2) approximately the same probability that the design ground shaking will be exceeded for all parts of the United States, and (3) the frequency of occurrence of earthquakes in various regions of the country. A detailed discussion of the development of the seismic risk maps and the associated design spectrum is given in Sec. C3.2.

Although the probability is quite small, it is possible that in highly seismic areas near active faults the ground motions could exceed the design earthquake ground shaking. For locations inside the 0.40 contour of Fig. 3, it is recommended that a qualified professional be consulted to determine an appropriate value for the Acceleration Coefficient, A.

SOIL EFFECTS ON GROUND MOTION

It is generally recognized that the effects of local soil conditions on ground motion characteristics should be considered in structural design. Three fundamentally different approaches have been used:

- The first approach was based on the concept of potential resonance of a structure with the underlying soil. In the SEAOC building seismic requirements[5] the seismic site-structure resonance coefficient varies from 1.0 to 1.5 depending on the ratio of the fundamental building period to the characteristic site period.
- In a second approach, the computer program SHAKE[6] was used by CalTrans to develop soil amplification factors for its design criteria. The program analyzes a one-dimensional soil column for shear wave motions propagating from the rock level to the top of the soil column. The CalTrans approach is limited because only vertically propagating one-dimensional soil effects are considered and several parameters which could have significant effects are not considered. These parameters include surface waves, oblique transmission of waves through the soil and the effects of reflection and refraction at the interfaces of different material layers.
- For the third approach representative ground motion spectral shapes were modified in ATC-3-06[4] to determine corresponding

values of effective peak ground acceleration and smoothed spectral shapes for three typical site conditions. These modifications were based on a study of ground motions recorded at locations with different site conditions and the exercise of experienced judgment in extrapolating beyond the data base. Coefficients were developed for each of three typical soil conditions.

The ATC-3-06 approach for considering soil effects on ground motion is used in these Guidelines and is discussed in more detail in Sec. C3.2.

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4. Applied Technology Council, "Tentative Provisions for the Development of Seismic Regulations for Buildings," ATC Report No. 3-06, Berkeley, California, June 1978.

5. Structural Engineers Association of California, "Recommended Lateral Force Requirements and Commentary," 1975 Edition.

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COMMENTARY

CHAPTER 3 - GENERAL REQUIREMENTS

C3.1 APPLICABILITY OF GUIDELINES

The Guidelines present seismic design and construction requirements applicable to the majority of highway bridges to be constructed in the United States. Bridges not covered by these provisions probably constitute 5 to 15% of the total number of bridges designed.

The Project Engineering Panel (PEP) decided that special seismic design provisions would not be required for buried type structures. It was recognized by the PEP, however, that this decision may need reconsideration as more research data on the seismic performance of this type of structure becomes available.

The Guidelines specify minimum requirements. More sophisticated design and/or analysis techniques may be utilized if deemed appropriate by the design engineer.

A cautionary note was added to this section to alert the bridge designer that the Acceleration Coefficient within the 0.40 contour of Figs. 3 and 4 may be higher than 0.40. This possibility occurs because these contour maps were developed on a macro rather than micro scale and the influence of many smaller faults within the 0.40 contour was not considered.

For bridge types not covered by these Guidelines the following factors should be considered.

1. The recommended elastic design force levels of the Guidelines should be applicable because force levels are largely independent of the type of bridge structure, although a project may warrant a site-specific study to determine appropriate design force levels. If the site is near an active fault zone it is also recommended that qualified professionals familiar with local conditions be consulted, especially for locations within the 0.40 contour of Figs. 3 and 4.

   It should be noted that the elastic design force levels of the Guidelines are part of a design philosophy described in the introduction to this Commentary. The appropriateness of both the design force levels and the design philosophy must be assessed before they are used for bridges that are not covered by these Guidelines.

2. The Multimode Spectral Procedure described in Sec. 5.4 should be considered, especially if the Acceleration Coefficient for the bridge site is greater than 0.20. The designer should consider the pros and cons of using elastic and/or inelastic methods of time history analysis for larger and more complex types of

bridges. If these methods are used, appropriate time histories must be determined as part of the site specific study. It is recommended that at least three ground motion time histories be used in this type of analysis.

3. Design displacements are as important as design forces and, where possible, the design methodology should consider displacements arising from the effects discussed in Sec. C4.9.

4. If a design methodology similar to that used in these Guidelines is deemed desirable, the design requirements of Chapters 6, 7 and 8 should be used to ensure compliance with the design philosophy.

C3.2, C3.5 and C5.2 ACCELERATION COEFFICIENT, SITE EFFECTS AND ELASTIC SEISMIC RESPONSE COEFFICIENT AND SPECTRUM

The ground motion coefficient to be used with the Guidelines was originally developed as part of a similar but even more extensive study for buildings entitled "Tentative Provisions for the Development of Seismic Regulations for Buildings" (ATC-3-06).[1] Since the ground motion coefficient and associated elastic response spectrum are independent of the structural system, the ATC-3-06 values are used in these Guidelines.

Two coefficients and two corresponding maps were developed in the ATC-3-06 provisions. The two coefficients are the Effective Peak Acceleration Coefficient, A_a, and the Effective Peak Velocity-Related Acceleration Coefficient A_v. County-by-county and contour maps of the United States for each of the two coefficients are included in the ATC-3-06 report.

A major policy decision in the development of the bridge Guidelines was to use only the Effective Peak Velocity-Related Acceleration Coefficient A_v and to identify it as the Acceleration Coefficient A. Further it was decided that the contour maps for A_v would be used rather than the county-by-county maps of ATC-3-06. The decision to use only one coefficient was made to simplify the bridge guidelines. The decision to use a contour rather than county-by-county map was made because it was felt that the local jurisdictional problems with buildings were not of major importance for bridges.

The following is pertinent text extracted from the Commentary on ATC-3-06 provisions. For a more complete discussion see the ATC-3-06 Commentary.[1]

A. INTRODUCTION

It must be emphasized at the outset that the specification of earthquake ground shaking cannot be achieved solely by following a set of scientific principles. First, the causes of earthquakes are still not well understood and experts do not

fully agree as to how available knowledge should be interpreted to specify ground motions for use in design. Second, to achieve workable bridge design provisions it is necessary to simplify the enormously complex matter of earthquake occurrence and ground motions. Finally, any specification of a design ground shaking involves balancing the risk of that motion occurring against the cost to society of requiring that structures be designed to withstand that motion. Hence judgment, engineering experience, and political wisdom are as necessary as scientific knowledge. In addition, it must be remembered that design ground shaking alone does not determine how a bridge will perform during a future earthquake; there must be a balance of the specified shaking with the rules used to assess structural resistance to that shaking.

The recommended regionalization maps and seismic design coefficients and spectra are the work of several committees and are based upon the best scientific knowledge available in 1976, adjusted and tempered by experience. The following sections explain the bases for the various recommendations, as a guide both to the user of the provisions and to those who will improve the provisions in the future. It is expected that the maps and coefficients will change with time, as the profession gains more knowledge about earthquakes and their resulting ground motions and as society gains greater insight into the process of establishing acceptable risk.

B. POLICY DECISIONS

The recommended ground shaking regionalization maps are based upon two major policy decisions. The first is a departure from past practice in the United States whereas the second is a currently accepted practice.

The first policy decision was that the probability of exceeding the design ground shaking should, as a goal, be assumed to be equal in all parts of the country. The desirability of this goal is accepted within the profession; however, there is some disagreement as to the accuracy of estimates of probability of ground motion as determined from current knowledge and procedures. Use of a contour map based on uniform probability of occurrence is a departure from the use of the present zone maps which are based on estimates of maximum ground shaking experienced during the recorded historical period without any consideration of how frequently such motions might occur. It is also recognized that the real concern is with the probability of structural failures and resultant casualties and that the geographical distribution of that probability is not necessarily the same as the distribution of the probability of exceeding some ground motion. Thus the goal as stated is the

most workable one for the present but not necessarily the ideal one for the future.

The second policy decision was that the regionalization maps should not attempt to microzone. In particular, there was to be no attempt to locate actual faults on the regionalization maps, and variations of ground shaking over short distances--about 10 miles or less--were not to be considered. Any such microzoning must be done by qualified professionals who are familiar with localized conditions. Many local jurisdictions may find it expedient to undertake microzoning.

C. DESIGN EARTHQUAKE GROUND MOTION

The previous sections have discussed design ground shaking in general without being specific as to the meaning of the phrase. To state the concept rather than a precise definition, the design ground shaking for a location is the ground motion that the engineer should consider when designing a structure to provide a specified degree of protection for life safety and to prevent collapse.

At present, the best workable tool for describing design ground shaking is a smoothed elastic response spectrum for single degree-of-freedom systems.(2) Such a spectrum provides a quantitative description of both the intensity and frequency content of a ground motion. Smoothed elastic response spectra for 5% damping were used as a basic tool for the development of the regionalization maps and for the inclusion of the effects of local ground conditions. In effect, the first policy decision was reinterpreted to mean the probability of exceeding the ordinates of the design elastic response spectrum for all structural periods for a given location would be roughly equal. Again, this concept should be looked upon as a gradual goal, and not one that can be strictly met on the basis of present knowledge.

This should not be interpreted to mean that a structure can necessarily be designed for the forces implied by an elastic response spectrum. The design philosophy associated with the elastic response spectrum is at least as important as the level of the response spectrum.

A smoothed elastic response spectrum is not necessarily the ideal means for describing the design ground shaking. A time history analysis would be better, but a single time history motion generally is not adequate. It would be better to use a set of three or more acceleration time histories with an average elastic response spectrum similar to the design spectrum. This approach may be desirable for structures of special importance

but is not feasible for the vast majority of structures. This discussion is intended to emphasize that the design ground shaking is not a single motion, but rather a concept that encompasses a family of motions having the same overall intensity and frequency content but differing in some potentially important details of the time sequences of the motions.

A significant deficiency of the response spectrum is that it does not by itself include the duration of the shaking. The extent that duration affects elastic response is accounted for by the spectrum. However, the major effect of duration is upon possible loss of strength once a structure yields. Duration effects have not been explicitly considered in drawing up the recommended provisions, although in a general way it was envisioned that the design ground shaking might have a duration of 20 to 30 sec. The possibility that the design motion might be longer in highly seismic areas and shorter in less seismic areas was one of the considerations which influenced the design provisions for the various Seismic Performance Categories.

D. GROUND MOTION PARAMETERS

In developing the design provisions for buildings, two parameters were used to characterize the level of design ground shaking. These parameters are called the Effective Peak Acceleration (EPA) and the Effective Peak- Velocity-Related Acceleration (EPV). These parameters do not at present have precise definitions in physical terms but their significance may be understood from the following paragraphs.

The meaning of EPA and EPV is better understood if they are considered as normalizing factors for construction of smoothed elastic response spectra[2] for ground motions of normal duration (see Fig. 8). The EPA is proportional to spectral acceleration ordinates for periods in the range of 0.1 to 0.5 sec., while the EPV is proportional to spectral velocity ordinates at a period of about 1 sec.[3] The constant of proportionality (for a 5% damping spectrum) is set at a standard value of 2.5 in both cases.

For purposes of computing the lateral force coefficient in Sec. 3.3 of the ATC-3-06 provisions, EPA and EPV are replaced by dimensionless acceleration coefficients A_a and A_v, respectively. The coefficient A_a is numerically equal to EPA when EPA is expressed as a decimal fraction of the acceleration of gravity; e.g., if EPA = 0.20 g then A_a = 0.20. The coefficient A_v is proportional to EPV as explained in Sec. 1.4.1(F) of the ATC-3-06 Commentary.

E. RISK ASSOCIATED WITH THE CONTOUR MAP

The probability that the recommended EPA and EPV at a given location will not be exceeded during a 50-year period is estimated to be about 90%. At present, this probability cannot be estimated precisely. Moreover, since the maps were adjusted and smoothed by the committee after consultation with seismologists, the risk may not be the same at all locations. It is believed that this probability of the design ground motion not being exceeded is in the range of 80% to 90%. The use of a 50-year interval to characterize the probability is a rather arbitrary convenience, and does not imply that all structures are thought to have a useful life of 50 years.

The probability that an ordinate of the design elastic response spectrum will not be exceeded, at any period, is approximately the same as the probability that the EPA and the EPV will not be exceeded. The veracity of this statement lies in the fact that the uncertainty in the EPA and EPV that will occur in a future earthquake is much greater than the uncertainty in spectral ordinates, given the EPA and EPV. Thus the probability that the ordinates of the design elastic response spectrum will not be exceeded during a 50-year interval is also roughly 90%, or in the general range of 80 to 95%.

F. SITE EFFECTS AND ELASTIC SEISMIC RESPONSE COEFFICIENT AND SPECTRUM (Sec. 3.5 and 5.2)

At the present time there is a high degree of agreement that the characteristics of ground shaking and the corresponding spectra are influenced by:

1. The characteristics of the soil deposits underlying the proposed area.
2. The magnitude of the earthquake producing the ground motions.
3. The source mechanism of the earthquake producing the ground motions.
4. The distance of the earthquake from the proposed site and the geology of the travel path.

While it is conceptually desirable to include specific consideration of all four of the factors listed above it is not possible to do so at the present time because of lack of adequate data. Sufficient information is available to characterize in a general way the effects of specific soil conditions on effective peak acceleration and spectral shapes. The effects of the other factors are so little understood at this time that they are often not considered in spectral studies.

The present recommendations therefore only consider effects of site conditions and distance from the seismic source zone. At such times that potential effects of other significant parameters can be delineated and quantified, the current recommendations can be modified to reflect these effects.

Thus, the starting points in the development of the ground motion spectra are the seismic design regionalization maps that express, by contours, the EPA and the EPV that would be developed on firm ground.

SITE EFFECTS

The fact that the effects of local soil conditions on ground motion characteristics should be considered in structural design has long been recognized in many countries of the world. Most countries considering these effects have developed different design criteria for several different soil conditions. Typically these criteria use up to four different soil conditions. In the early part of the ATC-3-06 study consideration was given to four different conditions of local site geology.

On the basis of available data, the following four conditions were selected:

1. Rock - of any characteristic, whether it be shalelike or crystalline in nature. In general, such material is characterized by a shear wave velocity greater than about 2,500 ft/sec (762 m/sec).
2. Stiff soil conditions or firm ground - including any site where soil depth is less than 200 ft (61 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.
3. Deep cohesionless or stiff clay soil conditions - including sites where the soil depth exceeds about 2,500 ft (762 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.
4. Soft to medium-stiff clays and sands - characterized by several tens of feet of soft to medium-stiff clay with or without intervening layers of sand or other cohesionless soils.

Effective Peak Accelerations for Different Site Conditions

The values of EPA for rock conditions were first modified to determine corresponding values of EPA for the three other site conditions outlined above. This modification was based on a statistical study of peak accelerations developed at locations

with different site conditions and the exercise of judgment to extrapolate beyond the data base.

After evaluating these effects and rounding out the results obtained, the values of EPA were modified as follows. For the first three soil types (rock, shallow stiff soils and deep cohesionless or stiff clay soils) there is no reduction. For the fourth soil type (soft to medium-stiff clays) a reduction factor of 0.8 is used for all seismicity index areas. It should be pointed out that statistical data show that the reduction effect is not constant for all ground motion levels and that the value of the reduction factor is generally smaller than is recommended here.

Spectral Shapes

Spectral shapes representative of the different soil conditions discussed above were selected on the basis of a statistical study of spectral shapes developed on such soils close to the seismic source zone in past earthquakes. The mean spectral shapes determined directly from the study by Seed et al.[4] based on 104 records, primarily from earthquakes in the Western United States, are shown in Fig. 9. These spectral shapes were also compared with spectral shapes from studies conducted by Blume,[5] Newmark,[6] and Mohraz.[7] It was considered appropriate to simplify the curves to a family of three by combining the spectra for rock and stiff soil conditions; the normalized spectral curves are shown in Fig. 10. The curves in this figure thus apply to the following three soil conditions.

> Soil Profile Type I: Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2,500 ft/sec (762 m/sec)); or stiff soil conditions where the soil depth is less than 200 ft (61 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.
>
> Soil Profile Type II: Deep cohesionless or stiff clay soil conditions, including sites where the soil depth exceeds 200 ft (61 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.
>
> Soil Profile Type III: Soft to medium-stiff clays and sands, characterized by 30 ft (9.1 m) or more of soft to medium-stiff clay with or without intervening layers of sand or other cohesionless soils.

Ground motion spectra for 5% damping for the different map areas are thus obtained by multiplying the normalized

spectral values shown in Fig. 10 by the appropriate EPA and by the correction factor of 0.8 if Soil Profile Type III exists. The resulting ground motion spectra for an EPA of 0.4 are shown in Fig. 11. The spectra from Fig. 11 are shown in Fig. 12 plotted in tripartite form. It can be readily seen in Fig. 12 that for all soil conditions the response spectra for periods near 1 sec. are horizontal or equivalent to a constant spectral velocity. It should also be noted that these spectra are modified as discussed in the following section before they are used in the design provisions. On the basis of studies of spectral shapes conducted by Blume[5] and Newmark,[6] spectra for 2% damping may be obtained by multiplying the ordinates of Fig. 10 by a factor of 1.25.

Spectra for vertical motions may be determined with sufficient accuracy by multiplying the ordinates of the spectra for horizontal motions by a factor of 0.67.

Elastic Seismic Response Coefficient and Spectra

The equivalent lateral force method of design requires that a horizontal force be accommodated in the structural design. The magnitude of this force is a function of several parameters including the Acceleration Coefficient, the type of site soil profile, and the fundamental period of the structure.

For use in a design provision or code it is distinctly advantageous to express the lateral design force coefficient in as simple a manner as possible. The recommended procedure for determining the lateral design force coefficient C_s is given in Sec. 5.2 as follows:

$$C_s = \frac{1.2AS}{T^{2/3}} \qquad (5\text{-}1)$$

The value of C_s need not exceed 2.5A for Type I, II or III soils. The soil coefficient S is given in Table 2. The use of a simple soil factor in Eq. 5-1 directly approximates the effect of local site conditions on the design requirements. This direct method eliminates the need for the estimation of a predominant site period and the computation of a soil factor based on the site period and the fundamental period of the bridge.

This concludes the text abstracted from the Commentary of the ATC-3-06 provisions.

It is apparent from the discussion on spectral shapes in the foregoing paragraphs and Figs. 10 and 11 that the recommended elastic acceleration response spectrum decreases approximately as 1/T for longer periods. However, because of the concerns associated with inelastic response of longer period

bridges it was decided that the ordinates of the design coefficients and spectra should not decrease as rapidly as 1/T but should be proportional to $1/T^{2/3}$ in Eqs. 5-1 and 5-2.

A comparison of the spectra resulting from Eqs. 5-1 and 5-2 and those of the ATC-3-06 recommended elastic acceleration response spectra is given in Fig. 13. The PEP decided that the elastic seismic response coefficient and spectra should be approximately 50% greater at a period of 2 sec. for the stiff soil condition than would be obtained by direct use of the recommended elastic acceleration response spectra. This increase should gradually decrease as the period of the bridge shortens. The two major reasons for introducing this convervatism in the design of long period bridges is:

1. The fundamental period of a bridge increases as the column height increases, the span length increases and the number of columns per bent decreases. Hence the longer the period the more likely that high ductility requirements will be concentrated in a few columns.
2. Instability of a bridge is more of a problem as the period increases.

Other factors associated with modification of the elastic seismic response spectra are given in Sec. C5.2.2.

C3.3 IMPORTANCE CLASSIFICATION

The Importance Classification (IC) is used in conjunction with the Acceleration Coefficient (A) to determine the Seismic Performance Category (SPC) for bridges with an Acceleration Coefficient greater than 0.29. The SPC controls the degree of complexity and sophistication of the analysis and design requirements.

Two Importance Classifications are specified. An IC of I is assigned for essential bridges and II for all others. Essential bridges are those that must continue to function after an earthquake. The determination of the Importance Classification of a bridge is necessarily subjective. Consideration should be given to the following Social/Survival and Security/Defense requirements. An additional consideration would be average annual daily traffic.

The Social/Survival evaluation is largely concerned with the need for roadways during the period immediately following an earthquake. In order for civil defense, police, fire department or public health agencies to respond to a disaster situation a continuous route must be provided. Bridges on such routes should be classified as essential.

Survival and mitigation of the effects of the earthquake are of primary concern following a seismic event. Transportation routes to critical facilities such as hospitals, police and fire stations and communication centers must continue to function and bridges required for this purpose should be classified as essential. In addition a bridge that has the

potential to impede traffic if it collapses onto an essential route should also be classified as essential.

The health and well-being of the community is another major concern. Victims with critical injuries or illnesses must be treated; food, water and shelter provided and utilities restored. Routes to such facilities as schools, arenas, etc., which could provide shelter or be converted to aid stations must suffer little or no damage and bridges on such routes should be classified as essential. Access must be available to power installations, water treatment plants, etc. and bridges required for these purposes should also be classified as essential.

The importance evaluation of a bridge for Social/Survival significance in a disaster situation depends on the range of options available and the possibility of a bridge being in parallel or series with other bridges in a roadway network. Discussion may be required with highway, civil defense and police officials.

A basis for the Security/Defense evaluation is the 1973 Federal-Aid Highway Act which required that a plan for defense highways be developed by each state. This plan had to include, as a minimum, the interstate and Federal-Aid primary routes; however, some of these routes can be deleted when such action is considered appropriate by a state. The defense highway network provides connecting routes to important military installations, industries and resources not covered by the Federal-Aid primary routes and includes:

1. Military bases and supply depots and National Guard installations.
2. Hospitals, medical supply centers and emergency depots.
3. Major airports.
4. Defense industries and those that could easily or logically be converted to such.
5. Refineries, fuel storage, and distribution centers.
6. Major railroad terminals, railheads, docks, and truck terminals.
7. Major power plants including nuclear power facilities and hydroelectric centers at major dams.
8. Major communication centers.
9. Other facilities that the state considers important from a national defense viewpoint or during emergencies resulting from natural disasters or other unforeseen circumstances.

Bridges serve as important links in the Security/Defense roadway network and such bridges should be classified as essential.

C3.4 SEISMIC PERFORMANCE CATEGORIES

A basic premise in developing the Guidelines was that they be applicable to all parts of the United States. The seismic risk varies from very small to high across the country and design requirements applicable to the higher risk areas are not always appropriate for the lower risk areas. In order to provide flexibility in specifying design provisions associated with areas of different seismic risk, four Seismic Performance Categories (SPC) were defined. The four categories permit variation in the requirements for methods of analysis, minimum support lengths, column design details, foundation and abutment design requirements in accordance with the seismic risk associated with a particular bridge location.

The Seismic Performance Category is determined from the Importance Classification of Sec. 3.3 and the Acceleration Coefficient of Sec. 3.2. Thus the importance of a bridge in a road network and the level of seismic exposure at a bridge site are used to determine the SPC. Different degrees of complexity in analysis and design requirements are specified for each SPC. Bridges classified as SPC D are those designed for the highest level of seismic performance and bridges classified as SPC A are those designed for the lowest level of seismic performance.

C3.5 SITE EFFECTS

See Sec. C3.2(F).

C3.6 RESPONSE MODIFICATION FACTORS

Response modification factors (R) shown in Table 3 are used to modify the component forces obtained from the elastic analysis. Inherent in the R values is the assumption that columns will yield when subjected to forces induced by the design ground motions and that connections and foundations are designed to accommodate the design ground motion forces with little, if any, damage.

The rationale used in the development of the R-Factors for columns, piers and pile bents was based on considerations of redundancy and ductility provided by the various supports. The wall type pier was judged to have minimal ductility capacity and redundancy in its strong direction and was assigned an R-Factor of 2. A multiple column bent with well-detailed columns, as specified in Chapter 8, was judged to have good ductility capacity and redundancy and was assigned the highest value of 5. The ductility capacity of single columns is similar to that of columns in a multiple column bent; however, there is no redundancy and therefore a lower R-Factor of 3 was assigned to single columns to provide a level of performance similar to that of multiple column bents. Unfortunately little information was available on the performance of pile bent substructures in actual earthquakes and the R-Factors were based on the PEP's judgment of potential pile bent performance in comparison to that of the other three

types of substructure. It was believed that there would be a reduction in the ductility capacity of pile bents with batter piles and therefore lower R-Factors were assigned to these systems.

The R-Factors of 1.0 and 0.8 assigned to connections mean that the connections are designed for the design elastic forces and for greater than the design elastic forces in the case of abutments. This approach was adopted in part to accommodate the redistribution of forces that occurs when a bridge responds inelastically.[8] The other reason for adopting these values was to maintain the overall integrity of the bridge structure at these important joints. Increased protection can be obtained for a minimum increase in construction cost by designing connections for these larger force levels. However it should be noted that for bridges classified as SPC C and D the recommended design forces for column connections are the forces that can be developed by plastic hinging of the columns. Since these are the maximum forces that can be developed and are generally smaller than the elastic values the desired integrity will be obtained at lower cost. The connection design forces associated with plastic hinging are not specified for bridges classified as SPC B because plastic hinging requires a more detailed analysis as per Sec. C4.8.2.

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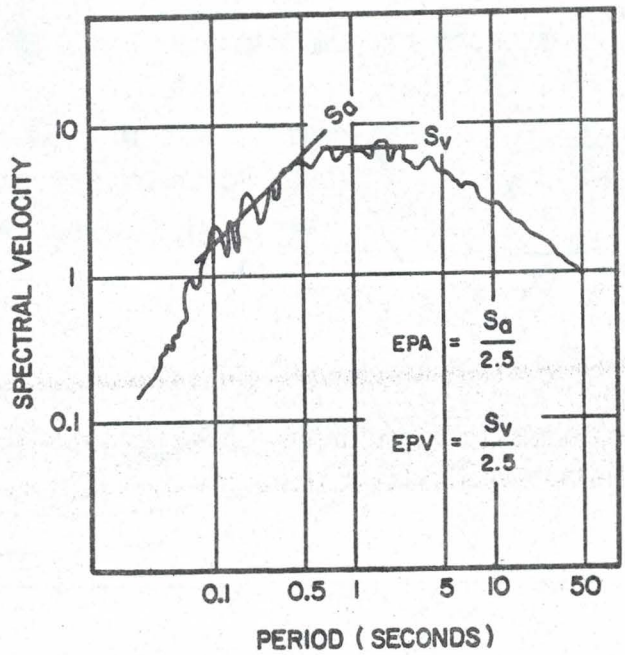

FIGURE 8. SCHEMATIC REPRESENTATION SHOWING HOW EFFECTIVE PEAK ACCELERATION AND EFFECTIVE PEAK VELOCITY ARE OBTAINED FROM A RESPONSE SPECTRUM

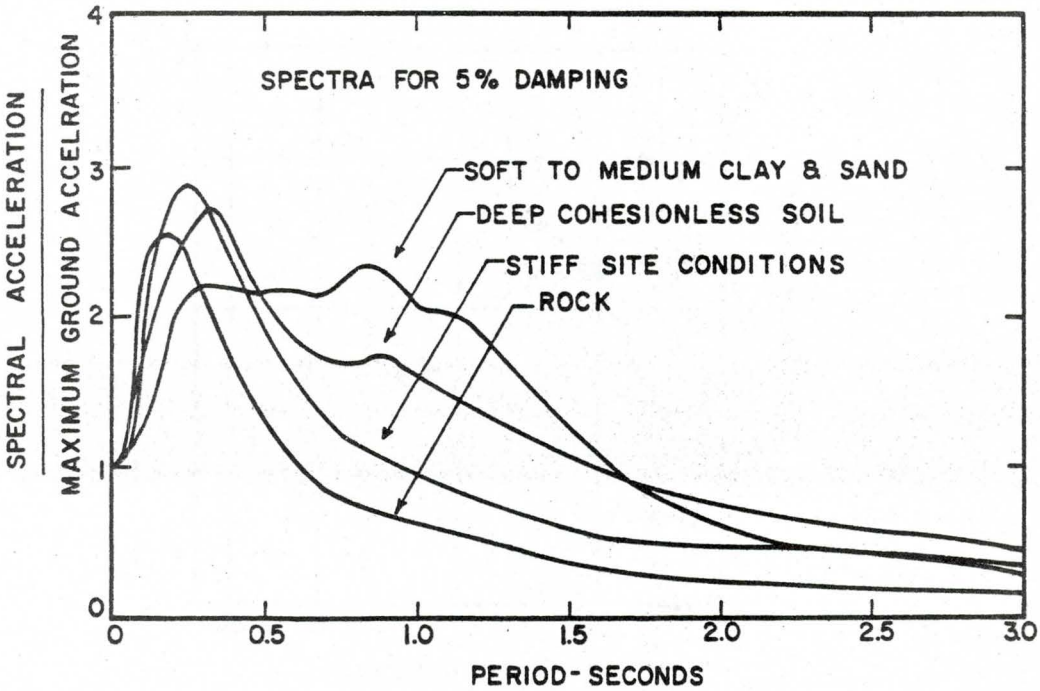

FIGURE 9. AVERAGE ACCELERATION SPECTRA FOR DIFFERENT SITE CONDITIONS (after SEED, et al., 1976)

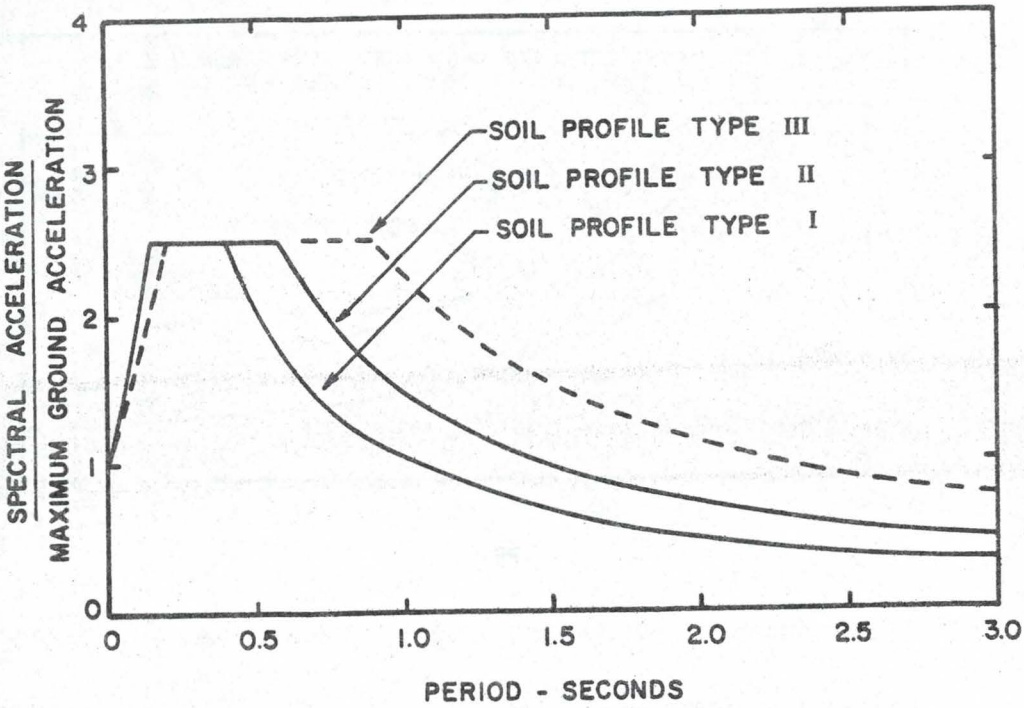

FIGURE 10. NORMALIZED RESPONSE SPECTRA

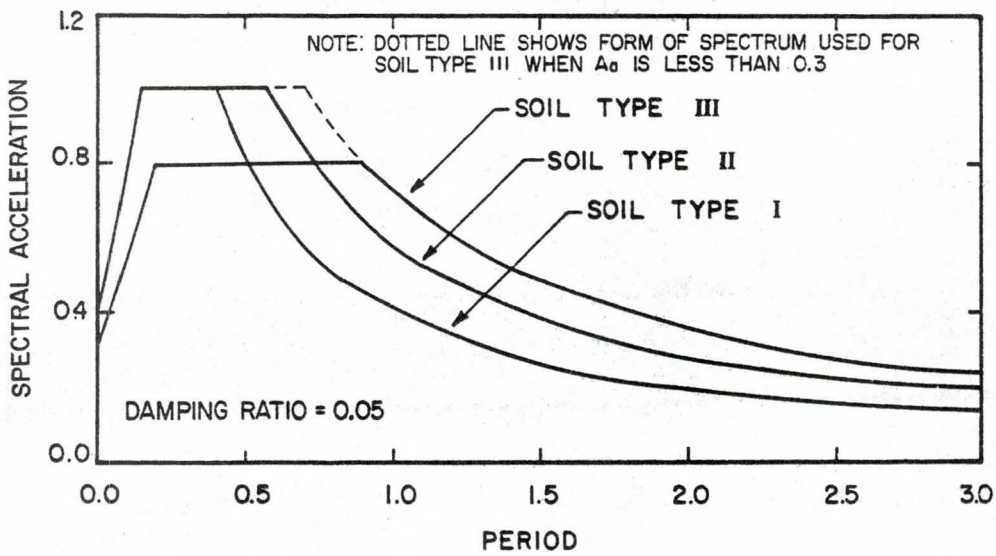

FIGURE 11. GROUND MOTION SPECTRA FOR A = 0.4

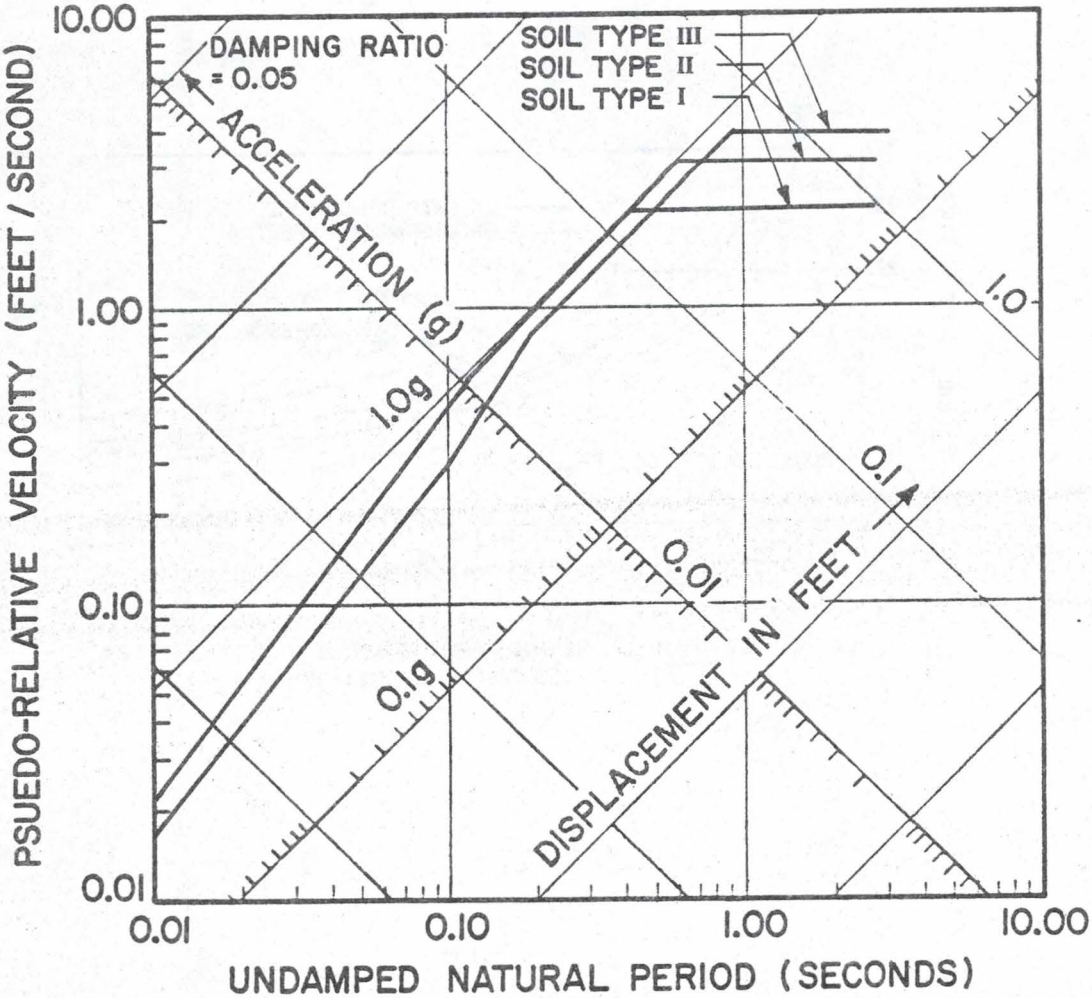

FIGURE 12. GROUND MOTION SPECTRA FOR A = 0.4

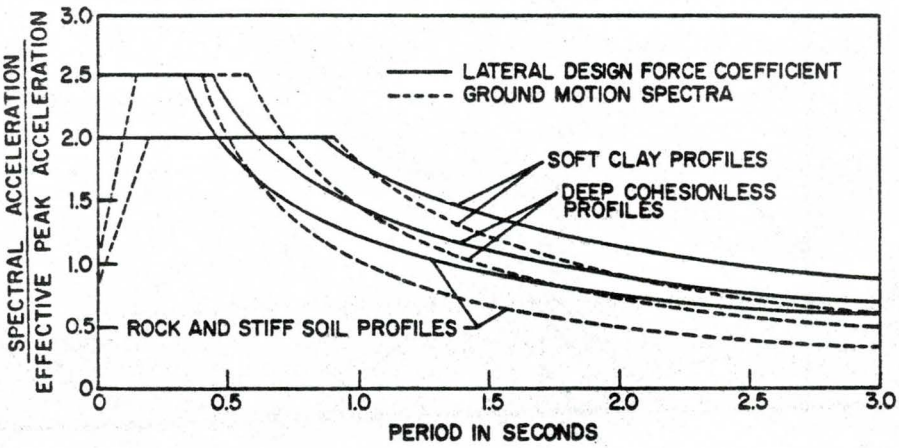

FIGURE 13. COMPARISON OF FREE FIELD GROUND MOTION SPECTRA AND LATERAL DESIGN FORCE COEFFICIENTS

COMMENTARY

CHAPTER 4 - ANALYSIS AND DESIGN REQUIREMENTS

C4.1 GENERAL

These Guidelines provide for bridge design (sizing of individual members, connections and supports) based on internal forces derived by modifying the results from a linear elastic analysis. The provisions of the Guidelines assume that the columns may yield during an earthquake but that damage to connections and foundations will be minimized.

C4.2 ANALYSIS PROCEDURE

An elastic analysis procedure is used for the seismic design of bridges to give the designer an indication of the force distribution to the structural members and to give him some indication of the relative deformations. It also provides the basis for the design of the components. The actual forces and displacements in a bridge subjected to the design ground motions may be quite different from those obtained from the elastic analysis because at these high levels of excitation the bridge may respond inelastically.

Two analytical procedures are specified. Procedure 1, the single-mode spectral method of analysis, requires calculation of the fundamental period (T) of the bridge. A reasonable estimate of the elastic forces and displacements can then be made for regular bridges. Limits on the applicability of the method have not been exhaustively determined and the definition of a regular bridge given in Table 4 needs further study. The limits on the use of this procedure are imposed because higher modes of vibration are believed to affect the distribution of forces and resulting displacements significantly if the bridge does not meet the definition of a regular bridge. In these instances the multimode spectral method should be used.

Procedure 2, the multimode spectral method of analysis, is the more sophisticated of the two procedures and generally requires the use of a digital computer. It is very effective for analyzing the response of any linearly elastic structure to any prescribed dynamic excitation. Two multimode methods of analysis are generally used; one is spectral analysis and the other is time history analysis. The spectral analysis does not directly account for the phase relationships between the modes of vibration whereas the time history analysis does. A statistical approach (square root of the sum of the squares) is used to combine the contributions of different modes of vibration in the spectral analysis. This is a major limitation since the accuracy of this approach for bridges has not been thoroughly validated. The time history method accounts for the phase relationships between modes but requires the determination of appropriate ground motion time histories. Its major limitation is due to the uncertainties in ground

motion studies. If a time history elastic or inelastic analysis is performed, a determination of appropriate time histories must be made as part of the site specific study and it is recommended that a minimum of three different time histories be used.

The details of the two procedures are presented in the commentary of Chapter 5. For bridges classified as SPC C and D with three or more spans, the designer should seriously consider including the flexibility of the foundations and abutments in the analysis.

C4.3 DETERMINATION OF ELASTIC FORCES AND DISPLACEMENTS

Current knowledge of earthquake ground motions indicates that structures will be subjected to simultaneous ground motion in three orthogonal directions.[1] For many bridges the effect of the vertical component of motion may not be important and a detailed analysis in the vertical direction is not required. However, for bridges classified as SPC C and D the effect is accounted for by the design requirements of Sec. 4.8.5(B).

To account for the two horizontal components of motion, an analysis is required in two orthogonal directions, generally the longitudinal and transverse directions of the bridge. Forces and moments resulting from these analyses are then combined as specified in Sec. 4.4 to account for the simultaneous occurrence of forces in two horizontal directions.

The forces and displacements obtained from an elastic analysis should be similar to those to which the bridge would be subjected if it responded elastically and the actual ground motion had similar characteristics to the design ground motion. Thus the displacements resulting from this analysis are used as a lower bound for the design displacements.

C4.4 COMBINATION OF ORTHOGONAL SEISMIC FORCES

The method of combining forces for each of the load cases is given by means of an example. The two principal transverse axes of a column, abutment, pier, etc. may be designated as the z and y axes. The shear (V), moment (M), and axial (P) forces resulting from an analysis of the bridge subjected to loads in the transverse direction are designated as V_z^T, V_y^T, M_z^T, M_y^T, and P^T, respectively. The corresponding forces resulting from an analysis of loads in the longitudinal direction are designated V_z^L, V_y^L, M_z^L, M_y^L, and P^L, respectively. The design shear (V_z^D, V_y^D), moment (M_z^D, M_y^D) and axial (P^D) forces for the z and y axes of the member for the two load cases are as follows:

LOAD CASE 1

$$V_z^D = 1.0 \left|V_z^L\right| + 0.3 \left|V_z^T\right|$$

$$V_y^D = 1.0 \left|V_y^L\right| + 0.3 \left|V_y^T\right|$$

$$M_z^D = 1.0 \left|M_z^L\right| + 0.3 \left|M_z^T\right|$$

$$M_y^D = 1.0 \left|M_y^L\right| + 0.3 \left|M_y^T\right|$$

$$P^D = 1.0 \left|P^L\right| + 0.3 \left|P^T\right|$$

LOAD CASE 2

$$V_z^D = 0.3 \left|V_z^L\right| + 1.0 \left|V_z^T\right|$$

$$V_y^D = 0.3 \left|V_y^L\right| + 1.0 \left|V_y^T\right|$$

$$M_z^D = 0.3 \left|M_z^L\right| + 1.0 \left|M_z^T\right|$$

$$M_y^D = 0.3 \left|M_y^L\right| + 1.0 \left|M_y^T\right|$$

$$P^D = 0.3 \left|P^L\right| + 1.0 \left|P^T\right|$$

The symbol | | denotes the absolute value or the magnitude of the force or moment without regard to its sign since a seismic force can act in either direction. It should be noted that, for a straight bridge with no skewed piers, columns or abutments, the above combinations simplify significantly because a transverse load will primarily produce moments and shear forces in the z direction of the structural member and the longitudinal load will primarily produce moments and shear forces in the y direction.

C4.5 DESIGN REQUIREMENTS FOR SINGLE SPAN BRIDGES

Single span bridges were separated from bridges with two or more spans for analysis and design requirements because of their response to seismic loads. This response was judged, on the basis of past performance, to be satisfactory provided there is sufficient support for the girders in both longitudinal and transverse directions. The design requirements for the connections are necessary to prevent damage and excessive deflections. The design forces are based on the premise that the bridge is very stiff. This

assumption on the stiffness of the bridge also acknowledges the fact that the period of vibration is difficult to calculate because of significant interaction with the abutments.

C4.6 DESIGN FORCES FOR SEISMIC PERFORMANCE CATEGORY A

Prior to the redesign phase of the project the PEP thought that the design of connections for wind forces would be satisfactory for anticipated seismic forces for bridges classified as SPC A. However when the magnitude of the wind and seismic forces were compared for six bridges, it was found in almost all cases that, for an Acceleration Coefficient of 0.10, seismic forces were greater than wind forces. In some cases the difference was significant. Hence it was deemed necessary to include the requirement of this section for the design of the connections. The requirement is simple and somewhat conservative, especially for more flexible bridges, since the forces are based on the maximum elastic response coefficient. If the design forces are difficult to accommodate it is recommended that SPC B analysis and design procedures be used.

C4.7 DESIGN FORCES FOR SEISMIC PERFORMANCE CATEGORY B

The seismic design forces specified for bridges classified as SPC B are intended to be relatively simple but consistent with the overall design concepts and methodology. Inherent in any simplification of a design procedure, however, is a degree of conservatism and for SPC B this occurs in the determination of the design forces for the foundations and connections to columns. If these forces appear to be excessive then the method specified for bridges classified as SPC C and D in Sec. 4.8.5 and 4.8.6 should be used. The major difference is that, for SPC C and D, foundations and connections to columns are designed for the maximum forces that a column can transmit to these components. In some cases these may be considerably less then the design forces specified in Sec. 4.7.

Section 4.7.1 specifies the design forces for the structural components of the bridge. In the first step the elastic forces of Load Cases 1 and 2 of Sec. 4.4 are divided by the appropriate R-Factors of Sec. 3.6. These forces are combined with those from other loads and the group loading combination is the same as that used in the current AASHTO Specifications with all γ and β factors equal to 1.0. The major difference between the Guidelines and the current AASHTO Interim Specifications is the R-Factors of the Guidelines and the Z-Factors of the AASHTO Interim Specifications by which the elastic forces are divided. Furthermore, in the Guidelines each component shall be designed to resist the two seismic group load combinations of Sec. 4.4, one including Load Case 1 and the other including Load Case 2. Each load case incorporates different proportions of bidirectional seismic loading whereas in the AASHTO requirements only unidirectional loading is considered. This may be important for some components (e.g., biaxial design of columns) and unimportant for others. In the design loads for each component the sign of the seismic forces and moments obtained from Sec. 4.4 can be taken as either

positive or negative. The sign of the seismic force or moment that gives the maximum magnitude for the design force (either positive or negative) shall be used.

Either the load factor or service load method of design according to the Standard Specifications for Highway Bridges, 12th Edition, adopted by AASHTO, can be used with the specified forces. For essential bridges in SPC B a designer may wish to consider the column design requirements for SPC C and D in Chapters 7 and 8 to enhance the column ductility capacity. However, for most bridges the AASHTO requirements and the additional requirements of Chapter 8 were deemed reasonable in view of the seismic risk level associated with SPC B. The ductility capacity of a column designed to the AASHTO Specifications is difficult to estimate because the potential mode of failure could be shear, flexure, compression, or loss of anchorage or a combination of any two or more. The design requirements of Chapters 7 and 8 for bridges classified as SPC C and D are specified such that the potential for a shear, compression or loss of anchorage mode of failure is minimized and the column is forced to yield in flexure with reasonable ductility capacity when subjected to significant seismic force levels.

Section 4.7.2 specifies the design forces for foundations which include the footings, pile caps and piles. The design forces are essentially twice the seismic design forces of the columns. This will generally be conservative and was adopted to simplify the design procedure for bridges classified as SPC B. However if seismic forces do not govern the design of columns and piers there is a possibility that during an earthquake the foundations will be subjected to forces larger than the design forces. This will occur if the columns remain elastic throughout the duration of the seismic ground motion. Thus for important bridges classified as SPC B consideration should be given to the use of the forces specified in Sec. 4.8.6 for foundations in SPC C and D. It should be noted that ultimate soil and pile strengths are to be used with the specified foundation seismic design forces.

C4.8 DESIGN FORCES FOR SEISMIC PERFORMANCE CATEGORIES C AND D

For bridges classified as SPC C and D two sets of design forces are defined and either one, or both sets, is specified as the design force for different components. If two sets are specified the designer has an either/or option with one set being more conservative than the other. The major difference between these design forces and those specified for bridges classified as SPC B is that one set of these defined forces corresponds to forces resulting from plastic hinging in the columns. The design forces for the various components are specified in Sec. 4.8.3 to 4.8.7.

C4.8.1 MODIFIED DESIGN FORCES

Section 4.8.1 defines the modified design forces which are used for the design of some components of the bridge. In the first step the

elastic forces of Load Cases 1 and 2 of Sec. 4.4 are divided by the appropriate R-Factor of Sec. 3.6. In combining these forces with those from other load types the group loading combination is the same as that used in Group VII of the current AASHTO Specifications with all γ and β factors equal to 1.0. The major difference between the Guidelines and the current AASHTO Interim Specifications is the R-Factors of the Guidelines and the Z-Factors of the AASHTO Interim Specifications by which the elastic forces are divided. Furthermore, in the Guidelines, two seismic group load combinations are defined; one for Load Case 1 of Sec. 4.4 and the other for Load Case 2. Each load case incorporates different proportions of bidirectional seismic loading whereas AASHTO only considers unidirectional loading. This may be important for some components (e.g., biaxial design of columns) and unimportant for others. In the design loads for each component for the group load combination, the sign of the seismic forces and moments obtained from Sec. 4.4 can be either positive or negative. The sign of the seismic force or moment that gives the maximum magnitude for the design force (either positive or negative) shall be used. The exception to this is for column axial loads in which the seismic axial load is considered alternately as a positive and negative load so that a minimum and maximum axial force is calculated for each load case.

C4.8.2 FORCES RESULTING FROM PLASTIC HINGING IN COLUMNS, PIERS OR BENTS

Section 4.8.2 defines the forces resulting from plastic hinging (a column reaching its ultimate moment capacity) in the columns and presents two procedures: One is for a single column hinging about its two principal axes; this is also applicable for piers and bents acting as single columns. The other procedure is for a multiple column bent in the plane of the bent. The forces are based on the potential overstrength capacity of the materials and to be valid the design detail requirements of Chapters 7 and 8 must be used so that plastic hinging of the columns can occur. The overstrength capacity results from actual material strengths (steel yield strength, concrete compressive strength) being greater than the minimum specified strengths. This fact must be accounted for when forces generated by yielding of the column are used as design forces.

The shear mode of failure in a column or pile bent will probably result in a partial or total collapse of the bridge; therefore, the design shear force must be calculated conservatively. In calculating the column or pile bent shear force, consideration must be given to the potential locations of plastic hinges. For flared columns these may occur at the top or bottom of the flare. For multiple column bents with a partial-height wall the plastic hinges will probably occur at the top of the wall unless the wall is structurally separated from the column. For columns with deeply embedded foundations the plastic hinge may occur above the foundation mat or pile cap. For pile bents the plastic hinge may occur above the calculated point of fixity. Because of the consequences of a shear failure it is recommended that conservatism be used in locating possible plastic hinges such that the smallest potential column length be used with the plastic moments to calculate the largest potential shear force for design.

C4.8.3 COLUMN AND PILE BENT DESIGN FORCES

The design forces for columns specified in Sec. 4.8.3 are based on the design philosophy of the Guidelines discussed in the Introduction to the Commentary. The design moments are specified on the assumption that the column is expected to yield when subjected to the forces of the design earthquake.

The design axial forces which control both the flexural design of the column and the shear design requirements are either the maximum or minimum of the unreduced design forces or the values corresponding to plastic hinging of the columns. In most cases the values corresponding to plastic hinging of the columns will be lower than the unreduced design forces. The design shear forces are specified so that the possibility of a shear failure in the column is minimized.

C4.8.4 PIER DESIGN FORCES

The design forces for piers specified in Sec. 4.8.4 are based on the assumption that a pier has low ductility capacity and no redundancy. As a result a low Response Modification Factor of 2 is used in determining the reduced design forces and it is expected that only a small amount of inelastic deformation will occur in the response of a pier when subjected to the forces of the design earthquake. If a pier is designed as a column in its weak direction then both the design forces and, more important, the design requirements of Sec. 4.8.3 and Chapter 8 are applicable.

C4.8.5 CONNECTION DESIGN FORCES

Connections are important elements in maintaining the overall integrity of a bridge structure. Therefore, specific attention was given to the displacements that occur at moveable supports (Sec. 4.9) and, for fixed connections, reasonably conservative design forces are specified to provide increased protection at minimum increase in construction cost.

The recommended design forces specified in Sec. 4.8.5 are such that column connections are designed for the maximum forces that a column can transmit to the connection (Sec. 4.8.5(C)). The design forces for other connections and the alternate forces for column connections are the elastic seismic forces specified in Sec. 4.8.1. Forces greater than the elastic seismic forces are specified in the case of abutment connections. An additional requirement to prevent significant relative displacements at connections is given in Sec. 4.8.5(A). Positive horizontal linkage shall be provided by cables or an equivalent mechanism. Friction shall not be considered as positive linkage. As a further safety measure, minimum bearing support lengths are required. The problem of relative displacement is discussed in more detail in Sec. C4.9.

Section 4.8.5(B) presents the only provision included in the Guidelines to minimize the potential adverse effects of vertical seismic excitation as discussed in Sec. C4.3. This is a reasonably straightforward requirement and will be subject to refinement as the state-of-the-art in the effects of vertical seismic excitation develops.

C4.8.6 FOUNDATION DESIGN FORCES

The foundation design forces specified in Sec. 4.8.6 are consistent with the design philosophy of minimizing damage that would not be readily detectable. The recommended design forces are the maximum forces that can be transmitted to the footing by plastic hinging of the column. The alternate design forces are the elastic design forces. It should be noted that these may be considerably greater than the recommended design forces although where architectural considerations govern the design of a column the alternate elastic design forces may be less than the forces resulting from column plastic hinging.

C4.9 DESIGN DISPLACEMENTS

In developing the Guidelines the PEP considered design displacements to be as important as design forces because many of the loss-of-span type failures in past earthquakes have been attributed in part to relative displacement effects.

The length of support provided at abutments, columns and hinge seats must accommodate displacements resulting from the overall inelastic response of the bridge structure, possible independent movement of different parts of the substructure, and out-of-phase rotation of abutments and columns resulting from traveling surface wave motions.

A reasonable estimate of the displacements resulting from the overall elastic dynamic response of the bridge structure can be obtained from the multimode spectral method of analysis if the flexibility of the foundations is included. Better estimates can be obtained if an inelastic time history analysis is performed; however, this is not recommended in the Guidelines because of the complexities involved in performing this method of analysis. Either the elastic or inelastic time history analysis will give reasonable estimates of the out-of-phase movements of different parts of the substructure whereas the multimode method of spectral analysis will not. The recent work of Elms et al.[(2),(3)] can be used to give the order of magnitude of abutment movement and the recent work of Werner et al.[(4),(5)] gives some indication of the effects of traveling waves on the responses of a limited number of bridges. However, much research remains to be done in both these areas.[(1)]

In summary, the current state of the art precludes a good estimate of the differential column and abutment displacements to be expected when a bridge is subjected to an earthquake. The PEP believes it necessary to

specify minimum support lengths at abutments, piers and hinge seats to provide for the effects discussed above. If the displacements resulting from the elastic analysis of Sec. 4.3 exceed the minimum specified values, the values resulting from the elastic analysis must be used in the design. The minimum support lengths specified are dependent on the deck length between expansion joints and the column height since both dimensions influence one or more of the factors that cause the differential displacements. Although a considerable amount of judgment was exercised on the basis of current knowledge, the proposed criteria should be refined as the state of the art develops.

C4.9.1 SEISMIC PERFORMANCE CATEGORY A

Since an elastic analysis is not required for bridges classified as SPC A the minimum support lengths specified in Sec. 4.9.1 are the only design displacement requirements for these bridges.

C4.9.2 and C4.9.3 SEISMIC PERFORMANCE CATEGORIES B, C AND D

For bridges classified as SPC B, C and D the design displacements are specified as the maximum of those determined from the elastic analysis of Sec. 4.3 or the minimum specified support lengths given by Eqs. 4-3 and 4-4. This either/or specification was introduced to account for larger displacements that may occur from the analysis of more flexible bridges. It was the opinion of the PEP that displacements obtained from the elastic analysis of bridges should provide a reasonable estimate of the displacements resulting from the inelastic response of the bridge. However, it must be recognized that displacements are very sensitive to the flexibility of the foundation and if the foundation is not included in the elastic analysis of Sec. 4.3, consideration should be given to increasing the specified displacements for bridges founded on very soft soils. This increase may be of the order of 50% or more but as with any generalization considerable judgment is required. A better method is to determine upper and lower bounds from an elastic analysis which incorporates foundation flexibility. Special care in regard to foundation flexibility is required for bridges with high piers.

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COMMENTARY

CHAPTER 5 - ANALYSIS PROCEDURES

C5.1 GENERAL

This chapter of the Guidelines presents two analytical procedures to determine the distribution of forces for the prescribed seismic loadings. Both are based on linear elastic analysis techniques.

C5.2 ELASTIC SEISMIC RESPONSE COEFFICIENT AND SPECTRUM

C5.2.1 ELASTIC SEISMIC RESPONSE COEFFICIENT

See Sec. C3.2(F)

C5.2.2 ELASTIC SEISMIC RESPONSE SPECTRUM

Equation 5-4 is to be used if a modal period exceeds 4 sec. It can be seen that Eqs. 5-4 and 5-2 coincide at $T_m = 4$ sec, so that the effect of using Eq. 5-4 is to provide a more rapid decrease in C_{sm} as a function of T_m than implied by Eq. 5-2. This modification is introduced in consideration of the known characteristics of earthquake response spectra at intermediate and long periods. At intermediate periods the average velocity spectrum of strong earthquake motions from large earthquakes (magnitude 6.5 and larger) is approximately horizontal; this implies that C_{sm} should decrease as $1/T_m$. In Eq. 5-2 C_{sm} decreases as $1/T_m^{2/3}$ for reasons discussed in Sec. C3.2(F), and this slower rate of decrease, if extended to very long periods, would result in an unbalanced degree of conservatism in the modal force for very flexible bridges. In addition, for very long periods, the average displacement spectrum of strong earthquake motions becomes horizontal; this implies that C_{sm}, which is a form of acceleration spectrum, should decay as $1/T_m^2$. The period at which the displacement response spectrum becomes horizontal depends on the size of the earthquake, being longer for large earthquakes, and a representative period of 4 sec was chosen to make the transition.

A central feature of modal analysis is that the earthquake response is considered as a combination of the independent responses of the bridge vibrating in each of its important modes. As the bridge vibrates back and forth in a particular mode at the associated period, it experiences maximum values of member forces and displacements. The coefficient C_{sm} is determined for each mode from Eq. 5-2 using the associated period of the mode, T_m, in addition to the factors A and S, which are discussed elsewhere in this Commentary. An exception to this procedure occurs for higher modes of those bridges which have periods shorter than 0.3 sec and which are founded on Type III soils. For such modes, Eq. 5-3 is used. Equation 5-3 gives values ranging from 0.8 A for very short periods to 2.0 A for

T_m = 0.3. Comparing these values with the limiting value of C_s of 2.0 A for Type III soils as specified following Eq. 5-2, it is seen that the use of Eq. 5-3, when applicable, reduces the modal base shear. This is an approximation introduced in consideration of the conservatism embodied in using the spectral shape specified by Eq. 5-2 and its limiting values. This shape is a conservative approximation to that of average spectra which are known to first ascend, then level off, and then decay as period increases - see Figs. 9 and 10. Equation 5-2 and its limiting values conservatively replace the ascending portion for small periods by a level portion. For Type I or II soils, the ascending portion of the spectrum is completed by the time the period reaches a value near 0.1 or 0.2 sec. On the other hand, for soft soils the ascent may not be completed until a larger period is reached. Equation 5-3 is then a replacement for Type III soils and short periods, which is more consistent with spectra for measured accelerations. It was introduced because it was judged unnecessarily conservative to use Eq. 5-2 for modal analysis in the case of Type III soils.

C5.3 SINGLE MODE SPECTRAL ANALYSIS METHOD

The single mode spectral analysis method is used to calculate the seismic design forces for bridges that respond predominantly in the first mode of vibration. The method, although completely rigorous from a structural dynamics point of view, reduces to a problem in statics after the introduction of inertia forces. The method, as formulated, can be applied to many types of bridges which have both continuous and non-continuous superstructures. Boundary conditions at the abutments and piers can also be modeled to include the effects of foundation flexibility.

Bridges are generally continuous systems consisting of many components which contribute to the overall resistance capacity of the system. Consider a bridge subjected to a transverse earthquake ground motion. The bridge is composed of several spans restrained transversely at the end abutments and intermediate piers, as shown in Fig. 14. Typically the bridge deck may have expansion joints at the piers or within the spans. These expansion joints do not have the capability to transmit transverse deck moments between adjacent deck sections. The equation of motion for a continuous system representing this system is conveniently formulated using energy principles. The

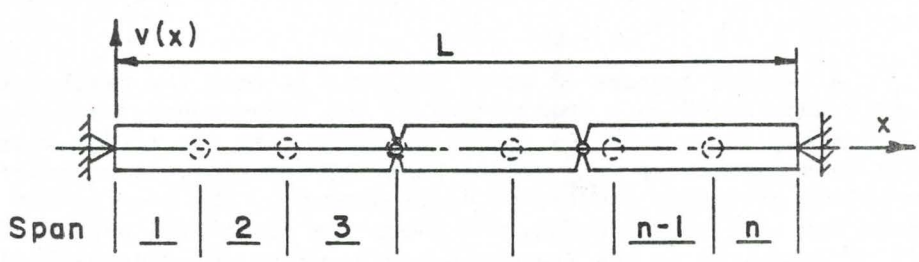

FIGURE 14. PLAN VIEW OF A BRIDGE SUBJECTED TO A TRANSVERSE EARTHQUAKE MOTION

principle of virtual displacements may be used to formulate a generalized parameter model of a continuous system in a manner which approximates the overall behavior of the system. Assuming transverse motion in a single mode shape, a single degree-of-freedom "generalized parameter" model may be formulated. To obtain an approximation to this mode shape, a uniform static loading, p_o, is applied to the superstructure and the resulting deflection, $v_s(x)$, is obtained. The dynamic deflection, $v(x,t)$, of the structure under seismic excitation as shown in Fig. 15 is then approximated by the shape function multiplied by a generalized amplitude function, $v(t)$, as shown by Eq. C5-1.

$$v(x,t) = v_s(x)v(t) \ . \qquad \text{(C5-1)}$$

This function will describe the deformed bridge structure in a manner which is consistent with the support conditions and intermediate expansion joint hinges in the deck. Note that it is an admissible function which satisfies the geometric boundary conditions of the system.

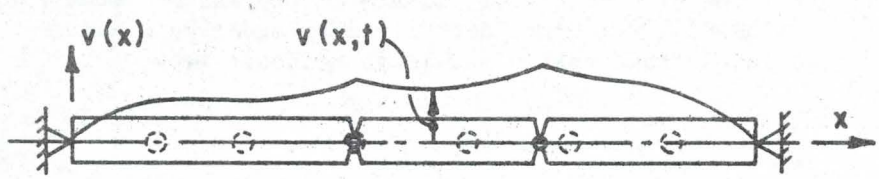

FIGURE 15. DISPLACEMENT FUNCTION DESCRIBING THE TRANSVERSE POSITION OF THE BRIDGE DECK

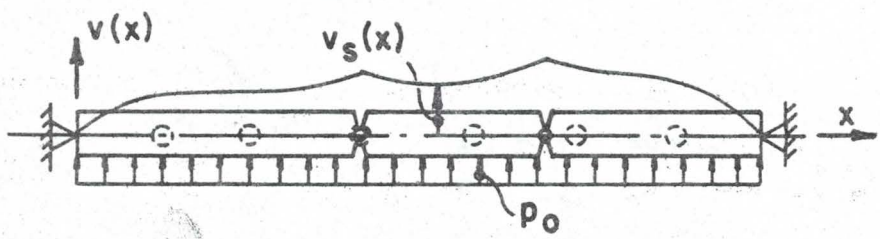

FIGURE 16. DEFLECTED SHAPE DUE TO UNIFORM STATIC LOADING

Initially, to establish the deflected shape for the generalized parameter model, apply a uniform loading p_o to the structure as shown in Fig. 16. Assume that the loading is applied gradually so that the kinetic energy of the mass of the structure is zero. The external work, W_E, done by the uniformly applied loading in deforming the structure is given by

$$W_E = \frac{p_o}{2}\int_o^L v_s(x)dx = \frac{p_o}{2}\alpha \qquad \text{(C5-2)}$$

where

$$\alpha \equiv \int_o^L v_s(x)dx \ . \qquad \text{(C5-3)}$$

This work will be stored internally in the elastic structure in the form of strain energy U; thus,

$$U = W_E \ . \qquad \text{(C5-4)}$$

After $v_s(x)$ is determined using any standard static analysis approach, the integral in Eq. C5-3, appearing in Step 2 of the Guidelines, may be evaluated numerically.

If the uniform loading p_o is suddenly removed, and the effects of damping are neglected, the structure will vibrate in the assumed mode shape shown in Fig. 17 at a natural frequency determined by equating maximum kinetic energy to maximum strain energy (Rayleigh method); i.e.

$$T_{max} = U_{max} \ . \qquad \text{(C5-5)}$$

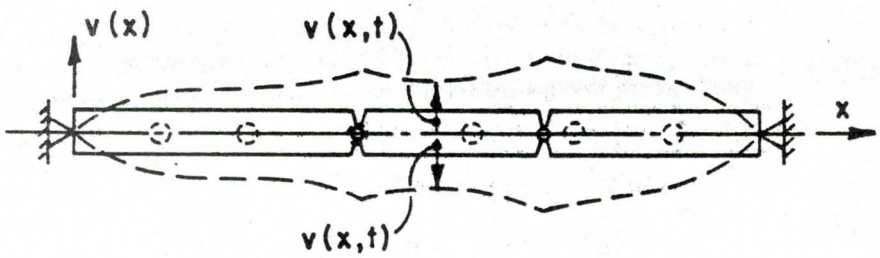

FIGURE 17. TRANSVERSE FREE VIBRATION OF THE BRIDGE IN ASSUMED MODE SHAPE

The maximum kinetic energy of the system is given by

$$T_{max} = \frac{\omega^2}{2g}\int_o^L w(x)v_s(x)^2dx = \frac{\omega^2\gamma}{2g} \qquad \text{(C5-6)}$$

where

$$\gamma \equiv \int_o^L w(x)v_s(x)^2dx \qquad \text{(C5-7)}$$

and ω is the frequency of the vibrating system. The factor γ defined in Eq. C5-7 and appearing in Step 2 of the Guidelines, is evaluated numerically.

The maximum strain energy stored in the system is

$$U_{max} = W_E \ . \tag{C5-8}$$

Using Eqs. C5-2, C5-6 and C5-8, Eq. C5-5 becomes

$$p_o \alpha = \frac{\omega^2 \gamma}{g} \ . \tag{C5-9}$$

Introducing $\omega = 2\pi/T$ into Eq. C5-9 and solving for the period T, yields

$$T = 2\pi \sqrt{\frac{\gamma}{p_o g \alpha}} \ . \tag{C5-10}$$

The generalized equation of motion for the single degree-of-freedom system subjected to a ground acceleration $\ddot{v}_g(t)$ may be written as

$$\ddot{v}(t) + 2\xi\omega\, \dot{v}(t) + \omega^2 v(t) = \frac{-\beta \ddot{v}_g(t)}{\gamma} \tag{C5-11}$$

where

$$\beta \equiv \int_o^L w(x) v_s(x)\, dx \tag{C5-12}$$

and ξ is the damping ratio to be prescribed. For most structures, a value of 0.05 is recommmended. Using the standard acceleration response spectral value C_s in its dimensionless form,

$$C_s = \frac{S_a(\xi, T)}{g} \tag{C5-13}$$

where $S_a(\xi, T)$ is the pseudo acceleration spectral value.

The maximum response of the system is given by

$$v(x,t)_{max} = v(t)_{max} v_s(x) \tag{C5-14}$$

where

$$v(t)_{max} = \frac{C_s g \beta}{\omega^2 \gamma} \ . \tag{C5-15}$$

Thus

$$v(x,t)_{max} = \frac{C_s g \beta}{\omega^2 \gamma} v_s(x) \ . \qquad \text{(C5-16)}$$

The static loading $P_e(x)$ which yields the displacement $v(x,t)_{max}$ is shown in Fig. 18 and is given by

$$p_e(x) = \frac{\beta C_s}{\gamma} w(x) v_s(x) \ . \qquad \text{(C5-17)}$$

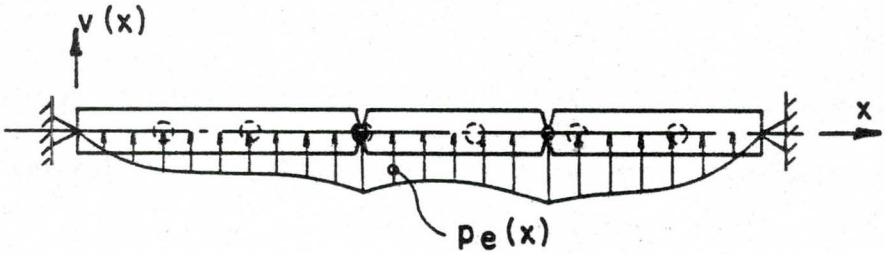

FIGURE 18. CHARACTERISTIC STATIC LOADING APPLIED TO THE BRIDGE SYSTEM

C5.4 MULTIMODE SPECTRAL ANALYSIS METHOD

C5.4.1 GENERAL

The multimode response spectrum analysis should be performed with a suitable linear dynamic analysis computer program. Programs generally available with these capabilities include: STRUDL, SAP4, SAP6, ANSYS, STARDYN, NASTRAN, EASE, and MARC.

C5.4.2 MATHEMATICAL MODEL

The model type and degree of refinement depends on the complexity of the actual structure and the results desired in the analysis. Modeling a bridge for a dynamic analysis is currently more an art than a science. The overall objective is to produce a mathematical model that will represent the dynamic characteristics of the structure and produce realistic results consistent with the input parameters. This section is intended to provide some basic guidelines which will yield realistic results for most bridge structures. Although the terms "joint" and "node" are generally used interchangeably, for the purposes of these Guidelines the term "node" is used to indicate the use of a joint specifically for the purposes of mathematically modelling mass or inertia characteristics. Condensation of mass terms should be done with care to prevent the loss of the inertia effects of the structure.

The force-displacement relationship at bridge abutments is a highly complex nonlinear problem and will be affected by the abutment design. In the absence of more accurate information, the following iterative technique may be used to determine an equivalent elastic transverse and longitudinal stiffness at the abutments to be used for the analysis of typical bridge structures. The procedure is outlined in the flowchart appearing in Fig. 19 and described in the following steps:

1. Assume an initial abutment design and stiffness.
2. Analyze the bridge and determine the forces at the abutment. Perform the appropriate following steps:
   (a) If the force levels exceed the acceptable capacity of the abutment fill and/or piles, reduce the stiffness of the abutments until the analysis indicates force levels below the acceptable capacity.
   (b) If the force levels are below the acceptable capacity of the abutments, proceed to Step 3.
3. Observe the analyzed displacements at the abutment and take the appropriate following step:
   (a) If displacements exceed acceptable levels, the assumed abutment design is inadequate. Redesign the abutment and return to Step 1.
   (b) If displacements are acceptable, the last assumed abutment stiffness is consistent with the assumed abutment design.

C5.4.3 MODE SHAPES AND PERIODS

The computer programs mentioned in Sec. C5.4.1 have the ability to calculate the mode shapes, frequencies and resulting member forces and displacements for a multimode spectral analysis. The following equations summarize the equations used in such an analysis.

Mode shapes and frequencies should be obtained from the equation

$$\left[\underline{k} - \omega^2 \underline{m}\right] \hat{\underline{v}} = \underline{0} \qquad \text{(C5-18)}$$

using standard eigenvalue computer programs; where $\underline{k}$ and $\underline{m}$ are the known stiffness and mass matrices of the mathematical model, respectively, $\hat{\underline{v}}$ is the displacement amplitude vector, and ω is the frequency. This analysis will yield the dimensionless mode shapes $\underline{\phi}_1, \underline{\phi}_2, \ldots, \underline{\phi}_n$ and their corresponding circular frequencies $\omega_1, \omega_2, \ldots, \omega_n$. The mode periods can then be obtained using

$$T_i = \frac{2\pi}{\omega_i} \quad (i=1,2,\ldots,n) \; . \qquad \text{(C5-19)}$$

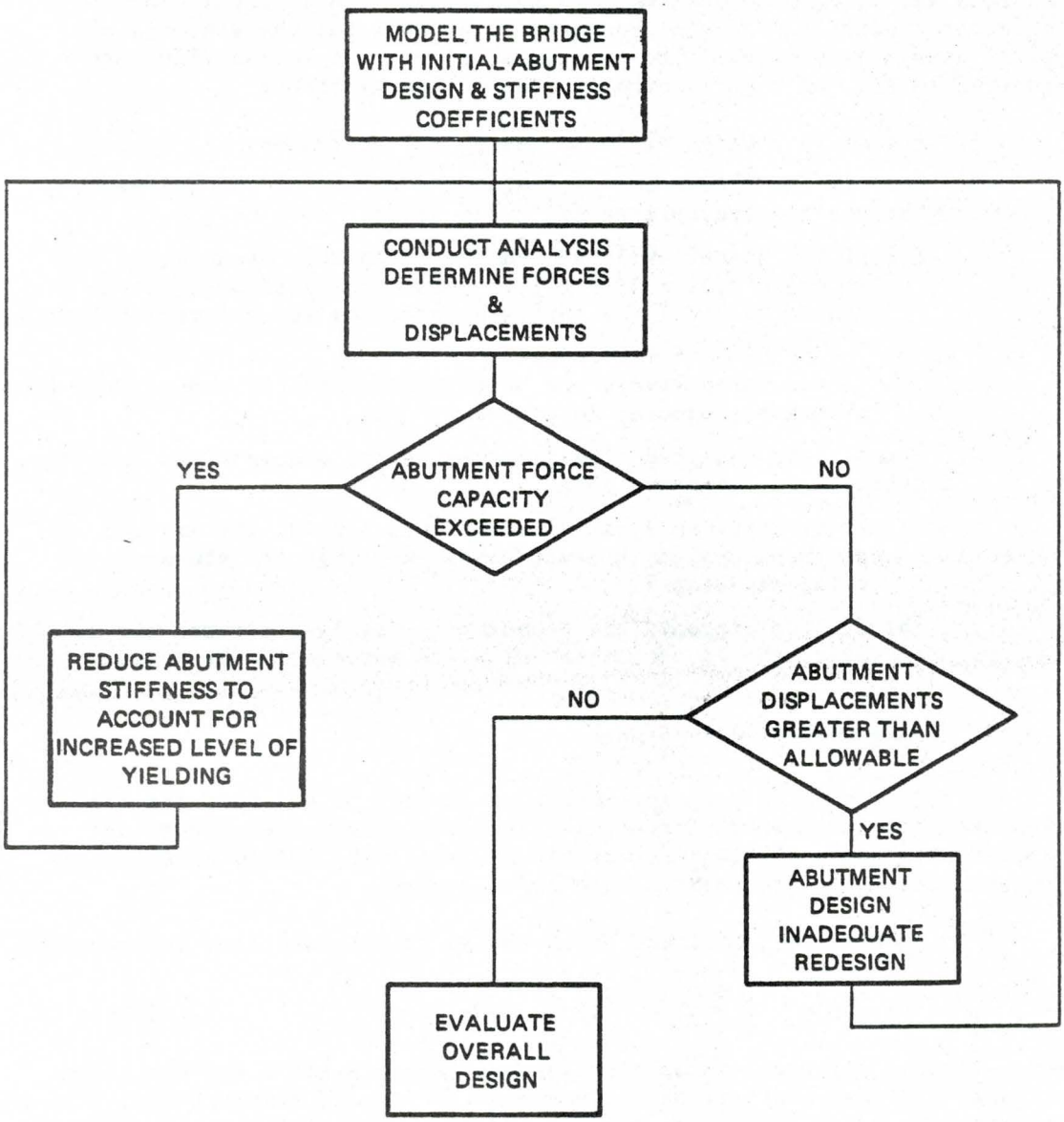

FIGURE 19. ITERATIVE PROCEDURE FOR INCLUDING ABUTMENT SOIL EFFECTS IN THE SEISMIC ANALYSIS OF BRIDGES

C5.4.4 MULTIMODE SPECTRAL ANALYSIS

The uncoupled normal mode equations of motion are of the form

$$\ddot{Y}_i(t) + 2\omega_i \xi_i \dot{Y}_i(t) + \omega_i^2 Y_i(t) = \frac{P_i(t)}{M_i} \qquad \text{(C5-20)}$$

$(i=1,2,\ldots,n)$

where the subscript i refers to the mode number, Y_i, ω_i and ξ_i are the mode amplitude, frequency, and damping ratios, respectively, and the effective modal load $P_i(t)$ and generalized mass M_i are given by

$$P_i(t) = \underline{\phi}_i^T \underline{m}\, \underline{B}\, \ddot{v}_g(t)$$

$$M_i = \underline{\phi}_i^T \underline{m}\, \underline{\phi}_i \qquad \text{(C5-21)}$$

where $\underline{B}$ is a vector containing ones and zeros corresponding to those components in the direction of excitation $\ddot{v}_g(t)$ and those components in the other orthogonal directions, respectively.

The maximum absolute value of $Y_i(t)$ during the entire time-history of earthquake excitation is given by

$$Y_i(t)_{max} = \frac{T_i^2 S_a(\xi_i, T_i)}{4\pi^2} \frac{\underline{\phi}_i^T \underline{m}\, \underline{B}}{\underline{\phi}_i^T \underline{m}\, \underline{\phi}_i} \qquad \text{(C5-22)}$$

where $S_a(\xi_i, T_i)$ is the acceleration response spectral value for the prescribed earthquake excitation. In these Guidelines $S_a(\xi_i, T_i)$ is obtained from the equation:

$$S_a(\xi_i, T_i) = gC_{sm} \qquad \text{(C5-23)}$$

where C_{sm} is defined through the empirical relation given by Eqs. 5-2, 5-3 or 5-4.

To determine the maxiumum value of any particular response quantity Z(t) (e.g., a shear, moment, displacement or relative displacement), use is made of the fact that it is linearly related to the normal mode amplitude, i.e.,

$$Z(t) = \sum_{i=1}^{n} A_i Y_i(t) \qquad \text{(C5-24)}$$

where coefficients A_i are known. The maximum value of Z(t) during the duration of the earthquake can be estimated using the square root of the sum of the squares (SRSS) method for systems having well-separated modes, i.e., using

$$|Z(t)|_{max} = \sqrt{\sum_{i=1}^{n} A_i^2 |Y_i(t)|_{max}^2} \quad . \qquad \text{(C5-25)}$$

C5.4.5 MEMBER FORCES AND DISPLACEMENTS

The member forces and displacements of an elastic structure are obtained by the superposition of the respective quantities of the individual modes of vibration. Generally, the maximum values for each mode do not occur simultaneously and thus the maximum value of each mode cannot be directly superimposed to obtain the maximum force or displacement of a member. The direct superposition (absolute sum) of the individual modal contributions thus provides an upper bound which is generally conservative and not recommended for design. A satisfactory estimate of the maximum value of a force or displacement can be obtained by taking the square root of the sum of the squares (SRSS) of the individual modal response as defined by Eq. C5-25.

The SRSS method is generally applicable to most bridges, however there are some bridges with unusual geometric features which cause some of the individual modes to have closely spaced periods to which this method may not be applicable. There are several methods currently available and new methods are emerging for combining these closely spaced modes. One of these methods, which may in some cases be conservative, suggests that the absolute values of closely spaced modes be added to the SRSS of the remaining modes. At present, however, there are not enough data available for bridges comparing these response spectrum results with time history analyses to provide a verifiable basis on which to make recommendations.

COMMENTARY

CHAPTER 6 - FOUNDATION AND ABUTMENT DESIGN REQUIREMENTS

C6.3, C6.4 and C6.5 SEISMIC PERFORMANCE CATEGORIES B, C and D

The Commentary on Chapter 6 is not broken down by Seismic Performance Category because most of the commentary on liquefaction, foundations, piles and abutments is applicable to all three categories.

C6.3.1, C6.4.1 and C6.5.1 FOUNDATIONS

C6.3.1(A), C6.4.1(A) and C6.5.1(A) Investigation

Slope instability, liquefaction, fill settlement and increases in lateral earth pressure have often been major factors in contributing to bridge damage in past earthquakes. These earthquake hazards may be significant design factors for peak earthquake accelerations in excess of 0.1g and should form part of a site specific investigation if the site conditions and the associated acceleration levels and design concepts suggest that such hazards may be of importance. Since liquefaction has contributed to many bridge failures, methods for evaluating site liquefaction potential are described in more detail below.

Liquefaction Potential. Liquefaction of saturated granular foundation soils has been a major source of bridge failures during historic earthquakes. For example, during the 1964 Alaska earthquake, 9 bridges suffered complete collapse, and 26 suffered severe deformation or partial collapse. Investigations indicated that liquefaction of foundation soils contributed to much of the damage, with loss of foundation support leading to major displacements of abutments and piers. A study of seismically induced liquefaction and its influence on bridges has been compiled by Ferritto and Forest in a report[1] to the Federal Highway Administration. A brief review of seismic design considerations for bridge foundations related to site liquefaction potential is given in reference 2. From the foundation failures documented in these reports and in the literature in general, it is clear that the design of bridge foundations in soils susceptible to liquefaction poses difficult problems. Where possible, the best design measure is to avoid deep, loose to medium-dense sand sites where liquefaction risks are high. Where dense or more competent soils are found at shallow depths, stabilization measures such as densification may be economical. The use of long ductile vertical steel piles to support bridge piers could also be considered. Calculations for lateral resistance would assume zero support from the upper zone of potential liquefaction, and the question of axial buckling would need to be addressed. Overall abutment stability would also

require careful evaluation, and it may be preferable to use longer spans and to anchor abutments well back from the end of approach fills.

A further design philosophy for bridges in liquefaction susceptible areas might be one of "calculated risk", at least for those bridges regarded as being less essential for communication purposes immediately after an earthquake. It may not be economically justifiable to design some bridges to survive a large earthquake in a liquefaction environment without significant damage. However, it may be possible to optimize a design so that the cost of repair of potential earthquake damage to those bridges does not exceed the cost of remedial measures and additional construction needed to avoid the damage. The approaches for determining the liquefaction potential at a site are outlined below.

A recent review of methodologies[3] identifies two basic approaches for evaluating the cyclic liquefaction potential of a deposit of saturated sand subjected to earthquake shaking:

1. Empirical methods based on field observations of the performance of sand deposits in previous earthquakes, and correlations between sites which have and have not liquefied and Relative Density or Standard Penetration Test (SPT) blowcounts.
2. Analytical methods based on the laboratory determination of the liquefaction strength characteristics of undisturbed samples and the use of dynamic site response analysis to determine the magnitude of earthquake-induced shearing stresses.

Both empirical and analytical methods require the level of ground acceleration at a site to be defined as a prerequisite for assessing liquefaction potential. This is often established from relationships between earthquake magnitude, distance from the epicenter and peak acceleration.

For conventional evaluations using a "total stress" approach the two methods are similar, but differ only in the manner in which the field liquefaction strength is determined. In the "total stress" approach, liquefaction strengths are normally expressed as the ratio of an equivalent uniform or average cyclic shearing stress τ_h acting on horizontal surfaces of the sand to the initial vertical effective stress σ_o'. As a first approximation, the cyclic stress ratio developed in the field because of earthquake ground shaking may be computed from an equation given by Seed and Idriss,[4] namely:

$$(\tau_h)_{av}/\sigma_o' = 0.65 r_d (a_{max}/g)/(\sigma_o/\sigma_o') \qquad \text{(C6-1)}$$

where

a_{max} = maximum or effective peak ground acceleration at the ground surface

σ_o = total overburden pressure on sand layer under consideration

σ_o' = initial effective overburden pressure on sand layer under consideration

r_d = stress reduction factor varying from a value of 1 at the ground surface to 0.9 at a depth of about 30 ft (9.1 m).

Empirical Methods. Values of the cyclic stress ratio defined by Eq. C6-1 have been correlated for sites which have and have not liquefied, with parameters such as relative density based on SPT data (Seed et al.,[5] Castro[6]). The latest form of this type of correlation (after Seed[3]) is expressed in Figs. 20 and 21. N_1 is the measured standard penetration resistance of the sand corrected to an effective overburden pressure of 1 ton/sq ft (95,800 N/m^2) using the relationship

$$N_1 = NC_N \qquad \text{(C6-2)}$$

Thus, for a given site and a given maximum ground surface acceleration, the average stress ratio developed during the earthquake, $(\tau_h)_{av}/\sigma_o'$, at which liquefaction may be expected to occur, is expressed by the empirical correlations shown by Fig. 20. The correlations for different magnitudes reflect the influence of earthquake duration on liquefaction potential. The factor of safety against liquefaction can be determined by comparing the stress ratio required to cause liquefaction with that induced by the design earthquake. It is suggested that a factor of safety of 1.5 is desirable to establish a reasonable measure of safety against liquefaction in the case of important bridge sites.

A further extension of the empirical approach has recently been described by Dezfulian and Prager,[7] where a correlation between cone penetrometer tests (CPT) and standard penetration tests (SPT) has enabled CPT measurements in sands (expressed as point resistance q_c) to be used as a measure of liquefaction potential. CPT have the advantage of being more economical than SPT, and since they can provide a continuous record of penetration resistance with depth, potentially liquefiable thin seams of sand can be identified more readily.

Whereas penetration tests have the clear advantage of being a field oriented liquefaction evaluation procedure, it must always be remembered that the empirical correlation has been established from a very limited data base restricted to sites comprising primarily deposits of fine silty sand. The correlation may break down for sandy silts and gravelly soils (where blowcount data are difficult to interpret), and for coarser sands where partial drainage of excess pore pressures may occur during an earthquake. Furthermore, for situations where additional stresses are imposed by construction operations, care is needed in interpreting the correlation.

Analytical Methods. The analytical approach for evaluating liquefaction potential is based on a comparison between field liquefaction

strengths established from cyclic laboratory tests on undisturbed samples, and earthquake-induced shearing stresses. In this approach it must be recognized that the development of a field liquefaction strength curve from laboratory test results, requires data adjustment to account for factors such as correct cyclic stress simulation, sample disturbance, aging effects, field cyclic stress history, and the magnitude of in situ lateral stresses. These adjustments require a considerable degree of engineering judgment. Also in many cases it is impossible to obtain undisturbed sand samples.

Once a liquefaction strength curve has been established, if a total stress analysis is used, liquefaction potential is evaluated from comparisons with estimated earthquake-induced shear stresses (as shown in Fig. 22).

The earthquake-induced shear stress levels may be established from a simplified procedure,[4] or more sophisticated assessments made using one dimensional "equivalent linear" dynamic response programs such as SHAKE. Average stress levels are established using the equivalent number of cycles concept (approximately 10 for M7 and 30 for M8.5 earthquakes). More recently, nonlinear programs have been introduced for response calculations.

An improved representation of the progressive development of liquefaction is provided by the use of an effective stress approach[8,9,10] where pore water pressure increases are coupled to nonlinear dynamic response solutions, and the influence of potential pore water pressure dissipation during an earthquake is taken into account. This approach provides data on the time history of pore water pressure increases during an earthquake, as shown in Fig. 23.

It is of interest to note that a rough indication of the potential for liquefaction may be obtained by making use of empirical correlations established between earthquake magnitude and the epicentral distance to the most distant field manifestations of liquefaction. Such a relationship has been described by Youd and Perkins[11] (Fig. 24), and has been used as a basis for preparation of liquefaction-induced ground failure susceptibility maps.

C6.3.1(B), C6.4.1(B) and C6.5.1(B) <u>Foundation Design</u>

The commonly accepted practice for the seismic design of foundations is to utilize a pseudo-static approach, where earthquake-induced foundation loads are determined from the reaction forces and moments necessary for structural equilibrium. Whereas traditional bearing capacity design approaches are also applied, with appropriate capacity reduction factors if a measure of safety against "failure" is desired, a number of factors associated with the dynamic nature of earthquake loading should always be borne in mind.

Under cyclic loading at earthquake frequencies, the strength capable of being mobilized by many soils is greater than the static strength. For unsaturated cohesionless soils the increase may be about 10%, while for cohesive soils, a 50% increase could occur. However, for softer saturated clays and saturated sands, the potential for strength and stiffness degradation under repeated cycles of loading must also be recognized. For bridges classified as SPC B, the use of static soil strengths for evaluating ultimate foundation capacity provides a small implicit factor of safety and, in most cases, strength and stiffness degradation under repeated loading will not be a problem because of the smaller magnitudes of seismic events. However for bridges classified as SPC C and D, some attention should be given to the potential for stiffness and strength degradation of site soils when evaluating ultimate foundation capacity for seismic design.

As earthquake loading is transient in nature, "failure" of soil for a short time during a cycle of loading may not be significant. Of perhaps greater concern is the magnitude of the cyclic foundation displacement or rotation associated with soil yield, as this could have a significant influence on structural displacements or bending moments and shear distributions in columns.

As foundation compliance influences the distribution of forces or moments in a structure and affects computation of the natural period, equivalent stiffness factors for foundation systems are often required. In many cases, use is made of various analytical solutions which are available for footings or piles, where it is assumed that soil behaves as an elastic medium. In using these formulae, it should be recognized that equivalent elastic moduli for soils are a function of strain amplitude, and for high seismic loads modulus values could be significantly less than those appropriate for low levels of seismic loading. Variation of shear modulus with shearing strain amplitude in the case of sands is shown in Fig. 25.

On the basis of field and experimental observations, it is becoming more widely recognized that transient foundation uplift or rocking during earthquake loading, resulting in separation of the foundation from the subsoil, is acceptable provided appropriate design precautions are taken (Taylor and Williams[12]). Experimental studies suggest that rotational yielding beneath rocking foundations can provide a useful form of energy dissipation. However, care must be taken to avoid significant induced vertical deformations accompanying possible soil yield during earthquake rocking, as well as excessive pier movement. These could lead to design difficulties with relative displacements.

Lateral Loading of Piles. Most of the well-known solutions for computing the lateral stiffness of vertical piles are based on the assumption of elastic behavior and utilize equivalent cantilever beam concepts,[13] the beam on an elastic Winkler foundation method[14] or elastic continuum solutions.[15] However, the use of methods incorporating nonlinear subgrade reaction behavior that allows for soil failure may be important for high

lateral loading of piles in soft clay and sand. Such a procedure is encompassed in the American Petroleum Institute (API) recommendations for offshore platform design.[16] The method utilizes nonlinear subgrade reaction or p-y curves for sands and clays which have been developed experimentally from field loading tests.

The general features of the API analysis in the case of sands are illustrated in Fig. 26. Under large loads, a passive failure zone develops near the pile head. Test data indicate that the ultimate resistance, p_u, for lateral loading is reached for pile deflections, y_u, of about 3d/80, where d is the pile diameter. Note that most of the lateral resistance is mobilized over a depth of about 5d. The API method also recognizes degradation in lateral resistance with cyclic loading, although in the case of saturated sands the degradation postulated does not reflect pore water pressure increases. The degradation in lateral resistance due to earthquake-induced free-field pore-water pressure increases in saturated sands, has been described by Finn and Martin.[17] A numerical method which allows the use of API p-y curves to compute pile stiffness characteristics forms the basis of the computer program BMCOL 76 described by Bogard and Matlock.[18]

The influence of group action on pile stiffness is a somewhat controversial subject. Solutions based on elastic theory can be misleading where yield near the pile head occurs. Experimental evidence tends to suggest that group action is not significant for pile spacings greater than 4d to 6d.

For batter pile systems, the computation of lateral pile stiffness is complicated by the stiffness of the piles in axial compression and tension. It is also important to recognize that bending deformations in batter pile groups may generate high reaction forces on the pile cap.

It should be noted that while batter piles are economically attractive for resisting horizontal loads, such piles are very rigid in the lateral direction if arranged so that only axial loads are induced. Hence, large relative lateral displacements of the more flexible surrounding soil may occur during the "free-field" earthquake response of the site (particularly if large changes in soil stiffness occur over the pile length), and these relative displacements may in turn induce high pile bending moments. For this reason, more flexible vertical pile systems where lateral load is resisted by bending near the pile heads, are recommended. However, such pile systems must be designed to be ductile, because large lateral displacements may be necessary to resist the lateral load. A compromise design using battered piles spaced some distance apart may provide a system which has the benefits of limited flexibility and the economy of axial load resistance to lateral load.

Soil-Pile Interaction. The use of pile stiffness characteristics to determine earthquake-induced pile bending moments based on a pseudo-static approach, assumes that moments are induced only by lateral loads arising from inertial effects on the bridge structure. However, it must be remembered

that the inertial loads are generated by interaction of the free-field earthquake ground motion with the piles, and that the free-field displacements themselves can influence bending moments. This is illustrated in an idealized manner in Fig. 27. The free-field earthquake displacement time histories provide input into the lateral resistance interface elements which in turn transfer motion to the pile. Near the pile heads, bending moments will be dominated by the lateral interaction loads generated by inertial effects on the bridge structure. At greater depth (e.g., greater than 10d) where soil stiffness progressively increases with respect to pile stiffness, the pile will be constrained to deform in a similar manner to that of the free field, and pile bending moments become a function of the curvatures induced by free-field displacements.

To illustrate the nature of free-field displacements reference is made to Fig. 28, which shows a 200 ft (61 m) deep cohesionless soil profile subjected to the El Centro Earthquake. The free-field response was determined using a nonlinear one-dimensional response analysis. From the displacement profiles shown at specific times, curvatures can be computed and pile bending moments calculated if it is assumed that the pile is constrained to displace in phase with the free-field response.

Large curvatures could develop at interfaces between soft and rigid soils and, clearly, in such cases emphasis should be placed on using flexible ductile piles. Margason[19] suggests that curvatures of up to 6×10^{-4} in.$^{-1}$ (15.24×10^{-3} mm^{-1}) could be induced by strong earthquakes but these should pose no problems to well-designed steel or prestressed concrete piles.

Studies incorporating the complete soil-pile-structure interaction system as presented by Fig. 27, have been described by Penzien[20] for a bridge piling system in a deep soft clay. A similar but somewhat simpler soil-pile-structure interaction system (SPASM) to that used by Penzien, has been described by Matlock et al.[21] The model used is, in effect, a dynamic version of the previously mentioned BMCOL program.

C6.3.1(C) and C6.4.1(C) **Special Pile Requirements**

The uncertainties of ground and bridge response characteristics lead to the desirability of providing tolerant pile foundation systems. Toughness under induced curvature and shears is required, and hence piles such as steel H-sections and concrete filled steel-cased piles are favored for highly seismic areas. Unreinforced concrete piles are brittle in nature, so nominal longitudinal reinforcing is specified to reduce this hazard. The reinforcing steel should be extended into the footing to tie elements together and to assist in load transfer from the pile to the pile cap.

Experience has shown that reinforced concrete piles tend to hinge or shatter immediately below the pile cap. Hence tie spacing is reduced in this area so that the concrete is better confined. Driven precast piles

should be constructed with considerable spiral confining steel to ensure good shear strength and tolerance of yield curvatures should these be imparted by the soil or structural response. Clearly, it is desirable to ensure that piles do not fail below ground level, and that flexural yielding in the columns is forced to occur above ground level. The additional pile design requirements imposed on piles for bridges classified as SPC C and D for which earthquake loading is more severe, reflect a design philosophy aimed at minimizing below ground damage which is not easily inspected following a major earthquake.

C6.3.2, C6.4.2 and C6.5.2 ABUTMENTS

The numerous case histories of damage to, or failure of, bridges induced by abutment failure or displacement during earthquakes have clearly demonstrated the need for careful attention to abutment design and detailing in seismic areas. Damage is typically associated with fill settlement or slumping, displacements induced by high seismically-induced lateral earth pressures, or the transfer of high longitudinal or transverse inertia forces from the bridge structure itself. Settlement of abutment backfill, severe abutment damage or bridge deck damage induced by the movement of abutments may cause loss of bridge access, and hence abutments must be considered as a vital link in the overall seismic design process for bridges.

The nature of abutment movement or damage during past earthquakes has been well documented in the literature. Evans[22] examined the abutments of 39 bridges within 30 miles (48.3 km) of the 1968 M7 Inangahua earthquake in New Zealand, of which 23 showed measurable movement and 15 were damaged. Movements of free standing abutments followed the general pattern of outward motion and rotation about the top after contact with and restraint by the superstructures. Fill settlements were observed to be 10 to 15% of the fill height. Damage effects on bridge abutments in the M7.1 Madang earthquake in New Guinea reported by Ellison[23] were similar; abutment movements as much as 20 in. (500 mm) were noted. Damage to abutments in the 1971 San Fernando earthquake is described by Fung et al.[24] Numerous instances of abutment displacement and associated damage have been reported in publications on the Niigata and Alaskan earthquakes. However, these failures were primarily associated with liquefaction of foundation soils.

Design features of abutments vary tremendously, and depend on the nature of the bridge site, foundation soils, bridge span length and load magnitudes. Abutment types include free-standing gravity walls, cantilever walls, tied back walls, and monolithic diaphragms. Foundation support may use spread footings, vertical piles or battered piles, while connection details to the superstructure may incorporate roller supports, elastomeric bearings or fixed bolted connections. Considering the number of potential design variables together with the complex nature of soil-abutment-superstructure interaction during earthquakes, it is clear that the seismic design of abutments necessitates many simplifying assumptions.

C6.3.2(A), C6.4.2(A) Free-Standing Abutments
C6.5.2(A)

For free-standing abutments such as gravity or cantilever walls, which are able to yield laterally during an earthquake (i.e., superstructure supported by bearings which are able to slide freely) the well-established Mononobe-Okabe pseudo-static approach outlined below, is widely used to compute earth pressures induced by earthquakes.

For free-standing abutments in highly seismic areas, design of abutments to provide zero displacement under peak ground accelerations may be unrealistic, and design for an acceptable small lateral displacement may be preferable. A recently developed method for computing the magnitude of relative wall displacement during a given earthquake is outlined in this subsection. Based on this simplified approach, recommendations are made for the selection of a pseudo-static seismic coefficient and the corresponding displacement level for a given effective peak ground acceleration.

Mononobe-Okabe Analysis

The method most frequently used for the calculation of the seismic soil forces acting on a bridge abutment is a static approach developed in the 1920s by Mononobe[25] and Okabe.[26] The Mononobe-Okabe analysis is an extension of the Coulomb sliding-wedge theory taking into account horizontal and vertical inertia forces acting on the soil. The analysis is described in detail by Seed and Whitman[27] and Richards and Elms.[28] The following assumptions are made:

1. The abutment is free to yield sufficiently to enable full soil strength or active pressure conditions to be mobilized. If the abutment is rigidly fixed and unable to move, the soil forces will be much higher than those predicted by the Mononobe-Okabe analysis.
2. The backfill is cohesionless, with a friction angle of ϕ.
3. The backfill is unsaturated, so that liquefaction problems will not arise.

Equilibrium considerations of the soil wedge behind the abutment (Fig. 29) then lead to a value, E_{AE}, of the active force exerted on the soil mass by the abutment (and vice versa), when the abutment is at the point of failure: E_{AE} is given by the expression

$$E_{AE} = 1/2\ \gamma H^2(1-k_v)K_{AE} \qquad \text{(C6-3)}$$

where the seismic active pressure coefficient K_{AE} is

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta \cos^2\beta \cos(\delta+\beta+\theta)\left[1 + \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta)\cos(i-\beta)}}\right]^2} \qquad \text{(C6-4)}$$

and where

γ = unit weight of soil

H = height of soil face

ϕ = angle of friction of soil

θ = $\arctan(\frac{k_h}{1-k_v})$

δ = angle of friction between soil and abutment

k_h = horizontal acceleration coefficient

k_v = vertical acceleration coefficient

i = backfill slope angle

β = slope of soil face.

The equivalent expression for passive force if the abutment is being pushed into the backfill is

$$E_{PE} = 1/2\ \gamma H^2(1-k_v)K_{PE} \qquad \text{(C6-5)}$$

where

$$K_{PE} = \frac{\cos^2(\phi-\theta+\beta)}{\cos\theta \cos^2\beta \cos(\delta-\beta+\theta)\left[1 - \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta+i)}{\cos(\delta-\beta+\theta)\sin(i-\beta)}}\right]^2} \qquad \text{(C6-6)}$$

As the seismic inertia angle θ increases, the values of K_{AE} and K_{PE} approach each other and, for a vertical backfill, become equal when $\theta = \phi$.

Despite the relative simplicity of the approach, the accuracy of Eq. C6-3 has been substantiated by model tests[27] and by back calculation from observed failures of flood channel walls.[29] In the latter case, however, the displacements were large; and this, as will be seen, can modify the effective values of k_h at which failure occurs.

The value of h, the height at which the resultant of the soil pressure acts on the abutment, may be taken as H/3 for the static case with no earthquake effects involved. However, it becomes greater as earthquake effects increase. This has been shown by tests and theoretically by Wood,[30] who found that the resultant of the dynamic pressure acted approximately at midheight. Seed and Whitman have suggested that h could be obtained by assuming that the static component of the soil force (computed from Eq. C6-3 with $\theta = k_v = 0$) acts at H/3 from the bottom of the abutment,

while the additional dynamic effect should be taken to act at a height of 0.6H. For most purposes it is sufficient to assume h = H/2, with a uniformly distributed pressure.

Although the Mononobe-Okabe expression for active thrust is easily evaluated for any particular geometry and friction angle, the significance of the various parameters is not obvious. Figure 30 shows the variation of K_{AE} against k_h for different values of ϕ and k_v; K_{AE} is obviously very sensitive to the value of ϕ. Also, for a constant value of ϕ, K_{AE} doubles as k_h increases from zero to 0.35 for zero vertical acceleration, and thereafter it increases more rapidly.

In order to evaluate the increase in soil active pressure due to earthquake effects more easily, K_{AE} can be normalized by dividing by its static value K_A to give a thrust factor

$$F_T = K_{AE}/K_A \ . \qquad \text{(C6-7)}$$

Whereas Fig. 30 shows that K_{AE} is sensitive to changes in the soil friction angle ϕ, the plots of F_T against ϕ in Fig. 31 indicate that the value of has little effect on the thrust factor until quite suddenly, over a short range of ϕ, F_T increases rapidly and becomes infinite for specific critical values of ϕ. The reason for this behavior may be determined by examining Eq. C6-4. The contents of the radical must be positive for a real solution to be possible, and for this it is necessary that

$$\phi \geq i + \theta = i + \text{arc tan}\left(\frac{k_h}{1-k_v}\right) \ . \qquad \text{(C6-8)}$$

This condition could also be thought of as specifying a limit to the horizontal acceleration coefficient that could be sustained by any structure in a given soil. The limiting condition is that

$$k_h \leq (1-k_v)\tan(\phi-i) \ . \qquad \text{(C6-9)}$$

For zero vertical acceleration and backfill angle and for a soil friction angle of 35°, the limiting value of k_h is 0.7. This is a figure of some interest in that it provides an absolute upper bound for the seismic acceleration that can be transmitted to any structure whatsoever built on soil with the given strength characteristics.

Figure 32 shows the effect on F_T of changes in the vertical acceleration coefficient k_v. Positive values of k_v have a significant effect for values of k_h greater than 0.2. The effect is greater than 10% above and to the right of the dashed line.

As is to be expected from Eq. C6-6, K_{AE} and F_T are also sensitive to variations in backfill slope, particularly for higher

values of horizontal acceleration coefficient when the limit implied by Eq. C6-6 is approached. This effect is shown in Fig. 33.

The effects of abutment inertia are not taken into account in the Mononobe-Okabe analysis. Many current procedures assume that the inertia forces due to the mass of the abutment itself may be neglected in considering seismic behavior and seismic design. This is not a conservative assumption, and for those abutments relying on their mass for stability it is also an unreasonable assumption, in that to neglect the mass is to neglect a major aspect of their behavior. The effects of wall inertia are discussed further by Richards and Elms,[28] who show that wall inertia forces should not be neglected in the design of gravity retaining walls.

Design for Displacement

If peak ground accelerations are used in the Mononobe-Okabe analysis method, the size of gravity retaining structures will often be excessively great. To provide a more economic structure, design for a small tolerable displacement rather than no displacement may be preferable.

Tests have shown that a gravity retaining wall fails in an incremental manner in an earthquake. For any earthquake ground motion, the total relative displacement may be calculated using the sliding block method suggested by Newmark.[31] The method assumes a displacement pattern similar to that of a block resting on a plane rough horizontal surface subjected to an earthquake, with the block being free to move against frictional resistance in one direction only. Figure 34 shows how the relative displacement relates to the acceleration and velocity time histories of soil and wall. At a critical value of k_h, the wall is assumed to begin sliding; relative motion will continue until wall and soil velocities are equal. Figures 35 and 36 show the results[28] of a computation of wall displacement for $k_h = 0.1$ for the El Centro 1940 N-S record.

Newmark computed the maximum displacement response for four earthquake records, and plotted the results after scaling the earthquakes to a common maximum acceleration and velocity. Franklin and Chang[32] repeated the analysis for a large number of both natural and synthetic records and added their results to the same plot. Upper bound envelopes for their results are shown in Fig. 37. All records were scaled to a maximum acceleration coefficient of 0.5 and a maximum velocity V of 30 in./sec (762 mm/sec). The maximum resistance coefficient N is the maximum acceleration coefficient sustainable by a sliding block before it slides. In the case of a wall designed using the Mononobe-Okabe method, the maximum coefficient is, of course, k_h.

Figure 37 shows that the displacement envelopes for all the scaled records have roughly the same shape.

An approximation to the curves for relatively low displacements is given by the relation, expressed in any consistent set of units,

$$d = 0.087 \frac{V^2}{Ag} \left(\frac{N}{A}\right)^{-4} \qquad \text{(C6-10)}$$

where d is the total relative displacement of a wall subjected to an earthquake ground motion whose maximum acceleration coefficient and maximum velocity are A and V, respectively. This is drawn as a straight line on Fig. 37. Note that as this expression has been derived from envelope curves, it will overestimate d for most earthquakes.

One possible design procedure would be to choose a desired value of maximum wall displacement d together with appropriate earthquake parameters, and to use Eq. C6-10 to derive a value of the seismic acceleration coefficient for which the wall should be designed. The wall connections, if any, could then be detailed to allow for this displacement.

By applying the above procedure to several simplified examples, Elms and Martin[33] have shown that a value of $k_h = A/2$ is adequate for most design purposes, provided that allowance is made for an outward displacement of the abutment of up to 10A in. (254A mm).

For bridges classified as SPC C and D, more detailed consideration of the mechanism of transfer of structural inertia forces through bridge bearings to free-standing abutments is required, particularly for bridges classified as SPC D where continued bridge accessibility after a major earthquake is required.

For sliding steel bearings or pot bearings, force diagrams describing limiting equilibrium conditions for a simple abutment are shown in Fig. 38. Where bearings comprise unconfined elastomeric pads, the nature of the forces transferred to the abutment becomes more complex, since such bearings are capable of transferring significant force. The magnitude of the force initially depends on the relative movement between the superstructure and the abutment, and force magnitudes can become quite large before slip will occur.

For bridges classified as SPC D, additional consideration should be given to the use of linkage bolts and buffers to minimize damage. A typical abutment support detail used by the New Zealand Ministry of Works is shown in Fig. 39. It may be seen that linkage bolts are incorporated to prevent spans dropping off supports. The rubber rings act as buffers to prevent impact damage in the event that the lateral displacement clearance provided is inadequate. The knock-off backwall accommodates differential displacement between the abutment and superstructure, with minimum structural damage. A more typical design provision in United States practice is to seal the gap between superstructure and abutment with bitumen to minimize impact damage. It must be recognized, however, that in this case some damage and possible abutment rotation will occur in strong earthquakes.

The use of a settlement or approach slab in Figs. 39 and 40 which has the effect of providing bridge access in the event of backfill settlement is also noted. The slab also provides an additional abutment friction anchorage against lateral movement.

Non-Yielding Abutments

As previously noted, the Mononobe-Okabe analysis assumes that the abutment is free to yield laterally a sufficient amount to mobilize peak soil strengths in the soil backfill. For granular soils, peak strengths can be assumed to be mobilized if deflections at the top of the wall are about 0.5% of the abutment height. For abutments which are restrained against lateral movement by tie backs or batter piles, lateral pressures induced by inertia forces in the backfill will be greater than those given by a Mononobe-Okabe analysis. Simplified elastic solutions presented by Wood[30] for rigid non-yielding walls, also indicate that pressures are greater than those given by Mononobe-Okabe. The use of a factor of 1.5 in conjunction with peak ground accelerations is suggested for design where doubt exists that an abutment can yield sufficiently to mobilize soil strengths.

C6.3.2(B), C6.4.2(B) Monolithic Abutments
C6.5.2(B)

Monolithic or end diaphragm abutments such as shown in Fig. 40 are commonly used for single and for two span bridges in California. As shown, the end diaphragm is cast monolithically with the superstructure and may be directly supported on piles, or provision may be made for beam shortening during post-tensioning. The diaphragm acts as a retaining wall with the superstructure acting as a prop between abutments.

Such abutments have performed well during earthquakes and avoid problems such as backwall and bearing damage associated with yielding abutments, and reduce the lateral load taken by columns or piers. On the other hand, higher longitudinal and transverse superstructure inertia forces are transmitted directly into the backfill and provision must be made for adequate passive resistance to avoid excessive relative displacements.

Whereas free-standing or seat type abutments allow the engineer more control over development of soil forces, the added joint introduces a potential collapse mechanism into the structure. To avoid this collapse mechanism, monolithic abutments are particularly recommended for bridges classified as SPC D. Whereas damage may be heavier than that for free- standing abutments because of the higher forces transferred to backfill soils, with adequate abutment reinforcement the collapse potential is low. In making estimates of monolithic abutment stiffness and associated longitudinal displacements during transfer of peak earthquake forces from the structure, it is recommended that abutments be proportioned to restrict displacements to 0.3 ft (91.4 mm) or less in order to minimize damage.

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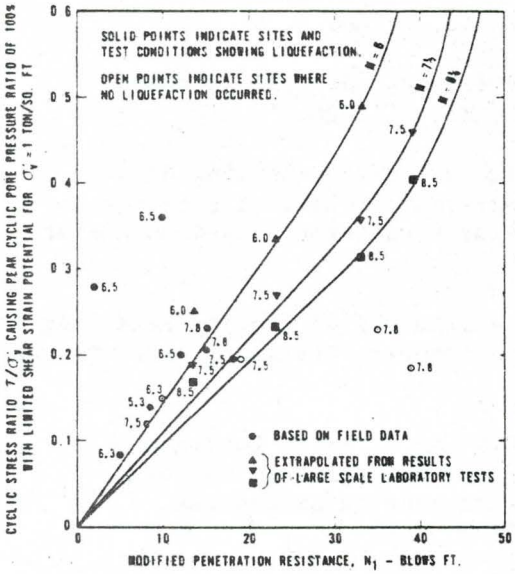

FIGURE 20. CORRELATION BETWEEN FIELD LIQUEFACTION BEHAVIOR AND PENETRATION RESISTANCE

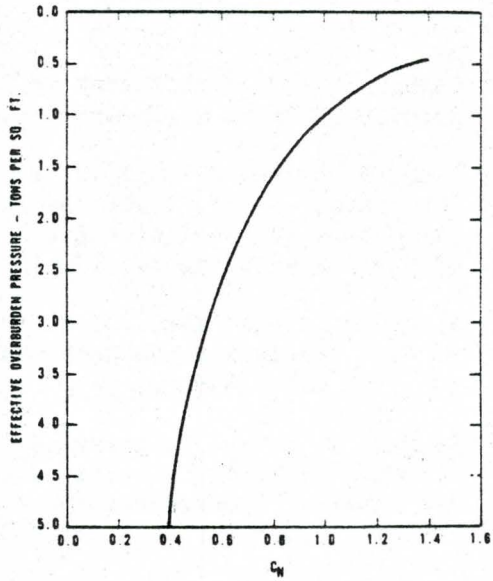

FIGURE 21. RELATIONSHIP BETWEEN C_N AND EFFECTIVE OVERBURDEN PRESSURE

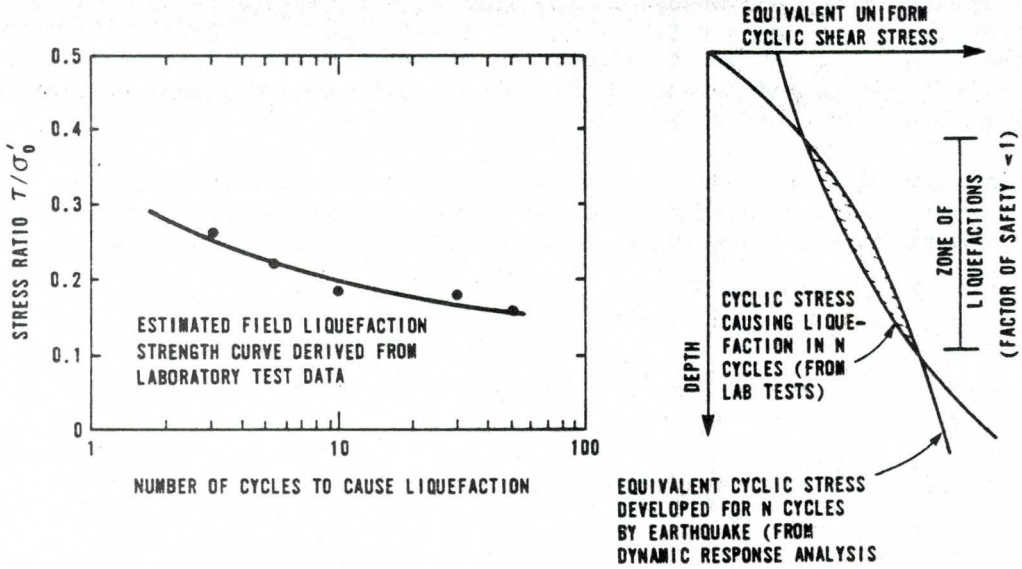

FIGURE 22. PRINCIPLES OF ANALYTICAL APPROACH (TOTAL STRESS) TO LIQUEFACTION POTENTIAL EVALUATION

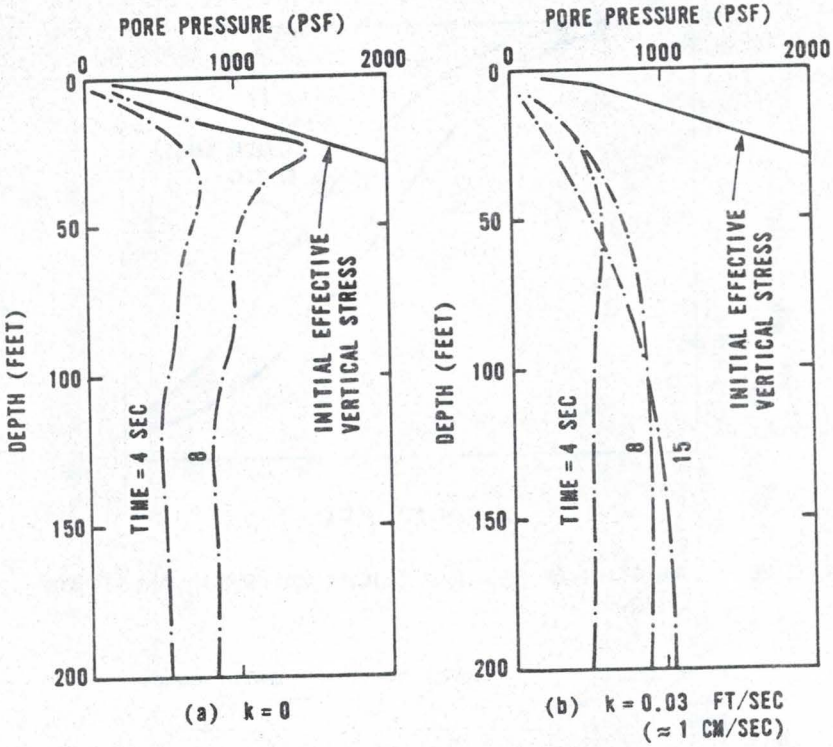

FIGURE 23. EFFECTIVE STRESS APPROACH TO LIQUEFACTION EVALUATION SHOWING EFFECT OF PERMEABILITY (After FINN et al., 1977)

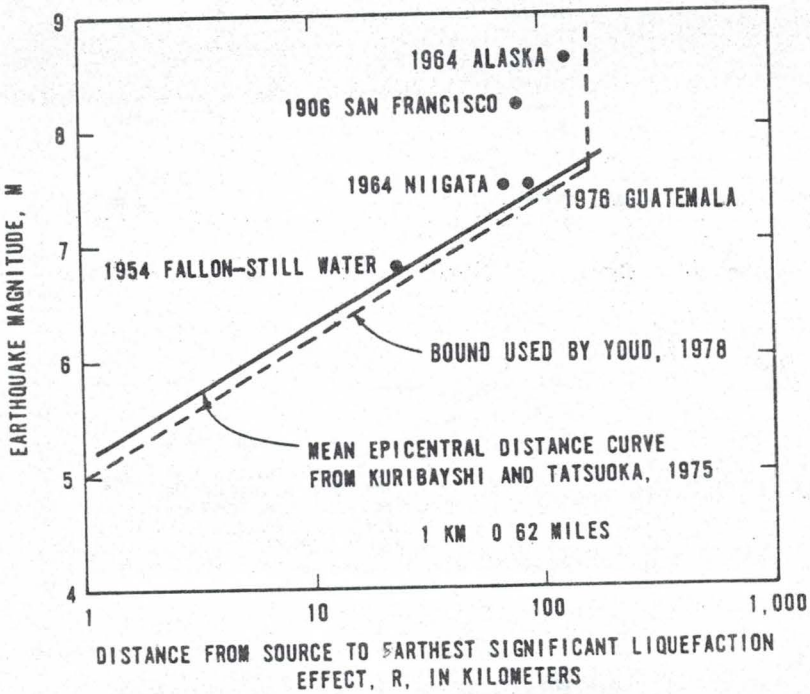

FIGURE 24. MAXIMUM DISTANCE TO SIGNIFICANT LIQUEFACTION AS A FUNCTION OF EARTHQUAKE MAGNITUDE

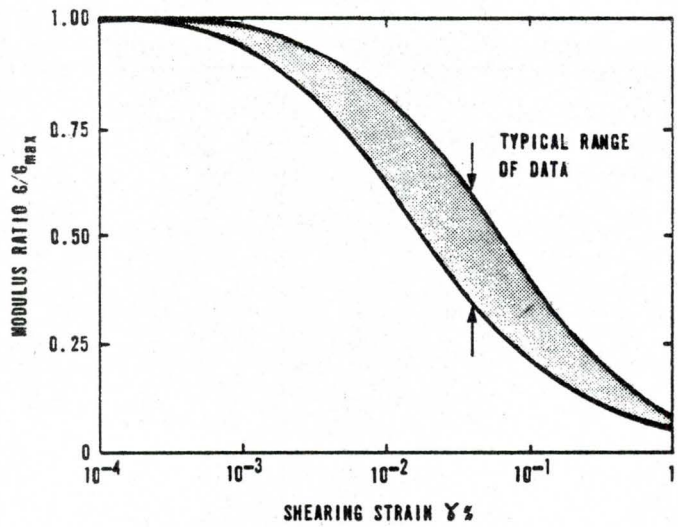

FIGURE 25. VARIATION OF SHEAR MODULUS WITH SHEARING STRAIN FOR SANDS

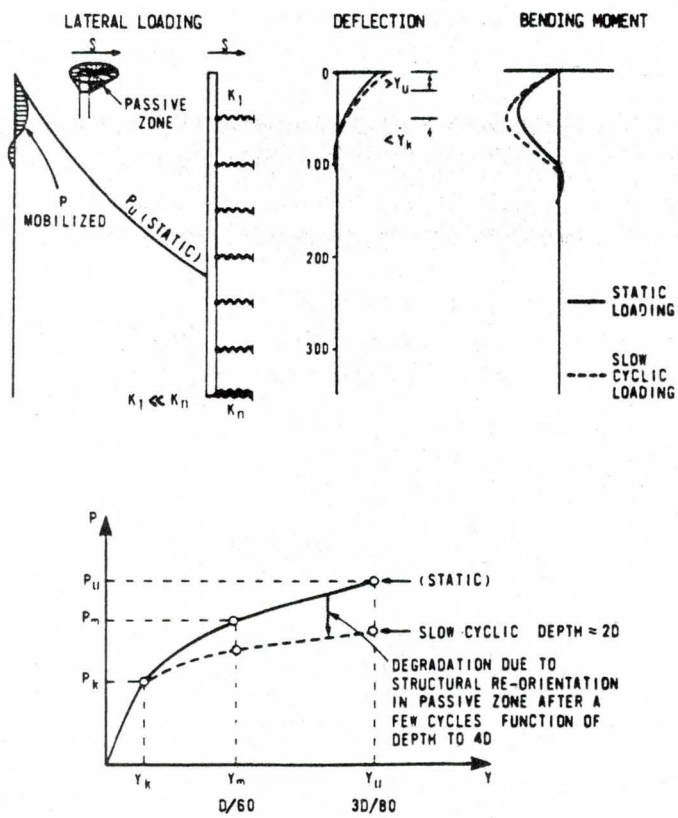

FIGURE 26. LATERAL LOADING OF PILES IN SAND USING API CRITERIA

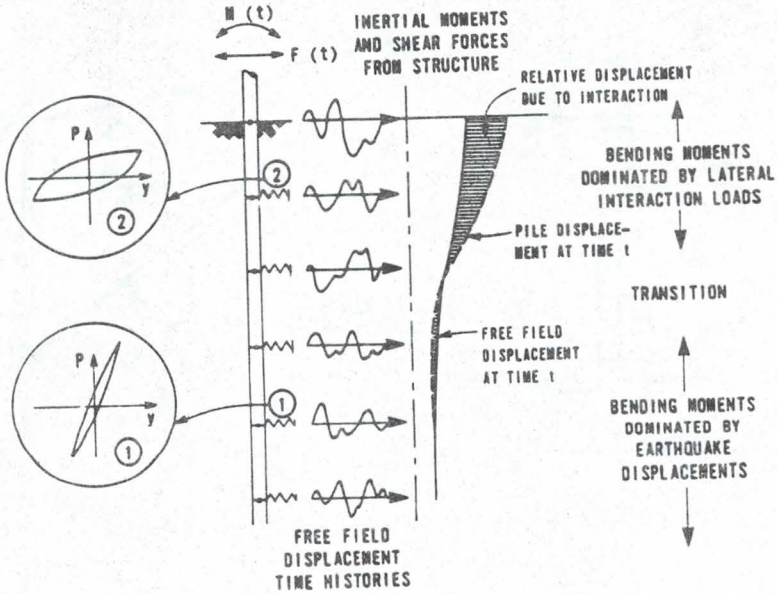

FIGURE 27. MECHANISM OF SOIL-PILE INTERACTION DURING SEISMIC LOADING

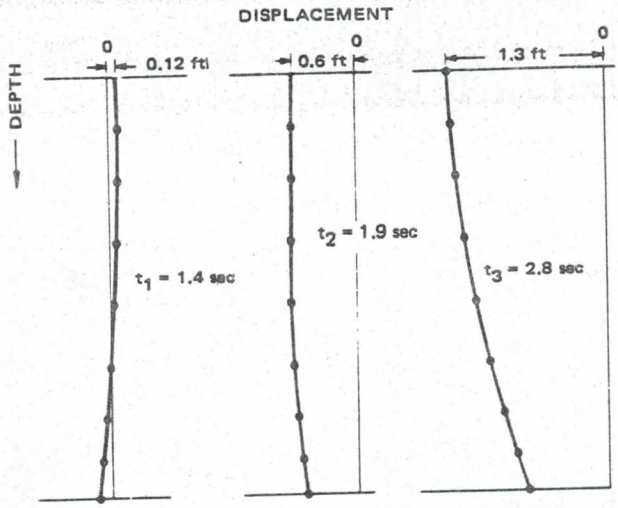

FIGURE 28. TYPICAL EARTHQUAKE DISPLACEMENT PROFILES

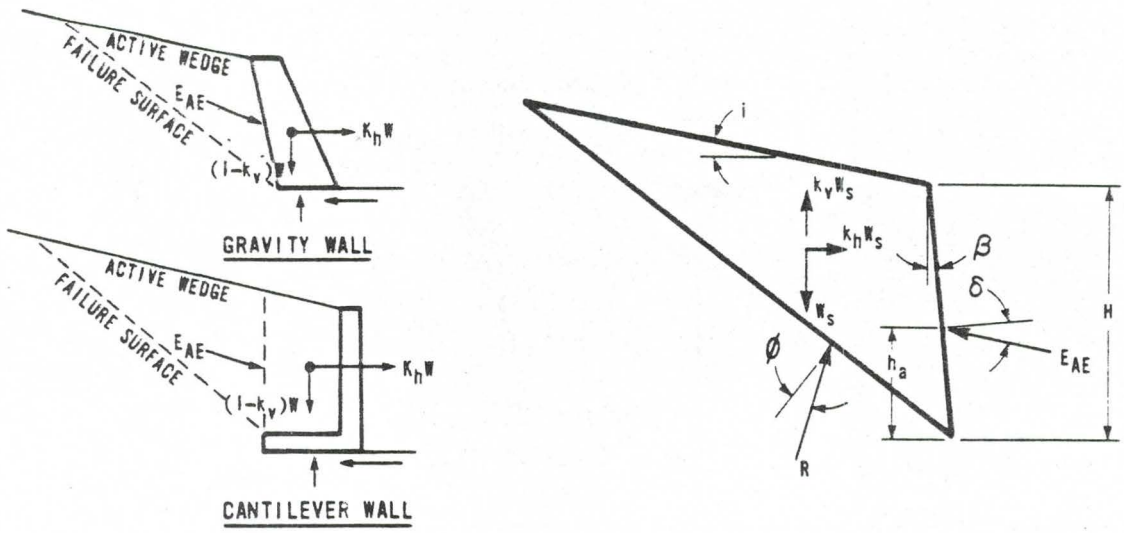

FIGURE 29. ACTIVE WEDGE FORCE DIAGRAM

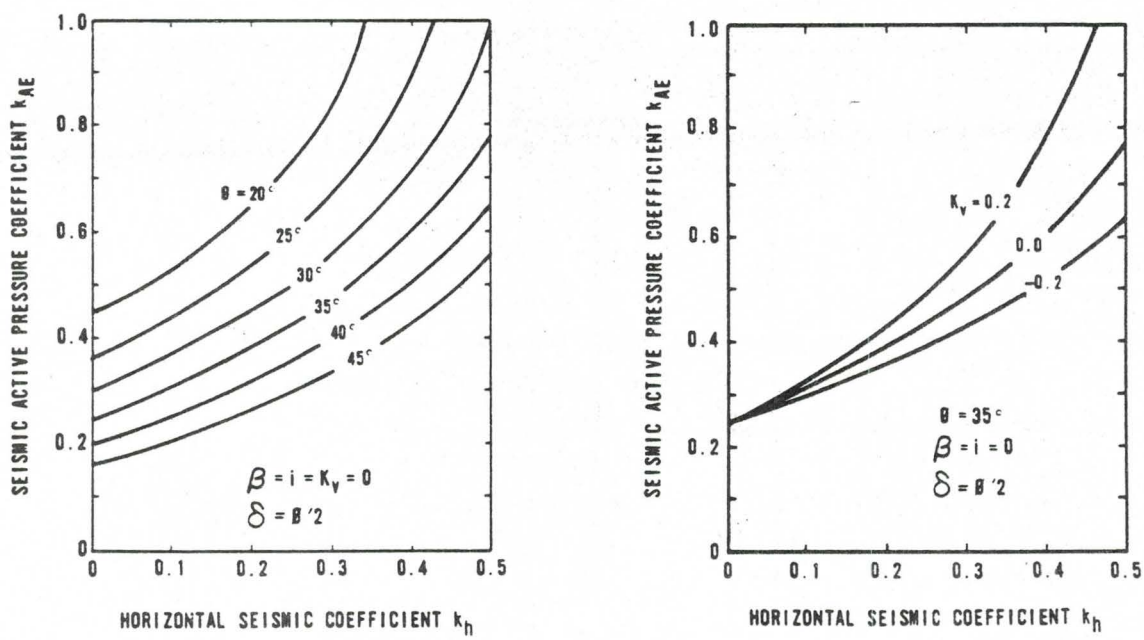

FIGURE 30. EFFECT OF SEISMIC COEFFICIENTS AND SOIL FRICTION ANGLE ON SEISMIC ACTIVE PRESSURE COEFFICIENT

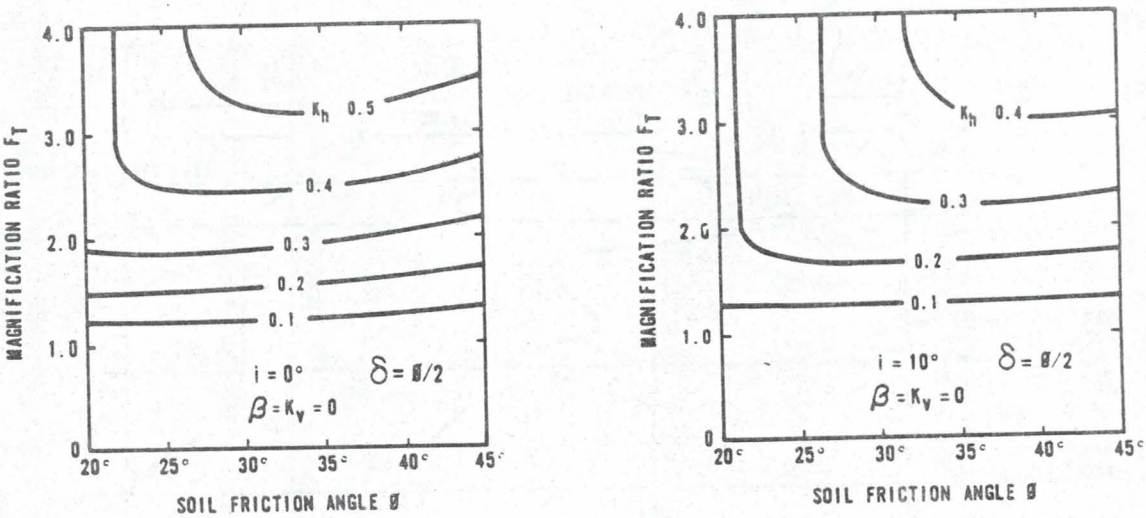

FIGURE 31. INFLUENCE OF SOIL FRICTION ANGLE ON MAGNIFICATION RATIO

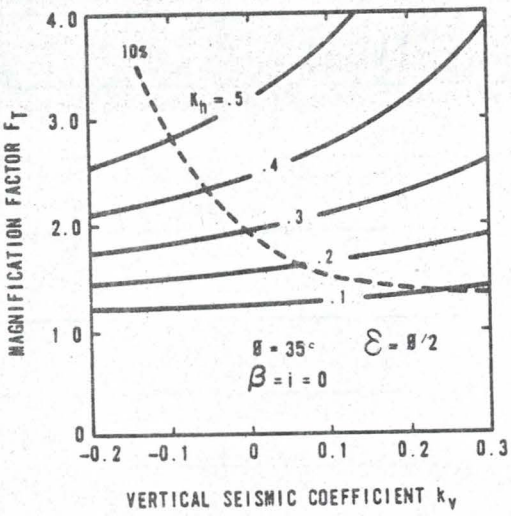

FIGURE 32. INFLUENCE OF VERTICAL SEISMIC COEFFICIENT ON MAGNIFICATION RATIO

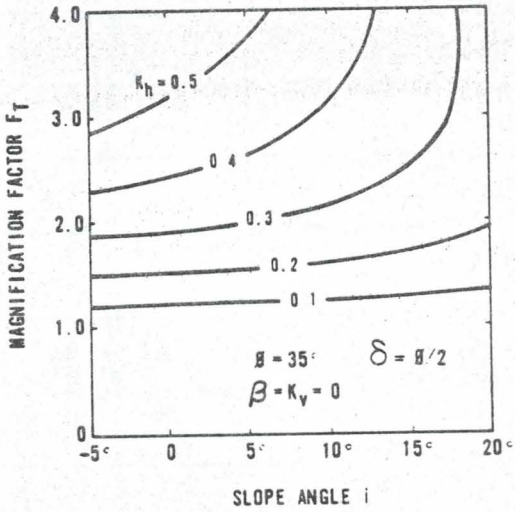

FIGURE 33. INFLUENCE OF BACKFILL SLOPE ANGLE ON MAGNIFICATION RATIO

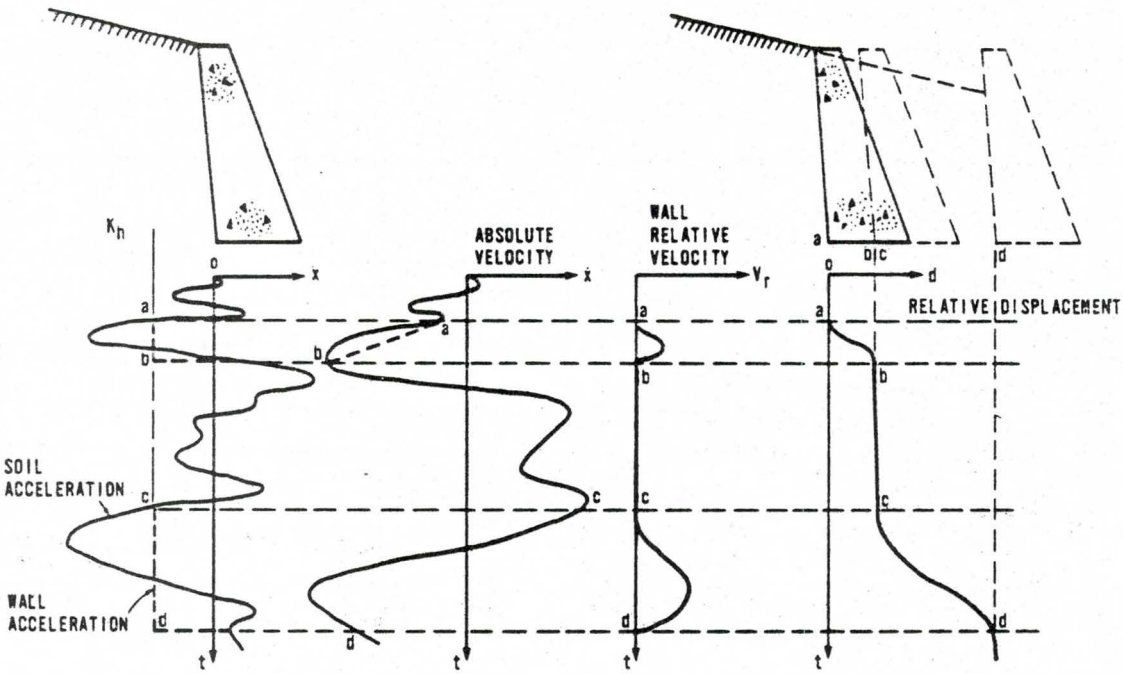

FIGURE 34. RELATION BETWEEN RELATIVE DISPLACEMENT AND ACCELERATION AND VELOCITY TIME HISTORIES OF SOIL AND WALL

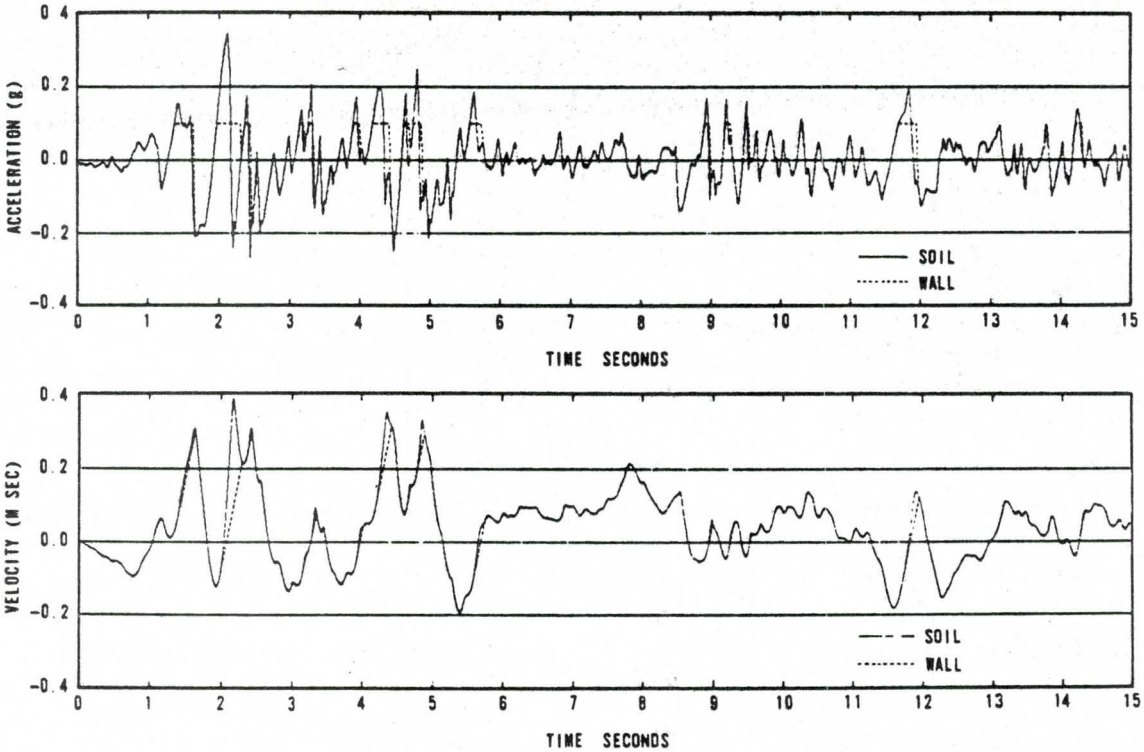

FIGURE 35. ACCELERATION AND VELOCITY TIME HISTORIES OF SOIL AND WALL (EL CENTRO 1940 N-S RECORD)

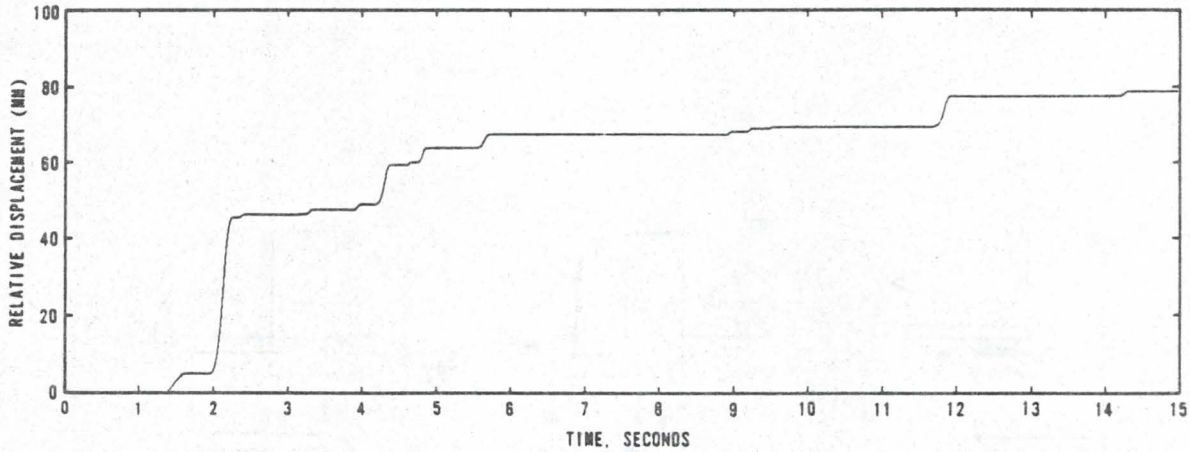

FIGURE 36. RELATIVE DISPLACEMENT OF WALL (EL CENTRO 1940 N-S RECORD)

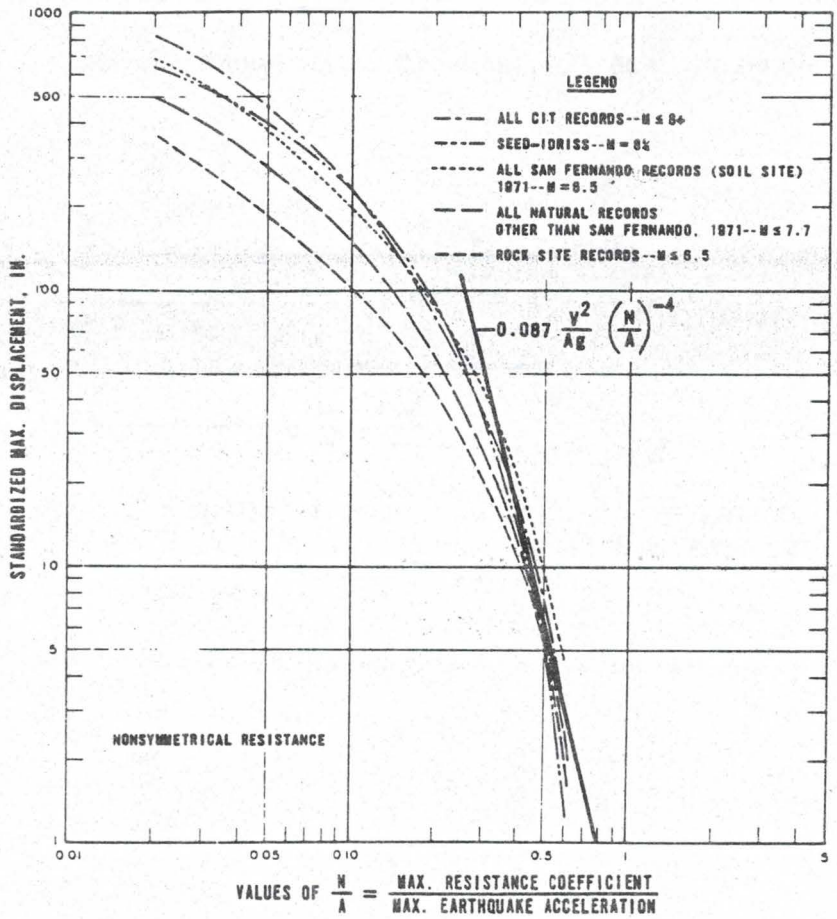

FIGURE 37. UPPER BOUND ENVELOPE CURVES OF PERMANENT DISPLACEMENTS FOR ALL NATURAL AND SYNTHETIC RECORDS ANALYZED BY FRANKLIN AND CHANG (1 IN. = 25.4 mm)

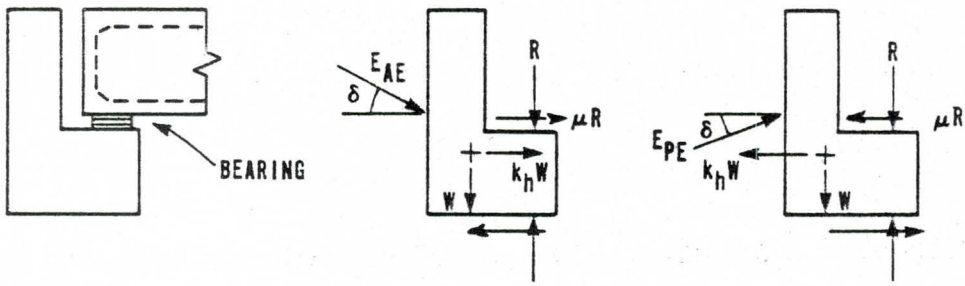

FIGURE 38. FORCE DIAGRAMS INCLUDING BEARING FRICTION

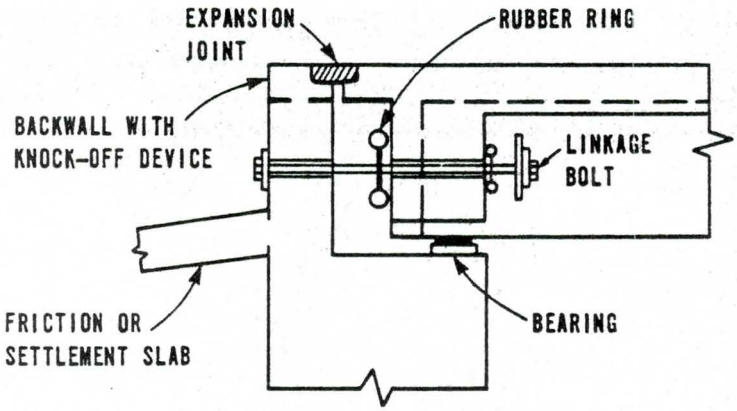

FIGURE 39. ABUTMENT SUPPORT DETAIL

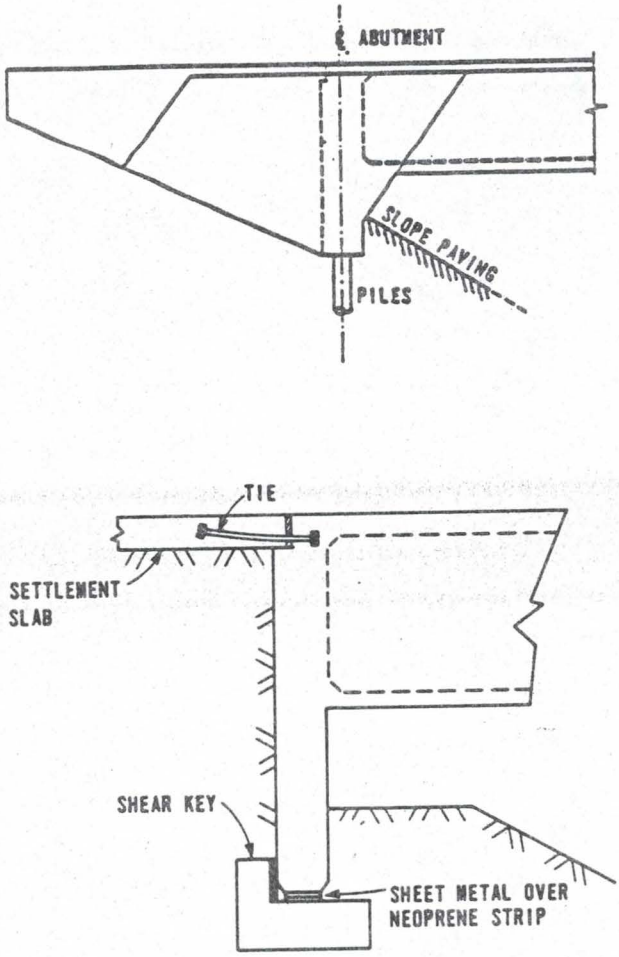

FIGURE 40. TYPICAL MONOLITHIC ABUTMENTS

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COMMENTARY

CHAPTER 7 - STRUCTURAL STEEL

C7.1 GENERAL

The 50% increase in allowable stresses for service load design is based on the following:

1. The margin of safety between the yield strength and allowable stress of short columns.
2. The margin of safety between the yield strength and allowable tensile stress.
3. The margin of safety of compression members, which varies between 1.7 and 1.9.[1,2]

C7.3 SEISMIC PERFORMANCE CATEGORIES B, C AND D

This subsection provides modifications to the interaction equations when the P-delta effects are explicitly determined. In columns, the reductions to the allowable stresses are in part a result of the consideration of member P-delta effects. These P-delta reductions are modified in AASHTO by a K-factor which is a recognition of the effect of end restraint in the member P-delta relationship. The bases for the values of this ratio where joint translation is prevented are well documented. The selection of the value of C_m where joint translation is permitted was an approximation applicable primarily to designs for which significant applied horizontal forces are not present. Since the advent of computer analysis, the solution of the interaction equations when secondary effects resulting from deflection are taken into account, has become much easier. In most cases with significant horizontal displacements the first iteration of deflection is sufficient. It is possible that for some members, such as weak axis columns depending on end support conditions, critical stress may occur at the midheight rather than the column ends. Thus the stress limits specified when joint translation is prevented should not be exceeded.

REFERENCES

1. "Manual of Steel Construction," American Institute of Steel Construction, Inc., 1979.
2. Johnson, B. G., Structural Stability Research Council's "Guide to Stability Design Criteria for Metal Structures," Third Edition, John Wiley and Sons, New York, 1976.

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COMMENTARY

CHAPTER 8 - REINFORCED CONCRETE

C8.1 GENERAL

The purpose of the additional design requirements of this chapter is to ensure, especially for bridges classified as SPC C and D, that the design of the components of a bridge are consistent with the overall design philosophy and that the potential for failures observed in past earthquakes is minimized. The additional column design requirements of this chapter for bridges classified as SPC C and D are such that a column is forced to yield in flexure with a reasonable ductility capacity and that the potential for a shear, compression or loss of anchorage mode of failure is minimized. The additional design requirements for piers provide for some inelastic capacity; however, the R-factor specified for piers is such that the anticipated inelastic capacity is significantly less than that of columns.

The actual ductility demand on a column or pier is a complex function of a number of variables[1,2] including the earthquake characteristics, design force level, period of the bridge, shape of the inelastic hysteresis loop of the columns, elastic damping coefficient, contributions of foundation and bearing compliance to structural flexibility, and plastic hinge length of the column. The damage potential of a column is also related to the ratio of the duration of strong motion shaking to the natural period of the bridge. This ratio will be an indicator of the number of yield excursions, and hence of the cumulative ductility. There are some grounds for considering the cumulative ductility to be a more useful index than the peak ductility level; for example, 10 cycles at a curvature ductility factor of 8 might be more damaging than one yield excursion at a curvature ductility factor of 10 or 12. However, there is little experimental evidence to support or contradict this view.

Both Service Load and Load Factor methods of design are permitted although it is recommended that the Load Factor method of design be used since it is consistent with the ultimate load capacity concept used in determining the design force levels. An increase in allowable stresses of 33-1/3% is permitted for Service Load design. This is consistent with the current AASHTO Interim Specifications.

C8.2 SEISMIC PERFORMANCE CATEGORY A

Consistent with the overall philosophy for bridges classified as SPC A, special seismic design requirements were eliminated because of the low level of seismic risk and the low probability that a column would be subjected to seismic forces that would cause yielding.

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C8.3 SEISMIC PERFORMANCE CATEGORY B

Bridges classified as SPC B have a reasonable probability of being subjected to seismic forces that will cause yielding of the columns. Thus it was deemed necessary that columns have some ductility capacity although it was recognized that the ductility demand will not be as great as for columns of bridges classified as SPC C and D. The most important requirement to ensure some level of ductility is the transverse reinforcement requirement specified.[1,2] This will prevent buckling of the longitudinal steel and provide confinement for the core of the column. The maximum spacing for the transverse reinforcement was increased to 6 in. (152 mm) because of the anticipated lower ductility demand.

C8.4 SEISMIC PERFORMANCE CATEGORIES C AND D

The requirements of this chapter were primarily developed for bridges classified as SPC C and D and a discussion of the provisions follows.

C8.4.1 COLUMN REQUIREMENTS

The definition of a column in this section is provided as a guideline to differentiate between the additional design requirements for a pier and for a column. This should be used as a guideline and, if a column or pier is above or below the recommended criterion, it can be considered either as a column or pier, provided the appropriate R-Factor of Sec. 3.6 and the appropriate requirements of either Sec. 8.4.1 or 8.4.2 are used. For columns with an H/d ratio less than 2-1/2, the forces resulting from plastic hinging will generally exceed the elastic design forces and consequently the forces of Sec. 4.8.2 would not be applicable.

C8.4.1(A) Vertical Reinforcement

This requirement is intended to apply for the full section of the columns. The lower limit on the column reinforcement reflects the traditional concern for the effect of time-dependent deformations as well as the desire to avoid a sizable difference between the flexural cracking and yield moments. The 6% maximum ratio is to avoid congestion and to permit anchorage of the longitudinal steel. If the effectiveness of higher percentages of reinforcement is substantiated by test results, relaxation of this requirement could be considered; however, the PEP gave serious consideration to reducing the upper limit to 4% and recommends that a lower value be used when feasible.

C8.4.1(B) Flexural Strength

Section C4.4 indicates that bridges will be subjected to the simultaneous occurrence of ground motion in three orthogonal directions.

Thus columns are required to be designed biaxially and checked for both the minimum and maximum axial forces. For columns with a maximum axial stress exceeding $0.20f'_c$ the strength reduction factor, ϕ, is reduced to 0.50. A linear interpolation is to be used for the value of ϕ (0.90) for pure flexure and the value at $0.20f'_c$. This requirement was added because of the trend towards a reduction in ductility capacity as the axial load increases. Implicit in this requirement is the recommendation that design axial stresses be less than $0.20f'_c$. Columns with axial stresses greater than this value are not prohibited but are designed for higher force levels (i.e., lower ϕ factor) in lieu of the lower ductility capacity.

C8.4.1(C) Column Shear and Transverse Reinforcement

The requirements of this section are to minimize the potential for a column shear failure.[1,2] The design shear force is specified as either that capable of being developed by flexural yielding of the columns or the elastic design shear force. This requirement was added because of the potential for superstructure collapse if a column fails in shear.

It should be noted that a column may yield in either the longitudinal or transverse direction and that the shear force corresponding to the maximum shear developed in either direction (for non-circular columns) should be used for the determination of the tranverse reinforcement.

The concrete contribution to shear capacity is undependable within the plastic hinge zone, particularly at low axial load levels, because of full section cracking under load reversals. As a result, the concrete shear contribution must be neglected for axial load levels less than $0.10f'_c A_g$. It is probable that the use of $v_c = 0$ for low load levels is overconservative; however, the test data available at present are too limited to evaluate a reasonable alternative.

C8.4.1(D) Transverse Reinforcement for Confinement at Plastic Hinges

The main function of the transverse reinforcement specified in this section is to ensure that the axial load carried by the column after spalling of the concrete cover will at least equal the load carried before spalling and that buckling of the longitudinal reinforcement is prevented.[1,2] Thus the spacing of the confining reinforcement is also important.

8.18

Equation 2-3 of AASHTO Sec. 1.5.11 and Eq. 8-1 of these Guidelines are based on the arbitrary concept that, under axial compressive loading, the maximum capacity of the helically reinforced column (spiral column) before loss of cover concrete is equal to that with the cover concrete destroyed and the helical reinforcement stressed to its useful limit. The toughness of the spiral column under axial loading is not directly relevant to its typical role in earthquake-resistant structures where toughness or ductility is likely to be related to performance of the column under large reversals of

moment as well as axial load. Nonetheless, without implicit quantitative relationships between performance criteria interpreted in terms of the quality of the confined concrete and the amount of spiral reinforcement, there has been no compelling reason to modify Eq. 8-1 for earthquake-resistant construction other than by adding Eq. 8-2 which provides a varying lower bound to the amount of transverse reinforcement and tends to govern for columns with large cross-sectional areas.

The confinement requirements for rectangular sections of Eq. 8-3 were developed from the requirements for spiral columns as follows.

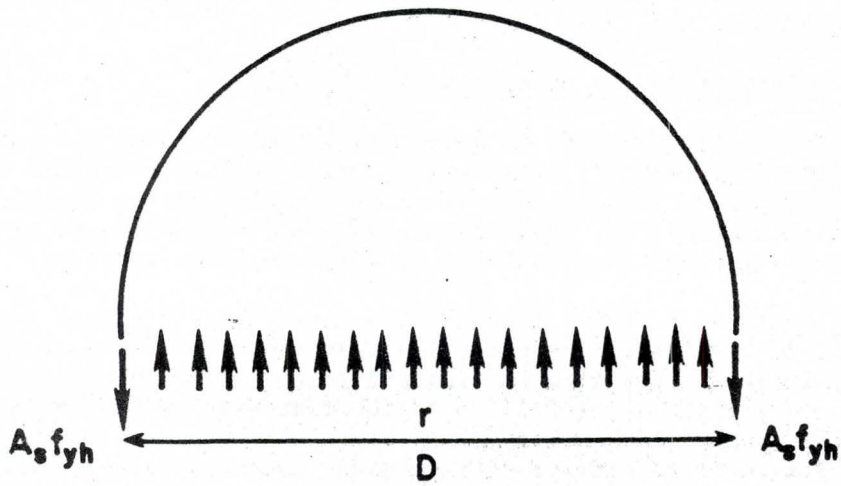

FIGURE 41. CONFINING PRESSURE PROVIDED BY A SPIRALLY REINFORCED COLUMN

The confining force P provided by a spirally reinforced column, shown in Fig. 41, is

$$P = rsD = 2A_s f_{yh} \quad ,$$

where

r = confining pressure

s = spacing of the spiral reinforcement

D = core diameter of the column

A_s = area of the spiral reinforcement

f_{yh} = yield strength of the spiral reinforcement.

Therefore,

$$r = \frac{2A_s f_{yh}}{sD} \quad . \qquad \text{(C8-1)}$$

The volumetric ratio ρ_s of spiral reinforcement is

$$\rho_s = \frac{4\pi DA_s}{\pi D^2 s} = \frac{4A_s}{Ds} \quad . \tag{C8-2}$$

Substituting Eq. C8-2 into Eq. C8-1,

$$r = \frac{\rho_s f_{yh}}{2} \quad . \tag{C8-3}$$

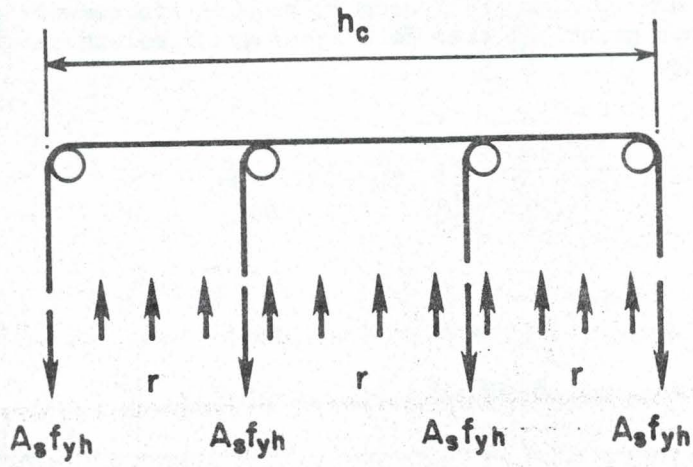

FIGURE 42. CONFINING PRESSURE PROVIDED BY A RECTANGULAR REINFORCED COLUMN

The confining force provided by a rectangular column, shown in Fig. 42, is

$$P = rah_c = \Sigma A_s f_{yh} \tag{C8-4}$$

where a is the spacing of the hoop reinforcement, and h_c is the core dimension of the column for the direction under consideration; $\Sigma A_s f_{yh}$ is the force resistance capability of the hoop reinforcement crossing the section under consideration. Therefore,

$$r = \frac{f_{yh}\,\Sigma A_s}{ah_c} \quad . \tag{C8-5}$$

Thus, if the two columns provide equal confining pressure, from Eq. C8-3 and Eq. C8-5

$$\Sigma A_s = ah_c\,\rho_s/2 \quad . \tag{C8-6}$$

Then, by substituting Eq. 8-1 into Eq. C8-6,

$$\Sigma A_s = 0.225\ ah_c \left[\frac{A_g}{A_c} - 1\right] \frac{f'_c}{f_{yh}} . \qquad \text{(C8-7)}$$

The 0.225 coefficient for a rectangular column corresponds to the experimentally determined 0.45 coefficient of Eq. 8-1 for a spiral column. The PEP felt however, on the basis of a limited amount of experimental data, that a rectangular column was not as effective as a spiral column. So the coefficient for a rectangular column was increased from 0.225 to 0.30.

Figures 43 and 44 will aid the designer in the use of Eq. 8-3. It should be noted that A_s, the total area of hoop reinforcement, should be determined for both principal axes of a rectangular column and the maximum value should be used.

NOTE:
AT SPLICES, CROSSTIES SHALL BE SUPPORTED OR SECURED TO PREVENT DISPLACEMENT DURING CONCRETE PLACEMENT.

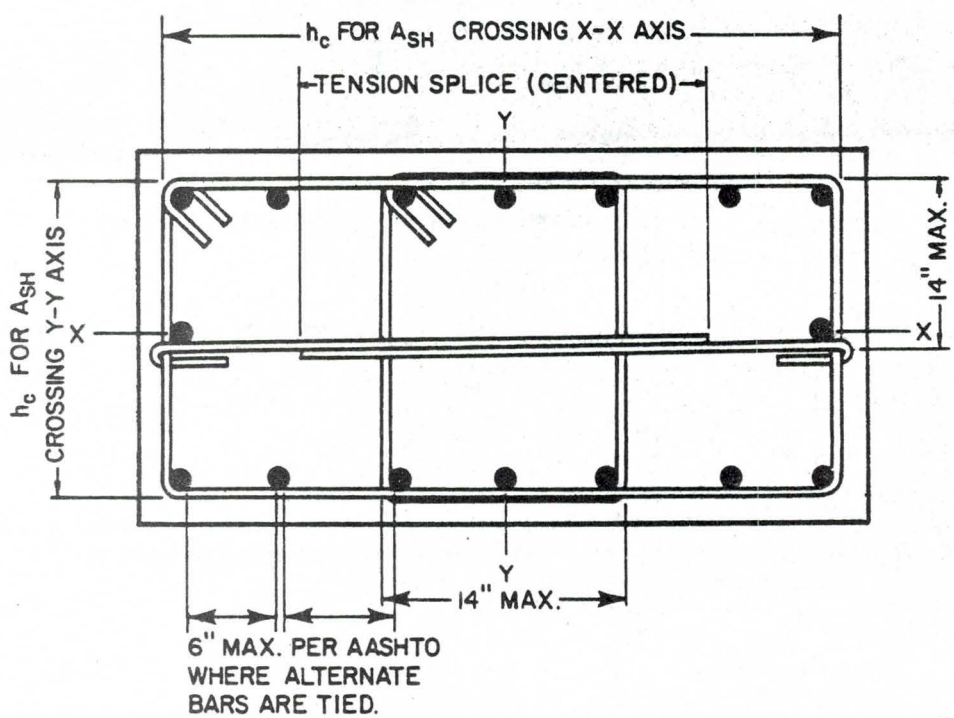

FIGURE 43. COLUMN TIE DETAILS

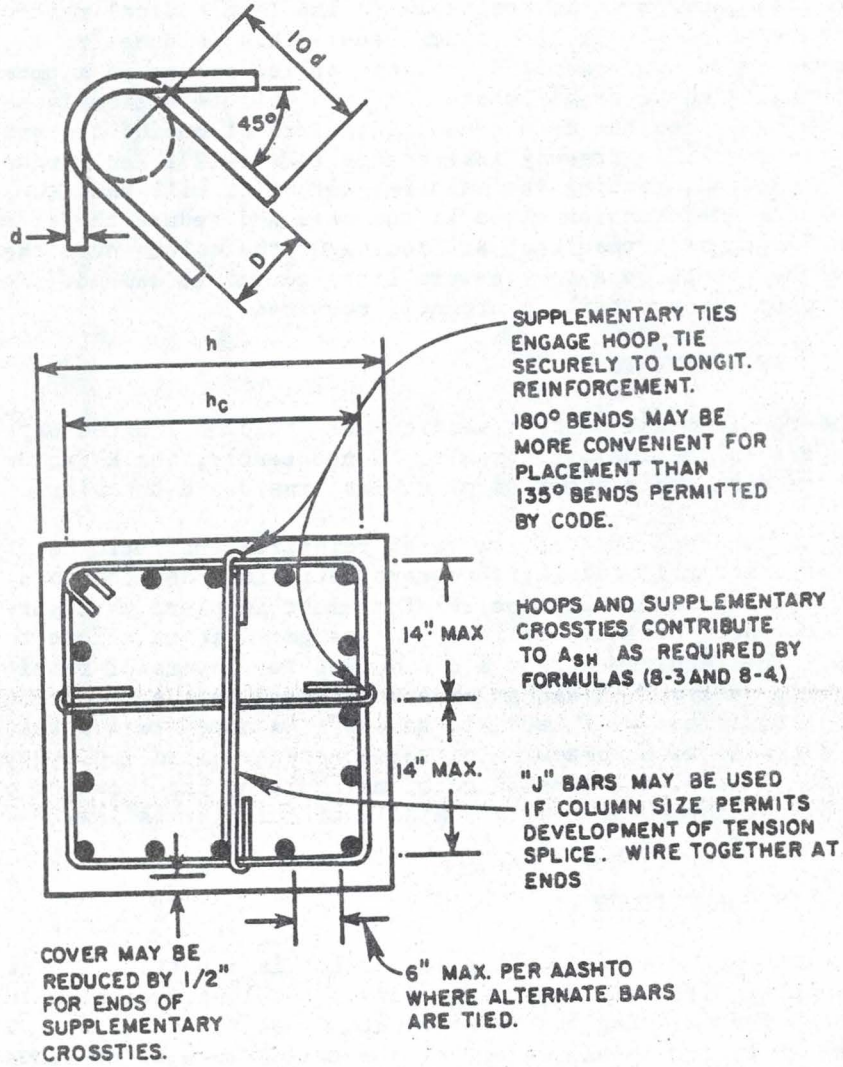

FIGURE 44. COLUMN TIE DETAILS

Loss of concrete cover in the plastic hinge zone, as a result of spalling, requires careful detailing of the confining steel. It is clearly inadequate simply to lap the spiral reinforcement. If the concrete cover is going to spall, the spiral will be able to unwind. Therefore under these conditions full strength lap welds are required. Similarly, rectangular hoops must be adequately anchored by bending ends back into the core. Thus the requirement of at least a 135° bend with an extension of at least 10 tie bar diameters back into the core, or an equivalent welded anchorage, was specified.

C8.4.1(F) Splices

In construction it is desirable to lap longitudinal reinforcement with starter bars or dowels at the column base. This is undesirable for seismic performance on two counts; first, the splice occurs in a potential plastic hinge region where requirements for bond will be extremely severe.[1] This appears to have been the main cause of failure of one of the bridges of the Golden-State-Foothills freeway interchange in the 1971 San Fernando earthquake.[3] Second, lapping the main reinforcement will tend to concentrate plastic deformation close to the base and reduce the effective plastic hinge length as a result of stiffening of the column over the lapping region. This may result in a very severe local curvature demand. Testing of this common construction detail is urgently required.

C8.4.2 PIER REQUIREMENTS

The requirements of this section are based on limited data on the behavior of piers in the inelastic range. Consequently, the R-Factor of 2 for piers is based on the assumption of minimal inelastic behavior.

It is required that the vertical reinforcement ratio be equal to or in excess of the horizontal reinforcement ratio in order to avoid the possibility of having inadequate web reinforcement in piers which are short in comparison to their height. Splices are staggered in an effort to avoid weak sections. The requirement for a minimum of two layers of reinforcement in walls carrying substantial design shears is based on the premise that two curtains of reinforcement will tend to "basket" the concrete and retain the integrity of the wall after cracking of the concrete. Also, under typical construction conditions, the likelihood of maintaining the location of a single layer of reinforcement near the middle of the pier is low.

C8.4.3 COLUMN CONNECTIONS

The integrity of the column connection is important if the columns are to develop their flexural capacity. First, the longitudinal reinforcement must be capable of developing its overstrength capacity of $1.25f_y$. Second, the transverse confining reinforcement of the column must be continued a sufficient distance into the joint to avoid a plane of weakness at the interface.

For column connections in a column cap an evaluation of existing and new data on the strength of joints subjected to moment reversals has indicated that the strength of the joint is relatively insensitive to the amount of transverse reinforcement, provided there is a minimum amount, and that a limiting shear stress of $12\sqrt{f'_c}$ for unconfined joints may be used for normal-weight aggregate concrete. The allowable stress for joints made with lightweight aggregate concrete has been based on the observation that shear transfer in such concrete has been measured to be approximately 75% of that in normal-weight aggregate concrete.

C8.4.4 CONSTRUCTION JOINTS IN PIERS AND COLUMNS

This section requires that construction joints be designed and constructed to resist seismic design forces at the joint. Equation 8-6 is based on Eq. 11-30 of ACI 318-73 but is restated to reflect dowel action and frictional resistance.

REFERENCES

1. Priestley, M.J.N. and Park, R. "Seismic Resistance of Reinforced Concrete Bridge Columns," Proceedings of a Workshop on the Earthquake Resistance of Highway Bridges, Applied Technology Council, Berkeley, California, January 1979.

2. Jirsa, J. O., "Applicability to Bridges of Experimental Seismic Test Results Performed on Subassemblages of Buildings," Proceedings of a Workshop on the Earthquake Resistance of Highway Bridges, Applied Technology Council, Berkeley, California, January 1979.

3. Fung, G., LeBeau, R. F., Klein, E. D., Belvedere, J. and Goldschmidt, A. G., "Field Investigation of Bridge Damage in the San Fernando Earthquake," Bridge Department, Division of Highways, California Department of Transportation, Sacramento, California, 1971.

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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.

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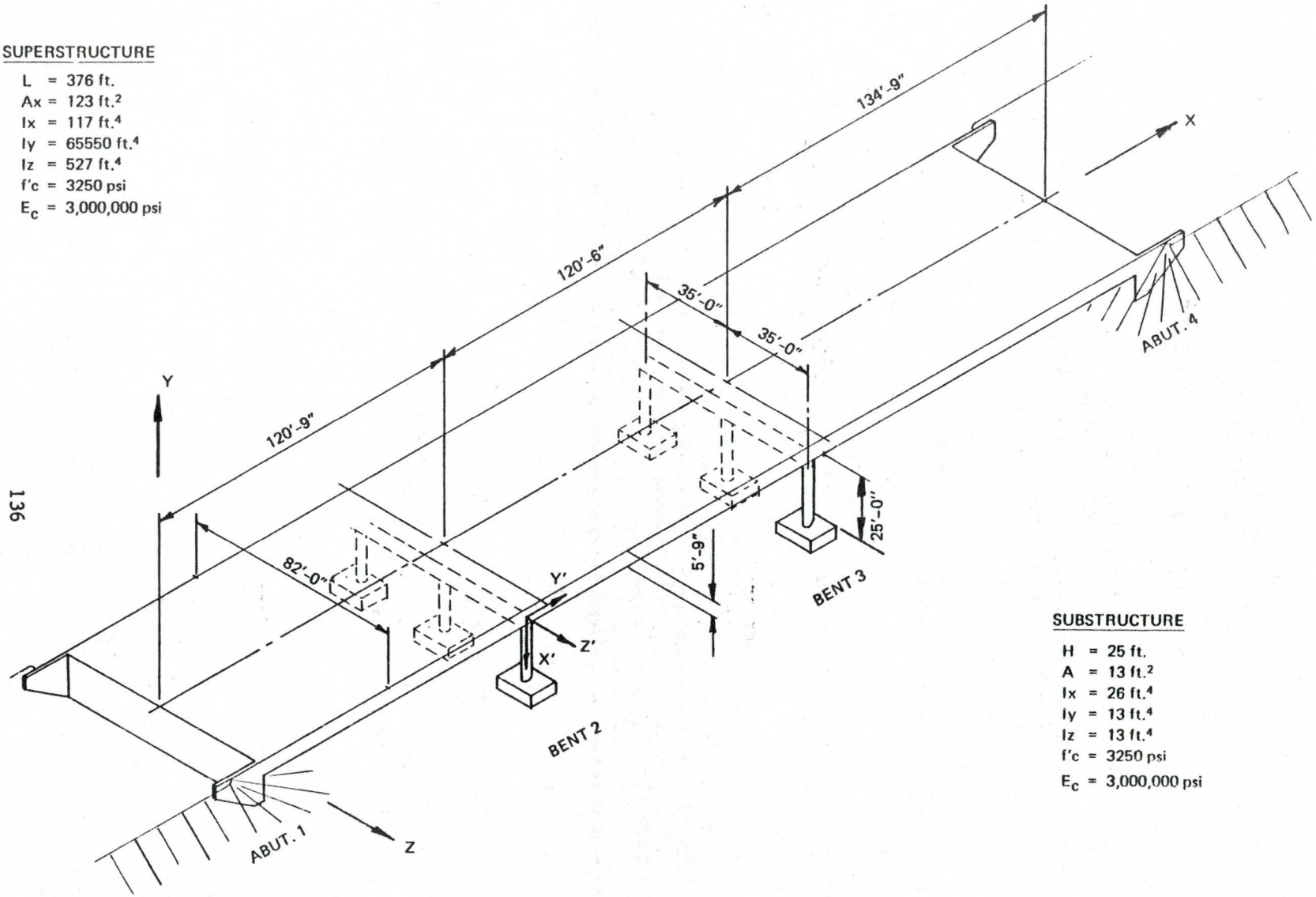
SUPERSTRUCTURE

L = 376 ft.
Ax = 123 ft.^{2}
Ix = 117 ft.^{4}
Iy = 65550 ft.^{4}
Iz = 527 ft.^{4}
f'c = 3250 psi
E_c = 3,000,000 psi



SUBSTRUCTURE

H = 25 ft.
A = 13 ft.^{2}
Ix = 26 ft.^{4}
Iy = 13 ft.^{4}
Iz = 13 ft.^{4}
f'c = 3250 psi
E_c = 3,000,000 psi

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 bends.

FIGURE A-1. DIMENSIONS OF EXAMPLE BRIDGE

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.

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.

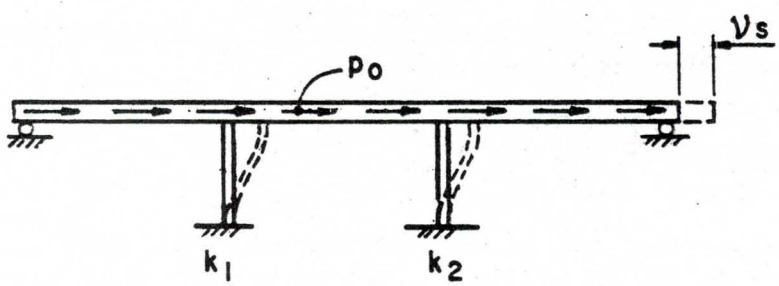


FIGURE A-2. STRUCTURAL IDEALIZATION AND APPLICATION OF ASSUMED UNIFORM LOADING FOR LONGITUDINAL MODE OF VIBRATION

Applying the assumed uniform longitudinal loading 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:

$$k_1=k_2=3\frac{(12EI)}{H^3}=3x\frac{12x432000}{25^3}x13=12940 \text{ kips/ft}$$

which yields a displacement of:

$$v_s = \frac{P_o L}{k_1+k_2} = \frac{1x376}{2x12940} = 0.0145 \text{ ft}$$

STEP 2: Assuming a weight density of the superstructure at 165 lb/ft^3, yields a dead weight per unit length of superstructure of: $w(x) = 0.165\ Ax = 0.165(123) = 20.3$ 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:

$$\alpha = \int_{\text{Abut. 1}}^{\text{Abut. 4}} v_s(x)dx = v_s L = 0.0145x376 = 5.46 \text{ ft}^2$$

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

$$\beta = \int_{\text{Abut. 1}}^{\text{Abut. 4}} w(x)v_s(x)dx = wv_s L = 20.3x0.0145x376 = 110.9 \text{ kip-ft}$$

$$\gamma = \int_{\text{Abut. 1}}^{\text{Abut. 4}} w(x)v_s(x)^2dx = wv_s^2 L = 20.3x(.0145)^2x376 = 1.61 \text{ kip-ft}^2$$

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

$$T = 2\pi\sqrt{\frac{\gamma}{p_o g\alpha}} = 2\pi\left[\frac{1.61}{1.0x32.2x5.46}\right]^{\frac{1}{2}}$$

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

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

$$C_s = \frac{1.2AS}{T^{2/3}} = \frac{1.2x0.4x1.2}{(0.60)^{2/3}} = 0.81$$

Since the seismic response coefficient does not exceed 2.5A (2.5x0.4 = 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:

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

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

$$M_{Z'Z'}\text{-Moment per Column} = 1030 \times 12.5 = 12900 \text{ kip-ft}$$

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

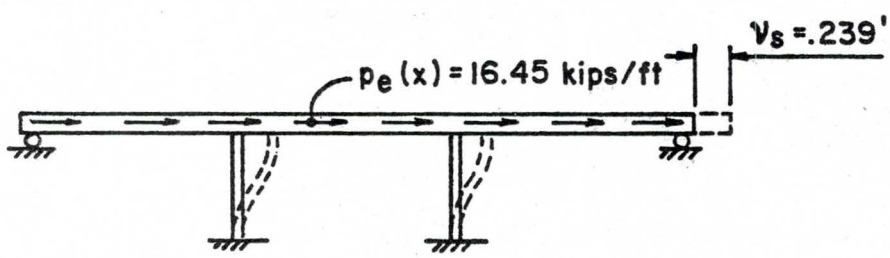
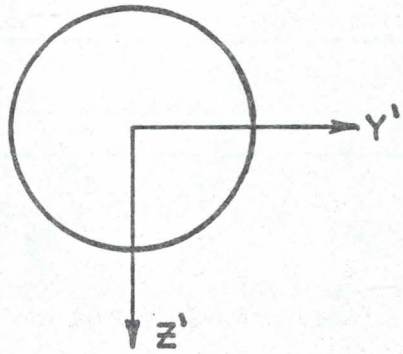


FIGURE A-3. DISPLACEMENTS AND SEISMIC LOADING INTENSITY FOR LONGITUDINAL LOADING

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 tabulated 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.

TABLE A-1: ELASTIC AND MODIFIED FORCES DUE TO LONGITUDINAL EARTHQUAKE MOTION



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.

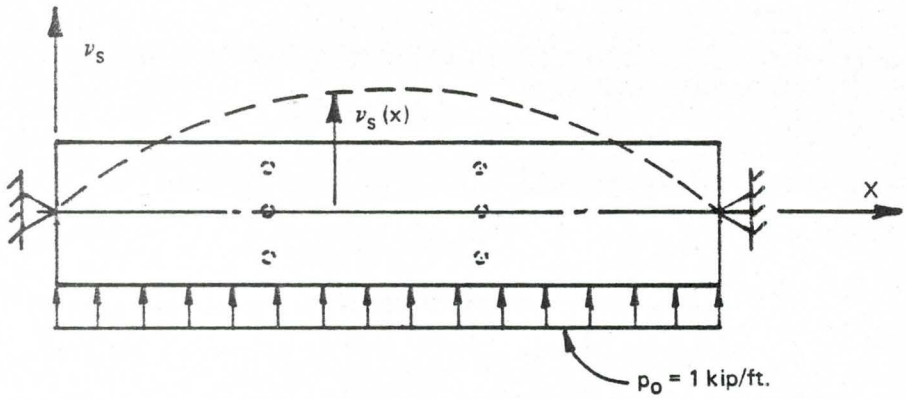


FIGURE A-4. PLAN VIEW OF THREE SPAN BRIDGE SUBJECTED TO ASSUMED TRANSVERSE LOADING

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

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

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

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

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

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

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

$$C_s = 1.2\frac{AS}{T^{2/3}} = \frac{1.2x0.4x1.2}{(0.314)^{2/3}} = 1.24$$

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:

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

$$= \frac{24.5 \times 1.0 \times 20.3}{0.096} v_s(x)$$

$$= 5157\ v_s(x)\ \text{kips/ft}^2$$

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.

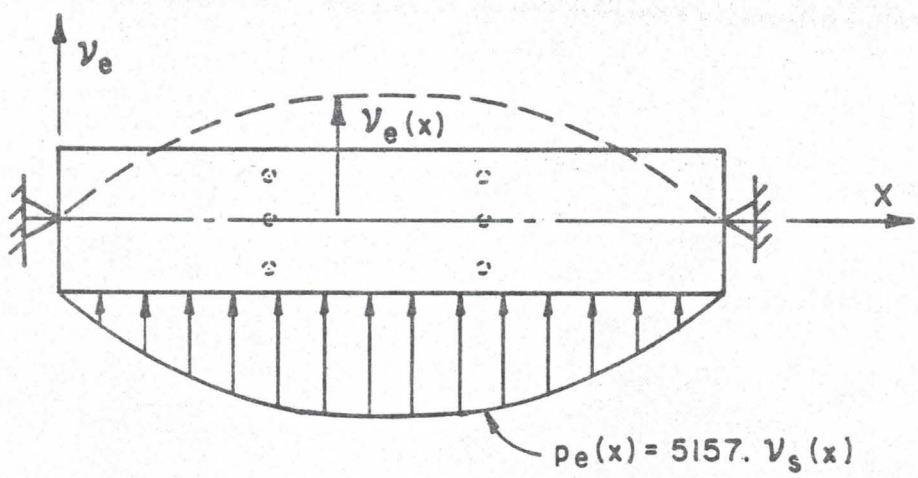


FIGURE A-5. PLAN VIEW OF THREE SPAN BRIDGE SUBJECTED TO EQUIVALENT STATIC SEISMIC LOADING

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

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

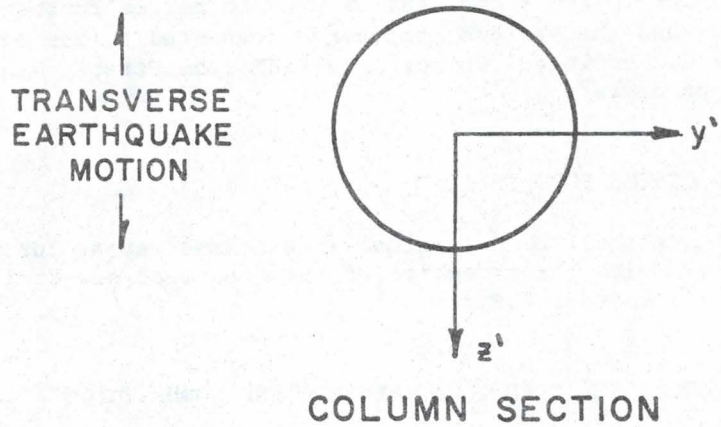
$$\beta = \int w(x)v_s(x)dx = 24.5 \text{ kip-ft}$$

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

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

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

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.

The transverse deck displacements are:

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

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.

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-Shear	685 kips	2283 kips
P_X-Axial Force	±106 kips*	±32 kips
Bents		
$V_{Y'}$-Shear	(1030+18) = 1048 kips	(59+309) = 368 kips
$M_{Z'Z'}$-Moment	(2580+42) = 2622 kip-ft	(141+774) = 915 kip-ft
$P_{X'}$-Axial Force	±(115+66) = ±181 kips	±(219+35) = ±254 kips
$V_{Z'}$-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
$V_{Y'}$-Shear	69 kips	0
$M_{Z'Z'}$-Moment	1170 kip-ft	0
$P_{X'}$-Axial Force	960 kips	624
$V_{Z'}$-Shear	0	0
$M_{Y'Y'}$-Moment	0	0

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

Modified Design Forces - Column

By inspection Load Case 1 controls:

$V_{Y'}$-Shear = 1.0(D+B+SF+E+EQM)

= 1.0(69+1048) = 1117 kips

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

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

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

$M_{Y'Y'}$-Moment = 1.0(305+0) = 305 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:

$V_{Z'}$-Shear = 1.0(D+B+SF+E+EQM)

= 1.0(0+2283) = 2283 kips.

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:

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

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

8-16.5.2 The magnification of moment due to slenderness effects is specified in AASHTO Sec. 1.5.34(B) for compression members not braced against sidesway. As specified the effects of slenderness may be neglected when $k\ell_u/r$ is less than 22. For these columns, $k\ell_u/r$ 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.

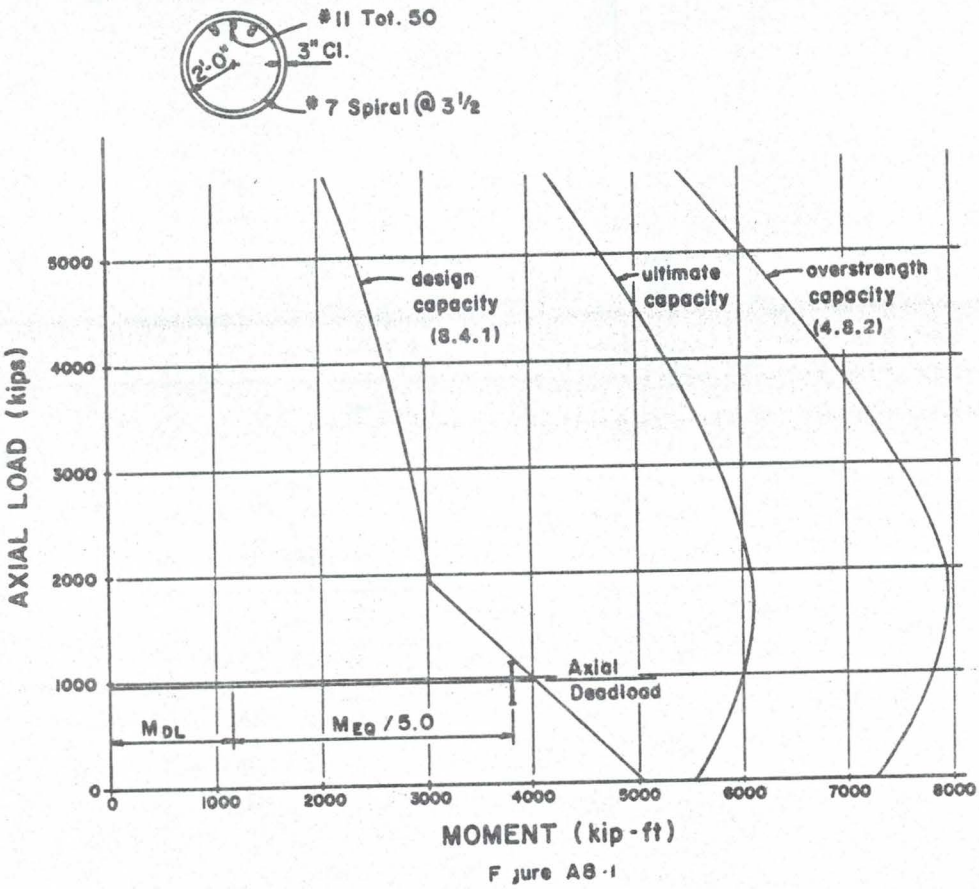
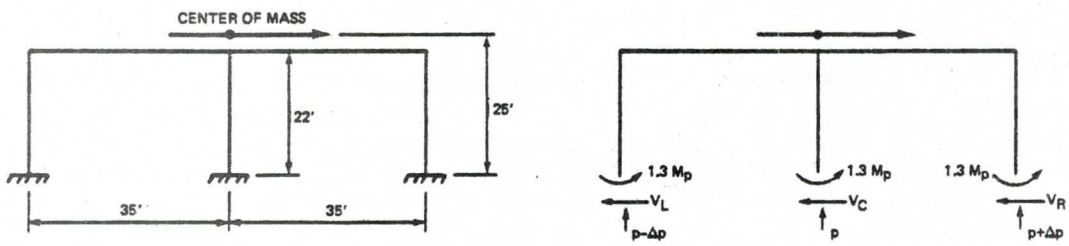


FIGURE A-6. COLUMN INTERACTION DIAGRAM

TABLE A-6: CALCULATION OF FORCES RESULTING FROM PLASTIC HINGING IN COLUMNS (4.8.2B)



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

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. 1.5.30 8.16 of AASHTO and Eq. 6-20 of Sec. 1.5.35 the factored shear stress for a circular column is 8.46 8.16.6

$$v_u = \frac{v_u}{\phi b_\omega d} = \frac{718}{0.85 \times 48 \times 43} = 409 \text{ psi} \quad .$$

The shear stress carried by the concrete outside the column end regions is (see AASHTO Sec. 1.5.35(B)). 8.16.6.2

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

Using Eq. 6-24 8.50 of AASHTO Sec. 1.5.35(C) 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

$$A_v = \frac{(v_u - v_c)}{f'y} b_\omega S = \frac{(409 - 114)}{60{,}000} 48 \times 3.5$$

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

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

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

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

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:

$$A_v = \frac{v_u}{fy} b_\omega S = \frac{394}{60{,}000} \text{ x } 48\text{x}3.5 = 1.1 \text{ in.}^2 \text{ total area required.}$$

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

Thus, a #7 spiral with a 3-1/2 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,

$$\rho_s = 0.45 \left(\frac{A_g}{A_c} - 1\right) \frac{f_c'}{f_{yh}}$$

$$= 0.45 \left(\frac{12.57}{9.62} - 1\right) \frac{3250}{60{,}000} = 0.0075$$

or

$$\rho_s = 0.12 \frac{f_c'}{f_{yh}} = \frac{0.12\text{x}3250}{60{,}000} = 0.0065$$

The cross-sectional area of a spiral at 3-1/2 in. pitch is given by:

$$A_{sp} = \frac{\rho_s sds}{4} = \frac{0.0075x3.5x41.25}{4} = 0.270 \text{ in.}^2$$

Since this is less than the shear reinforcement, there is no additional requirement for confinement at the plastic hinges; thus use #7 spiral at 3-1/2 in. in the 4 ft-0 in. end regions and #6 spiral at 3-1/2 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 x 624 = 312 kips

312 > 106 None Required

Abutment 4

0.5 x 701 = 350 kips

350 > 92 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:

Axial-bearings 701+92 = 793 kips

Shear-keys = 2283 kips

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:

$$N = 12+0.03L + 0.12H$$

$$= 12 + 0.03 \times 376 + 0.12 \times 25$$

$$= 26 \text{ in.}$$

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

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

APPENDIX B

SUMMARY OF REDESIGNS

B1 INTRODUCTION

The redesign phase of the project consisted of using the third draft of the Guidelines to redesign existing bridges seismically. Five state highway departments (California, Idaho, New York, Oklahoma, and Washington) and four consulting firms (De Leuw Cather, Inc. of San Francisco; Howard, Needles, Tammen and Bergendorff of Milwaukee; Modjeski and Masters of Pennsylvania and Moffat and Nichol of Long Beach) participated in the redesigns. A total of 21 bridges were seismically redesigned using the third draft of the Guidelines. All bridge loadings remained the same for the redesigns except for the seismic loads. Each state redesigned its bridges for Acceleration Coefficients and Seismic Performance Categories applicable to that state. The consultants redesigned their bridges for four different Acceleration Coefficients (0.10, 0.20, 0.30 and 0.40) and three different Seismic Performance Categories (B, C and D). A brief description of the bridges and a brief summary of the redesigns are presented in Table B1. A more detailed description and summary of the results for the bridges redesigned by the consultants are given in Tables B2 through B7. Results for those bridges redesigned by the states are described in Sec. B4.

B2 CHANGES TO THE THIRD DRAFT OF THE GUIDELINES RESULTING FROM THE REDESIGNS

A number of significant changes to the third draft of the Guidelines resulted from the redesigns. These are discussed prior to the presentation of the results of the redesigns because several of the state redesigns were not reassessed to determine the impact of the changes. The more significant changes are as follows:

(1) The third draft contained three methods of analysis. Methods 2 and 3 in the third draft are now procedures 1 and 2, respectively, of Sec. 4.2. Method 1 of the third draft was a simplified but conservative method of analysis and was specified for regular bridges classified as SPC B.

The method did not require calculation of the period of the bridge. In effect, the method was designed to specify a lateral (i.e., longitudinal or transverse) force equal to some fraction of the bridge weight. The bridge, or bridge section, was designed to resist a lateral seismic load, W_L, applied through the center of mass of the superstructure.

For the analysis, the structural idealization consisted of the entire structure or the structure subdivided into sections in a manner analogous to that used to analyze the bridge for lateral wind loadings. When the idealization consisted of the entire structure, the seismic load was distributed according to the distribution of the structure mass. When

subdividing the superstructure into sections at each bent, the superstructure lateral load was apportioned to each section in accordance with the superstructure vertical dead load reaction at the bent or the dead load from tributary areas adjacent to the bent. The seismic load, W_L, was given by:

$$W_L = C_s W \quad , \qquad (6\text{-}1)$$

where

W_L = total lateral load in kips/ft applied along the center of mass of the superstructure,

C_s = 2.5A,

W = the gravity load of the bridge in kips/ft. W shall be taken equal to the weight of the superstructure of the bridge and applicable portions of other components.

Most redesigns performed with this method resulted in conservative design values especially in the longitudinal direction. The ease of using the method in areas where engineers were not familiar with seismic design was considerably offset by the significant increase in costs that resulted from its inherent conservatism. Consequently the PEP recommended that the method not be included in the Guidelines. The analysis methods referred to in Tables B1 to B7 are the three methods used in the third draft of the Guidelines (i.e., Method 1 is as outlined, and Methods 2 and 3 which are Procedures 1 and 2 of Sec. 4.2 in the final Guidelines).

(2) The third draft of the Guidelines specified a minimum gap (N') requirement at all discontinuous segments of a bridge. The requirement N' equals N/4 - the minimum seat width requirement given by Eqs. 4-3 and 4-4. The purpose of this proposed requirement was to minimize damage due to impact at these joints. In almost all redesigns this proved to be a very expensive requirement and the PEP assessed that the cost/benefit ratio was not sufficient to justify retaining the requirement.

(3) The third draft of the Guidelines required that foundations for bridges classified as SPC B be designed for the elastic forces. This was included so that a designer would not have to calculate the forces resulting from plastic hinging. This proved to be a costly requirement and the foundation forces for bridges classified as SPC B were reduced to twice the column design force.

(4) The third draft of the Guidelines included a provision for the use of ultimate capacities of both soils and piles with the specified foundation design forces. This was not used in all redesigns and as a result the provision was given more emphasis in the Guidelines.

(5) Uplift of the foundations was not permitted in the third draft of the Guidelines. This decision was reassessed in light of the large increases in foundation costs. Uplift with certain limitations is now permitted.

(6) The third draft of the Guidelines had only one provision for bridges classified as SPC A, that of minimum support lengths. It was thought that designing for wind would be sufficient to cover the seismic design of connections. As part of the consultants' redesign effort, wind design forces were compared with seismic design forces for Acceleration Coefficients of 0.05 and 0.10. It was found in most instances that the seismic design forces were greater than the wind forces and consequently the requirement in Sec. 4.6 was added.

(7) The R-Factor for multiple column bents has been decreased to 5 from the value of 6 given in the third draft. This change will not have a significant cost impact although its impact was not included in the redesigns.

B3 SUMMARY OF COST IMPACT

The cost impact of the Guidelines is based on comments received from the engineers who redesigned the 21 bridges listed in Table B1. The engineers varied considerably in experience from those having little or no seismic experience to those who, routinely, seismically design bridges. Engineers with little experience tended to make the most conservative interpretation of the Guidelines and not all had time to reevaluate the impact of the recommended changes to the third draft of the Guidelines.

A summary of the types of bridges included in the redesigns is given in Fig. B-1. The cost impact by bridge type using the third draft of the Guidelines is given in Fig. B-2 and a bar chart summary is given in Fig. B-3. Following the changes to the third draft of the Guidelines (Sec. B2), the cost impact of the redesigns was re-evaluated. A summary of the cost impact after these changes were incorporated is shown in Fig. B-4. It should be noted that the cost impacts shown in Fig. B-4 are applicable to the guidelines presented in this report. The average cost increase for all bridges is 6.3%. If the three bridges with pier supports are excluded then the average cost increase for all other bridge types is 1.7%.

In the case of pier supports the cost impact was significant, especially for higher Acceleration Coefficients. The cause of the high cost increase for foundations is the design philosophy which attempts to minimize damage to the foundations. This is achieved in the Guidelines by designing the foundations for the maximum expected forces (either from column hinging or the elastic forces). This requirement was subsequently reduced for bridges classified as SPC B. To minimize the cost impact from the foundations the engineer should be sure that ultimate soil capacities are used and, where permitted, uplift should be accounted for in the design. In addition, wherever feasible, forces resulting from column or pier hinging should be used. These factors will considerably decrease the cost impact on the foundations.

The cost impact on columns varied but was not more than 5% of the total bridge cost. The cost impact on abutments varied from a decrease in

cost in some cases to increases in other cases. However, in most instances the abutment was not as thoroughly redesigned as other elements.

B4 DETAILED RESULTS OF REDESIGNS

More detailed results of the redesigns performed by the consultants are given in Tables B2 to B7. The results of redesigns performed by the states follow. The bridge numbers in the following summary refer to the bridge numbers in Table B1.

Bridge Nos. 7 to 11 - California

The effect of the Guidelines on five bridges designed by CalTrans was assessed. Unfortunately because of time pressures a thorough redesign was not performed on any of the bridges, although the effect on all the major structural elements was assessed. Current CalTrans criteria use a maximum Coefficient of 0.7 and three of the five bridges were redesigned using this coefficient for both the ATC and CalTrans provisions.

The three-span Kern River bridge which has pier supports had a 3.3% increase in pier costs with little or no increase in the foundation costs. Based on the total bridge cost, the Route 80 on-ramp, which had single column supports, had a 1.5% increase in column costs and a 1.0% increase in foundation costs. There was also a 1.5% increase in the cost of hinge restrainers.

The three bridges with multiple column bents showed little or no increase in cost. For the Milliken Avenue bridge, the designer stated that seismic design did not govern in the original design or the design using the draft Guidelines.

In summary it is apparent that, although the philosophy and some of the detailed provisions of the Guidelines differ from those of the CalTrans requirements, the difference in the costs of implementing the two sets of provisions is within 5% based on the total bridge cost and the design force levels applicable to California.

Bridge Nos. 12 and 13 - Idaho

The Lewiston Hill bridge was redesigned as a bridge classified as SPC A with an Acceleration Coefficient of 0.05. In the third draft of the criteria the only design requirement for this category was a minimum support length, and this had no impact on the original design. The Guidelines now contain a requirement for designing connections between the super and substructure. The effect of this requirement was not evaluated for this bridge.

In the initial redesign the Heyburn bridge had a 20% cost increase. Five percent of this cost was attributable to the gap requirement while the other 15% occurred because of the increased number of piles. This increase was caused by two factors. First, analysis method 1 was used to determine the forces; and second, the third draft of the Guidelines did not specify an R-Factor for pile bents. Consequently the R-Factor of 1 for foundations was used. Following clarification of the R-Factor for pile bents, the use of analysis method 2 and the removal of the gap requirement, no increase in cost resulted from use of the revised Guidelines.

Bridge Nos. 14 and 15 - Oklahoma

The Verdigris bridge was redesigned using an Acceleration Coefficient of 0.10 and analysis method 1. The designer was not able to complete the redesign but determined for one bent there would be a $20,000 increase in the foundation cost. Unfortunately because of time constraints the designer was not able to evaluate the impact of using analysis method 2 or the recommended changes in the foundation forces for SPC B. It is assumed that both these factors would reduce the cost increase obtained for the one foundation that was redesigned.

The Poteau River bridge was redesigned using an Acceleration Coefficient of 0.10 and analysis method 1. This resulted in a $40,000 increase in the cost of the foundation for each bent. No cost increases were reported for the columns. The designer assessed the impact of using analysis method 2 and twice the column design forces for the foundation design as recommended in the Guidelines for SPC B. This decreased the cost increase for each foundation from $40,000 to $4,000.

Bridge Nos. 16 to 19 - New York

The four redesigns performed by the State of New York for an Acceleration Coefficient of 0.10 indicated that for three of the four bridges the original bearings would have to be redesigned. A cost estimate for the redesigned bearings was not provided although it was stated that it would not be significant. Analysis method 1 was evaluated in two of the redesigns and the results indicated the method produced very conservative design forces when compared to those obtained from analysis method 2. This and similar results from other redesigns convinced the PEP to remove the easily applied but overly conservative analysis method 1 of the third draft of the Guidelines.

The results from the Future Route 225 bridge indicated that additional reinforcement would be required in the columns and footings although a cost impact was not provided.

Bridge Nos. 20 and 21 - Washington

The two span bridge was originally designed using the 1973 AASHTO specifications and the interim specifications through 1977. An increase of 1.1% was reported in the foundation cost, which is a significantly smaller increase than for redesigns of other bridges with pier substructures. This is attributed to the fact that the original design forces were similar to those used in the redesigns.

The six span bridge was originally designed using the 1973 AASHTO specifications and the interim specifications through 1977. The reported cost increase of 1.9% does not include any cost increases for the foundation. The designer estimated that this increase would be of the order of 5% of the cost of the bridge but he indicated that if the forces resulting from column plastic hinging were used this cost would be significantly decreased. In the redesign the designer estimated that the cost of the gap requirement would be approximately 1.7% of the cost of the bridge. This provision has now been removed from the Guidelines.

SUBSTRUCTURE (BENTS AND PIERS)	SUPERSTRUCTURE			
	STEEL	CONCRETE	STEEL	CONCRETE
PIER	1	2	2	2
MULTI COLUMN BENT	2	1	3	6
SINGLE COLUMN BENT	-	-	-	2
OTHER	-	2	-	-

FIGURE B-1. BRIDGE TYPES

SUBSTRUCTURE (BENTS AND PIERS)	SUPERSTRUCTURE			
	STEEL	CONCRETE	STEEL	CONCRETE
PIER	5.1	5.1	4.5	44.8
MULTI COLUMN BENT	0.5	1.9	0.3	4.7
SINGLE COLUMN BENT	-	-	-	3.6
OTHER	-	0.0	-	-

FIGURE B-2. COST INCREASE BY BRIDGE TYPES (3RD DRAFT)

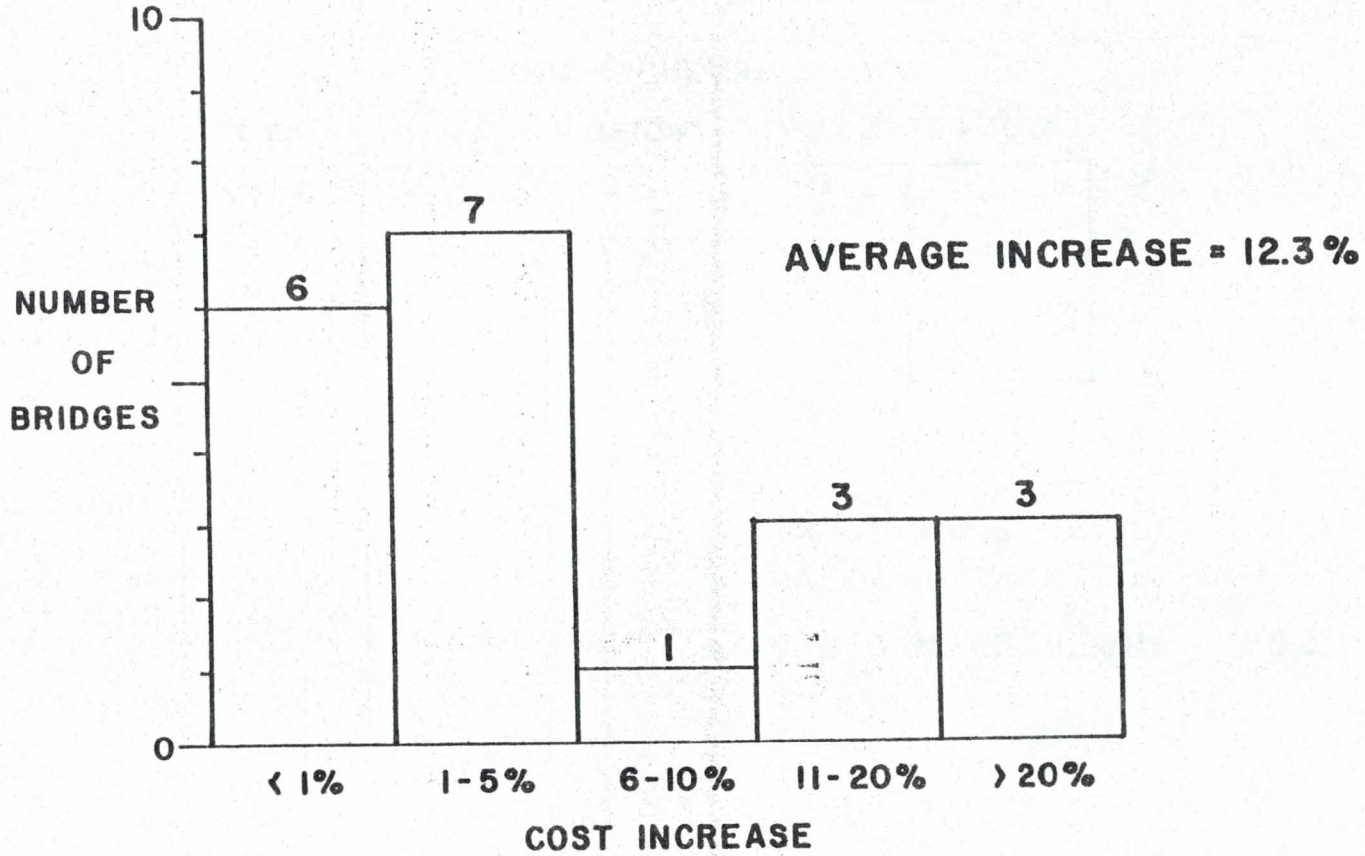

FIGURE B-3. COST IMPACT OF REDESIGNS (3RD DRAFT)

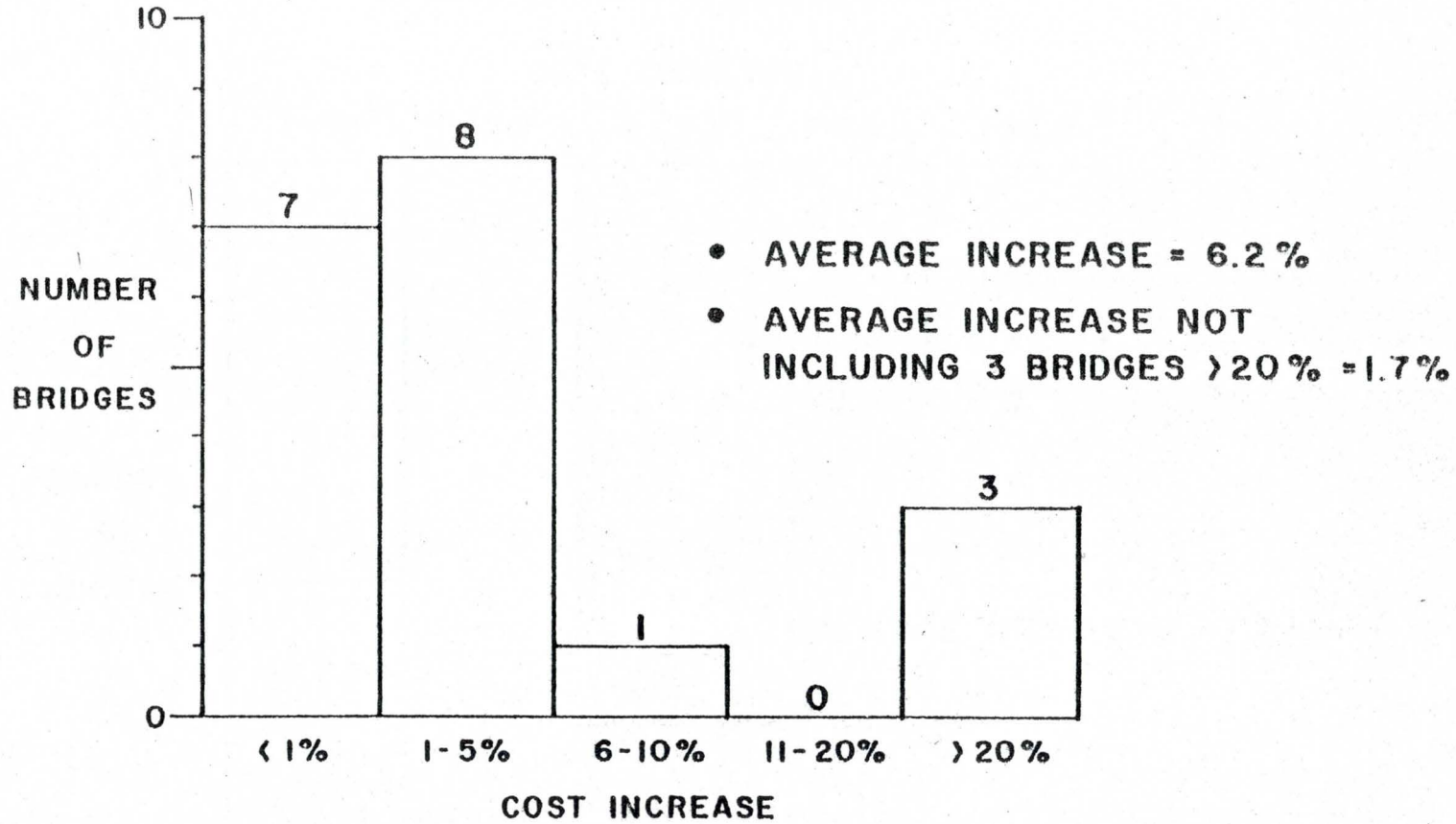

FIGURE B-4. COST IMPACT OF REDESIGNS (AFTER CHANGES TO 3RD DRAFT)

TABLE B-1: SUMMARY OF BRIDGE REDESIGNS USING THIRD DRAFT OF SEISMIC DESIGN GUIDELINES FOR HIGHWAY BRIDGES

Bridge Name & Location	Length Feet (Meters)	No. of Spans	A	SPC	Analysis Method[1]	Superstructure	Substructure	Foundation	Abutments	Comments
1. Hudson River Valley New York	357 (109)	5	0.10 0.20 0.30 0.40	B C C D	1 2 2 2	Continuous rolled beam - concrete structure 36'(11m) wide	3 Column Reinf. Conc. Bents Squ. Columns w/pier caps	Pile Supported Concrete Slabs	Stub Abutments w/wing walls supported by piles	Costs increased up to 5% for SPC D. Costs were incurred in columns, footings, and abutments as the force level increased. No original seismic design. Designed by 1957 AASHTO.
2. Humbolt River Nevada	290 (88)	3	0.10 0.20 0.30 0.40	B C C D	1 2 2 2	Cast-in-place conc. box girder 57' (17m) wide w/1300' (396m) radius of curvature	Four Column Bents Skew varies between 30 to 40 degrees	Pile footings	Open end seat with spread footings	Costs increased up to 12% for SPC D designs. 70% of cost was in the abutment. Designed by 1977 AASHTO.
3. Durham Road California	202 (62)	2	0.10 0.20 0.30 0.40	B C C D	1 2 2 2	Cast-in-place conc. box girder 84' (26m) wide	Four Column Bents	Pile footings	Monolithic w/deck on pile footings	Costs increased up to 24% for SPC D designs. Most of the increase was in foundation. Designed by 1977 AASHTO. CalTrans criteria.
4. Menomonee River Wisconsin	240 (73)	2	0.10 0.20 0.30 0.40	B C C D	1 2 2 2	Continuous welded steel composite plate girder 30' (9m) wide	4'(1.2m) wide x 20'(6m) long pier with 11 degree skew	Pile footings	1-free standing full retaining 1-free standing cellular - both pile supported	Costs increased up to 34% for SPC D. Most of the increase was in the foundation. No original seismic design. Designed by 1976 AASHTO.

Notes:
[1]The numbers refer to those used in the third draft. Analysis methods 2 and 3 of the third draft are the same as analysis procedures 1 and 2 of the Guidelines.

TABLE B-1: SUMMARY OF BRIDGE REDESIGNS USING THIRD DRAFT OF SEISMIC DESIGN GUIDELINES FOR HIGHWAY BRIDGES (continued)

Bridge Name & Location	Length Feet (Meters)	No. of Spans	A	SPC	Analysis[1] Method	Superstructure	Substructure	Foundation	Abutments	Comments
5. Michelson Dr. California	350 (107)	4	0.10 0.20 0.30 0.40	B C C D	2 2 2 2	Precast-prestressed concrete girders w/reinforced concrete deck 43'(13m) wide	Solid reinforced concrete walls	45 ton prestressed conc. piles	Open end seat type, free standing with back wall	Costs increased up to 47% for SPC D with 2/3 of the cost increase in the piers and foundations. Designed by 1973 AASHTO and CalTrans.
6. South E. Street California	640 (195)	8	0.10 0.20 0.30 0.40	B C C D	2 3 3 3	Cast-in-place R.C. T beams. 2 intermediate hinges and expansion joints at each abutment - 39'(12m) wide	Solid reinforced concrete walls	50 ton driven steel H piles	Open end seat type, free standing with back wall	Costs increased up to 45% for SPC D with 80% of the cost occurring in the piers and foundations. Designed by 1973 AASHTO and CalTrans.
7. Kern River California	404 (123)	3	0.70 0.20	C C	CalTrans 1	Composite concrete slab on steel plate girders 62'(19m) wide	Wall piers	Spread foundation	Cantilever seat type abutment	3.3% increase in pier costs; 0.03% increase in foundation costs; 0.25% increase in superstructure/pier connection. Increase based on going from CalTrans method to ATC-6 with A=0.7.
8. Route 80 On-Ramp California	694 (212)	7	0.70 0.20	C C	2	2 Frames-CIP/PS Box Girder and RC Box Girder 40'(12) wide	Single cantilever columns	Concrete cast in drilled holes	Expansion type with end diaphragm on piles	1.5% increase in column costs; 1.0% increase in foundation; 0.3% increase in restrainers; 1.5% increase in hinges. Increases based on going from CalTrans method to ATC-6 with A=0.7.

Notes:
[1]The numbers refer to those used in the third draft. Analysis methods 2 and 3 of the third draft are the same as analysis procedures 1 and 2 of the Guidelines.

TABLE B-1: SUMMARY OF BRIDGE REDESIGNS USING THIRD DRAFT OF SEISMIC DESIGN GUIDELINES FOR HIGHWAY BRIDGES (continued)

Bridge Name & Location	Length Feet (Meters)	No. of Spans	A	SPC	Analysis Method[1]	Superstructure	Substructure	Foundation	Abutments	Comments
9. Milliken Ave. California	455 (139)	7	0.40 0.20	C C	2 2	Reinforced concrete box girders 42' (13m) wide	2 column bents	Concrete footings on CIDH piles	Cantilever type with back wall	Redesign not completed and Designer states that seismic forces do not appear to govern.
10. Verdugo Blvd. California	523 (160)	5	0.70 0.40	C C	3 3	CIP reinforced box girders 16m wide	Multi-column bents	Spread footings and short steel H piles	Diaphragm type on spread footings	No changes required compared to design for comparable CalTrans force levels.
11. Camden Ave. California	225 (69)	2	0.40 0.30	C C	1 1	Composite concrete slab on steel box girders 67.5'(21m) wide	2 column bents	Spread footings	Monolithic abutment on spread footing	0.3% increase in column costs; 0.5% increase in footing costs; 1.4% decrease in abutment costs. Decrease of 0.6% based on going from CalTrans method to ATC-6 with A=0.4.
12. Lewiston Idaho	252 (77)	3	0.05	A	N/A	Cast-in-place conc. box girder 79'(24m) wide	3 column bents	Spread footings	Concrete cap supported on piles	No change in the design or related costs. The new SPC A requirements were not evaluated.
13. Heyburn Idaho	1312 (400)	32	0.15	C	1	Prestressed T-beams w/concrete deck 42'(13m) wide	Steel shell pile bents filled with concrete with 35° skew	Piles and columns are continuous	Beam type supported by piles	A 20% increase in total cost due to additional pile requirements. Changes to third draft reduced the increase to zero. Designed by 1975 AASHTO.

Notes:
[1]The numbers refer to those used in the third draft. Analysis methods 2 and 3 of the third draft are the same as analysis procedures 1 and 2 of the Guidelines.

TABLE B-1: SUMMARY OF BRIDGE REDESIGNS USING THIRD DRAFT OF SEISMIC DESIGN GUIDELINES FOR HIGHWAY BRIDGES (continued)

Bridge Name & Location	Length Feet (Meters)	No. of Spans	A	SPC	Analysis[1] Method	Superstructure	Substructure	Foundation	Abutments	Comments
14. Verdigris River Oklahoma	451 (137)	5	0.10	B	1	Prestressed conc. 1 beam 35'(11m) wide	2 columns w/pier cap and web wall	Spread footings	Cantilever on piles with wing walls	A $100,000 increase in foundation costs. This was not reevaluated for the recommended changes. Designed by 1976 AASHTO.
15. Poteau River Oklahoma	572 (174)	5	0.10	B	1	Reinforced concrete girders on approach spans. Haunched steel plate girder center 3 spans 35'(11m) wide	2 columns per bent w/a web wall and tie beam between columns	Two footing conditions were included: A drilled shaft and a spread footing	Cellular type abutments. The approach spans form the top part of the abutment.	Approx. 3.6% (of total cost) increase in cost of each foundation. This reduced to 0.4% with recommended changes. Designed by 1976 AASHTO.
16. Battenkill New York	182 (55)	1	0.10	B	1	Prestressed post tensioned concrete beams with 10cm concrete slab 14m wide	----------	----------	Open end seat type on spread foundations	A small increase in the cost of the abutment bearings. Designed by NYDOT spec.
17. Rt. 225 New York	234 (71)	2	0.10	B	1 and 2	Steel plate girders w/concrete slab 63'(19m) wide	3 column bents	Spread footings	Open end seat type on spread foundations	Major redesign of bearings. Foundation size increased. Designed by NYDOT spec.
18. Hulett Street New York	631 (192)	3	0.1	B	1 and 2	Steel plate girders w/concrete slab 42'(13m) wide	Two column bents	35 ton cast-in-place concrete piles	Open end abutments supported on piles	Bearings require redesigning. No change in column design. Designed by NYDOT spec.

Notes:
[1]The numbers refer to those used in the third draft. Analysis methods 2 and 3 of the third draft are the same as analysis procedures 1 and 2 of the Guidelines.

TABLE B-1: SUMMARY OF BRIDGE REDESIGNS USING THIRD DRAFT OF SEISMIC DESIGN GUIDELINES FOR HIGHWAY BRIDGES (continued)

Bridge Name & Location	Length Feet (Meters)	No. of Spans	A	SPC	Analysis Method[1]	Superstructure	Substructure	Foundation	Abutments	Comments
19. Rt. 28A New York	475 (145)	2	0.10	B	1 and 2	Steel plate girders w/composite slab 37'(11m) wide	Two column bent	Spread footings	Open seat abutment on spread foundations	No change in footing and column design. Designed by NYDOT spec.
20. Midway Avenue Washington	241 (73)	2	0.20	C	1	Precast girders continuous for liveload, 13m wide	Single column 7m high	Spread footings	Cantilever bearing wall w/spread footings	Small cost increase (1%) in footings. Designed by 1977 W DOT spec. for earthquake.
21. Yakima River Washington	570 (174)	6	0.15	C	2	Prestressed girder 16m wide	Two column bent	Spread footings	Stub wall	Approx. 8% increase in total cost compared to original structure. Designed by 1977 W DOT spec. for earthquake.

Notes:
[1]The numbers refer to those used in the third draft. Analysis methods 2 and 3 of the third draft are the same as analysis procedures 1 and 2 of the Guidelines.

TABLE B-2: BRIDGE NO. 1 - HUDSON RIVER VALLEY

GENERAL BRIDGE INFORMATION

GENERAL: Five span continuous rolled beam - concrete deck structure.

BRIDGE LENGTH: 357 ft (109 m)

SPANS: Four spans of 75 ft (23 m); one span of 57 ft (17 m)

CURVATURE: None

ROADWAY WIDTH: 30 ft (9 m)

TOTAL BRIDGE WIDTH: 35 ft 9 in. (10.9 m)

PIER SKEW: None

ABUTMENT SKEW: None

DESCRIPTION OF SUPERSTRUCTURE: Five 36 in. (914 mm) wide flange beams designed in composite action with the 7 in. (178 mm) thick slab. Transverse diaphragms are 18 in. (457 mm) structural channels spaced at 19 ft (5.8 m).

DESCRIPTION OF SUBSTRUCTURE: Three column bents of reinforced concrete with 36 in. (914 mm) square columns and pier caps. Clear column heights are 21 ft (6.4 m), 9.2 ft (2.8 m), 16 ft (4.9 m) and 15 ft (4.6 m).

DESCRIPTION OF ABUTMENTS: Stub abutments with wing walls are on pile supported footings in the upper portion of the embankment. Backwalls are 4.5 ft (1.4 m) high, bearing seats are 3 ft (0.9 m) high and the footings are 3 ft (0.9 m) thick for a total abutment height of 10.5 ft (3.2 m). Concrete filled steel pipe piles are embedded 6 in. (152 mm) into the footings.

TABLE B-2: BRIDGE NO. 1 - HUDSON RIVER VALLEY (continued)

DESCRIPTION OF FOUNDATIONS: Pile supported concrete slabs 3 ft (0.9 m) thick, 29 ft (8.8 m) long 10 to 12 ft (3 to 3.7 m) wide. Piles are concrete-filled steel pipe piles driven to rock with a design capacity of 45 tons (405 kN). The pile lengths vary from 13 to 35 ft (4 to 11 m).

COLUMN CONNECTIONS: Three of the piers are connected with rocker bearings for the steel stringers. The fixed pier bearings base casting is anchored to the pier with 2 in. (50.8 mm) diameter bolts.

ABUTMENT CONNECTIONS: Cast iron rocker bearings.

DESIGN SPECIFICATION: The bridge was designed in 1961 to the 1957 AASHTO Standard Specifications. Seismic forces were not considered in the design.

DESIGN LOADING: Two 24,000 lb (108 kN) axles at 4 ft (1.2 m) on center.

TABLE B-2: BRIDGE NO. 1 - HUDSON RIVER VALLEY (continued)

SEISMIC DESIGN DATA

ACCELERATION COEFFICIENT A	SOIL PROFILE	SEISMIC PERFORMANCE CATEGORY	RESPONSE MODIFICATION FACTOR			METHOD OF ANALYSIS	SEISMIC COEFFICIENT				PERIOD OF VIBRATION (SEC)			
			COLUMN OR PIER		ABUTMENT AND/OR EXP. JOINT		ORIGINAL DESIGN		GUIDELINE DESIGN		ORIGINAL DESIGN		GUIDELINE DESIGN	
			LONG	TRAN			LONG	TRAN	LONG	TRAN	LONG	TRAN	LONG	TRAN
0.10	I	B	6	6	0.8	1	-	-	0.25	0.25	-	-	-	-
0.20	I	C	6	6	0.8	2	-	-	0.23	0.50	-	-	1.04	0.19
0.30	I	C	6	6	0.8	2	-	-	0.35	0.75	-	-	1.04	0.19
0.40	I	D	6	6	0.8	2	-	-	0.47	1.00	-	-	1.04	0.19

TABLE B-2: BRIDGE NO. 1 - HUDSON RIVER VALLEY (continued)

SUMMARY OF EXTRA COSTS

SEISMIC PERFORMANCE CATEGORY	B	C	C	D
ACCELERATION COEFFICIENT	0.10	0.20	0.30	0.40
Columns & Piers	-	$14,715	$15,970	$17,735
Abutments	$ 9,322	$11,518	$11,598	$12,654
Foundations	$ 5,985	$13,458	$19,895	$29,600
Connections				
Other				
Total	$15,307	$39,798	$47,463	$59,989
% Increase Based on 1980 Construction Cost of $1,200,000	1.3%	3.3%	4.0%	5.0%

TABLE B-3: BRIDGE NO. 2 - HUMBOLT RIVER

GENERAL BRIDGE INFORMATION

GENERAL: Three span cast-in-place post tensioned concrete box girder bridge.

BRIDGE LENGTH: 292 ft (89 m)

SPANS: 3

CURVATURE: 1300 ft (396 m)

ROADWAY WIDTH:

TOTAL BRIDGE WIDTH: 57 ft (17 m)

PIER SKEW: 37° and 32°

ABUTMENT SKEW: 28° and 42°

DESCRIPTION OF SUPERSTRUCTURE: Cast-in-place post tensioned concrete box girder 4-1/2 ft (1.37 m) deep.

DESCRIPTION OF SUBSTRUCTURE: Two four column bents. Columns are 4 ft (1.2 m) in diameter 27 to 32 ft (8.2 to 9.8 m) long.

DESCRIPTION OF ABUTMENTS: Open end seat with 3 ft (914 mm) bearing seat and 1 ft (305 mm) backwall. The abutment is on a 9 ft wide by 2 ft (2.8 m x 0.6 m) thick spread footing.

DESCRIPTION OF FOUNDATIONS: Pile footing with a 9 ft x 7 ft x 3-1/4 ft (2.8 m x 2.1 m x 1.1 m) pile cap. There are six 40 ft (12.2 m) long piles with a 70 ton (630 kN) capacity.

TABLE B-3: BRIDGE NO. 2 - HUMBOLT RIVER (continued)

COLUMN CONNECTIONS: Integral with the superstructure and pinned at the column footing.

ABUTMENT CONNECTIONS: Elastomeric bearing pads.

DESIGN SPECIFICATION: 1973 AASHTO Specifications and Interim Specifications to 1977.

DESIGN LOADING: AASHTO HS 20-40. Seismic loading was for a coefficient of 0.066 in the longitudinal direction and 10% in the transverse.

TABLE B-3: BRIDGE NO. 2 - HUMBOLT RIVER (continued)

SEISMIC DESIGN DATA

ACCELERATION COEFFICIENT A	SOIL PROFILE	SEISMIC PERFORMANCE CATEGORY	RESPONSE MODIFICATION FACTOR			METHOD OF ANALYSIS	SEISMIC COEFFICIENT				PERIOD OF VIBRATION (SEC)			
			COLUMN OR PIER		ABUTMENT AND/OR EXP. JOINT		ORIGINAL DESIGN		GUIDELINE DESIGN		ORIGINAL DESIGN		GUIDELINE DESIGN	
			LONG	TRAN			LONG	TRAN	LONG	TRAN	LONG	TRAN	LONG	TRAN
0.10	II	B	6	6	0.8	1	0.066	0.10	0.25	0.25	-	-	-	-
0.20	II	C	6	6	0.8	2	0.066	0.10	0.25	0.50	1.23	-	1.23	0.33
0.30	II	C	6	6	0.8	2	0.066	0.10	0.38	0.75	1.23	-	1.23	0.33
0.40	II	D	6	6	0.8	2	0.066	0.10	0.50	1.00	1.23	-	1.23	0.33

TABLE B-3: BRIDGE NO. 2 - HUMBOLT RIVER (continued)

SUMMARY OF EXTRA COSTS

SEISMIC PERFORMANCE CATEGORY	B	C	C	D
ACCELERATION COEFFICIENT	0.10	0.20	0.30	0.40
Columns & Piers	-$2,780	-$2,830	-$2,800	-$2,740
Abutments	-	-	-	$34,920
Foundations	+$20,480[1]	$6,670	$13,550	$16,350
Connections	-$1,520	$440	$2,060	$3,180
Other	-	-	-	-
Total	$16,180	$4,280	$12,810	$51,710
% Increase Based on Construction Cost of $435,000.	3.7%	1.0%	2.9%	11.9%

Note: [1]The foundations were designed for the full elastic forces whereas in SPC C and D they were designed for the column plastic hinge forces.

TABLE B-4: BRIDGE NO. 3 - DURHAM ROAD

GENERAL BRIDGE INFORMATION

GENERAL: Two span cast-in-place post tensioned concrete box girder.

BRIDGE LENGTH: 202 ft (62 m)

SPANS: Two of 101 ft (31 m)

CURVATURE: None

ROADWAY WIDTH:

TOLL BRIDGE WIDTH: 84 ft (26 m)

PIER SKEW: None

ABUTMENT SKEW: None

DESCRIPTION OF SUPERSTRUCTURE: Cast-in-place post tensioned concrete box girder 4-1/2 ft (1.4 m) deep.

DESCRIPTION OF SUBSTRUCTURE: One four column bent with 4 ft (1.2 m) diameter columns 32-1/2 ft (9.9 m) long.

DESCRIPTION OF ABUTMENTS: Monolithic abutments 3 ft (0.9 m) wide with pile foundation. Piles are 71 ft (2.2 m) long with 70 ton (630 kN) capacity.

TABLE B-4: BRIDGE NO. 3 - DURHAM ROAD (continued)

DESCRIPTION OF FOUNDATIONS: Pile foundation with 9 ft x 9 ft x 3-3/4 ft (2.7 m x 2.7 m x 1.1 m) thick pile cap. Piles are 51 ft (15.5 m) long with 70 ton (630 kN) capacity.

COLUMN CONNECTIONS: Integral with superstructure and fixed at column footing.

ABUTMENT CONNECTIONS: Monolithic

DESIGN SPECIFICATION: 1977 AASHTO and State of California Division of Highways Bridge Planning and Design Manual.

DESIGN LOADING: AASHTO HS20-44

TABLE B-4: BRIDGE NO. 3 - DURHAM ROAD (continued)

SEISMIC DESIGN DATA

ACCELERATION COEFFICIENT A	SOIL PROFILE	SEISMIC PERFORMANCE CATEGORY	RESPONSE MODIFICATION FACTOR			METHOD OF ANALYSIS	SEISMIC COEFFICIENT				PERIOD OF VIBRATION (SEC)			
			COLUMN OR PIER		ABUTMENT AND/OR EXP. JOINT		ORIGINAL DESIGN		GUIDELINE DESIGN		ORIGINAL DESIGN		GUIDELINE DESIGN	
			LONG	TRAN			LONG	TRAN	LONG	TRAN	LONG	TRAN	LONG	TRAN
0.10	II	B	6	6	-	1	0.146	0.146	0.25	0.25	0.51	0.51	-	-
0.20	II	C	6	6	-	2	0.146	0.146	0.30	0.50	0.51	0.51	0.94	0.12
0.30	II	C	6	6	-	2	0.146	0.146	0.45	0.75	0.51	0.51	0.94	0.12
0.40	II	D	6	6	-	2	0.146	0.146	0.60	1.0	0.51	0.51	0.94	0.12

TABLE B-4: BRIDGE NO. 3 - DURHAM ROAD (continued)

SUMMARY OF EXTRA COSTS

SEISMIC PERFORMANCE CATEGORY	B	C	C	D
ACCELERATION COEFFICIENT	0.10	0.20	0.30	0.40
Columns & Piers	-$8,350	-$7,650	-$2,400	-$5,540
Abutments	-	-	-	-
Foundations	$3,000	$22,900	$76,620	$99,200
Connections	-	$800	$800	$1,500
Other	-$420	-$420	-$420	$28,380(1)
Total	-$5,770	$15,630	$74,600	$134,530
% Increase Based on 1980 Estimated Cost of $576,200	-1.0%	2.7%	13.0%	23.4%

Note: (1)Approach Slab Costs.

TABLE B-5: BRIDGE NO. 4 - MENOMONEE RIVER

GENERAL BRIDGE INFORMATION

GENERAL: Two span structure spanning the Menomonee River, two traffic lanes, pedestrian walkway on one side.

BRIDGE LENGTH: 240 ft (73 m)

SPANS: Two of 120 ft (36.6 m)

CURVATURE: 6°

ROADWAY WIDTH: 30 ft (9 m)

TOTAL BRIDGE WIDTH: 38 ft 6 in. (11.8 m)

PIER SKEW: 11° +

ABUTMENT SKEW: 11° +

DESCRIPTION OF SUPERSTRUCTURE: Two span continuous welded steel composite plate girder with 60 in. (1524 mm) web; girder spacing 8 ft (2.4 m) +; 8 in. (203 mm) concrete deck.

DESCRIPTION OF SUBSTRUCTURE: Concrete T-type pier, pile supported, with 4 ft x 20 ft (1.2 m x 6 m) shaft beveled at ends; shaft cap cantilevers 9 ft (2.7 m) each side of shaft; 4 ft (1.2 m) deep concrete seal poured beneath footing; pier height = 37 ft (11.3 m) from top of footing to top of pier; footing thickness = 4 ft (1.2 m).

DESCRIPTION OF ABUTMENTS: North Abutment - Free standing, full retaining abutment, pile supported; South Abutment - free standing, cellular abutment, pile supported.

DESCRIPTION OF FOUNDATIONS: All footings supported on HP 12 x 53 piles driven to 60 tons (540 kN). Piles designed for 45 tons (405 kN) per pile with an allowance of 15 tons (135 kN) per pile for drag load.

TABLE B-5: BRIDGE NO. 4 - MENOMONEE RIVER (continued)

COLUMN CONNECTIONS: Fixed steel rocker bearing welded to girder and anchored to pier with steel anchor bolts.

ABUTMENT CONNECTIONS: Expansion steel rocker bearings with lubricated bronze plate. Bearing plate anchored to abutment with steel anchor bolts.

DESIGN SPECIFICATION: AASHTO 1973 with interims through 1976 with no seismic design.

DESIGN LOADING: HS20-44

TABLE B-5: BRIDGE NO. 4 - MENOMONEE RIVER (continued)

SEISMIC DESIGN DATA

ACCELERATION COEFFICIENT A	SOIL PROFILE	SEISMIC PERFORMANCE CATEGORY	RESPONSE MODIFICATION FACTOR			METHOD OF ANALYSIS	SEISMIC COEFFICIENT				PERIOD OF VIBRATION (SEC)			
			COLUMN OR PIER		ABUTMENT AND/OR EXP. JOINT		ORIGINAL DESIGN		GUIDELINE DESIGN		ORIGINAL DESIGN		GUIDELINE DESIGN	
			LONG	TRAN			LONG	TRAN	LONG	TRAN	LONG	TRAN	LONG	TRAN
0.10	I	B	2	2	0.8	1	-	-	0.25	0.25	-	-	0.00	0.00
0.20	I	C	2	2	0.8	2	-	-	0.37	0.50	-	-	0.98	0.14
0.30	I	C	2	2	0.8	2	-	-	0.55	0.75	-	-	0.98	0.14
0.40	I	D	2	2	0.8	2	-	-	0.73	1.00	-	-	0.98	0.14

TABLE B-5: BRIDGE NO. 4 - MENOMONEE RIVER (continued)

SUMMARY OF EXTRA COSTS

SEISMIC PERFORMANCE CATEGORY	B	C	C	D
ACCELERATION COEFFICIENT	0.10	0.20	0.30	0.40
Columns & Piers	$ 5,000	$ 5,000	$ 6,000	$ 12,000
Abutments	-	$ 1,000	$ 1,000	$ 20,000
Foundations	$32,000	$60,000	$100,000	$140,000
Connections	$ 1,000	$ 4,000	$ 6,000	$ 8,000
Other	-	-	-	-
Total	$38,000	$70,000	$113,000	$160,000
% Increase Based on 1980 Estimated Cost of $475,000.	8%	15%	24%	34%

TABLE B-6: BRIDGE NO. 5 - MICHELSON DRIVE

GENERAL BRIDGE INFORMATION

GENERAL: Four span precast prestressed concrete "I" girder bridge.

BRIDGE LENGTH: 350 ft (107 m)

SPANS: Four of approximately 87 ft (27 m) each

CURVATURE: None

ROADWAY WIDTH: 29 ft (8.8 m)

TOTAL BRIDGE WIDTH: 43 ft (13 m)

PIER SKEW: 1-1/2°

ABUTMENT SKEW: 1-1/2°

DESCRIPTION OF SUPERSTRUCTURE: Precast prestressed "I" girders, continuous for live load with reinforced concrete deck slab.

DESCRIPTION OF SUBSTRUCTURE: Solid wall of reinforced concrete 44 ft long x 23 ft high, 20 in. thick (13.4 m x 7.0 m, 508 mm).

DESCRIPTION OF ABUTMENTS: Open end seat type, free standing with back wall, parallel wing walls integral with abutments.

DESCRIPTION OF FOUNDATIONS: 45 ton (405 kN) driven precast-prestressed concrete piles, 15 in. (381 mm) square.

TABLE B-6: BRIDGE NO. 5 - MICHELSON DRIVE (continued)

CONCRETE CONNECTIONS: Girders continuous at piers, depressed shear keys at top of piers to resist longitudinal and transverse loads.

ABUTMENT CONNECTIONS: Girders on elastomeric bearing pads, shear keys utilized to resist transverse lateral loads.

DESIGN SPECIFICATION: 1973 AASHTO supplemented by State of California Bridge Planning and Design Manual.

DESIGN LOADING: HS 20.

TABLE B-6: BRIDGE NO. 5 - MICHELSON DRIVE (continued)

SEISMIC DESIGN DATA

ACCELERATION COEFFICIENT A	SOIL PROFILE	SEISMIC PERFORMANCE CATEGORY	RESPONSE MODIFICATION FACTOR			METHOD OF ANALYSIS	SEISMIC COEFFICIENT				PERIOD OF VIBRATION (SEC)			
			COLUMN OR PIER		ABUTMENT AND/OR EXP. JOINT		ORIGINAL DESIGN		GUIDELINE DESIGN		ORIGINAL DESIGN		GUIDELINE DESIGN	
			LONG	TRAN			LONG	TRAN	LONG	TRAN	LONG	TRAN	LONG	TRAN
0.10	II	B	2	2	0.8	2	0.12	0.18	0.15	0.25	0.92	-	0.92	0.03
0.20	II	C	4	2	0.8	2	0.12	0.18	0.30	0.50	0.92	-	0.92	0.03
0.30	II	C	4	2	0.8	2	0.12	0.18	0.46	0.75	0.92	-	0.92	0.03
0.40	II	D	4	2	0.8	2	0.12	0.18	0.61	1.00	0.92	-	0.92	0.03

TABLE B-6: BRIDGE NO. 5 - MICHELSON DRIVE (continued)

SUMMARY OF EXTRA COSTS

SEISMIC PERFORMANCE CATEGORY	B	C	C	D
ACCELERATION COEFFICIENT	0.10	0.20	0.30	0.40
Columns & Piers	-	$18,000	$ 56,600	$ 88,600
Abutments	-	$ 9,500	$ 23,000	$ 51,200
Foundations			$ 24,000	$122,000
Connections				
Other				
Total	-	$27,500	$103,600	$261,600
% Increase Based on 1980 Estimated Cost of $600,500.		4.6%	17%	44%

TABLE B-7: BRIDGE NO. 6 - SOUTH E. STREET

GENERAL BRIDGE INFORMATION

GENERAL: Eight span cast in place reinforced contrete "T" beam.

BRIDGE LENGTH: 640 ft (195 m)

SPANS: Two of 64 ft (19.5 m), six of 85.3 ft (26 m)

CURVATURE: None

ROADWAY WIDTH: 26 ft (7.9 m)

TOTAL BRIDGE WIDTH: 39 ft (12 m)

PIER SKEW: 20°

ABUTMENT SKEW: 20°

DESCRIPTION OF SUPERSTRUCTURE: Cast in place reinforced concrete "T" beam with 2 intermediate hinges and expansion joints at each abutment.

DESCRIPTION OF SUBSTRUCTURE: Solid wall reinforced concrete piers 42.5 ft (13.0 m) long, 37 ft (11.3 m) high and 20 in. (508 mm) thick.

DESCRIPTION OF ABUTMENTS: Open end seat type, free standing with back wall. Settlement approach slab on back wall.

DESCRIPTION OF FOUNDATIONS: 50 ton (450 kN) driven "H" piles

TABLE B-7: BRIDGE NO. 6 - SOUTH E. STREET (continued)

COLUMN CONNECTIONS: Piers built integral with deck.

ABUTMENT CONNECTIONS: Expansion joints with longitudinal and transverse shear keys.

DESIGN SPECIFICATION: 1973 AASHTO supplemented by State of California Bridge Planning and Design Manual.

DESIGN LOADING: HS 20.

TABLE B-7: BRIDGE NO. 6 - SOUTH E. STREET (continued)

SEISMIC DESIGN DATA

ACCELERATION COEFFICIENT A	SOIL PROFILE	SEISMIC PERFORMANCE CATEGORY	RESPONSE MODIFICATION FACTOR			METHOD OF ANALYSIS	SEISMIC COEFFICIENT				PERIOD OF VIBRATION (SEC)			
			COLUMN OR PIER		ABUTMENT AND/OR EXP. JOINT		ORIGINAL DESIGN		GUIDELINE DESIGN		ORIGINAL DESIGN		GUIDELINE DESIGN	
			LONG	TRAN			LONG	TRAN	LONG	TRAN	LONG	TRAN	LONG	TRAN
0.10	II	B	4	2	0.8	2	0.14	0.25	0.14	0.23	0.82	0.4	1.0	0.49
0.20	II	C	4	2	0.8	3	0.14	0.25	0.28	0.46	0.82	0.4	1.0	0.49
0.30	II	C	4	2	0.8	3	0.14	0.25	0.43	0.70	0.82	0.4	1.0	0.49
0.40	II	D	4	2	0.8	3	0.14	0.25	0.57	0.93	0.82	0.4	1.0	0.49

TABLE B-7: BRIDGE NO. 6 - SOUTH E. STREET (continued)

SUMMARY OF EXTRA COSTS

SEISMIC PERFORMANCE CATEGORY	B	C	C	D
ACCELERATION COEFFICIENT	0.10	0.20	0.30	0.40
Columns & Piers	-	$137,000	$173,700	$241,500
Abutments	$ 4,000	$ 4,000	$ 15,000	$ 15,400
Foundations		$ 84,000	$140,000	$187,600
Connections				
Other	$ 8,000	$ 8,000	$ 8,000	$ 9,000
Total	$12,000	$233,000	$336,700	$453,500
% Increase Based on 1980 Estimated Cost of $990,000	1.2%	23.5%	34%	46%

APPENDIX C

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