Recommended LRFD Guidelines for the Seismic Design of Highway Bridges

Customary U.S. Units

Requested by:

American Association of State Highway
and Transportation Officials (AASHTO)

Highway Subcommittee on Bridge and Structures

Prepared by:

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Acknowledgements

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Preface

The seismic design specifications included in the current AASHTO LRFD Bridge Design Specifications, Third Edition (2004) with 2006 Interim Revisions and the AASHTO Standard Specifications for Highway Bridges, Division I-A, 17th Edition (2002) with Errata March 2005 are essentially the recommendations that were completed by the Applied Technology (ATC-6) in 1981 and adopted by AASHTO as a "Guide Specification" in 1983. In 1990 AASHTO adopted the Guide Specification (i.e., ATC-6/Division I-A) as part of the AASHTO Standard Specification for Highway Bridges. Some minor revisions were made for their inclusion into the AASHTO LRFD Bridge Design Specifications. There have been some significant changes that have occurred in seismic design since the adoption of ATC-6. Recognizing the availability of improvements as documented in NCHRP 12-49, Caltrans Seismic Design Criteria (SDC) 2004, SCDOT – Seismic Design Specifications for Highway Bridges, 2002 and related research projects, the T-3 AASHTO committee for seismic design has, with the financial support of NCHRP, initiated this project to update the Recommended LRFD Guidelines for the "Seismic Design of Highway Bridges" May 2006.

1. INTRODUCTION

1.1 BACKGROUND

The AASHTO LRFD Guidelines for Seismic Design of Highway Bridges is established in accordance with NCHRP 20-07/Task 193 Task 6 Report. Task 6 contains five (5) Sections corresponding to Tasks 1 to 5 as follows:

SECTION 1 includes a review of the pertinent documents and information that were available.

SECTION 2 presents the justification for the 975-year return period (i.e., 5% probability of exceedance in 50 years as recommended for the seismic design of highway bridges.

SECTION 3 includes a description of how the "no analysis" zone is expanded and how this expansion is incorporated into the displacement based approach.

SECTION 4 describes the two alternative approaches available for the design of highway bridges with steel superstructures and concludes with a recommendation to use a force based approach for steel girder superstructures.

SECTION 5 describes the recommended procedure for liquefaction design to be used for highway bridges. This aspect of the design is influenced by the recommended hazard level and the no analysis zone covered in Tasks 2 and 3, respectively. The recommendations proposed are made taking into account the outcome of these two tasks for Seismic Design Category D.

The following recommendations are documented.

<u>Task 2</u>

1. Adopt the 5% in 50 years hazard level for development of a design spectrum.
2. Ensure sufficient conservatism (1.5 safety factor) for minimum seat width requirement. This conservatism is needed to enable to use the reserve capacity of hinging mechanism of the bridge system. This conservatism shall be embedded in the specifications to address unseating vulnerability. At a minimum it is recommended to embed this safety factor for sites outside of California.

C1.1 Scope of Commentary

This commentary is included to provide additional information to clarify and explain the technical basis for the specifications provided in the LRFD Guidelines for Seismic Design of Highway Bridges. These guidelines are for the design of new bridges. It is envisioned that the commentary will be expanded and completed at the completion of the Test Designs being completed by the states that have volunteered to use the new guidelines on the trial designs.

The term "shall" denotes a requirement for compliance with these Guidelines.

The term "should" indicates a strong preference for a given criterion.

The term "may" indicates a criterion that is usable, but other local and suitably documented, verified, and approved criterion may also be used in a manner consistent with the LRFD approach to bridge design.

The term "recommended" is used to give guidance based on past experiences. Seismic design is a developing field of engineering, which has not been uniformly applied to all bridge types and thus the experiences gained to date on only a particular type are included as recommendations.

3. Partition Seismic Design Categories (SDCs) into four categories and proceed with the development of analytical bounds using the 5% in 50 years Hazard level.

Task 3

Establish four Seismic Design Categories with the following requirements.

1. SDC A
    a. No Displacement Capacity Check Needed
    b. No Capacity Design Required
    c. SDC A, Minimum Requirements
2. SDC B
    a. Implicit Displacement Capacity Check Required (i.e., use a Closed Form Solution Formula)
    b. No Capacity Design Required
    c. SDC B, Level of Detailing
3. SDC C
    a. Implicit Displacement Capacity Check Required
    b. Capacity Design Required
    c. SDC C, Level of Detailing
4. SDC D
    a. Pushover Analysis Required
    b. Capacity Design Required
    c. SDC D, Level of Detailing

Task 4

Recommend the following for SDC C & D.

1. Adopt AISC LRFD Specifications for design of single angle members and members with stitch welds.
2. Allow for three types of a bridge structural system as adopted in SCDOT Specifications.

    Type 1 – Design a ductile substructure with an essentially elastic superstructure.

    Type 2 – Design an essentially elastic substructure with a ductile superstructure.

Type 3 – Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.

3. Adopt a force reduction factor of 3 for design of normal end cross-frame.
4. Adopt NCHRP 12-49 for design of "Ductile End-Diaphragm" where a force reduction factor greater than 3 is desired.

Task 5

The following list highlights the main proposed liquefaction design requirements:

1. Liquefaction design requirements are applicable to SDC "D".
2. Liquefaction design requirements are dependent on the mean magnitude for the 5% PE in 50-year event and the normalized Standard Penetration Test (SPT) blow count [$(N_1)_{60}$].
3. If liquefaction occurs, then the bridge shall be designed and analyzed for the Liquefied and Non-Liquefied configurations.

Design requirements for lateral flow are still debatable and have not reached a stage of completion for inclusion in the Guidelines. Recommendations for foundation type are deemed appropriate at this stage to mitigate lateral flow hazard.

1.2 PROJECT ORGANIZATION

This NCHRP Project was organized to assist the "AASHTO T-3 Subcommittee for Seismic Design of Bridges" to complete another step towards producing an LRFD Seismic Design Specification for inclusion into the AASHTO Specifications. The T-3 Subcommittee defined very specific tasks as described in Article 1.1 above that they envisioned were needed to supplement the existing completed efforts (i.e., AASHTO Division I-A, NCHRP 12-49 Guidelines, SCDOT Specifications, Caltrans Seismic Design Criteria, NYDOT Seismic Intensity Maps and ATC-32) to yield an implementable specification for AASHTO. The tasks have now been completed by TRC/Imbsen & Associates, Inc. under the direction of the T-3 Subcommittee and the assistance of their Board of Reviewers to yield a stand-alone Guideline that can be evaluated by AASHTO and considered for adopting in 2007. This project was completed by Imbsen Consulting under a subcontract with TRC/Imbsen & Associates, Inc.

1.2.1 Project Direction from AASHTO T-3

The T-3 Working Group that defined the project objectives and directed the project include:

- Rick Land, CA (Past chair)
- Harry Capers, NJ (Current Co-chair)
- Richard Pratt, AK (Current chair)
- Ralph Anderson, IL
- Jerry Weigel, WA
- Ed Wasserman, TN
- Paul Liles, GA
- Kevin Thompson, CA

The project team members and reviewers that participated in the NCHRP 20-07/193 include:

- Roger Borcherdt, USGS
- Po Lam, Earth Mechanics, Inc.
- Ed V. Leyendecker, USGS
- Lee Marsh, Berger/Abam
- Randy Cannon, Site Blauvelt
- George Lee, MCEER, Chair
- Geoff Martin, MCEER
- Joe Penzien, HSRC, EQ V-team
- John Kulicki, HSRC
- Les Youd, BYU
- Joe Wang, Parsons, EQ V-team
- Lucero Mesa, SCDOT V-team
- Derrell Manceaux, FHWA
- Peter W. Osborn, FHWA
- Alexander K. Bardow, Mass. Highway
- Stephanie Brandenberger, Montana DOT
- Bruce Johnson, Oregon DOT
- Michael Keever, Calif. DOT
- Jerry O'Connor, MCEER
- Roland Nimis, FHWA
- W. Phil Yen, FHWA
- Firas Ibrhim, FHWA
- Shyam Gupta, MODOT
- Elmer E. Marx, Alaska DOT & PF

- William Crawford, Nevada DOT
- Jugesh Kapur, Washington State DOT
- John Jordan, Indiana DOT

1.2.2 Technical Assistance Agreement Between AASHTO and USGS

Under the agreement the USGS prepared two types of products for use by AASHTO. The first product was a set of paper maps of selected seismic design parameters for a 5% probability of exceedance in 50 years. The second product was a ground motion software tool to simplify determination of the seismic design parameters.

These guidelines use spectral response acceleration with a 5% probability of exceedance in 50 years as the basis of the seismic design requirements. As part of the National Earthquake Hazards Reduction Program, the U.S. Geological Survey's National Seismic Hazards Mapping Project prepares seismic hazard maps of different ground motion parameters with different probabilities of exceedance. However maps were not prepared for the probability level required for use by these guidelines. These maps were prepared by the U.S. Geological Survey under a separate Technical Assistance Agreement with the American Association of State Highway and Transportation Officials (AASHTO), Inc. for use by AASHTO and in particular the Highway Subcommittee on Bridges and Structures.

Maps

The set of paper maps covered the fifty states of the U.S. and Puerto Rico. Some regional maps were also included in order to improve resolution of contours. Maps of the conterminous 48 states were based on USGS data used to prepare maps for a 2002 update. Alaska was based on USGS data used to prepare a map for a 2006 update. Hawaii was based on USGS data used to prepare 1998 maps. Puerto Rico was based on USGS data used to prepare 2003 maps.

The maps included in the map package were prepared in consultation with the Subcommittee on Bridges and Structures. The package included a series of maps prepared for a short period (0.2 sec) value of spectral acceleration, S_S, and a longer period (1.0 sec) value of spectral acceleration S_1. The maps were for spectral accelerations for a reference Site Class B.

Ground Motion Tool

The ground motion software tool was packaged on a CD-ROM for installation on a PC using a Windows-based operating system. It includes features allowing the user to calculate Peak Ground Acceleration, (PGA) and the mapped spectral response accelerations as described below:

- PGA, S_S, and S_1 - Determination of the parameters PGA, S_S, and S_1 by latitude-longitude or zip code from the USGS gridded data. The peak ground acceleration, PGA,
- Design values of PGA, S_S, and S_1 – Modification of PGA, S_S, and S_1 by the site factors to obtain design values. These are calculated using the mapped parameters and the site coefficients for a specified site class.

In addition to calculation of the basic parameters, the CD allows the user to obtain the following additional information for a specified site:

- Calculation of a response spectrum – The user can calculate response spectra for spectral response accelerations and spectral displacements using design values of PGA, S_S, and S_1. In addition to the numerical data the tools include graphic displays of the data. Both graphics and data can be saved to files.
- Maps - The CD also include the 5% in 50 year maps in PDF format. A map viewer is included that allows the user to click on a map name from a list and display the map.

1.3 FLOW CHARTS

It is envisioned that the flow charts will provide the engineer with a simple reference to direct the design process needed for each of the four Seismic Design Categories (SDC).

Flow charts outlining the steps in the seismic design procedures implicit in these specifications are given in Figures 1.3A to 1.3G.

The flow chart in Figure 1.3A guides the designer on the applicability of the specifications and the breadth of the design procedure dealing with a single span bridge versus a multi-span bridge and a bridge in Seismic Design Category A versus a bridge in Seismic Design Category B, C, or D.

Figure 1.3B shows the core flow chart of procedures outlined for bridges in SDC B, C, and D. Figure 1.3D directs the designer to determine displacement capacity for SDC B or C using implicit procedures defined in Article 4.8. Since the displacement approach is the main thrust of this criteria, the flow chart in Figure 1.3C directs the designer to Figure 1.3E in order to establish the displacement demands on the subject bridge and Figure 1.3F and 1.3G in order to establish member requirements for SDC C or D based on the type of the structure chosen for seismic resistance.

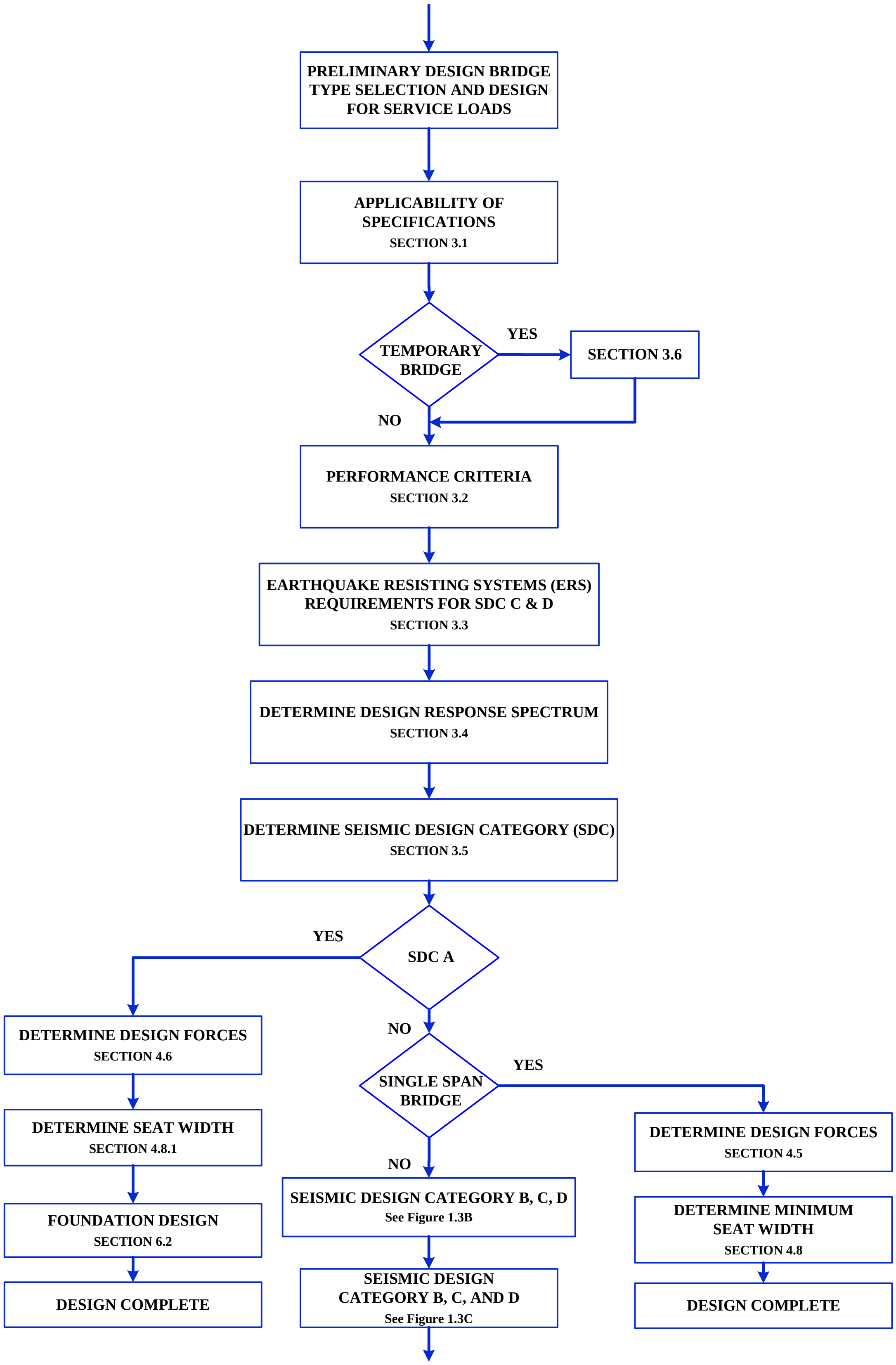

FIGURE 1.3A: Design Procedure Flow Chart A

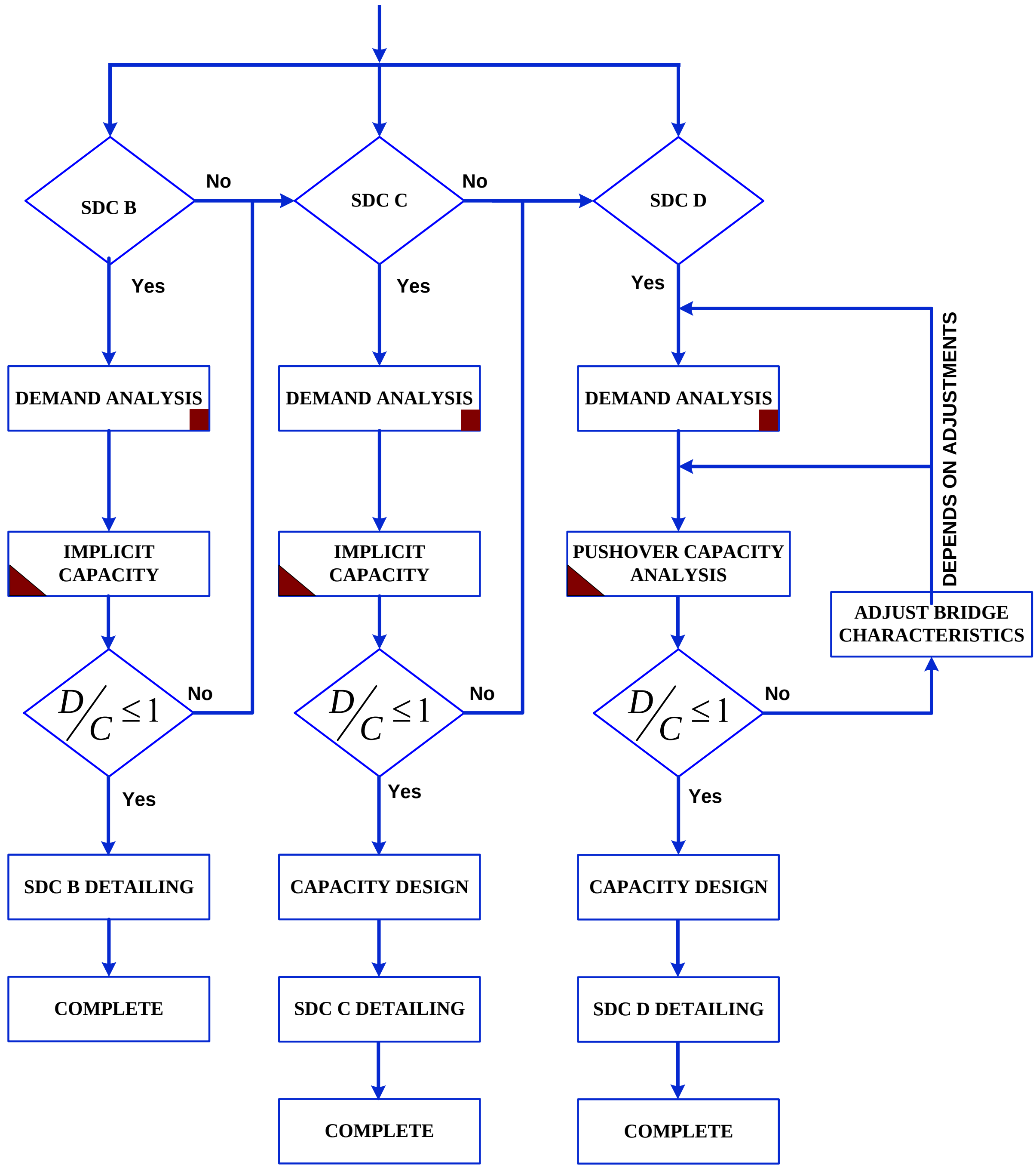

FIGURE 1.3B: Design Procedure Flow Chart B

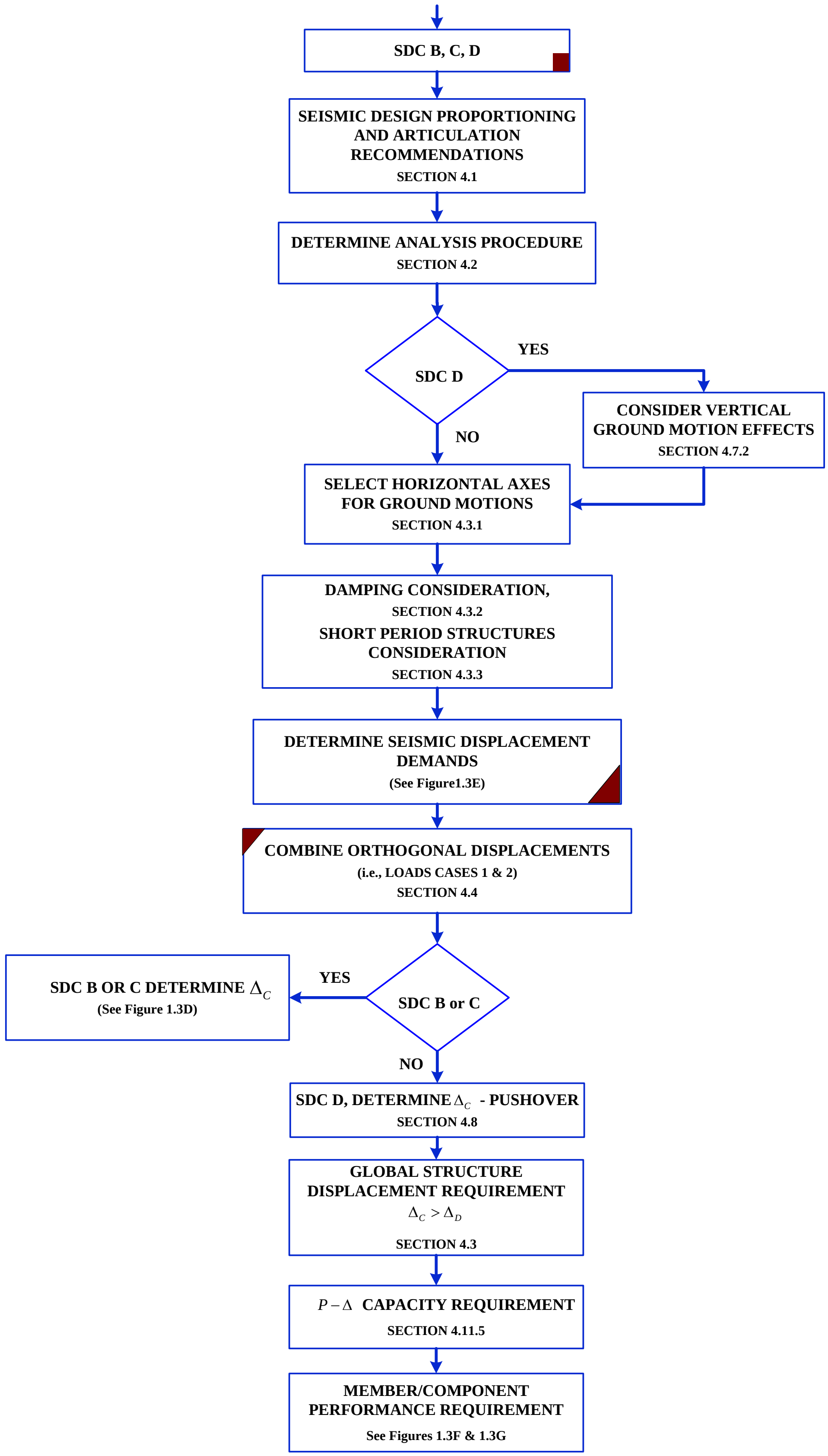

FIGURE 1.3C: Design Procedure Flow Chart C

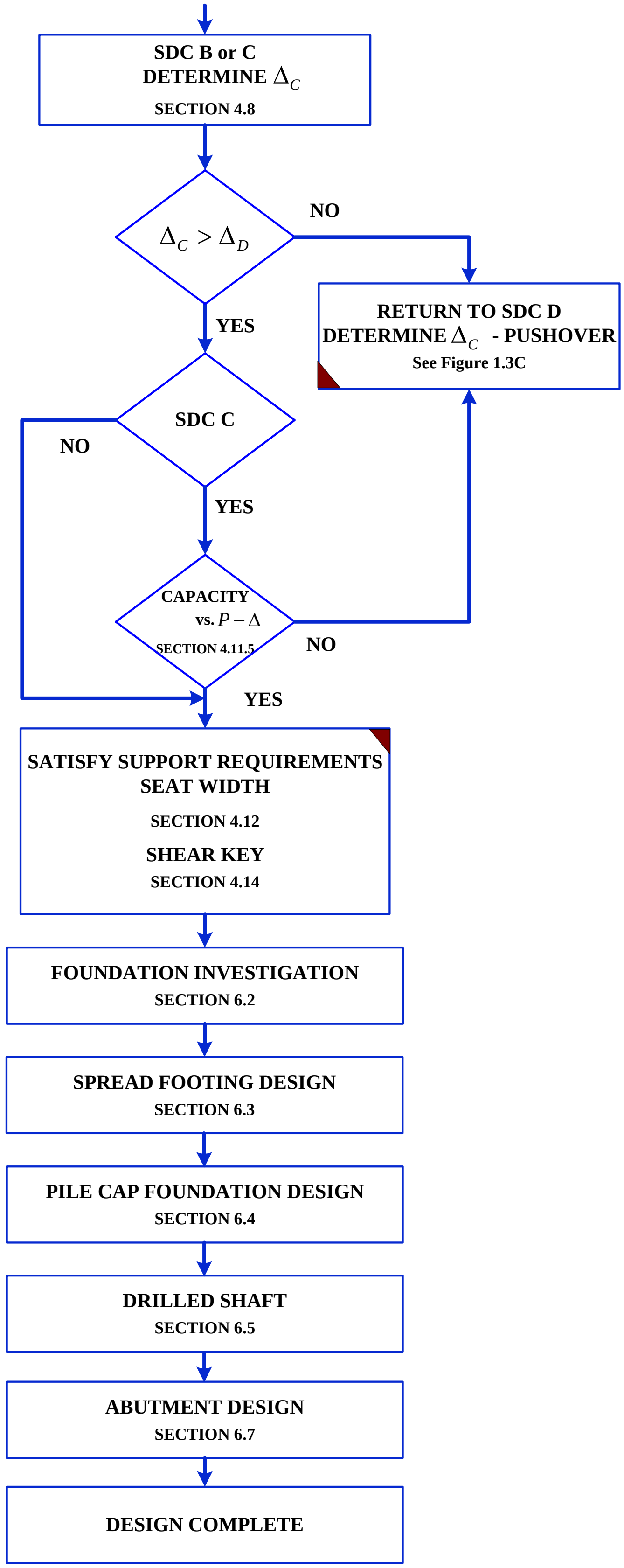

FIGURE 1.3D: Design Procedure Flow Chart D

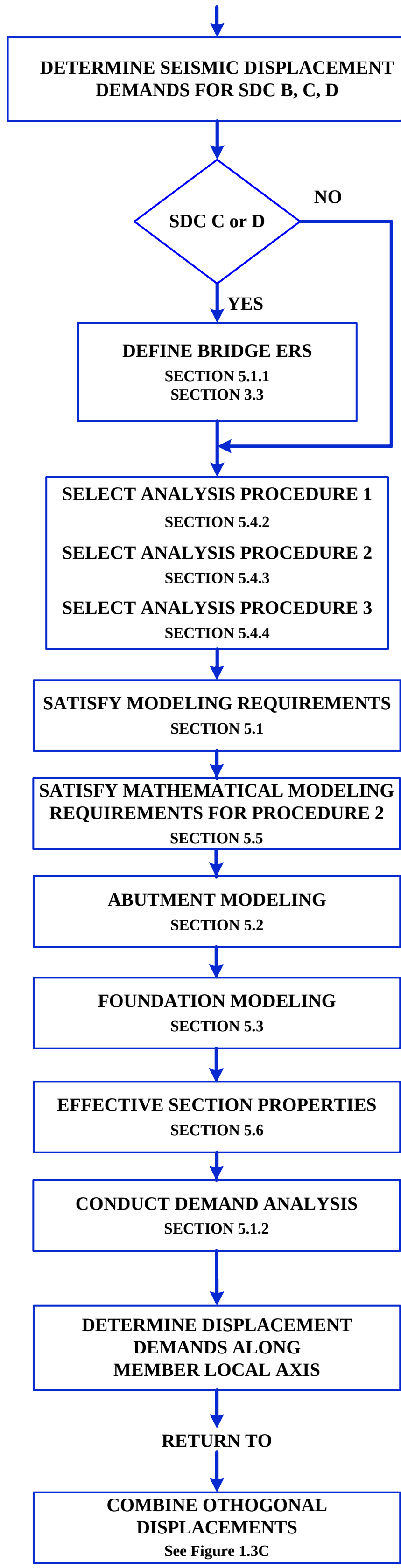

FIGURE 1.3E: Design Procedure Flow Chart E

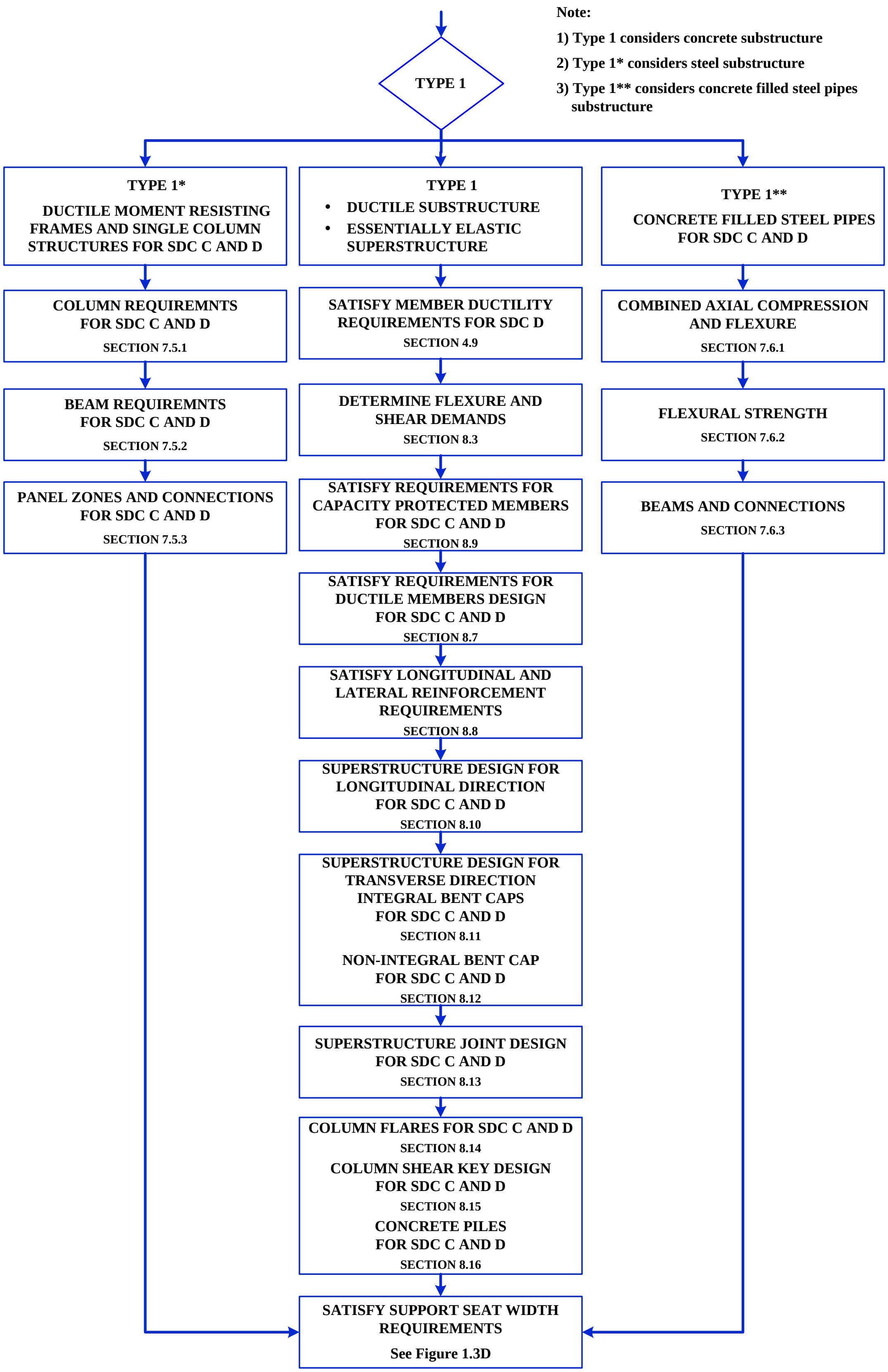

FIGURE 1.3F: Design Procedure Flow Chart F

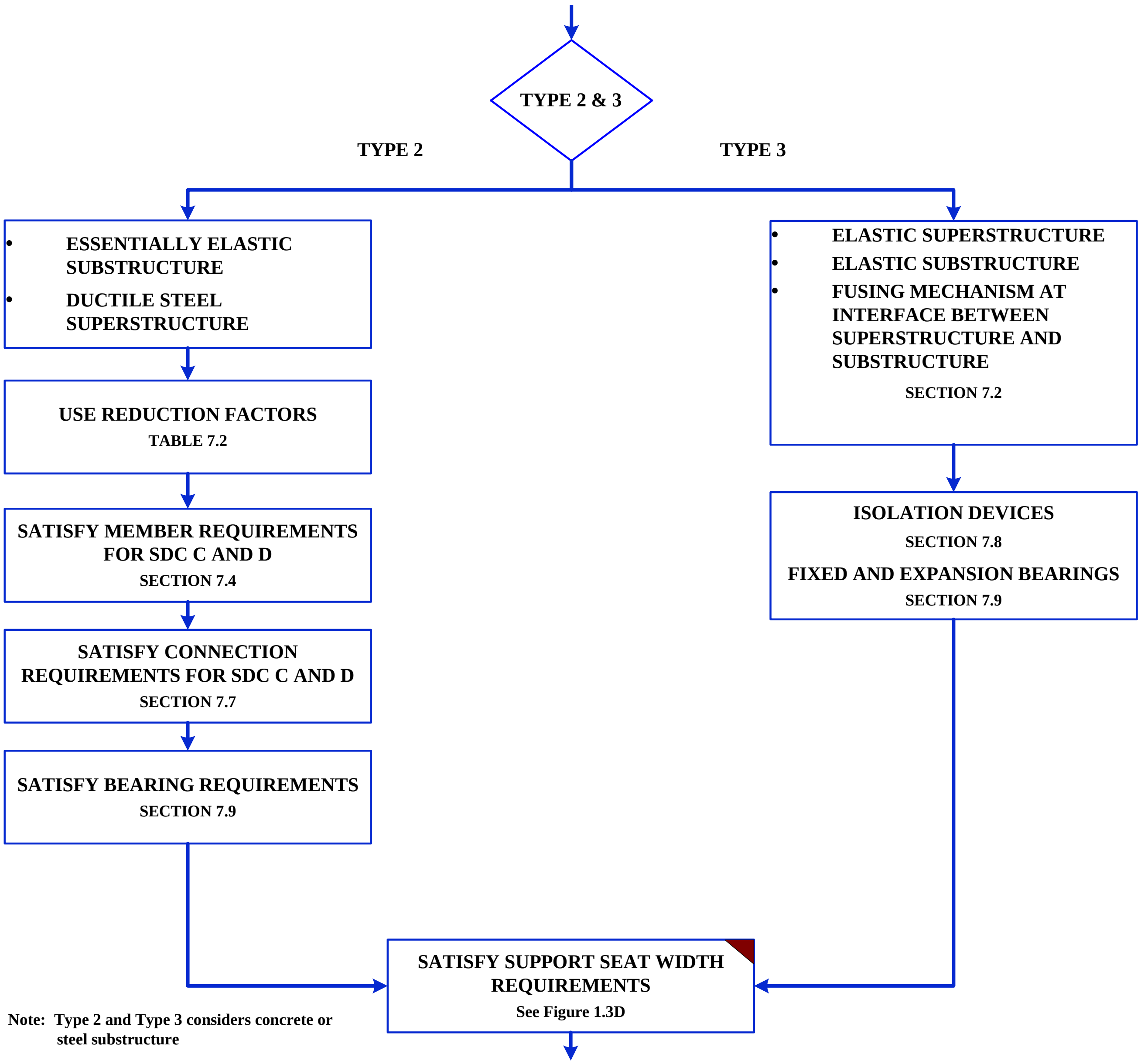

FIGURE 1.3G: Design Procedure Flow Chart G

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2. SYMBOLS AND DEFINITIONS

2.1 NOTATIONS

The following symbols and definitions apply to these Standards:

A = Cross-section area of a steel member

A_c = Area of reinforced concrete column core (in^2)

A_{cap}^{bot} = Area of bottom reinforcement in the bent cap (in^2)

A_{cap}^{top} = Area of top reinforcement in the bent cap (in^2)

A_e = Effective shear area (in^2)

A_{ew} = Cross-sectional area of pier wall

A_g = Gross area of reinforced concrete column (in^2)

A_{gg} = Gross area of gusset plate (in^2)

A_{jh} = The effective horizontal area of a moment resisting joint (in^2)

A_{jh}^{ftg} = The effective horizontal area at mid-depth of the footing, assuming a 45 degree spread away from the boundary of the column in all directions (in^2)

A_n = Net area of a gusset plate (in^2)

A_s^{jv} = Area of vertical stirrups required for joint reinforcement (in^2)

A_s^{jh} = Area of horizontal stirrups required for joint reinforcement (in^2)

A_s^{j-bar} = Area of J-dowels reinforcement required for joint reinforcement (in^2)

A_s^{sf} = Area of longitudinal side face reinforcement in the bent cap (in^2)

A_{sp} = Cross-Sectional area of a hoop or spiral bar (in^2)

A_{st} = Total area of column reinforcement anchored in the joint (in^2)

A_{tg} = Gross area along the plane resisting tension in a gusset plate (in^2)

A_{tn} = Net area along the plane resisting tension in a gusset plate (in^2)

A_v = Cross-Sectional area of web reinforcement

A_{vg} = Gross area along the plane resisting shear in a gusset plate (in^2)

A_{vn} = Net area along the plane resisting shear in a gusset plate (in^2)

B_c = Width of a rectangular column (in)

B_{cap} = Width of a bent cap (in)

B_{eff} = Effective width of a bent cap (in)

B_{eff}^{ftg} = Effective width of a footing (in)

B_o = Column gross width or diameter in the direction of bending (ft)

B_r = Footing width orthogonal to direction of rocking

$C_{(i)^{pile}}$ = Compression force in pile (i) (kips)

D' = Core diameter of a column (in)

D/C = Displacement Demand to Capacity Ratio

D/t = Diameter to thickness ratio of a tubular member

D^* = Diameter for circular shafts or the cross section dimension in direction being considered for oblong shafts (in)

D_c = Column diameter or depth

D_{cj} = Column width or diameter parallel to direction of bending

$D_{c,\max}$ = Largest cross-sectional dimension of the column (in)

D_{eff} = Effective yield displacement of soil behind the abutment backwall

D_{ftg} = Footing depth (in)

D_g = Abutment gap width

D_s = Superstructure depth (in)

E = Structural Steel Elastic Modulus

E_c = Concrete Elastic Modulus

E_cI_{eff} = Effective flexural rigidity (kips-in^2)

E_s = Steel elastic modulus (ksi)

F = Applied force at the superstructure level for a rocking column/footing system

F_a = Site coefficient defined in Table 3.3.3A based on the site class and the values of the response acceleration-parameter S_S

F_u = Specified minimum tensile strength of structural steel (ksi)

F_v = Site coefficient defined in Table 3.3.3.B based on the site class and the values of the response acceleration

F_y = Specified minimum yield strength of structural steel (ksi)

F_{ye} = Expected yield strength of structural steel

G = Soil shear modulus

$(GA)_{eff}$ = Shear stiffness parameter

G_c = Concrete shear modulus

$G_c J$ = Torsional rigidity

G_f = Gap between the isolated flare and the soffit of the bent cap (in)

$G_{\max}$ = Maximum soil shear modulus

H = Thickness of soil layer (ft)

H_h = Largest column height within the most flexible frame adjacent to the expansion joint, height from top of footing to top of the column (i.e., column clear height, ft.) or equivalent column height for pile extension column (ft.). For single spans seated on abutments, the term H is taken as the abutment height (ft.).

H_o = Height from top of footing to top of the column (i.e., column clear height, ft.).

H_r = Height of column/footing system used for rocking analysis

H_w = Wall height (ft)

H' = Length of pile shaft/column from point of maximum moment to point of contraflexure above ground (in)

I_{eff} = Effective flexural moment of inertia (in^4)

I_g = Gross flexural moment of inertia (in^4)

$I_{p.g.}$ = Moment of inertia of the pile group defined by Equation 6-3

J_{eff} = Effective torsional moment of inertia (in^4)

J_g = Gross torsional moment of inertia (in^4)

K = Effective length factor used in steel design and given in Article 7.4 (dimensionless)

K_i = Effective stiffness of abutment soil backwall corresponding to iteration (i)

KL/r = Slenderness ratio of a steel member (dimensionless)

K_r = Equivalent stiffness of a rocking system

L = The length of column from the point of maximum moments to the point of contra-flexure

L_c = Column clear height used to determine shear demand

L_F = Length of base of footing in the direction of rocking

L^{ftg} = The cantilever length of the pile cap measured from the face of the column to the edge of the footing (in)

L_g = Unsupported edge length of a gusset plate (in)

L_p = Analytical plastic hinge length (in)

L_{pr} = Plastic hinge region (in)

M = Flexural moment of a member due to seismic and permanent loads (kip-in)

M_g = Moment demand in a gusset plate (kip-in)

M_n = Nominal moment capacity of a member

M_{ne} = Nominal moment capacity of a reinforced concrete member based on expected materials properties (kip-in)

M_{ng} = Nominal moment strength of a gusset plate (kip-in)

M_{ns} = Nominal flexural moment strength of a steel member (kip-in)

M_o = Column over strength moment.

M_p = Idealized plastic moment capacity of a reinforced concrete member based on expected material properties (kip-in)

M_{po} = Overstrength plastic moment capacity (kip-in)

M_{pg} = Plastic moment of a gusset plate under pure bending (kip-in)

M_{px} = The column plastic moment under pure bending calculated using F_{ye}

M_r = Restoring moment of a rocking column/footing system

M_w = Mean Earthquake Moment Magnitude

M_y = Moment capacity of the section at first yield of the reinforcing steel

N = Minimum support length (in)

$\overline{N}$ = Average standard penetration resistance for the top 100 ft (blows/ft)

$\overline{N}_{ch}^{+}$ = Average standard penetration resistance of a cohesionless soil layer for the top 100 ft

N_i = Standard Penetration Resistance not to exceed 100 blows/ft as directly measured in the field

N_p = Total number of piles in the pile group

P = Axial load of a member due to seismic and permanent loads (kip)

P_{ac} = Axial force at top of the column including the effects of overturning (kips)

P_b = Horizontal effective axial force at the center of the joint including prestressing

P_{bs} = Tensile strength of a gusset plate based on block-shear (kip)

P_c = The total axial load on the pile group including column axial load (dead load +EQ load), footing weight, and overburden soil weight

P_{col} = Axial force including the effects of overturning at the base of the column (kip)

P_{dl} = Axial dead load at the bottom of the column (kip)

P_g = Axial load in a gusset plate (kip)

PI = Plasticity Index

P_n = Nominal axial strength of a member (kip)

P_{ng} = Nominal compressive or tensile strength of a gusset plate

P_p = Passive force behind backwall

P_u = Maximum strength of concentricity loaded steel columns (kips)

P_y = Yield axial strength of a member (kips)

P_{yg} = Yield axial strength in a gusset plate (kips)

R = Force reduction factor is obtained by dividing the elastic spectral force by the plastic yield capacity

R_D = Reduction factor to account for increased damping

R_d = Magnification factor to account for short period structure

R_y = Overstrength factor of Structural Steel

S_a = The design spectral response acceleration

S_1 = The mapped design spectral acceleration for the one second period as determined in Sections 3.4.2 and 3.4.3 (for Site Class B: Rock Site)

S_{D1} = Design spectral response acceleration parameter at one second

S_{DS} = Design short-period (0.2-second) spectral response acceleration parameter

S_k = Angle of skew of support in degrees, measured from a line normal to the span

S_S = The mapped design spectral acceleration for the short period (0.2 second) as determined in Sections 3.4.2 and 3.4.3 (for Site Class B: Rock Site)

S_{sm} = Elastic section modulus about strong axis for a gusset plate (in^2)

T = Fundamental period of the structure (second)

T_1 = Fundamental period from frame 1 (second)

T_2 = Fundamental period of frame 2 (second)

T_c = Column tensile force obtained from a section analysis corresponding to the overstrength column moment capacity (kips)

T_F = Fundamental Period of the subject bridge

T_i = Fundamental period of the less flexible frame (second)

$T_{(i)^{pile}}$ = The tensile axial demand in a pile (kip)

T_j = Fundamental period of the more flexible frame (second)

T_{jv} = Critical shear force in the column footing connection (kips)

T_o = Structure period defining the design response spectrum as shown in Figure 3.4.1 (second)

T_S = Structure period defining the design response spectrum as shown in Figure 3.4.1 (second)

T^* = Characteristic Ground Motion Period (second)

V_c = Concrete shear contribution (kip)

V_d = Shear demand for a column

V_g = Shear force in a gusset plate (kip)

V_n = Nominal shear capacity (kip)

V_{ng} = Nominal shear strength of a gusset plate (kip)

V_{nk} = Nominal shear capacity of a shear key

V_o = Column shear demand corresponding to column overstrength capacity

V_{ok} = Overstrength shear capacity of a shear key

V_{pg} = plastic shear capacity of gusset plate ($0.58A_{gg}F_y$) (kips)

V_{po} = Overstrength plastic shear demand (kip)

V_s = Transverse steel shear contribution (kip)

V_u = Maximum shear demand in a column or a pier wall

W_{COLUMN} = Column weight of a rocking column/footing system

W_{COVER} = Cover weight of a rocking column/footing system

$W_{FOOTING}$ = Footing weight of a rocking column/footing system

W_T = Total weight of a rocking column/footing system

W_s = Superstructure weight of a rocking column/footing system

b = Width of tied column

b_{eff} = Effective joint width for footing joint stress calculation

b/t = Width to thickness ratio for a stiffened or unstiffened element

c = Damping ratio (maximum of 10%)

$c_{x(i)}$ = Distance from column centerline to pile centerline along x-axis (in)

$c_{y(i)}$ = Distance from column centerline to pile centerline along y-axis (in)

d = Pier wall depth (in)

d_{bl} = Longitudinal reinforcement bar diameter (in)

d_i = Thickness of any layer between 0 and 100 ft depth (ft)

f_c' = Specified compressive strength of concrete (psi or MPa)

f_{cc}' = Compressive strength of confined concrete

f_{ce}' = Expected compressive strength of concrete

f_h = Horizontal effective compressive stress in a joint (ksi)

f_{ps} = Prestressing steel stress

f_{ue} = Expected tensile strength (ksi)

f_v = Vertical effective compressive stress in a joint (ksi)

f_y = Specified minimum yield strength of reinforcing steel (ksi)

f_{ye} = Expected yield strength of reinforcing steel (ksi)

f_{yh} = Yield strength of transverse reinforcement (ksi)

h/t_w = Web slenderness ratio

k_i^e = The smaller effective bent or column stiffness

k_j^e = The larger effective bent or column stiffness

l_{ac} = The anchorage length for longitudinal column bars (in)

m_i = Tributary mass of column or bent (i)

m_j = Tributary mass of column or bent (j)

n = The total number of piles at distance $c_{x(i)}$ or $c_{y(i)}$ from the centroid of the pile group

p_b = Ultimate compressive bearing pressure

p_c = Principal compressive stress (psi)

p_p = Passive pressure behind backwall

p_t = Principal tensile stress (psi)

r = Radius of gyration (in)

r_y = Radius of gyration about weak axis (in)

s = Spacing of transverse reinforcement in reinforced concrete columns (in)

$\bar{s}_u$ = Average undrained shear strength in the top 100 ft

s_{ul} = Undrained shear strength not to exceed 5000 psf ASTM D2166-91 or D2850-87 (psf)

t = Thickness of a gusset plate (in)

v_c = Concrete shear stress (psi)

v_{jv} = Vertical joint shear stress (ksi)

$\bar{v}_s$ = Average shear wave velocity (ft/sec)

v_{si} = Shear wave velocity of layer "i" (ft/sec)

w = Moisture content

ε_{cc} = Compressive strain for confined concrete corresponding to ultimate stress in concrete

ε_{co} = Compressive strain for unconfined concrete corresponding to ultimate stress in concrete

ε_{cu} = Ultimate compressive strain in confined concrete

ε_{ps} = Prestressing steel strain

$\varepsilon_{ps,EE}$ = Essentially Elastic prestress steel strain

ε_{psu} = Ultimate prestress steel strain

$\varepsilon^{R}_{ps,u}$ = Reduced ultimate strain of prestressing steel reinforcement

ε_{sh} = Onset of strain hardening of steel reinforcement

ε_{sp} = Ultimate unconfined compression spalling strain

ε_{su} = Ultimate strain of steel reinforcement

$\varepsilon^{R}{}_{su}$ = Reduced ultimate strain of steel reinforcement

ε_{ye} = Yield strain at expected yield stress of steel reinforcement

Δ = Total Displacement of a rocking column/footing system

Δ_b = Displacement Demand due to flexibility of essentially elastic components, i.e., bent caps

Δ_C = Corresponding displacement capacity obtained along the same axis as the displacement demand

Δ_{col} = The portion of global displacement attributed to the elastic displacement Δ_y and plastic displacement Δ_p of an equivalent member from the point of maximum moment to the point of contra-flexure

Δ_{cr+sh} = Displacement due to creep and shrinkage

Δ_D = Displacement along the local principal axes of a ductile member under the Design Earthquake applied to the structural system

Δ_{eq} = Seismic displacement demand of the long period frame on one side of the expansion joint (in.)

Δ_f = Displacement demand due to foundation flexibility; pile cap displacements

Δ_{fo} = Column flexural displacement of a rocking column/footing system

Δ_{ot} = Movement attributed to prestress shortening creep, shrinkage and thermal expansion or contraction to be considered no less than one inch per 100 feet of bridge superstructure length between expansion joints. (in.)

Δ_{pc} = Plastic displacement capacity

Δ_{pd} = Plastic displacement demand

$\Delta_{p/s}$ = Displacement due to prestress shortening

Δ_r = The relative lateral offset between the point of contra-flexure and the end of the plastic hinge.

Δ_{ro} = Rigid body rotation of a rocking column/footing system

Δ_S = The pile shaft displacement at the point of maximum moment

Δ_{temp} = Displacement due to temperature variation

Δ_y = Elastic displacement

Δ_{ycol} = Column yield displacement

Δ_μ = Pile cap displacement

Λ = Fixity factor for a column

β = Stability term of a rocking column/footing system

ϕ = Shear strength reduction factor (dimensionless)

ϕ_b = Resistance factor used for limiting width-thickness ratios

ϕ_{bs} = 0.8 for block shear failure

ϕ_{tf} = 0.8 for fracture in net section

ϕ_u = Ultimate curvature

ϕ_y = Yield Curvature

μ = Ductility parameter of a rocking column/footing system

ρ_{fs} = Transverse reinforcement ratio in a column flare

ρ_h = The ratio of horizontal shear reinforcement area to gross concrete area of vertical section in pier wall

ρ_n = The ratio of vertical shear reinforcement area to gross concrete area of horizontal section pier walls

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

ρ_w = Web reinforcement ratio in the direction of bending

λ_b = Slenderness parameter of flexural moment dominant members

λ_{bp} = Limiting slenderness parameter for flexural moment dominant members

λ_c = Slenderness parameter of axial load dominant members

λ_{cp} = Limiting slenderness parameter for axial load dominant members

λ_{mo} = Moment overstrength factor

λ_p = Limiting width-thickness ratio for ductile component

λ_r = Limiting width-thickness ratio for essentially elastic component

μ_D = Local member maximum ductility demand under the Design Earthquake

ξ = damping ratio (maximum of 0.1)

2.2 DEFINITIONS

Capacity Design – A method of component design that allows the designer to prevent damage in certain components by making them strong enough to resist loads that are generated when adjacent components reach their overstrength capacity.

Capacity Protected Element – Part of the structure that is either connected to a critical element or within its load path and that is prevented from yielding by virtue of having the critical member limit the maximum force that can be transmitted to the capacity protected element.

Collateral Seismic Hazard – Seismic hazards other than direct ground shaking such as liquefaction, fault rupture, etc.

Complete Quadratic Combination (CQC) – A statistical rule for combining modal responses from an earthquake load applied in a single direction to obtain the maximum response due to this earthquake load.

Critical or Ductile Elements – Parts of the structure that are expected to absorb energy, undergo significant inelastic deformations while maintaining their strength and stability.

Damage Level – A measure of seismic performance based on the amount of damage expected after one of the design earthquakes.

Displacement Capacity Verification – Seismic Design and Analysis Procedure (SDAP) E – A design and analysis procedure that requires the designer to verify that his or her structure has sufficient displacement capacity. It generally involves a non-linear static (i.e. "pushover") analysis.

Ductile Substructure Elements – See Critical or Ductile Elements

Earthquake Resisting Element (ERE) – The individual components, such as columns, connections, bearings, joints, foundation, and abutments, that together constitute the Earthquake Resisting System (ERS).

Earthquake Resisting System (ERS) – A system that provides a reliable and uninterrupted load path for transmitting seismically induced forces into the ground and sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements.

Life Safety Performance Level – The minimum acceptable level of seismic performance allowed by this specification. It is intended to protect human life during and following a rare earthquake.

Liquefaction – Seismically induced loss of shear strength in loose, cohesionless soil that results from a build up of pore water pressure as the soil tries to consolidate when exposed to seismic vibrations.

Liquefaction-Induced Lateral Flow. – Lateral displacement of relatively flat slopes that occurs under the combination of gravity load and excess porewater pressure (without inertial loading from earthquake). Lateral flow often occurs after the cessation of earthquake loading.

Liquefaction-Induced Lateral Spreading – Incremental displacement of a slope that occurs from the combined effects of pore water pressure buildup, inertial loads from the earthquake, and gravity loads.

Maximum Considered Earthquake (MCE) – The upper level, or rare, design earthquake having ground motions with a 3% chance of being exceeded in 75 years. In areas near highly-active faults, the MCE ground motions are deterministically bounded to ground motions that are lower than those having a 3% chance of being exceeded in 75 years.

Minimum Seat Width – The minimum prescribed width of a bearing seat that must be provided in a new bridge designed according to these specifications.

Nominal resistance – Resistance of a member, connection or structure based on the expected yield strength (F_{ye}) or other specified material properties, and the nominal dimensions and details of the final section(s) chosen, calculated with all material resistance factors taken as 1.0.

Operational Performance Level – A higher level of seismic performance that may be selected by a bridge owner who wishes to have immediate service and minimal damage following a rare earthquake.

Overstrength Capacity – The maximum expected force or moment that can be developed in a yielding structural element assuming overstrength material properties and large strains and associated stresses.

Performance Criteria – The levels of performance in terms of post earthquake service and damage that are expected to result from specified earthquake loadings if bridges are designed according to this specification.

Plastic Hinge – The region of a structural component, usually a column or a pier in bridge structures, that undergoes flexural yielding and plastic rotation while still retaining sufficient flexural strength.

Pushover Analysis – See Displacement Capacity Verification

Plastic Hinge Zone – Those regions of structural components that are subject to potential plastification and thus must be detailed accordingly.

Response Modification Factor (R-Factor) – Factors used to modify the element demands from an elastic analysis to account for ductile behavior and obtain design demands.

Seismic Hazard Level – One of four levels of seismic ground shaking exposure measured in terms of the rare earthquake design spectral accelerations for 0.2 and 1.0 second.

Service Level – A measure of seismic performance based on the expected level of service that the bridge is capable of providing after one of the design earthquakes.

Site Class – One of six classifications used to characterize the effect of the soil conditions at a site on ground motion.

Square Root of the Sum of the Squares (SRSS) Combination – In this specification, this vertical combination rule is used for combining forces resulting from two or three orthogonal ground motion components.

Tributary Weight – The portion of the weight of the superstructure that would act on a pier participating in the ERS if the superstructure between participating piers consisted of simply supported spans. A portion of the weight of the pier itself may also be included in the tributary weight.

3. GENERAL REQUIREMENTS

3.1 APPLICABILITY OF SPECIFICATIONS

These Specifications are for the design and construction of new bridges to resist the effects of earthquake motions. The provisions apply to bridges of conventional slab, beam, girder and box girder superstructure construction with spans not exceeding 500 ft. For other types of construction (e.g., suspension bridges, cable-stayed bridges, truss bridges, arch type and movable bridges) and spans exceeding 500 ft, the Owner shall specify and/or approve appropriate provisions.

Seismic effects for box culverts and buried structures need not be considered, except when they are subject to unstable ground conditions (e.g., liquefaction, landslides, and fault displacements) or large ground deformations (e.g., in very soft ground).

The provisions specified in the specifications are minimum requirements. Additional provisions are needed to achieve higher performance criteria for repairable or minimum damage attributed to essential or critical bridges. Those provisions are site/project specific and are tailored to a particular structure type.

No detailed seismic analysis is required for a single span bridge or for any bridge in Seismic Design Category A. Specific Detailing Requirements are applied for SDC A. For single simple span bridge, minimum seat width requirement shall apply according to Article 4.12.

3.2 PERFORMANCE CRITERIA

Bridges shall be designed for the life safety performance objective considering a seismic hazard corresponding to a 5% probability of exceedance in 50 years. Higher levels of performance, such as the operational objective, may be used with the authorization of the bridge owner. Development of design earthquake ground motions for the 5% probability of exceedance in 50 years are given in Article 3.4.

Life Safety for the Design Event infers that the bridge has a low probability of collapse but, may suffer significant damage and significant disruption to service. Partial or complete replacement may be required.

C3.1 APPLICABILITY OF SPECIFICATIONS

Commentary to be added.

C3.2 PERFORMANCE CRITERIA

The design earthquake ground motions specified herein are based on a probability of exceedance of 5% in 50 years for a nominal life expectancy of a bridge. As a minimum, these specifications are intended to achieve minimal damage to the bridge during moderate earthquake ground motions and to prevent collapse during rare, high-amplitude earthquake. Bridge owners may choose to mandate higher levels of bridge performance for a special bridge.

Allowable displacements are constrained by geometric, structural and geotechnical considerations. The most restrictive of these constraints will govern displacement capacity. These displacement constraints may apply to either transient displacements as would occur during ground shaking, or permanent displacements as may occur due to seismically induced ground failure or permanent structural deformations or dislocations, or a combination. The extent of allowable displacements depends on the desired performance level of the bridge design.

Allowable displacements shown in Table C3.2-1 were developed at a Geotechnical Performance Criteria Workshop conducted by MCEER on September 10 and 11, 1999 in support of the NCHRP 12-49 project.

Geometric constraints generally relate to the usability of the bridge by traffic passing on or under it. Therefore, this constraint will usually apply to permanent displacements that occur as a result of the earthquake. The ability to repair such displacements or the desire not to be required to repair them should be considered when establishing displacement capacities. When uninterrupted or immediate service is desired, the permanent displacements should be small or non-existent, and should be at levels that are within an accepted tolerance for normally operational highways of the type being considered. A bridge

Significant Damage Level includes permanent offsets and damage consisting of cracking, reinforcement yielding, major spalling of concrete and extensive yielding and local buckling of steel columns, global and local buckling of steel braces, and cracking in the bridge deck slab at shear studs. These conditions may require closure to repair the damages. Partial or complete replacement of columns may be required in some cases. For sites with lateral flow due to liquefaction, significant inelastic deformation is permitted in the piles. Partial or complete replacement of the columns and piles may be necessary if significant lateral flow occurs. If replacement of columns or other components is to be avoided, the design strategy producing minimal or moderate damage such as seismic isolation or the control and repairability design concept should be assessed.

Significant Disruption to Service Level includes limited access (reduced lanes, light emergency traffic) on the bridge. Shoring may be required.

3.3 EARTHQUAKE RESISTING SYSTEMS (ERS) REQUIREMENTS FOR SDC C & D

For SDC C or D (see Article 3.5), all bridges and their foundations shall have a clearly identifiable Earthquake Resisting System (ERS) selected to achieve the Life Safety Criteria defined in Section 3.2. The ERS shall provide a reliable and uninterrupted load path for transmitting seismically induced forces into the surrounding soil and sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements. All structural and foundation elements of the bridge shall be capable of achieving anticipated displacements consistent with the requirements of the chosen design strategy of seismic resistance and other structural requirements.

For the purposes of encouraging the use of appropriate systems and of ensuring due consideration of performance for the owner, the ERS and earthquake resisting elements (ERE) are categorized as follows:

- Permissible
- Permissible with Owner's Approval
- Not Recommended for New Bridges

These terms apply to both systems and elements. For a system to be in the permissible category, its primary EREs must all be in the permissible category. If any ERE is not permissible, then the entire system is not permissible.

designed to a performance level of no collapse could be expected to be unusable after liquefaction, for example, and geometric constraints would have no influence. However, because life safety is at the heart of the no collapse requirement, jurisdictions may consider establishing some geometric displacement limits for this performance level for important bridges or those with high average daily traffic (ADT). This can be done by considering the risk to highway users in the moments during or immediately following an earthquake. For example, an abrupt vertical dislocation of the highway of sufficient height could present an insurmountable barrier and thus result in a collision that could kill or injure. Usually these types of geometric displacement constraints will be less restrictive than those resulting from structural considerations and for bridges on liquefiable sites it may not be economic to prevent significant displacements from occurring.

C3.3 EARTHQUAKE RESISTING SYSTEMS

Bridges are seismically designed so that inelastic deformation (damage) intentionally occurs in columns in order that the damage can be readily inspected and repaired after an earthquake. Capacity design procedures are used to prevent damage from occurring in foundations and beams of bents and in the connections of columns to foundations and columns to the superstructure. There are two exceptions to this design philosophy. For pile bents and drilled shafts, some limited inelastic deformation is permitted below the ground level. The amount of permissible deformation is restricted to ensure that no long-term serviceability problems occur from the amount of cracking that is permitted in the concrete pile or shaft. The second exception is with lateral spreading associated with liquefaction. For the life-safety performance level, significant inelastic deformation is permitted in the piles. It is a costly and difficult problem to achieve a higher performance level from piles. There are a number of design approaches that can be used to achieve the performance objectives. These are given in Figure C3.3-1 and discussed briefly below.

Conventional Ductile Design - Caltrans first introduced this design approach in 1973 following the 1971 San Fernando earthquake. It was further refined and applied nationally in the 1983 AASHTO *Guide Specification for Seismic Design of Highway Bridges*, which was adopted directly from the ATC-6 report, *Seismic Design Guidelines for Highway Bridges* (ATC, 1981). These provisions were adopted by AASHTO in 1991 as their standard seismic provisions.

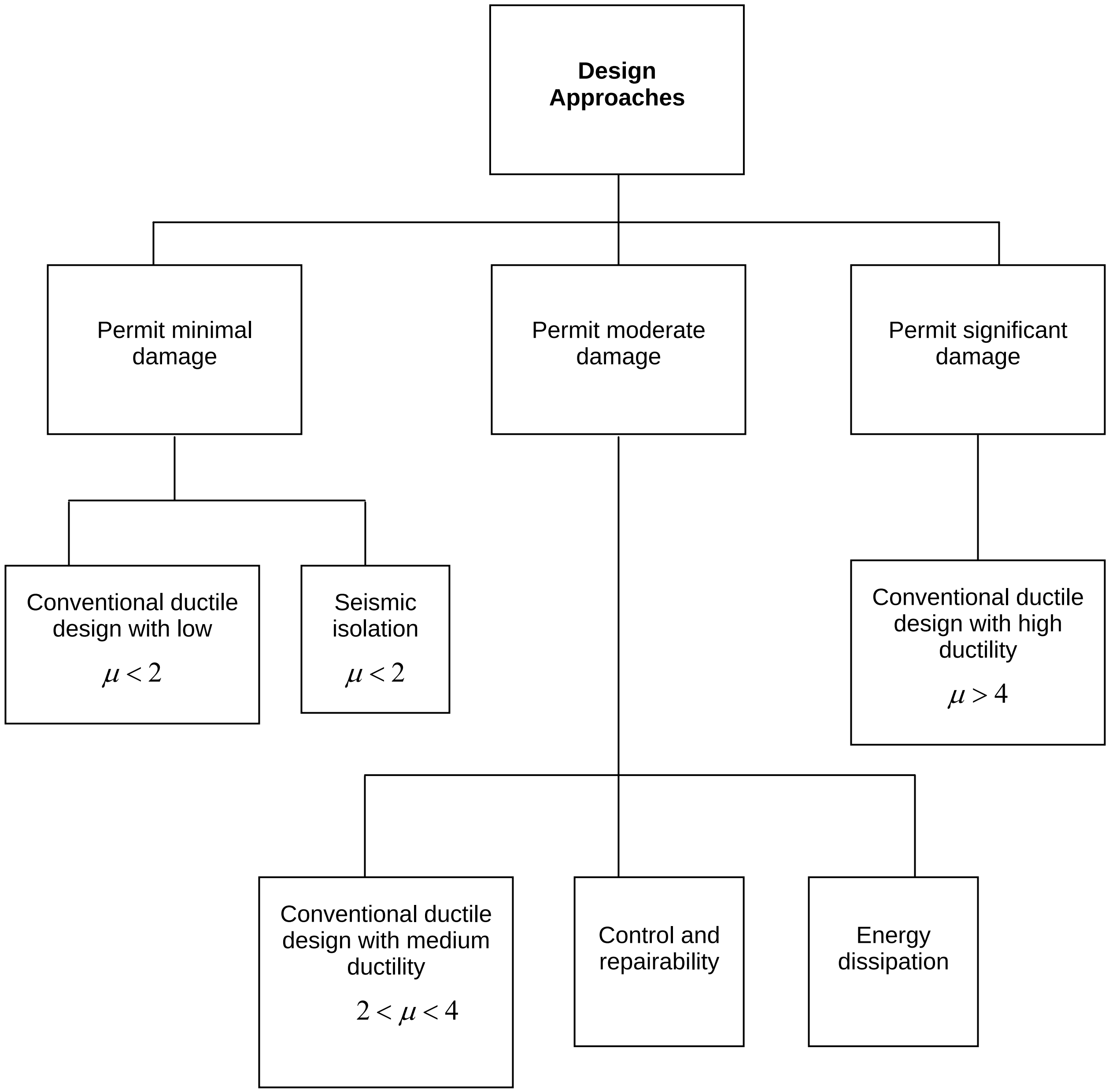


Figure C3.3-1 Design Approaches

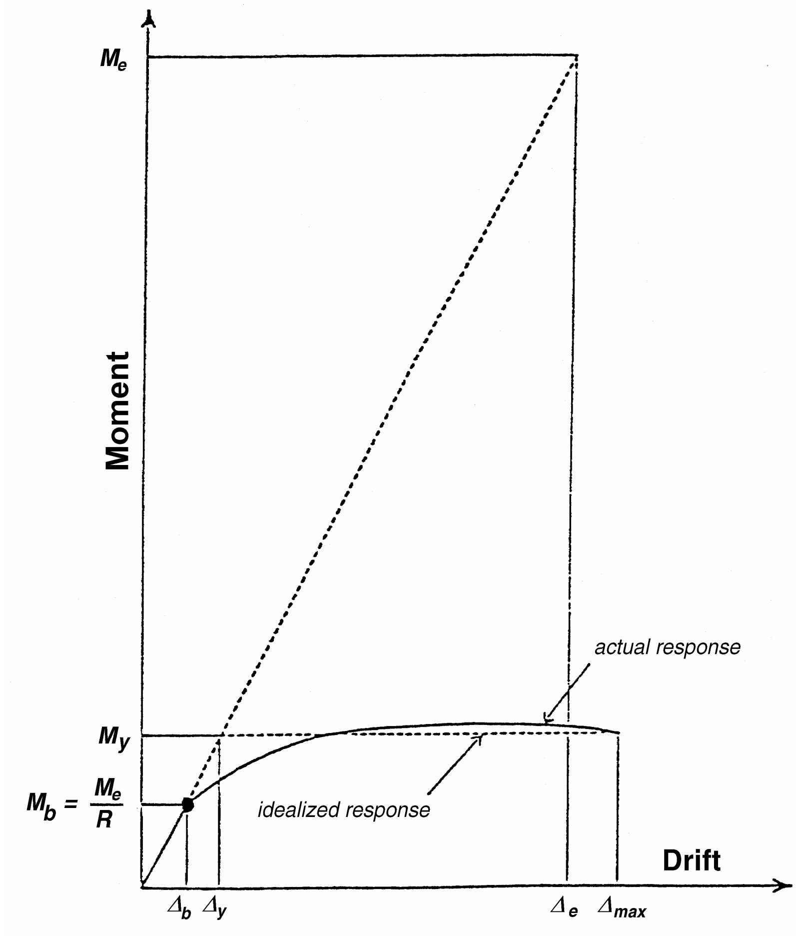
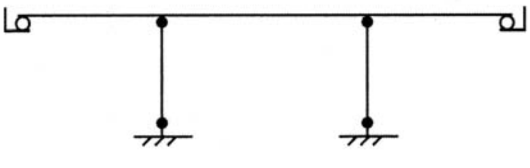


Figure C3.3-2 Basis for Conventional Ductile Design

Longitudinal Response

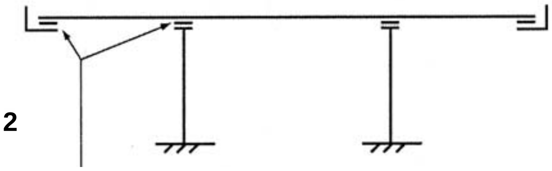
1



- Plastic hinges in inspectable locations or elastic design of columns.
- Abutment resistance not required as part of ERS
- Knock-off backwalls permissible

Longitudinal Response

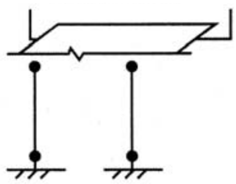
2



- Isolation bearings accommodate full displacement
- Abutment not required as part of ERS

Transverse Response

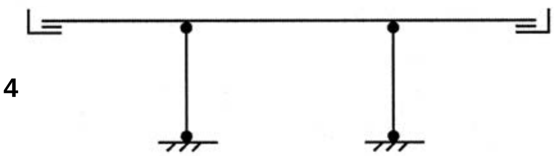
3



- Plastic hinges in inspectable locations or elastic design of columns
- Abutment not required in ERS, breakaway shear keys permissible

Transverse or Longitudinal Response

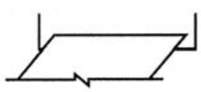
4



- Plastic hinges in inspectable locations or elastic design of columns
- Isolation bearings with or without energy dissipaters to limit overall displacements

Transverse or Longitudinal Response

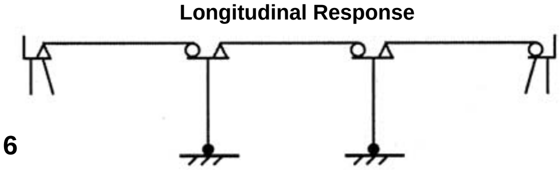
5



- Abutment required to resist the design earthquake elastically
- Longitudinal passive soil pressure must be less than 0.70 of the value obtained using the procedure given in Article 5.2.3

Longitudinal Response

6



- Multiple simply-supported spans with adequate seat widths
- Plastic hinges in inspectable locations or elastic design of columns

FIGURE 3.3.1a Permissible Earthquake Resisting Systems (ERS)

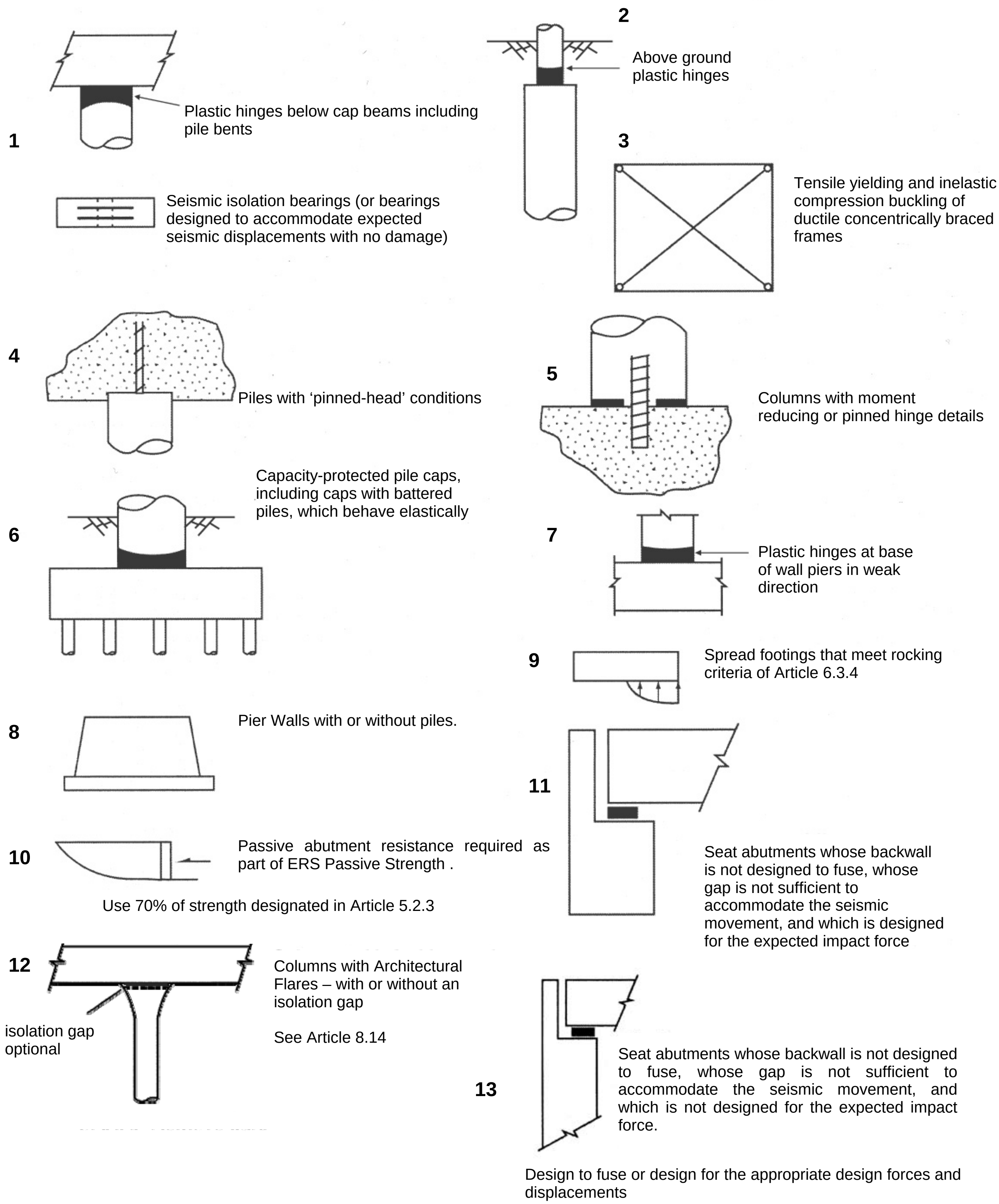


FIGURE 3.3.1b Permissible Earthquake Resisting Elements (ERE)

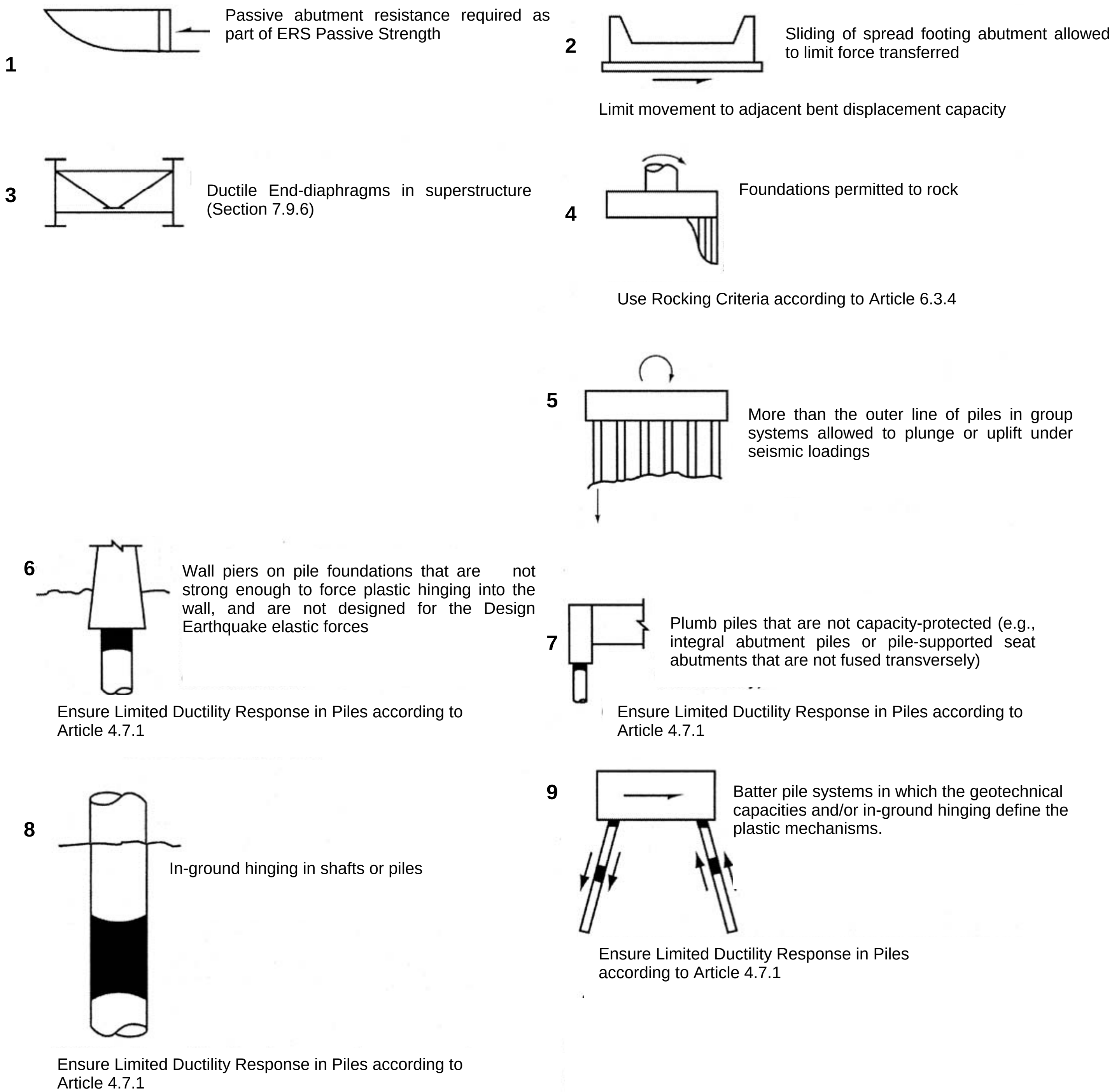


FIGURE 3.3.2 Permissible Earthquake Resisting Elements that Require Owner's Approval

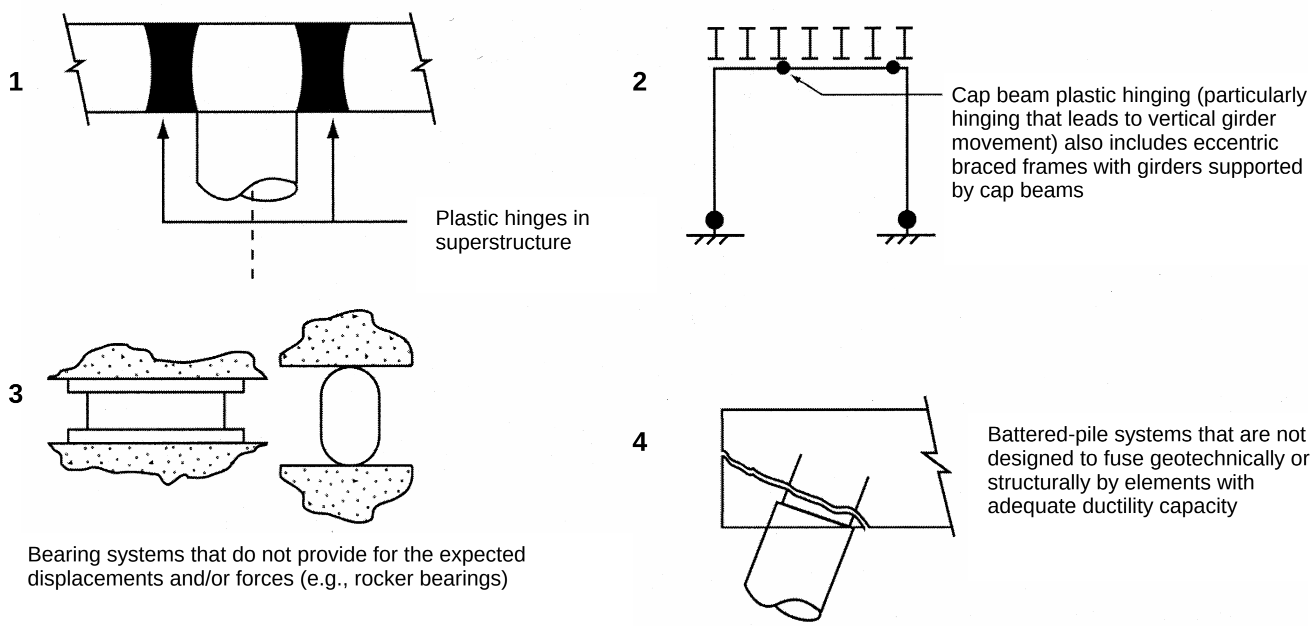


FIGURE 3.3.3 Earthquake Resisting Elements that are not Recommended for New Bridges

Permissible systems and elements (Figure 3.3.1a and 3.3.1b) have the following characteristics:

1. All significant inelastic action shall be ductile and occur in locations with adequate access for inspection and repair. Piles subjected to lateral movement from lateral flow resulting from liquefaction are permitted to hinge below the ground line provided the owner is informed and does not require any higher performance criteria for a specific objective. If all structural elements of a bridge are designed elastically then no inelastic deformation is anticipated and elastic elements are permissible, but minimum detailing is required according to the bridge Seismic Design Category (SDC).
2. Inelastic action of a structural member does not jeopardize the gravity load support capability of the structure (e.g. cap beam and superstructure hinging)

Permissible systems that require owner's approval (Figure 3.3.2) are those systems that do not meet either item (1) or (2), above.

In general, systems that do not fall in either of the two permissible categories (i.e., those in the not permitted category, see Figure 3.3.3) are not recommended. However, if adequate consideration is given to all potential modes of behavior and potential undesirable

This approach is based on the expectation of significant inelastic deformation (damage) associated with ductility equal or greater than 4.

The other key premise of the Part I *Specifications* is that displacements resulting from the inelastic response of a bridge are approximately equal to the displacements obtained from an analysis using the linear elastic response spectrum. As diagrammatically shown in Figure C3.3-2 this assumes that Δ_{max} (or $\Delta_{inelastic}$) is equal to Δ_e (or $\Delta_{elastic}$). Work by Miranda and Bertero (1994a, b) and by Chang and Mander (1994) indicates that this is a reasonable assumption except for short period structures for which it is non-conservative. A correction factor to be applied to elastic displacements to address this issue is given in Article 4.7. A more detailed discussion on the basis of the conventional design provisions can be found in the ATC-18 report (ATC, 1997).

Seismic Isolation. This design approach reduces the seismic forces a bridge must resist by introducing an isolation bearing with an energy dissipation element at the bearing location. The isolation bearing intentionally lengthens the period of a relatively stiff bridge and this results in lower design forces provided the design is in the decreasing portion of the acceleration response spectrum. This design alternative was first applied in the United States in 1984 and has been extensively reported on at technical conferences and seminars (e.g., the 1989, 1991 and 1993 ASCE Structures Congresses), and in failure mechanisms are suppressed, then such systems may be used with the owner's approval.

3.4 SEISMIC GROUND SHAKING HAZARD

The ground shaking hazard prescribed in these Specifications is defined in terms of acceleration response spectra and site coefficients. They shall be determined in accordance with the general procedure of Section 3.4.1 or the site-specific procedure of Section 3.4.3.

In the general procedure, the spectral response parameters are defined using the USGS/AASHTO seismic hazard maps produced by the U.S. Geological Survey depicting probabilistic ground motion and spectral response for 5% probability of exceedance in 50 years.

A site-specific procedure shall be used if any of the following apply:

- Soils at the site require site-specific evaluation (i.e., Site Class F soils, Article 3.4.2.1); unless a determination is made that the presence of such soils would not result in a significantly higher response of the bridge.
- The bridge is considered to be critical or essential according to Article 4.2.2 for which a higher degree of confidence of meeting the seismic performance objectives of Article 3.2 is desired.
- The site is located within 6 miles of a known active fault and its response could be significantly and adversely influenced by near-fault ground motion characteristics.

3.4.1 Design Spectra Based on General Procedure

Design response spectra shall be constructed using response spectral accelerations taken from national ground motion maps described in this section and site factors described in Article 3.4.2. The construction of the response spectra shall follow the procedures described below and illustrated in Figure 3.4.1-1.

the technical literature (e.g. ATC, 1986 and 1993; EERI, 1990). As of January 1, 1999 there were over 120 bridges constructed in the U.S. and over 300 worldwide using this concept. AASHTO adopted *Guide Specifications for Seismic Isolation Design of Highway* Bridges in 1991 and these were substantially revised in 1997. The 1997 and 2000 revisions are now incorporated in these provisions. Elastic response of the substructure elements is possible with seismic isolation, since the elastic forces resulting from seismic isolation are generally less than the reduced design forces required by conventional ductile design.

Energy Dissipation. This design approach adds energy-dissipation elements between the deck and the column, and between the deck and abutment, or in the end diaphragm of a steel girder bridge, with the intent of dissipating energy in these elements. This eliminates the need for the energy needing dissipation in the plastic hinge zones of columns. This design approach differs from seismic isolation in that additional flexibility is generally not part of the system and thus the fundamental period of vibration is not changed. If the equivalent viscous damping of the bridge is increased above 5% then the displacement of the deck will be reduced. In general the energy dissipation design concept does not result in reduced design forces but it will reduce the ductility demand on columns due to the reduction in deck displacement (ATC, 1993 and EERI, 1998). If the energy dissipation is in the end diaphragm of a steel girder bridge then the diaphragm acts as a force-limiting fuse in the transverse direction.

Control and Repairability Design. This design approach is based on the conventional ductile design concept that permits significant inelastic deformation in the plastic hinge zone of a column. The difference from conventional ductile design is that construction details in the hinge zone of reinforced concrete columns provide a replaceable or renewable sacrificial plastic-hinge element. Hinge zones are deliberately weakened with respect to their adjoining elements and all regions outside the hinge zone are detailed to remain elastic and undamaged during seismic loading. The concept has been extensively tested but has not been widely used in practice. Cheng and Mander (1997) provide the details for the implementation of this design concept.

Abutments as an Additional Energy-Dissipation Mechanism

1. In the early phases of the development of the *Specifications*, there was serious debate as to whether or not the abutments would be included and relied upon in the earthquake

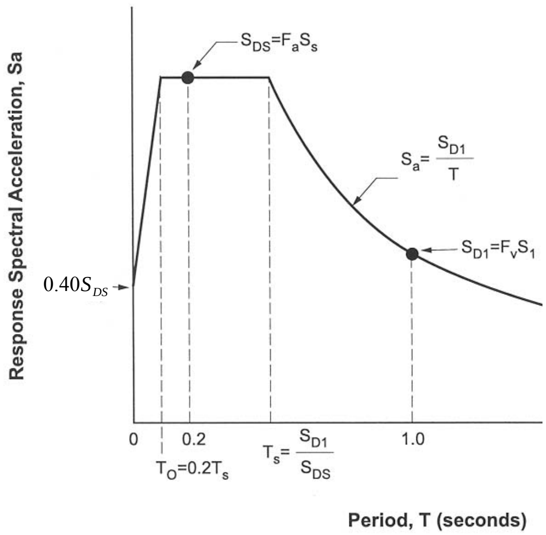


FIGURE 3.4.1-1 Design Response Spectrum, Construction Using Two-Point Method

Design earthquake response spectral acceleration at short periods, S_{DS}, and at 1 second period, S_{D1}, shall be determined from Equations 3.1 and 3.2, respectively:

$$S_{DS} = F_a S_s \tag{3.1}$$

and

$$S_{D1} = F_v S_1 \tag{3.2}$$

where S_s and S_1 are the 0.2-second period spectral acceleration and 1-second period spectral acceleration, respectively, on Class B rock from ground motion maps, shown in Figures 3.4.1-2a through 3.4.1-22, and F_a and F_v are site coefficients described in Article 3.4.2.3. Alternatively, values of S_s and S_1 may also be obtained using the CD-ROM *Bridge Design Ground Motion* published by the U.S. Geological Survey (USGS) for site coordinates specified by latitude and longitude or zip code.

The design response spectrum curve shall be developed as indicated in Figure 3.4.1-1 and as described on the next page:

resisting system (ERS). Some states may require the design of a bridge where the substructures are capable of resisting all the lateral load without any contribution from the abutments. In this design approach, the abutments are included in a mechanism to provide an unquantifiable higher level of safety. Rather than mandate this design philosophy here, it was decided to permit two design alternatives. The first is where the ERS does not include the abutments and the substructures are capable of resisting all the lateral loads. In the second alternative the abutments are an important part of the ERS and, in this case, a higher level of analysis is required (SDC D). Furthermore, this design option requires a continuous superstructure to deliver longitudinal forces to the abutment. If these conditions are satisfied, the abutments can be designed as part of the ERS and become an additional source for dissipating the bridge's earthquake energy. In the longitudinal direction the abutment may be designed to resist the forces elastically utilizing the passive pressure of the backfill. In some cases the longitudinal displacement of the deck will cause larger soil movements in the abutment backfill, exceeding the passive pressures there. This requires a more refined analysis to determine the amount of expected movement. In the transverse direction the abutment is generally designed to resist the loads elastically. The design objective when abutments are relied upon to resist either longitudinal or transverse loads is either to minimize column sizes or reduce the ductility demand on the columns, accepting that damage may occur in the abutment.

2. The performance expectation is that inelastic deformation will occur in the columns as well as the abutments. If large ductility demands occur in the columns then the columns may need to be replaced. If large movements of the superstructure occur the abutment back-wall may be damaged and there may be some settlement of the abutment backfill. Large movements of the superstructure can be reduced with use of energy dissipators and isolation bearings at the abutments and at the tops of the columns. Replacement of columns can be avoided with the use of the control and repairability design approach ductility with the use of the seismic isolation design

1. For periods less than or equal to T_0, the design response spectral acceleration, S_a, shall be defined by Equation 3.3:

$$S_a = 0.60 \frac{S_{DS}}{T_0} T + 0.40 S_{DS} \tag{3.3}$$

T and T_0 are defined below.

Note that for $T = 0$ seconds, the resulting value of S_a is equal to $0.4S_{DS}$.

2. For periods greater than or equal to T_0 and less than or equal to T_s, the design response spectral acceleration, S_a, shall be defined by Equation 3.4:

$$S_a = S_{DS} \tag{3.4}$$

where $T_0 = 0.2T_s$, and $T_s = S_{D1}/S_{DS}$, and T = period of vibration (sec).

3. For periods greater than T_s, the design response spectral acceleration, S_a, shall be defined by Equation 3.5:

$$S_a = \frac{S_{D1}}{T} \tag{3.5}$$

Response spectra constructed using maps and procedures described in Article 3.4.1 are for a damping ratio of 5%.

3.4.2 Site Effects on Ground Motions

The generalized site classes and site factors described in this section shall be used with the general procedure for constructing response spectra described in Article 3.4.1. Site-specific analysis of soil response effects shall be conducted where required by Article 3.4 and in accordance with the requirements in Article 3.4.3.Table 3.4.2-1: Site Classification

alternative to reduce the demand on the columns.

C3.3 Earthquake-Resisting Systems (ERS)

Selection of an appropriate ERS is fundamental to achieving adequate seismic performance. To this end, the identification of the lateral-force-resisting concept and the selection of the necessary elements to fulfill the concept should be accomplished in the conceptual design phase, or the type, size and location phase, or the design alternative phase of a project.

Seismic performance is typically better in systems with regular configurations and evenly distributed stiffness and strength. Thus, typical geometric configuration constraints, such as skew, unequal pier heights, and sharp curves, may conflict with seismic design goals. For this reason, it is advisable to resolve potential conflicts between configuration and seismic performance early in the design effort. For example, resolution may lead to decreased skew angles at the expense of longer end spans. The resulting trade-off between performance and cost should be evaluated in the type, size, and location phase, or design alternative phase, of a project, when design alternatives are viable from a practical viewpoint.

The classification of ERS and ERE into permissible and not recommended categories is meant to trigger due consideration of seismic performance that leads to the most desirable outcome, that is, seismic performance that ensures, wherever possible, post-earthquake serviceability. To achieve such an objective, special care in detailing the primary energy-dissipating elements is necessary. Conventional reinforced concrete construction with ductile plastic-hinge zones can continue to be used, but designers should be aware that such detailing, although providing desirable seismic performance, will leave the structure in a damaged state following a large earthquake. It may be difficult or impractical to repair such damage.

Under certain conditions the use of EREs that require owners' approval will be necessary. In previous AASHTO seismic specifications some of the EREs in the owners' approval category were simply not permitted for use (e.g., in-ground hinging of piles and shafts, and foundation rocking). These elements are now permitted, provided their deformation performance is assessed.

This approach of allowing their use with additional analytical effort was believed to be preferable to an outright ban on their use. Thus, it is not the objective of this specification to discourage

Table 3.4.2-1: Site Classification

Site Class	$\overline{v}_s$	$\overline{N}$ or $\overline{N}_{ch}^{+}$	$\overline{s}_u$ *
A	> 5000 ft/sec	_	_
B	2500 to 5000 ft/sec	_	_
C	1200 to 2500 ft/sec	> 50	> 2000 psf
D	600 to 1200 ft/sec	15 to 50	1000 to 2000 psf
E	<600 ft/sec	<15 blows/ft	<1000 psf

Table note: If the $\overline{s}_u$ method is used and the $\overline{N}_{ch}$ and $\overline{s}_u$ criteria differ, select the category with the softer soils (for example, use Site Class E instead of D).
* Average undrained shear strength in the top 100 ft. (Section 3.4.2.2)
\+ $\overline{N}$ Average standard penetration resistance for the top 100 ft.
$\overline{N}_{ch}$ Average standard and penetration resistance of cohesionless soil layers for the top 100 ft. (Section 3.4.2.2)

3.4.2.1 Site Class Definitions

The site shall be classified as one of the following classes (Table 3.4.2-1) according to the average shear wave velocity, Standard Penetration Test (SPT) blow count (N-value), or undrained shear strength in the upper 100 ft of site profile. Procedures given in Article 3.4.2.2 shall be used to determine the average condition for varying profile conditions. The Site Class shown in Table 3.4.2-1 are described in further detail below:

A. Hard rock with measured shear wave velocity, $\overline{v}_s > 5000$ ft/sec

B. Rock with 2500 ft/sec < $\overline{v}_s \leq 5000$ ft/sec

C. Very dense soil and soft rock with 1200 ft/sec < $\overline{v}_s \leq 2500$ ft/sec or with either $\overline{N} > 50$ blows/ft or $\overline{s}_u > 2000$ psf

D. Stiff soil with 600 ft/sec ≤ $\overline{v}_s \leq 1200$ ft/sec or with either 15 ≤ $\overline{N} \leq$ 50 blows/ft or 1000 psf ≤ $\overline{s}_u \leq 2000$ psf

E. A soil profile with $\overline{v}_s$ < 600 ft/sec or with either $\overline{N}$ < 15 blows/ft or $\overline{s}_u$ < 1000 psf, or any profile with more than 10 ft of soft clay defined as soil with $PI > 20$, the moisture content, $w \geq 40\%$, and $\overline{s}_u < 500$ psf

the use of systems that require owner approval. Instead, such systems may be used, but additional design effort and consensus between the designer and owner are required to implement such systems.

Common examples from each of the three ERS and ERE categories are shown in Figures 3.3.1a through 3.3.1b.

In general, the soil behind an abutment is capable of resisting substantial seismic forces that may be delivered through a continuous superstructure to the abutment. Furthermore, such soil may also substantially limit the overall movements that a bridge may experience. This is particularly so in the longitudinal direction of a straight bridge with little or no skew and with a continuous deck. The controversy with this design concept is the scenario of what may happen if there is significant abutment damage early in the earthquake ground-motion duration and if the columns rely on the abutment to resist some of the load. This would be a problem in a long-duration, high-magnitude (greater than magnitude 7), earthquake. Unless shock transmission units (STUs) are used, a bridge composed of multiple simply supported spans cannot effectively mobilize the abutments for resistance to longitudinal force. It is recommended that simply supported spans not rely on abutments for any seismic resistance.

Because structural redundancy is desirable (Buckle et al., 1987), good design practice dictates the use of the design alternative where the intermediate substructures, between the abutments, are designed to resist all seismic loads, if possible. This ensures that in the event abutment resistance

F. Soils requiring site-specific evaluations:
   a. Peats and/or highly organic clays (H > 10 ft of peat and/or highly organic clay where H = thickness of soil)
   b. Very high plasticity clays (H > 25 ft with PI > 75)
   c. Very thick soft/medium stiff clays (H > 120 ft

For preliminary design Site Classes E or F need not be assumed unless the authority having jurisdiction determines that Site Classes E or F could be present at the site or in the event that Site Classes E or F are established by geotechnical data.

The shear wave velocity for rock, Site Class B, shall be either measured on site or estimated on the basis of shear wave velocities in similar competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Site Class C.

The hard rock, Site Class A, category shall be supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 ft surficial shear wave velocity measurements may be extrapolated to assess $\overline{v}_s$.

The rock categories, Site Classes A and B, shall not be used if there is more than 10 ft of soil between the rock surface and the bottom of the spread footing or mat foundation.

3.4.2.2 Definitions of Site Class Parameters

The definitions presented below apply to the upper 100 ft of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there are a total of n distinct layers in the upper 100 ft. The subscript i then refers to any one of the layers between 1 and n.

becomes ineffective, the bridge will still be able to resist the earthquake forces and displacements. In such a situation, the abutments provide an increased margin against collapse. The same arguments can be made for allowing damage in locations that are very difficult to inspect. For instance, the first approach to a design using drilled shafts is to keep plastic hinging above the ground, and some states mandate this design concept. However, situations arise where this is impractical. In such situations, the ERS would require owner approval.

The flow chart in Figure X.X helps facilitate the decision-making process for assessing and accommodating restricted behavior.

C3.4 SEISMIC GROUND SHARING HAZARD

Using either the general procedure or the site-specific procedure, a decision as to whether the design motion is defined at the ground surface or some other depth needs to be made as an initial step in the design process. Article C3.4.2 provides a commentary on this issue.

Examples of conditions that could lead to a determination that Site Class F soils would not result in a significantly higher bridge response are (1) localized extent of Site Class F soils and (2) limited depth of these soft soils.

As discussed in Article C3.4.2.2, for short bridges (with a limited number of spans) having earth approach fills, ground motions at the abutments will generally determine the response of the bridge. If Site Class F soils are localized to the intermediate piers and are not present at the abutments, the bridge engineer and geotechnical engineer might conclude that the response of interior piers would not significantly affect bridge response.

Article C3.4.2.2 also describes cases where the effective depth of input ground motion is determined to be in stiffer soils at depth, below a soft surficial layer. If the surficial layer results in a classification of Site Class F and the underlying soil profile classifies as Site Class E or stiffer, a determination might be made that the surficial soils would not significantly increase bridge response.

For purposes of these provisions, an active fault is defined as a fault whose location is known or can reasonably be inferred, and which has exhibited evidence of displacement in Holocene (or recent) time (in the past 11,000 years, approximately). Active fault locations can be found from maps showing active faults prepared by state geological agencies or the U.S. Geological Survey. Article C3.4.3 describes near-fault ground-motion effects that are not included in national ground-motion mapping and could potentially increase the response of some bridges. Normally, site-specific evaluation of these effects would be considered only for essential or very critical bridges.

C3.4.1 Design Spectra Based on General Procedure

National ground-motion maps are based on probabilistic national ground motion mapping conducted by the U.S. Geological Survey (USGS) having a 5% chance of exceedance in 50 years.

In lieu of using national ground motion maps referenced in this *Guideline*, ground-motion response spectra may be constructed, based on approved state ground-motion maps. To be accepted, the development of state maps should conform to the following:

1. The definition of design ground motions should be the same as described in Article 3.2.
2. Ground-motion maps should be based on a detailed analysis demonstrated to lead to a quantification of ground motion, at a regional scale, that is as accurate or more so, as is achieved in the national maps. The analysis should include: characterization of seismic sources and ground motion that incorporates current scientific knowledge; incorporation of uncertainty in seismic source models, ground motion models, and parameter values used in the analysis; detailed documentation of map development; and detailed peer review as deemed appropriate by the Owner. The peer review process should preferably include one or more individuals from the U.S. Geological Survey who participated in the development of the national maps.

For periods exceeding approximately 3 seconds, depending on the seismic environment, Equation 3.5 may be conservative because the ground motions may be approaching the constant spectral displacement range for which S_a decays with period as $1/T^2$.

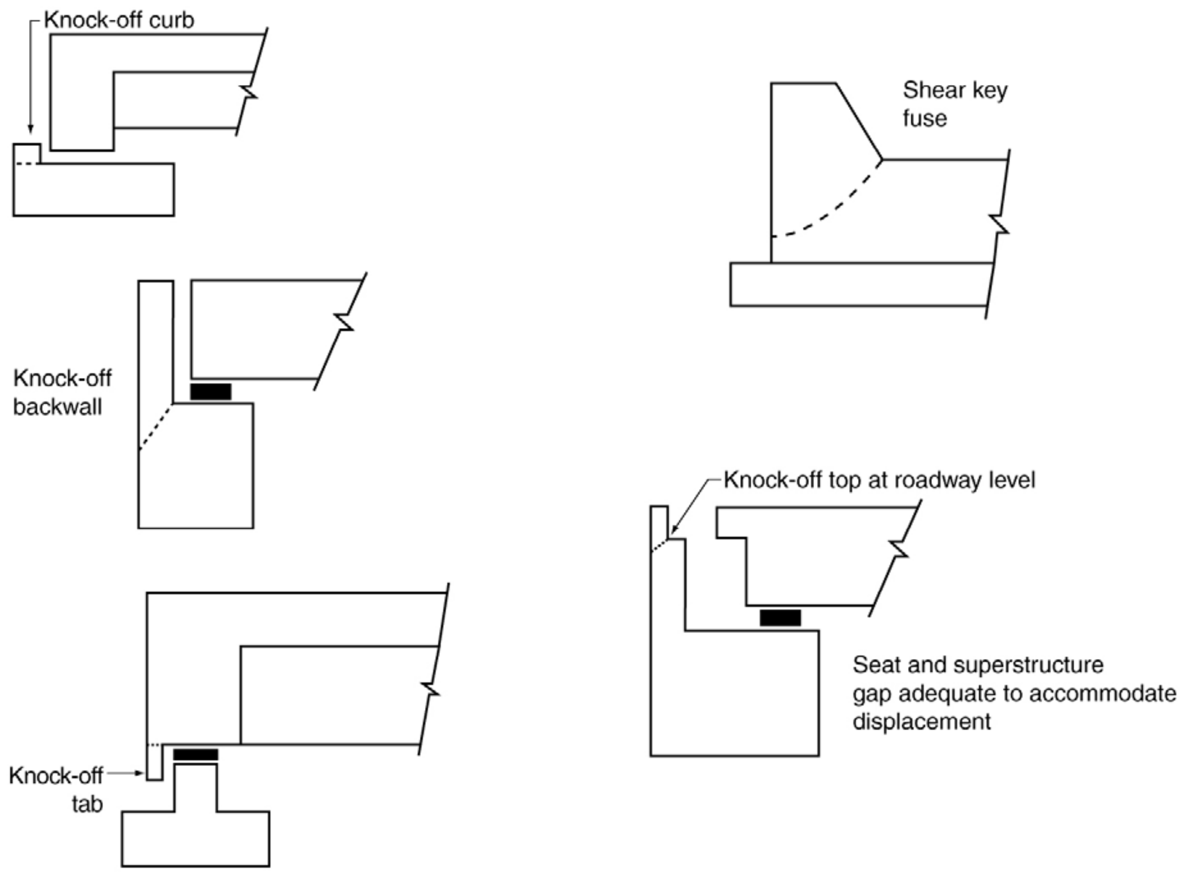


Figure C3.3.1-4 Methods of Minimizing Damage to Abutment Foundation

The average $\overline{v}_s$ for the layer is as follows:

$$\overline{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \tag{3.6}$$

where

$\sum_{i=1}^{n} d_i$ is equal to 100 ft, v_{si} is the shear wave velocity in ft/sec of the layer, and d_i is the thickness of any layer between 0 and 100 ft.

N_i is the Standard Penetration Resistance (ASTM D1586-84) not to exceed 100 blows/ft as directly measured in the field without corrections.

$\overline{N}$ is:

$$\overline{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \tag{3.7}$$

$\overline{N}_{ch}$ is

$$\overline{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \tag{3.8}$$

where

$\sum_{i=1}^{m} d_i = d_s$, and d_i and N_i are for cohesionless soils only, and d_s is the total thickness of cohesionless soil layers in the top 100 ft.

s_{ul} is the undrained shear strength in psf, not to exceed 5,000 psf, ASTM D2166-91 or D2850-87.

$\overline{s}_u$ is:

$$\overline{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ul}}} \tag{3.9}$$

C3.4.2 Site Effects on Ground Motions

The site classes and site factors described in this article were originally recommended at a site response workshop in 1992 (Martin, ed., 1994). Subsequently they were adopted in the seismic design criteria of Caltrans (1999), the 1994 and the 1997 edition of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (BSSC, 1995, 1998), the 1997 *Uniform Building Code* (UBC) and the 2000 *International Building Code* (ICC, 2000). The bases for the adopted site classes and site factors are described by Martin and Dobry (1994), Rinne (1994), and Dobry et al. (2000).

Procedures described in this article were originally developed for computing ground motions at the ground surface for relatively uniform site conditions. Depending on the site classification and the level of the ground motion, the motion at the surface could be different from the motion at depth. This creates some question as to the location of the motion to use in the bridge design. It is also possible that the soil conditions at the two abutments are different or they differ at the abutments and interior piers. An example would be where one abutment is on firm ground or rock and the other is on a loose fill. These variations are not always easily handled by simplified procedures described in this commentary. For critical bridges it may be necessary to use more rigorous numerical modeling to represent these conditions. The decision to use more rigorous numerical modeling should be made after detailed discussion of the benefits and limitations of more rigorous modeling between the bridge and geotechnical engineers.

Geologic Differences: If geotechnical conditions at abutments and intermediate piers result in different soil classifications, then response spectra should be determined for each abutment and pier having a different site classification. The design response spectra may be taken as the envelope of the individual spectra. However, if it is assessed that the bridge response is dominated by the abutment ground motions, only the abutment spectra need be enveloped (Article C3.4.2.2).

where

$\sum_{i=1}^{k} d_i = d_c$, and d_c is the total thickness 100-d_s ft of cohesive soil layers in the top 100 ft.

PI is the plasticity index, ASTM D4318-93.

w is the moisture content in percent, ASTM D2216-92.

3.4.2.3 Site Coefficients

Site coefficients for the short-period range (F_a) and for the long-period range (F_v) are given in Tables 3.4.2.3-1 and 3.4.2.3-2, respectively. Application of these coefficients to determine elastic seismic response coefficients of ground motion is described in Article 3.4.1.

C3.4.2.1 Site Class Definitions

Steps for Classifying a Site (also see Table 3.4.2-1)

Step 1: Check the site against the three categories of Site Class F, requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific evaluation.

Step 2: Categorize the site using one of the following three methods, with $\bar{v}_s$, $\bar{N}$, and $\bar{s}_u$ computed in all cases as specified by the definitions in Article 3.4.2.2:

Method a: $\bar{v}_s$ for the top 100 ft ($\bar{v}_s$ method)

Method b: $\bar{N}$ for the top 100 ft ($\bar{N}$ method)

Method c: $\bar{N}_{ch}$ for cohesionless soil layers (PI <20) in the top 100 ft and average $\bar{s}_u$ for cohesive soil layers (PI > 20) in the top 100 ft ($\bar{s}_u$ method)

$\bar{N}_{ch}$ and $\bar{s}_u$ are averaged over the respective thickness of cohesionless and cohesive soil layers within the upper 100 ft. Refer to Article 3.4.2.2 for equations for calculating average parameter values for the methods a, b, and c above. If method c is used, the site class is determined as the softer site class resulting from the averaging to obtain $\bar{N}_{ch}$ and $\bar{s}_u$ (for example, if $\bar{N}_{ch}$ were equal to 20 blows/ft and $\bar{s}_u$ were equal to 800 psf, the site would classify as E in accordance with Table 3.4.2-1). Note that when using method b, $\bar{N}$ values are for both cohesionless and cohesive soil layers within the upper 100 feet.

Table 3.4.2.3-1: Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration

Site Class	Mapped Spectral Response Acceleration at Short Periods				
	$S_s \leq 0.25$ g	$S_s = 0.50$ g	$S_s = 0.75$ g	$S_s = 1.00$ g	$S_s \geq 1.25$ g
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*a*	*a*	*a*	*a*	*a*

Table notes: Use straight line interpolation for intermediate values of S_s, where S_s is the spectral acceleration at 0.2 second obtained from the ground motion maps.

a Site-specific geotechnical investigation and dynamic site response analyses shall be performed (Article 3.4.3).

Table 3.4.2.3-2: Values of F_v as a Function of Site Class and Mapped 1 Second Period Spectral Acceleration

Site Class	Mapped Spectral Response Acceleration at 1 Second Periods				
	$S_1 \leq 0.1$ g	$S_1 = 0.2$ g	$S_1 = 0.3$ g	$S_1 = 0.4$ g	$S_1 \geq 0.5$ g
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	*a*	*a*	*a*	*a*	*a*

Table notes: Use straight line interpolation for intermediate values of S_1, where S_1 is the spectral acceleration at 1.0 second obtained from the ground motion maps.

a Site-specific geotechnical investigation and dynamic site response analyses shall be performed (Article 3.4.3).

3.4.3 Response Spectra Based on Site-Specific Procedures

A site-specific procedure to develop design response spectra of earthquake ground motions shall be performed when required by Article 3.4 and may be performed for any site. The objective of the site-specific probabilistic ground-motion analysis is to generate a uniform-hazard acceleration response spectrum considering a 5% probability of exceedance in 50 years for spectral values over the entire period range of interest. This analysis involves establishing

(1) the contributing seismic sources, (2) an upper-bound earthquake magnitude for each source zone, (3) median attenuation relations for acceleration response spectral values and their associated standard deviations, (4) a magnitude-recurrence relation for each source zone and (5) a fault-rupture-length relation for each contributing fault. Uncertainties in source modeling and parameter values shall be taken into consideration. Detailed documentation of ground-motion analysis is required and shall be peer reviewed. (Appendix A).

Where analyses to determine site soil response effects are required by Articles 3.4 and 3.4.2.1 for Site Class F soils, the influence of the local soil conditions shall be determined based on site-specific geotechnical investigations and dynamic site response analyses. (Appendix B).

For sites located within 6 miles of an active surface or shallow fault, as depicted in the USGS Active Fault Map, studies shall be considered to quantify near-fault effects on ground motions to determine if these could significantly influence the bridge response. The fault-normal component of near-field (D < 6 miles) motion may contain relatively long-duration velocity pulses which can cause severe nonlinear structural response, predictable only through nonlinear time-history analyses. For this case the recorded near-field horizontal components of motion need to be transformed into principal components before modifying them to be response-spectrum-compatible.

A deterministic spectrum may be utilized in regions having known active faults if the deterministic spectrum is no less than 2/3 of the probabilistic spectrum in the region of $0.5T_F$ to $2T_F$ of the spectrum where T_F is the bridge fundamental period. The deterministic spectrum shall be the envelope of a median spectra calculated for characteristic maximum magnitude earthquakes on known active faults. Alternatively, deterministic spectra may be defined for each fault, and each spectrum, or the spectrum that governs bridge response should be used.

As described in Article C3.4.2.2, it may be appropriate in some cases to define the ground motion at depth, below a soft surficial layer, if the surficial layer would not significantly influence bridge response. In this case, the Site Class may be determined on the basis of the soil profile characteristics below the surficial layer.

Within Site Class F (soils requiring site-specific evaluation), one category has been deleted in these specifications from the four categories contained in the previously cited codes and documents. This category consists of soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible, weakly cemented soils. It was judged that special analyses for the purpose of refining site ground-motion amplifications for these soils was too severe a requirement for ordinary bridge design because such analyses would require utilization of effective stress and strength-degrading nonlinear analyses that are difficult to conduct. Also, limited case-history data and analysis results indicate that liquefaction reduces spectral response rather than increases it, except at long periods in some cases. Because of the general reduction in response spectral amplitudes due to liquefaction, the designer may wish to consider special analysis of site response for liquefiable soil sites to avoid excessive conservatism in assessing bridge inertia loads when liquefaction occurs. Site-specific analyses are required for major or very important structures in some cases (Article 3.4), so that appropriate analysis techniques would be

When response spectra are determined from a site-specific study, the spectra shall not be lower than two-thirds of the response spectra determined using the general procedure of Article 3.4.1 in the region of $0.5T_F$ to $2T_F$ of the spectrum where T_F is the bridge fundamental period.

3.4.4 Acceleration Time-Histories

The development of time histories shall meet the requirements of this section. The developed time histories shall have characteristics that are representative of the seismic environment of the site and the local site conditions.

Response-spectrum-compatible time histories shall be used as developed from representative recorded motions. Analytical techniques used for spectrum matching shall be demonstrated to be capable of achieving seismologically realistic time series that are similar to the time series of the initial time histories selected for spectrum matching.

When using recorded time histories, they shall be scaled to the approximate level of the design response spectrum in the period range of significance. Each time history shall be modified to be response-spectrum compatible using the time-domain procedure.

At least three response-spectrum-compatible time histories shall be used for each component of motion in representing the Design Earthquake (ground motions having 5% probability of exceedance in 50 years). The issue of requiring all three orthogonal components (x, y, and z) of design motion to be input simultaneously shall be considered as a requirement when conducting a nonlinear time-history analysis. The design actions shall be taken as the maximum response calculated for the three ground motions in each principal direction. If a minimum of seven time histories are used for each component of motion, the design actions may be taken as the mean response calculated for each principal direction.

For near-field sites (D < 6 miles) the recorded horizontal components of motion selected should represent a near-field condition and that they should be transformed into principal components before making them response-spectrum-compatible. The major principal component should then be used to represent motion in the fault-normal direction and the minor principal component should be used to represent motion in the fault-parallel direction.

used for such structures. The deletion of liquefiable soils from Site Class F only affects the requirement to conduct site-specific analyses for the purpose of determining ground motion amplification through these soils. It is still required to evaluate liquefaction occurrence and its effect on a bridge as specified in Article 6.8.

C3.4.2.2 Definitions of Site Class Parameters

An alternative to applying Equations 3.6, 3.7, and 3.8 to obtain values for $\overline{N}$, $\overline{N}_{ch}$ and $\overline{s}_u$ is to convert the N-values or s_u values into estimated shear wave velocities and then to apply Equation 3.6. Procedures given in Kramer (1996) can be used for these conversions.

If the site profile is particularly non-uniform, or if the average velocity computed in this manner does not appear reasonable, or if the project involves special design issues, it may be desirable to conduct shear-wave velocity measurements, using one of the procedures identified in Appendix ~~B~~. In all evaluations of site classification, the shear-wave velocity should be viewed as the fundamental soil property, as this was used when conducting the original studies defining the site categories.

Depth of Motion Determination

For short bridges that involve a limited number of spans, the motion at the abutment will generally be the primary mechanism by which energy is transferred from the ground to the bridge superstructure. If the abutment is backed by an earth approach fill, the site classification should be determined at the base of the approach fill. The potential effects of the approach fill overburden pressure on the shear-wave velocity of the soil should be accounted for in the determination of site classification.

For long bridges it may be necessary to determine the site classification at an interior pier. If this pier is supported on spread footings, then the motion computed at the ground surface is appropriate. However, if deep foundations (i.e., driven piles or drilled shafts) are used to support the pier, then the location of the motion will depend on the horizontal stiffness of the soil-cap system relative to the horizontal stiffness of the soil-pile system. If the pile cap is the stiffer of the two, then the motion should be defined at the pile cap. If the pile cap provides little horizontal stiffness or if there is no pile cap (i.e., pile extension), then the controlling motion will likely be at some depth below the ground surface. Typically

3.5 SELECTION OF SEISMIC DESIGN CATEGORY SDC

Each bridge shall be designed to one of four Seismic Design Categories (SDC), A through D, based on the one-second period design spectral acceleration for the Life Safety Design Earthquake (S_{D1} refer to Section 3.4.1) as shown in Table 3.5.1.

Table 3.5.1: Partitions for Seismic Design Categories A, B, C and D

Value of $S_{D1} = F_v S_1$	SDC
$S_{D1} < 0.15g$	A
$0.15g \leq S_{D1} < 0.30g$	B
$0.30g \leq S_{D1} < 0.50g$	C
$0.50g \leq S_{D1}$	D

The five requirements for each of the proposed Seismic Design Categories are shown in Figure 3.5.1 and described below. For both single span bridges and bridges classified as SDC A the connections must be designed for specified forces in Article 4.5 and Article 4.6 respectively, and must also meet minimum support length requirements of Article 4.12.

1. SDC A
   a. No identification of ERS according to Article 3.3
   b. No Demand Analysis
   c. No Implicit Capacity Check Needed
   d. No Capacity Design Required
   e. Minimum Detailing requirements for seat width and superstructure/substructure connection design force
2. SDC B
   a. No Identification of ERS
   b. Demand Analysis
   c. Implicit Capacity Check Required (displacement, $P-\Delta$, seat width)
   d. No Capacity Design Required except for column shear requirement
   e. SDC B Level of Detailing

this will be approximately 4 to 7 pile diameters below the pile cap or where a large change in soil stiffness occurs. The determination of this elevation requires considerable judgment and should be discussed by the geotechnical and bridge engineers.

For cases where the controlling motion is more appropriately specified at depth, site-specific ground response analyses can be conducted following guidelines given in Appendix to establish ground motions at the point of fixity. This approach or alternatives to this approach should be used only with the owner's approval.

C3.4.2.3 Site Coefficients

Commentary to be added.

C3.4.3 Response Spectra Based on Site-Specific Procedure

The intent in conducting a site-specific probabilistic ground motion study is to develop ground motions that are more accurate for the local seismic and site conditions than can be determined from national ground motion maps and the procedure of Article 3.4.1. Accordingly, such studies must be comprehensive and incorporate current scientific interpretations at a regional scale. Because there are typically scientifically credible alternatives for models and parameter values used to characterize seismic sources and ground-motion attenuation, it is important to incorporate these uncertainties formally in a site-specific probabilistic analysis. Examples of these uncertainties include seismic source location, extent and geometry; maximum earthquake magnitude; earthquake recurrence rate; and ground-motion attenuation relationship.

Guidelines are presented in Appendix for site-specific geotechnical investigations and dynamic site response analyses for Site Class F soils. These guidelines are applicable for site-specific determination of site response for any site class when the site response is determined on the basis of a dynamic site response analysis.

Near-fault effects on horizontal response spectra include: (1) higher ground motions due to the proximity of the active fault; (2) directivity effects that increase ground motions for periods greater than 0.5 second if the fault rupture propagates toward the site; and (3) directionality effects that increase ground motions for periods greater than 0.5 second in the direction normal (perpendicular) to the strike of the fault. If the active fault is included and appropriately modeled in the development of national ground motion maps, then effect (1) is already

3. SDC C
   a. Identification of ERS
   b. Demand Analysis
   c. Implicit Capacity Check Required (displacement, $P-\Delta$, seat width)
   d. Capacity Design Required including column shear requirement
   e. SDC C Level of Detailing
4. SDC D
   a. Identification of ERS
   b. Demand Analysis
   c. Displacement Capacity Required using Pushover Analysis (check $P-\Delta$ and seat width)
   d. Capacity Design Required including column shear requirement
   e. SDC D Level of Detailing

included in the national ground motion maps. Effects (2) and (3) are

not included in the national maps. These effects are significant only for periods longer than 0.5 second and normally would be evaluated only for essential or critical bridges having natural periods of vibration longer than 0.5 second. Further discussion of effects (2) and (3) are contained in Somerville (1997) and Somerville et al. (1997). The ratio of vertical-to-horizontal ground motions increases for short-period motions in the near-fault environment.

C3.4.4 Acceleration Time Histories

Characteristics of the seismic environment of the site to be considered in selecting time-histories include: tectonic environment (e.g., subduction zone; shallow crustal faults in western United States or similar crustal environment; eastern United States or similar crustal environment); earthquake magnitude; type of faulting (e.g., strike-slip; reverse; normal); seismic-source-to-site distance; local site conditions; and design or expected ground-motion characteristics (e.g., design response spectrum; duration of strong shaking; and special ground-motion characteristics such as near-fault characteristics). Dominant earthquake magnitudes and distances, which contribute principally to the probabilistic design response spectra at a site, as determined from national ground motion maps, can be obtained from deaggregation information on the U.S. Geological Survey website: http://geohazards.cr.usgs.gov/eq/.

It is desirable to select time-histories that have been recorded under conditions similar to the seismic conditions at the site listed above, but compromises are usually required because of the multiple attributes of the seismic environment and the limited data bank of recorded time-histories. Selection of time-histories having similar earthquake magnitudes and distances, within reasonable ranges, are especially important parameters because they have a strong influence on response spectral content, response spectral shape, duration of strong shaking, and near-source ground-motion characteristics. It is desirable that selected recorded motions be somewhat similar in overall ground motion level and spectral shape to the design spectrum to avoid using very large scaling factors with recorded motions and very large changes in spectral content in the spectrum-matching approach. If the site is located within 6 miles of an active fault, then intermediate-to-long-period ground-motion pulses that are characteristic of near-source time-histories should be included if these types of ground motion characteristics could significantly influence structural response. Similarly, the high

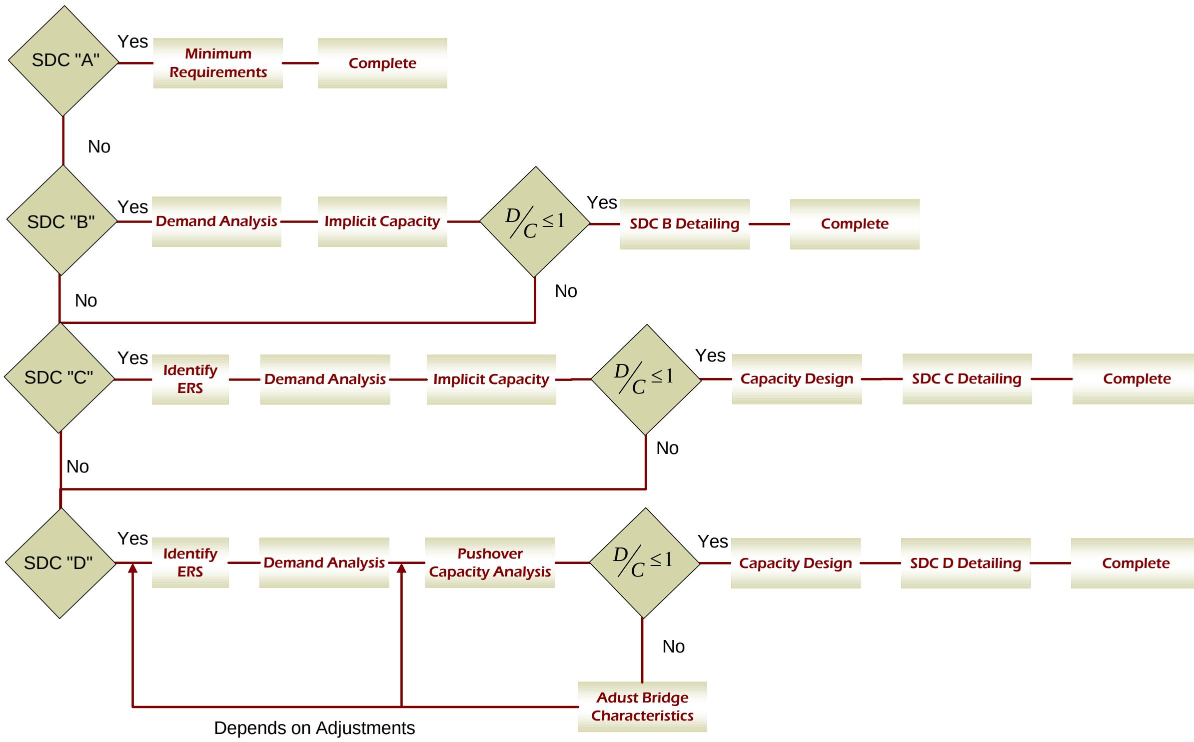


FIGURE 3.5.1: Seismic Design Category (SDC) Core Flowchart

3.6 TEMPORARY AND STAGED CONSTRUCTION

Any bridge or partially constructed bridge that is expected to be temporary for more than five years shall be designed using the requirements for permanent structures and shall not use the provisions of this Article.

Temporary bridges expected to carry vehicular traffic or pedestrian bridges over roads carrying vehicular traffic must satisfy the Performance Criteria defined in Section 3.2. The provisions also apply to those bridges that are constructed in stages and expected to carry traffic and/or pass over routes that carry traffic. The design response spectra given in Article 3.4 may be reduced by a factor of not more than 2.5 in order to calculate the component elastic forces and displacements. The Seismic Design Category of the temporary bridge shall be obtained based on the reduced/modified response spectrum except that a temporary bridge classified in SDC B, C or D based on the unreduced spectrum can not be

short-period spectral content of near-source vertical ground motions should be considered.

Ground-motion modeling methods of strong-motion seismology are being increasingly used to supplement the recorded ground-motion database. These methods are especially useful for seismic settings for which relatively few actual strong-motion recordings are available, such as in the central and eastern United States. Through analytical simulation of the earthquake rupture and wave-propagation process, these methods can produce seismologically reasonable time series.

Response spectrum matching approaches include methods in which time series adjustments are made in the time domain (Lilhanand and Tseng, 1988; Abrahamson, 1992) and those in which the adjustments are made in the frequency domain (Gasparini and Vanmarcke, 1976; Silva and Lee, 1987; Bolt and Gregor, 1993). Both of these approaches can be used to modify existing time-histories to achieve a close match to the design response spectrum while maintaining fairly well the basic time-domain character of the recorded or simulated time-histories. To minimize changes to the time-domain characteristics, it is desirable that the overall shape of the spectrum of the recorded timereclassified to SDC A based on the reduced/modified spectrum. The requirements for each of the Seismic Design Categories A through D shall be met as defined in Article 3.5. Response spectra for construction sites that are within 6 miles of an active fault (see Article 3.4) shall be the subject of special study.

3.7 LOAD FACTORS

Extreme Event I – Load combination including earthquake (see Table 3.4.1-1 Load Combinations and Load Factors) of the AASHTO LRFD Specifications. The load factors given for permanent loads, γ_p, are given in Table 3.4.1-2. Load Factors for Permanent Loads, γ_p, as shown in the table have ranges, which can vary from 1.25 for a maximum to 0.90 for a minimum. It is recommended that for the seismic response analysis that material unit weighs as conventionally used to compute the inertia effects be used with $\gamma_p = 1.0$. The load factor for live load for Extreme Event I, γ_{EQ}, should be determined on a project specific basis. The inertia effects of live load should not be considered when conducting a dynamic response analysis. Only the gravity effects of live load are considered for bridges, which carry large volumes of traffic in populated metropolitan areas.

history not be greatly different from the shape of the design response spectrum and that the time-history initially be scaled so that its spectrum is at the approximate level of the design spectrum before spectrum matching.

When developing three-component sets of time histories by simple scaling rather than spectrum matching, it is difficult to achieve a comparable aggregate match to the design spectra for each component of motion when using a single scaling factor for each time-history set. It is desirable, however, to use a single scaling factor to preserve the relationship between the components. Approaches for dealing with this scaling issue include: (1) use of a higher scaling factor to meet the minimum aggregate match requirement for one component while exceeding it for the other two; (2) use of a scaling factor to meet the aggregate match for the most critical component with the match somewhat deficient for other components; (3) compromising on the scaling by using different factors as required for different components of a time-history set. While the second approach is acceptable, it requires careful examination and interpretation of the results and possibly dual analyses for application of the horizontal higher horizontal component in each principal horizontal direction.

The requirements for the number of time histories to be used in nonlinear inelastic dynamic analysis and for the interpretation of the results take into account the dependence of response on the time domain character of the time histories (duration, pulse shape, pulse sequencing) in addition to their response spectral content.

Additional guidance on developing acceleration time histories for dynamic analysis may be found in publications by the Caltrans Seismic Advisory Board Adhoc Committee (CSABAC) on Soil-Foundation-Structure Interaction (1999) and the U.S. Army Corps of Engineers (2000). CSABAC (1999) also provides detailed guidance on modeling the spatial variation of ground motion between bridge piers and the conduct of seismic soil-foundation-structure interaction (SFSI) analyses. Both spatial variations of ground motion and SFSI may significantly affect bridge response. Spatial variations include differences between seismic wave arrival times at bridge piers (wave passage effect), ground motion incoherence due to seismic wave scattering, and differential site response due to different soil profiles at different bridge piers. For long bridges, all forms of spatial variations may be important. For short bridges, limited information appears to indicate that wave passage effects and incoherence are, in general, The γ_p for the dead load used in this combination should match that used for other Load Combinations.

relatively unimportant in comparison to effects of differential site response (Shinozuka et al., 1999; Martin, 1998). Somerville et al. (1999) provide guidance on the characteristics of pulses of ground motion that occur in time histories in the near-fault region.

C3.4.5 Vertical Acceleration Effects

The most comprehensive study (Button et al., 1999) performed to date on the impact of vertical acceleration effects indicates that for some design parameters (superstructure moment and shear, column axial forces) and for some bridge types the impact can be significant. The study was based on vertical response spectra developed by Silva (1997) from recorded western United States ground motions. Until more information is known about the characteristics of vertical ground motions in the central and eastern Untied States and those areas impacted by subductions zones in the Pacific, this Guideline does *not* provide specific recommendations. However, it is advisable for designers to be aware that vertical acceleration effects may be important (Button et al., 1999) and, for essential or critical bridges, should be assessed.

Recent studies (e.g. Abrahamson and Silva, 1997; Silva, 1997; Campbell and Bozorgnia, 2000) have shown that the ratio of the vertical response spectrum to the horizontal response spectrum of ground motions can differ substantially from the nominal two-thirds ratio commonly assumed in engineering practice. These studies show that the ratios of vertical to horizontal response spectral values are functions of the tectonic environment, subsurface soil or rock conditions, earthquake magnitude, earthquake-source-to-site distance, and period of vibration. Whereas the two-thirds ratio may be conservative for longer periods of vibration (say greater than 0.3 second) in many cases, at shorter periods, the ratio of vertical to horizontal response spectra may exceed two-thirds and even substantially exceed unity for close earthquake source-to-site distances and periods less than 0.2 second. At present, detailed procedures have not been developed for constructing vertical spectra having an appropriate relationship to the horizontal spectra constructed using the general procedure of Article 3.4.1.

At present, this guideline recommends implicit consideration of vertical acceleration effects in design only as a function of the distance of a bridge site from an active fault. As such, these requirements would generally not be applied to sites in the central and eastern United States. Also, because the characteristics of vertical ground motions in subduction zones have been the subject of only limited studies, the guideline does not at present impose requirements for vertical acceleration effects as a function of distance from subduction zone faults.

For use in Tables X.X and X.X, earthquake magnitude is taken as the largest (maximum) magnitude, based on the moment magnitude scale (rather than the Richter, or local, magnitude), of an earthquake considered capable of occurring on the active fault. Usually, maximum magnitude is estimated on the basis of the longest rupture length or the largest rupture area assessed as achievable during an earthquake on the fault (e.g., Wells and Coppersmith, 1994). Maximum magnitude should be estimated by a knowledgeable geologist or seismologist.

C3.5 SELECTION OF SEISMIC DESIGN CATEGORY (SDC)

The Seismic Hazard Level is defined as a function of the magnitude of the ground surface shaking as expressed by F_vS_1.

The Seismic Design Category reflects the variation in seismic risk across the country and ~~are~~ is used to permit different requirements for methods of analysis, minimum support lengths, column design details, and foundation and abutment design procedures.

C3.6 TEMPORARY AND STAGED CONSTRUCTION

The option to use a reduced acceleration coefficient is provided to reflect the limited exposure period.

C3.7 LOAD FACTORS

Extreme Event-I limit state includes water loads, WA. The probability of a major flood and an earthquake occurring at the same time is small. Therefore, basing water loads and scour depths on mean discharges may be warranted. Live load coincident with an earthquake is only included for bridges with heavy truck traffic (i.e., high ADTT) and for elements particularly sensitive to gravity loading.

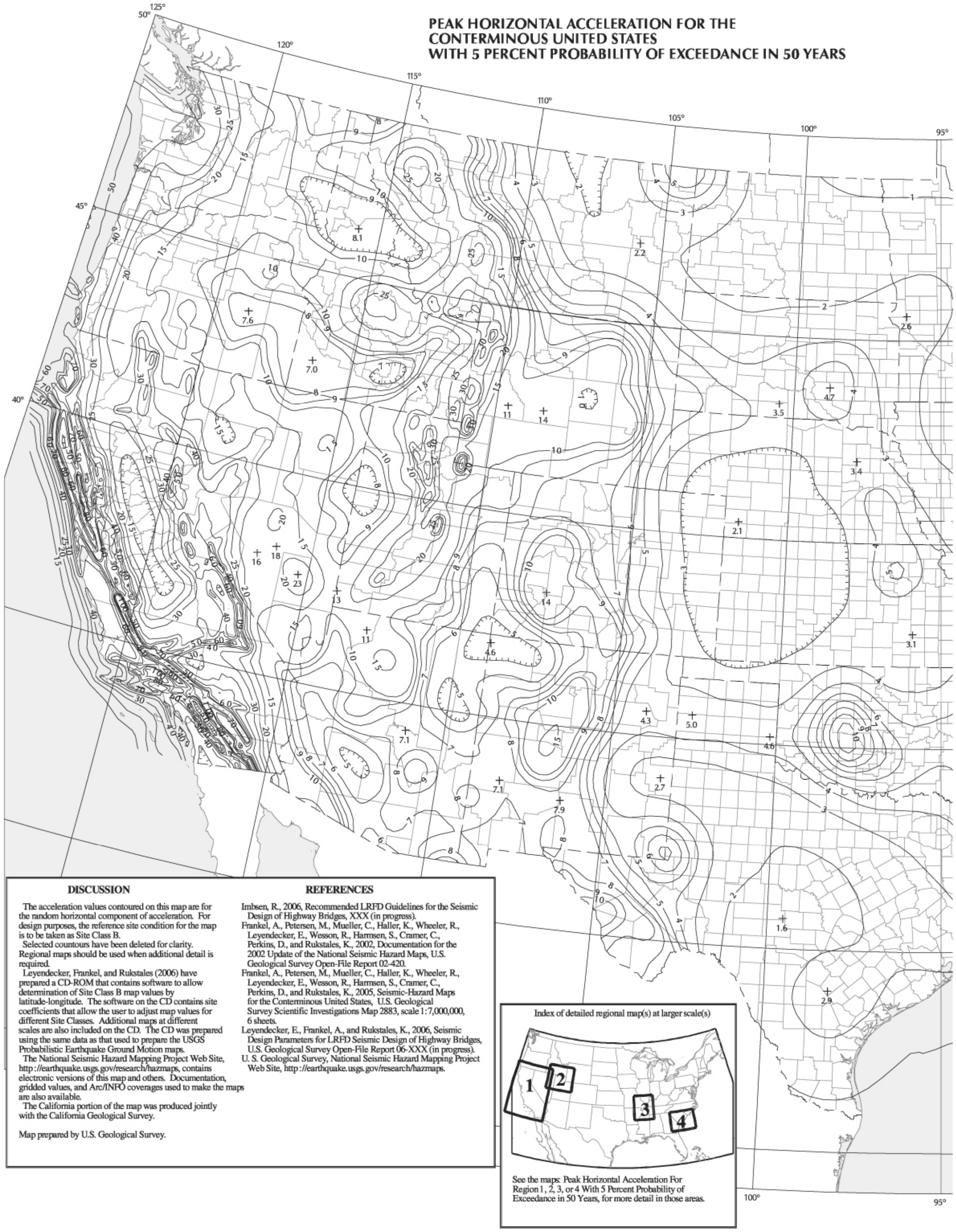


FIGURE 3.4.1-2a: Conterminous U.S. PGA – Western

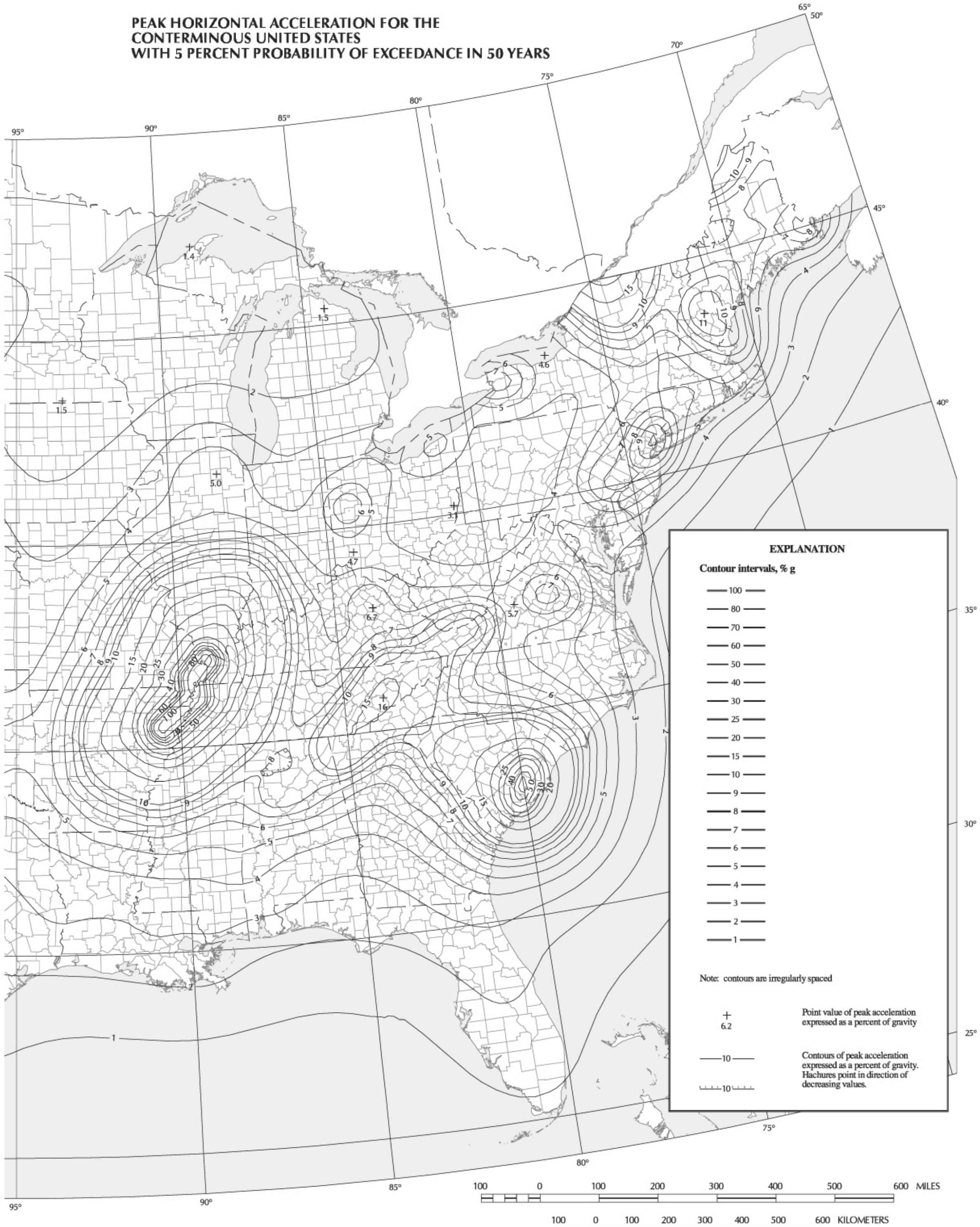


FIGURE 3.4.1-2b: Conterminous U.S. PGA – Eastern

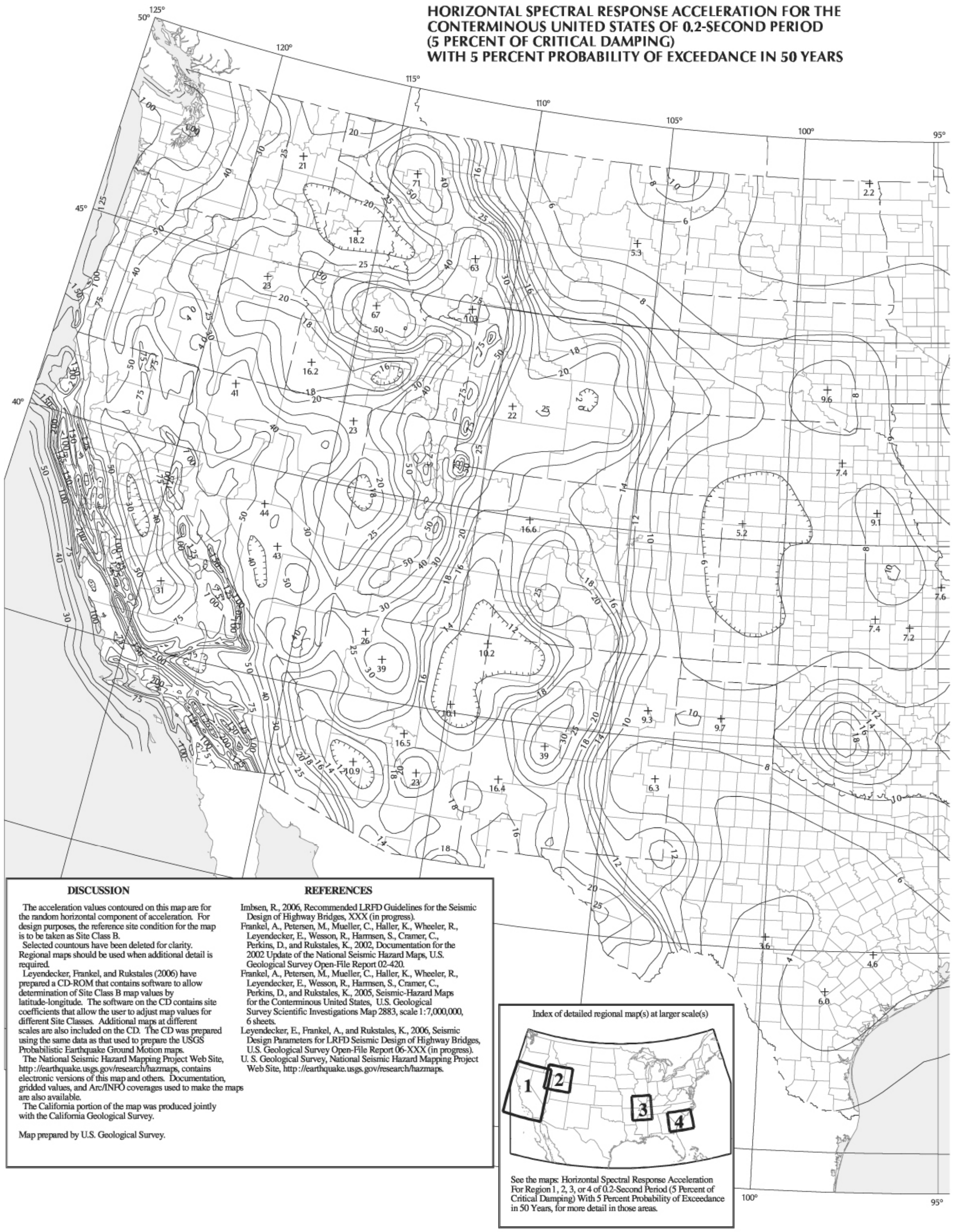


FIGURE 3.4.1-3a: Conterminous U.S. 0.2 Sec. – Western

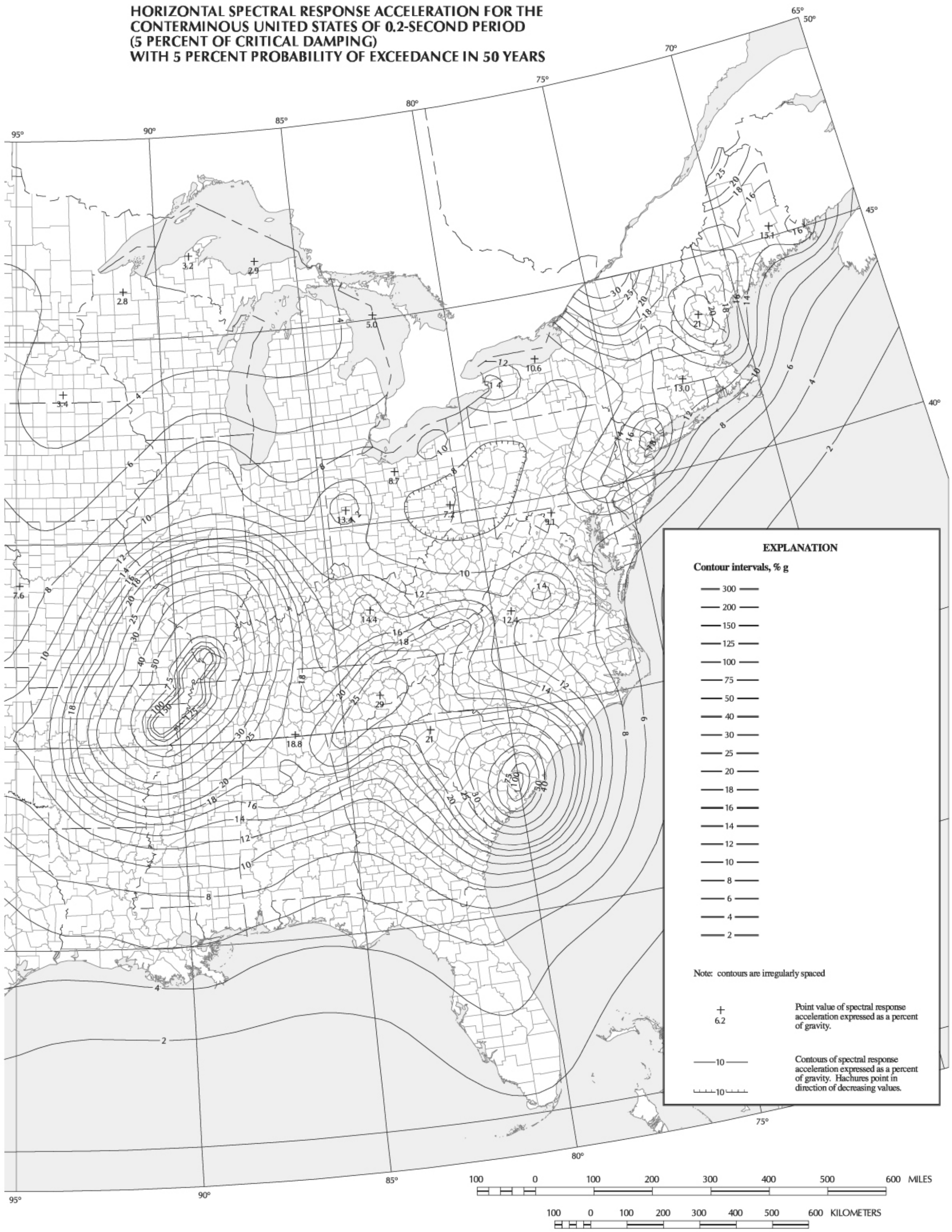


FIGURE 3.4.1-3b: Conterminous U.S. 0.2 Sec. – Eastern

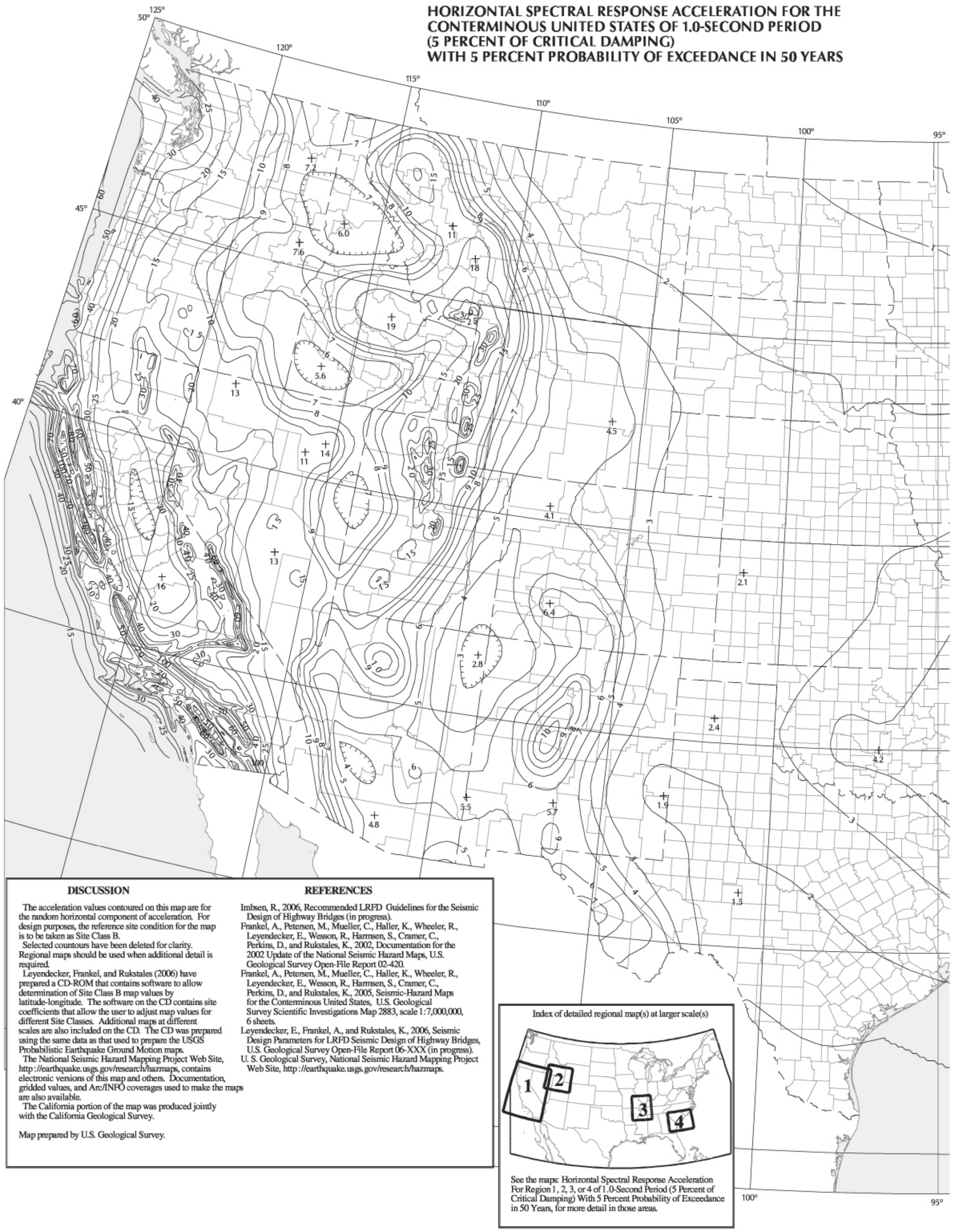


FIGURE 3.4.1-4a: Conterminous U.S. 1.0 Sec. – Western

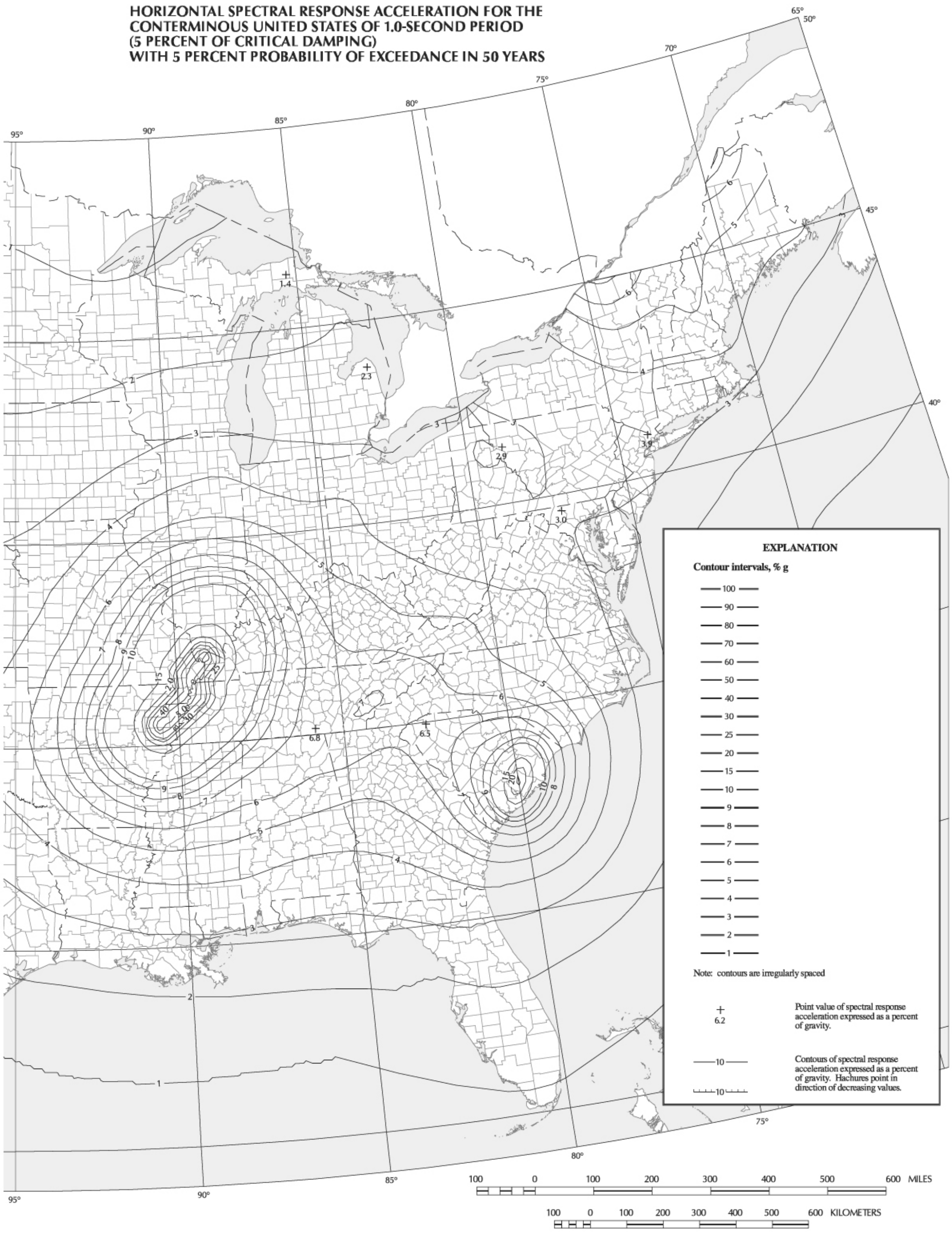


FIGURE 3.4.1-4b: Conterminous U.S. 1.0 Sec. – Eastern

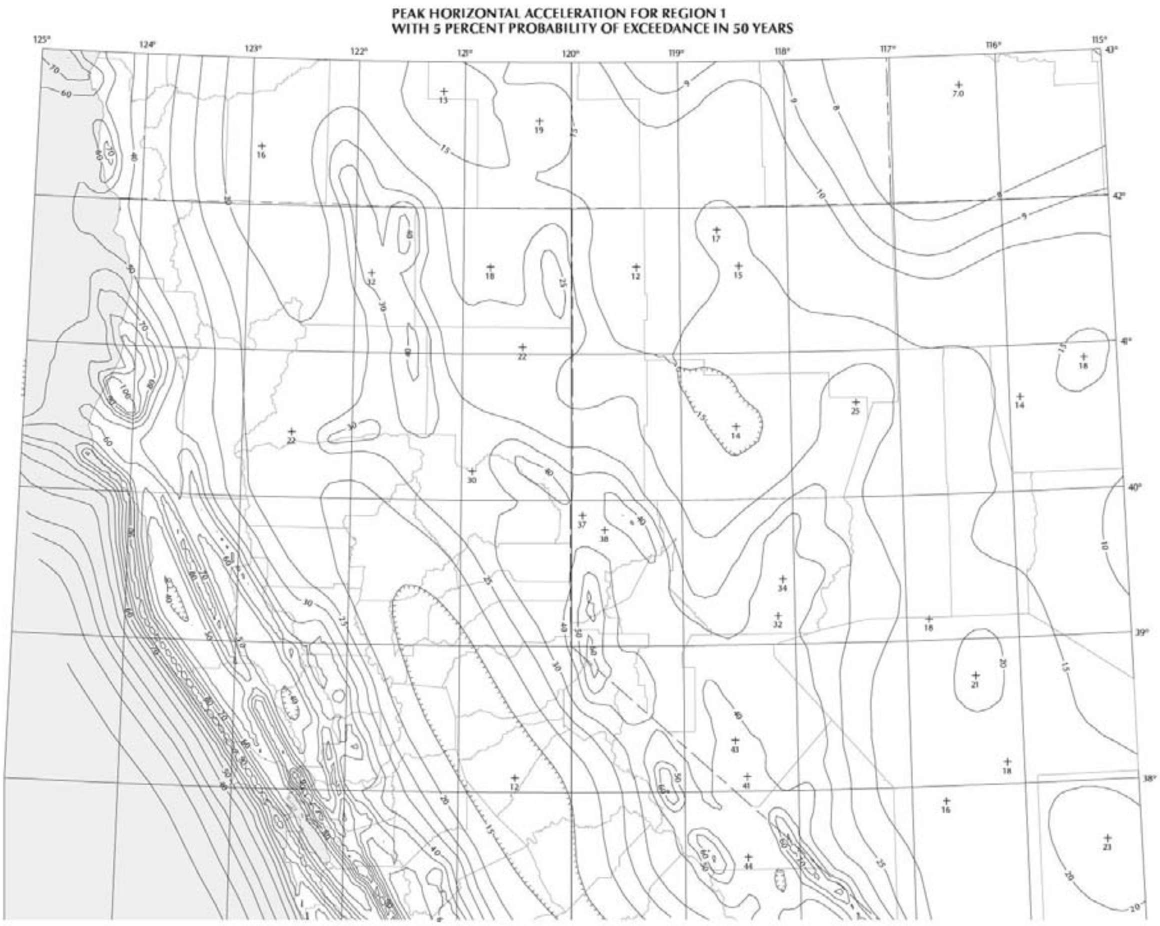
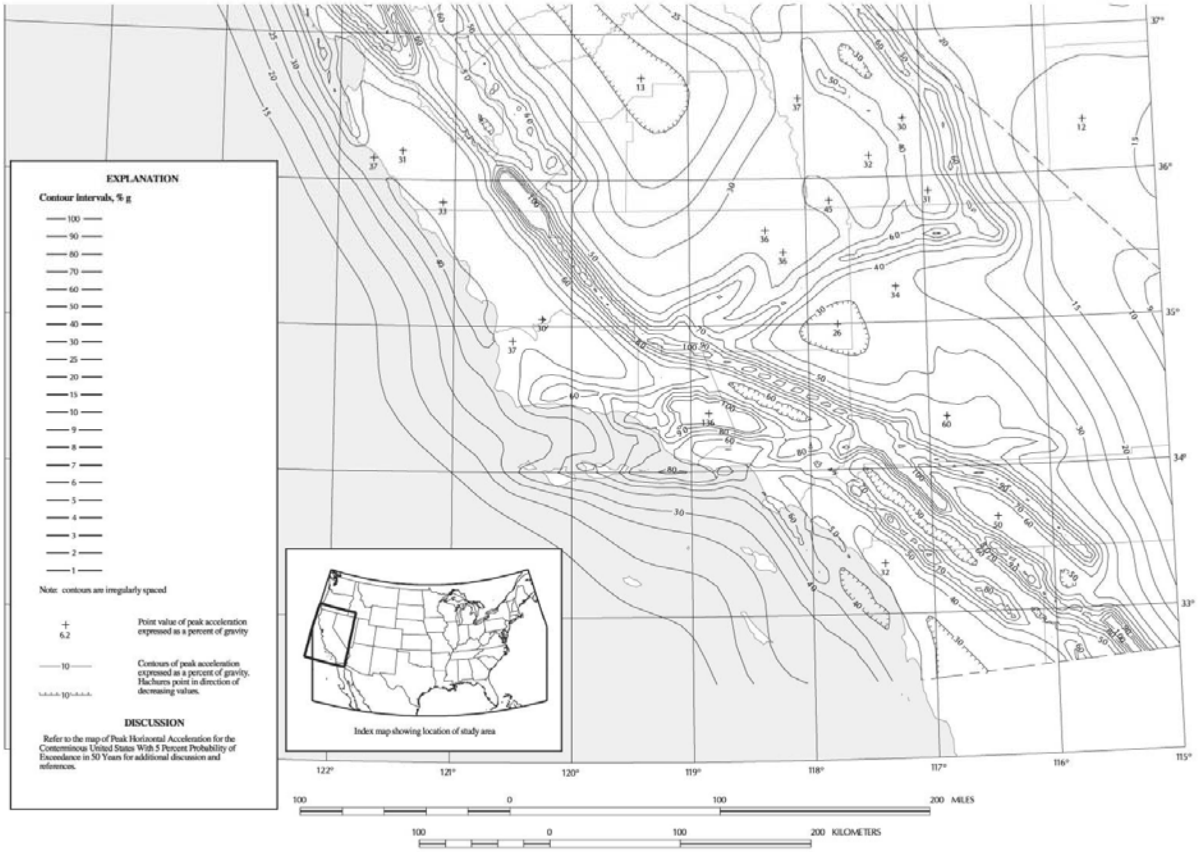


FIGURE 3.4.1-5a: Region 1 U.S. PGA – Upper Portion



PEAK HORIZONTAL ACCELERATION FOR REGION 1
WITH 5 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS

FIGURE 3.4.1-5b: Region 1 U.S. PGA – Lower Portion

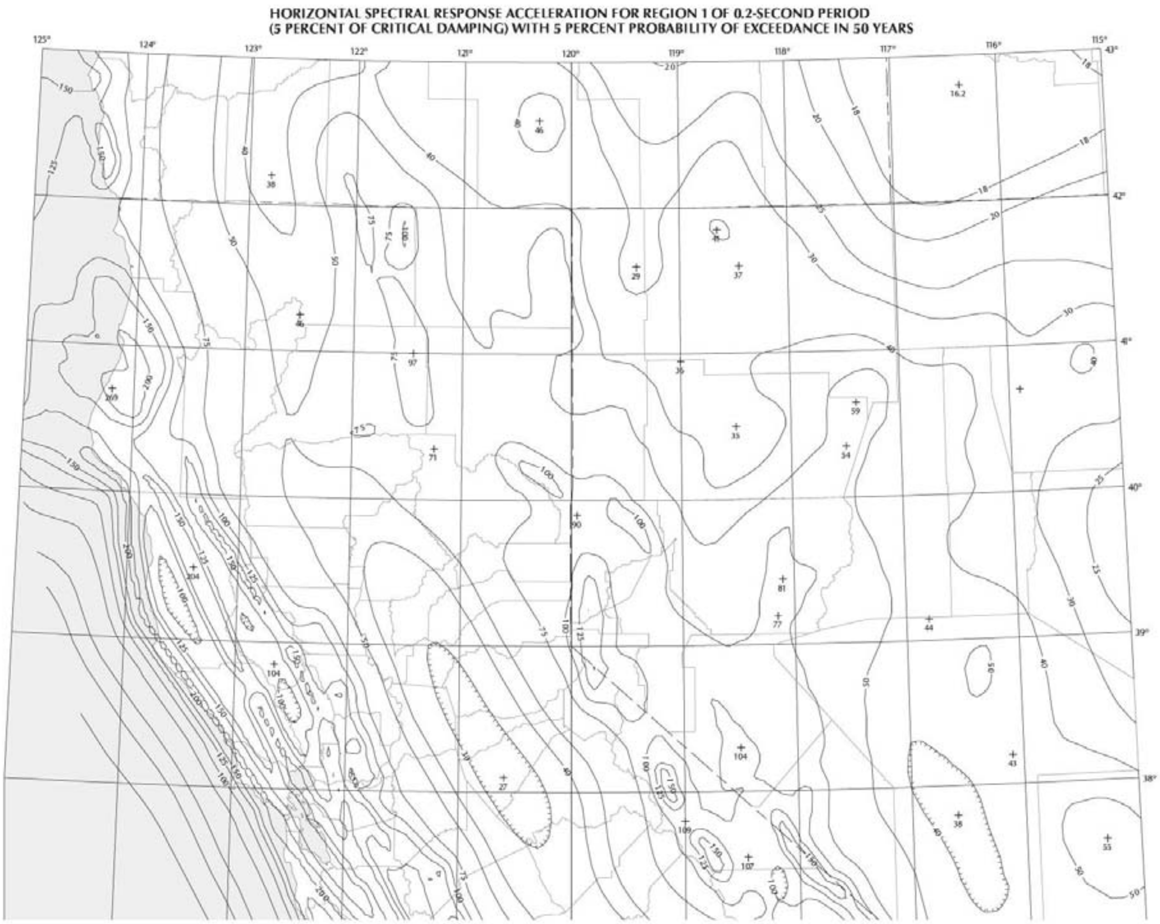


FIGURE 3.4.1-6a: Region 1 U.S. 0.2 Sec. – Upper Portion

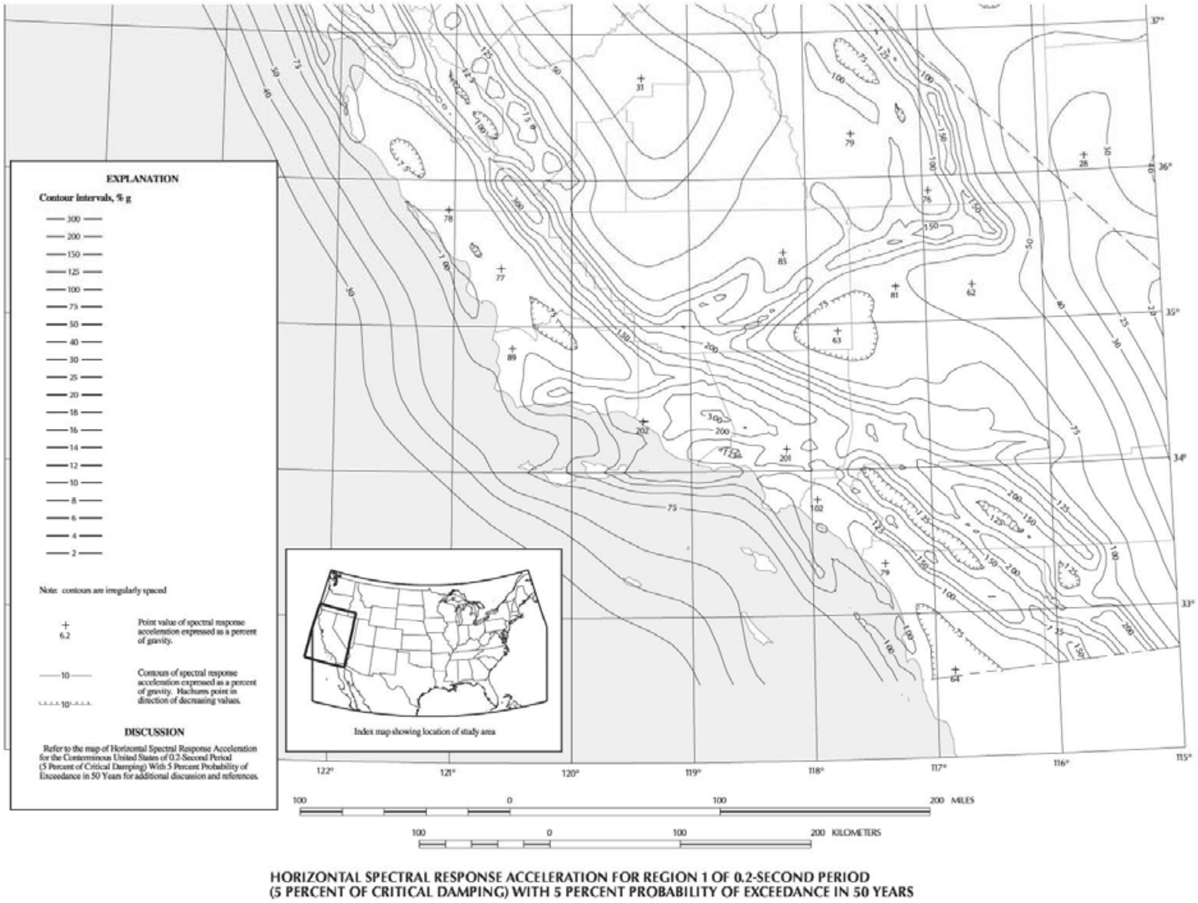


FIGURE 3.4.1-6b: Region 1 U.S. 0.2 Sec. – Lower Portion

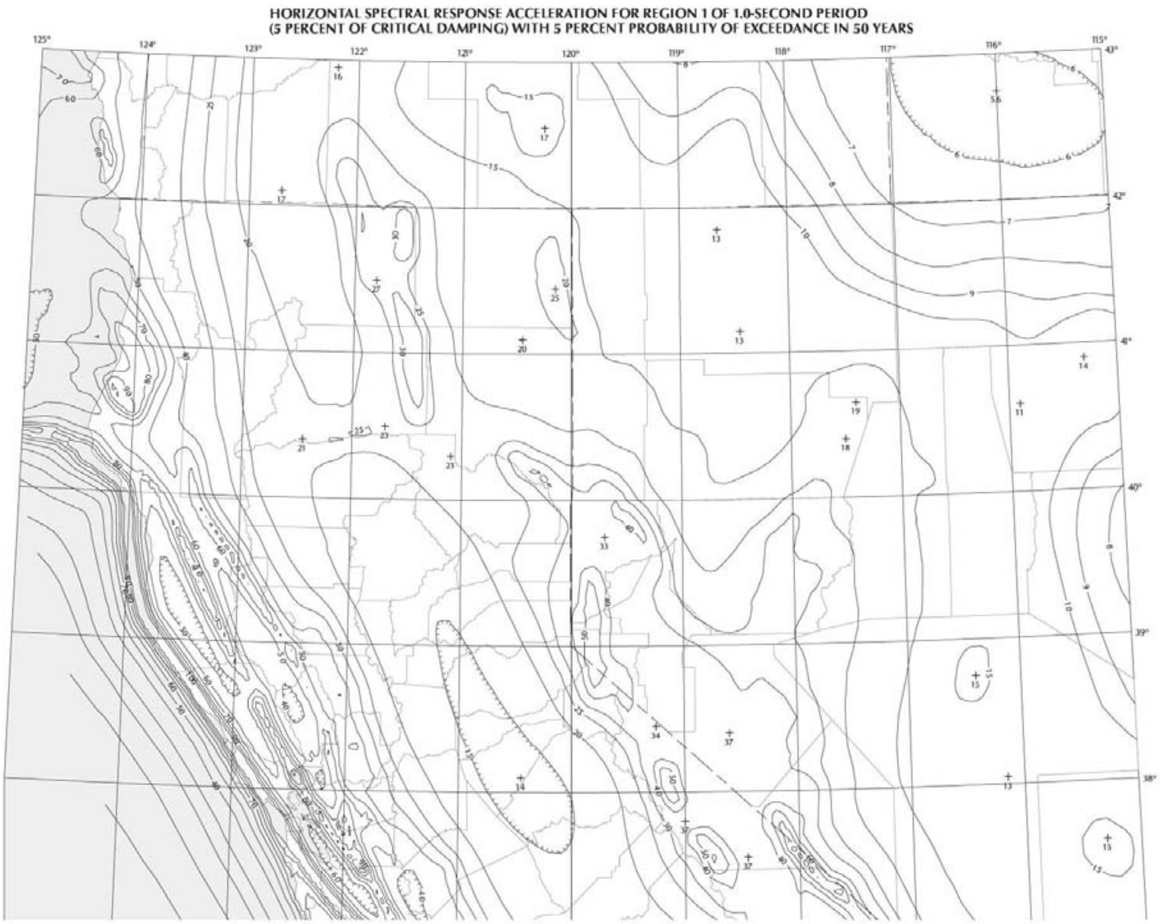
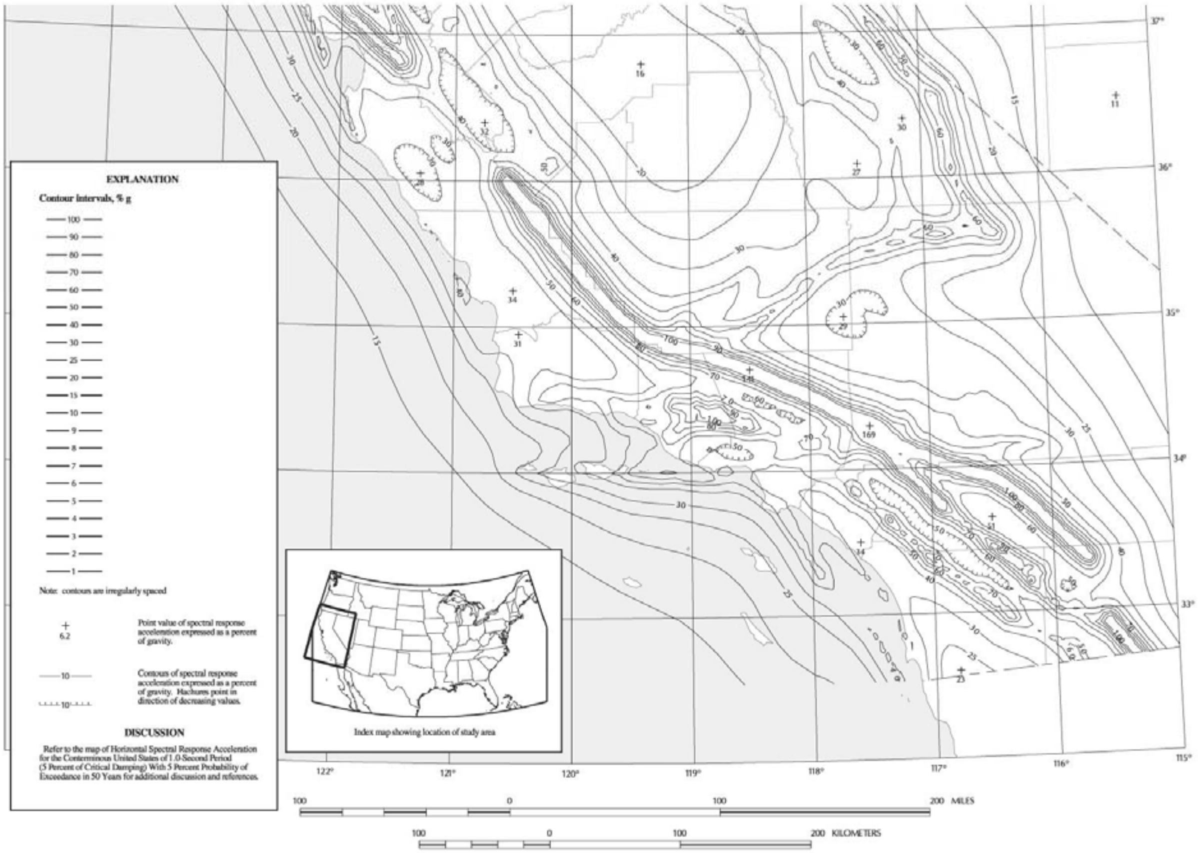


FIGURE 3.4.1-7a: Region 1 U.S. 1.0 Sec. – Upper Portion



HORIZONTAL SPECTRAL RESPONSE ACCELERATION FOR REGION 1 OF 1.0-SECOND PERIOD
(5 PERCENT OF CRITICAL DAMPING) WITH 5 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS

FIGURE 3.4.1-7b: Region 1 U.S. 1.0 Sec. – Lower Portion

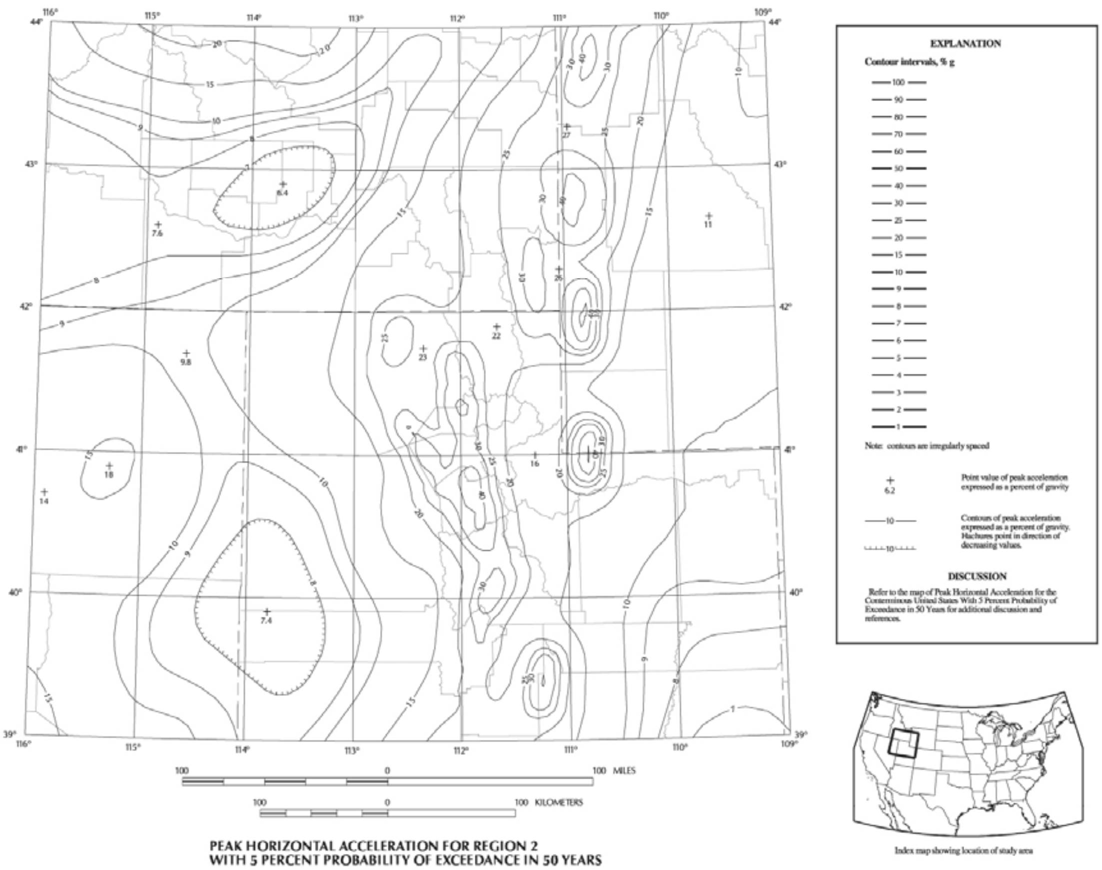


FIGURE 3.4.1-8: Region 2 U.S. PGA

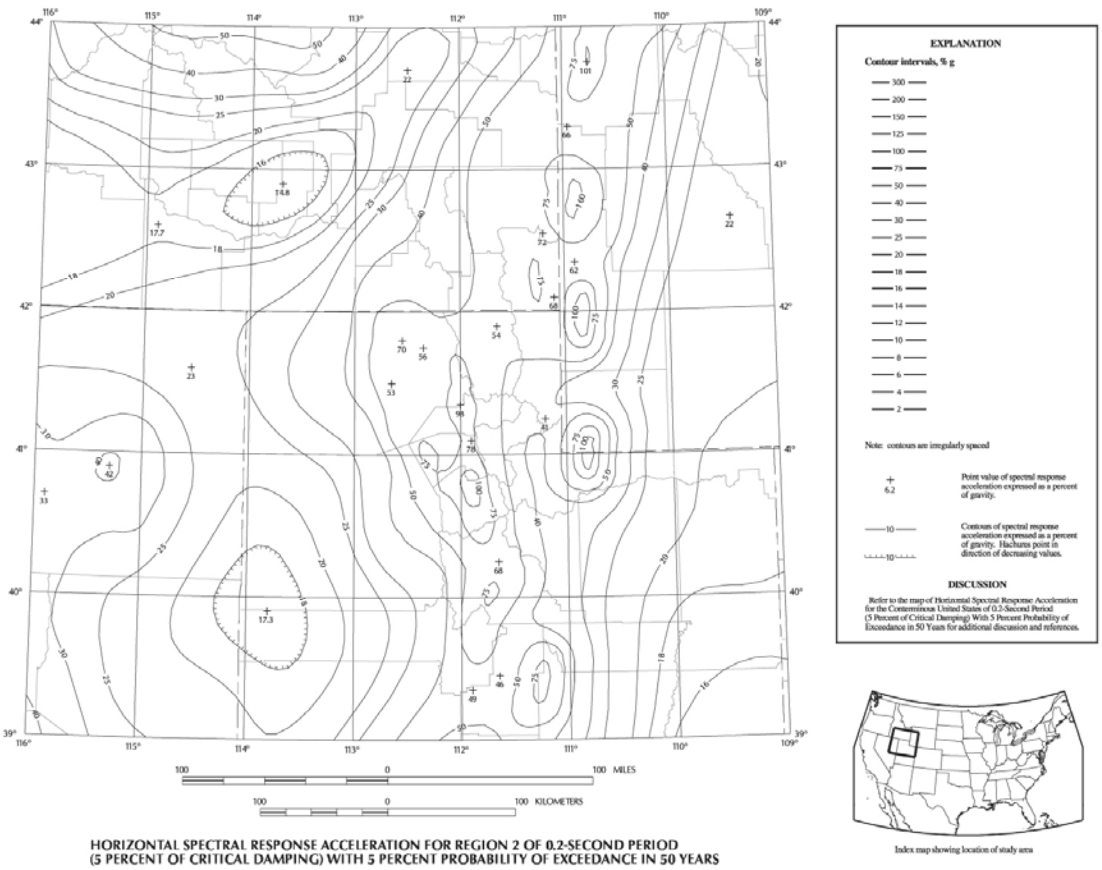


FIGURE 3.4.1-9: Region 2 U.S. 0.2 Sec.

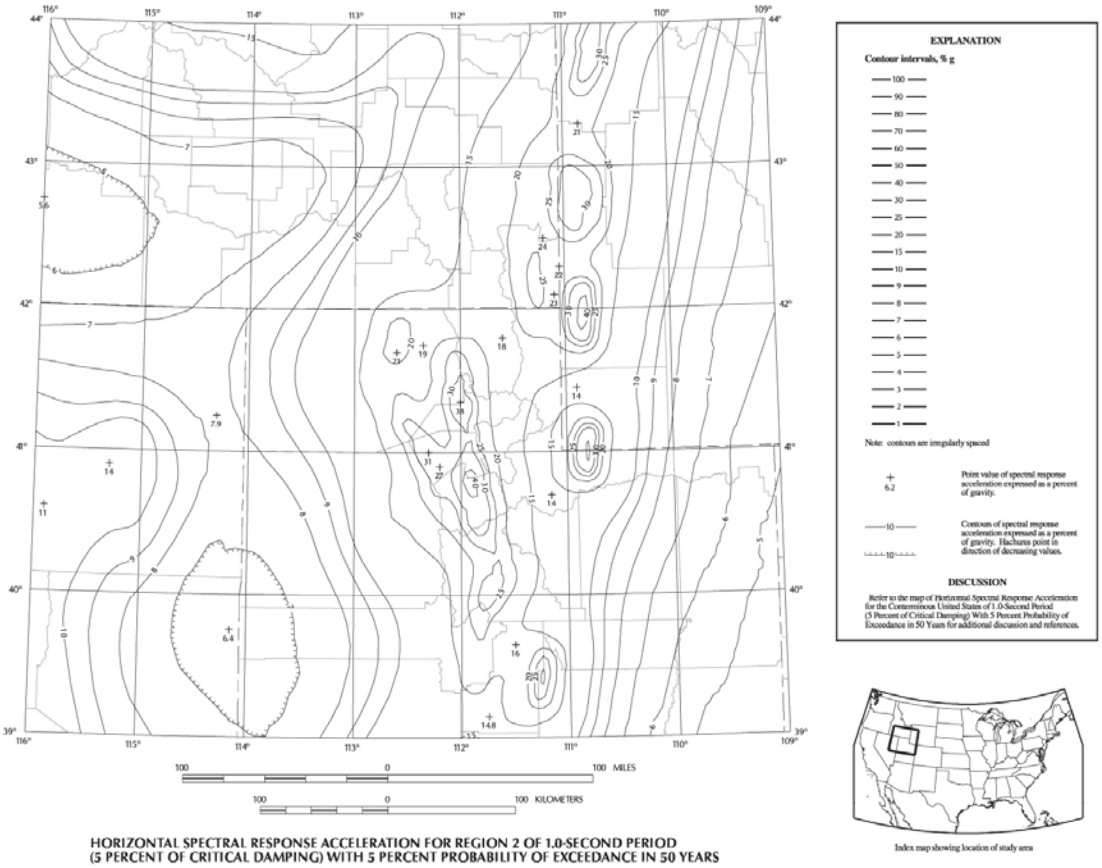


FIGURE 3.4.1-10: Region 2 U.S. 1.0 Sec.

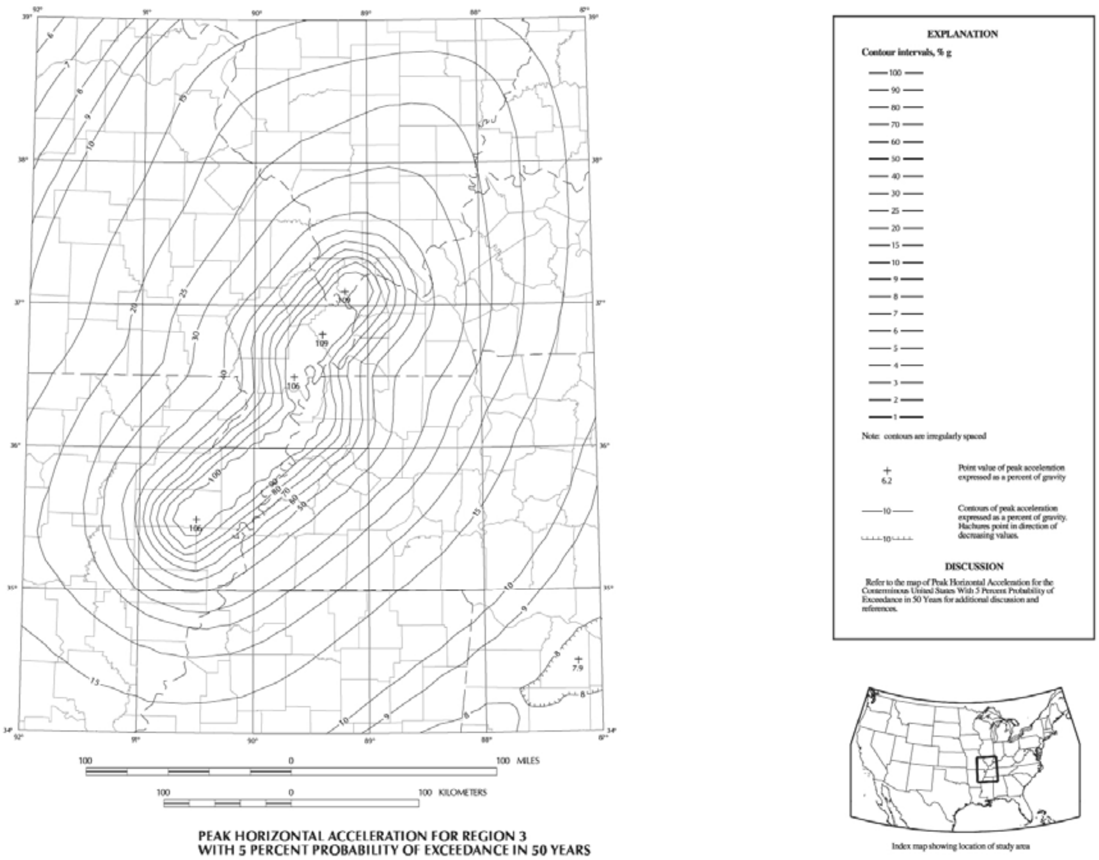
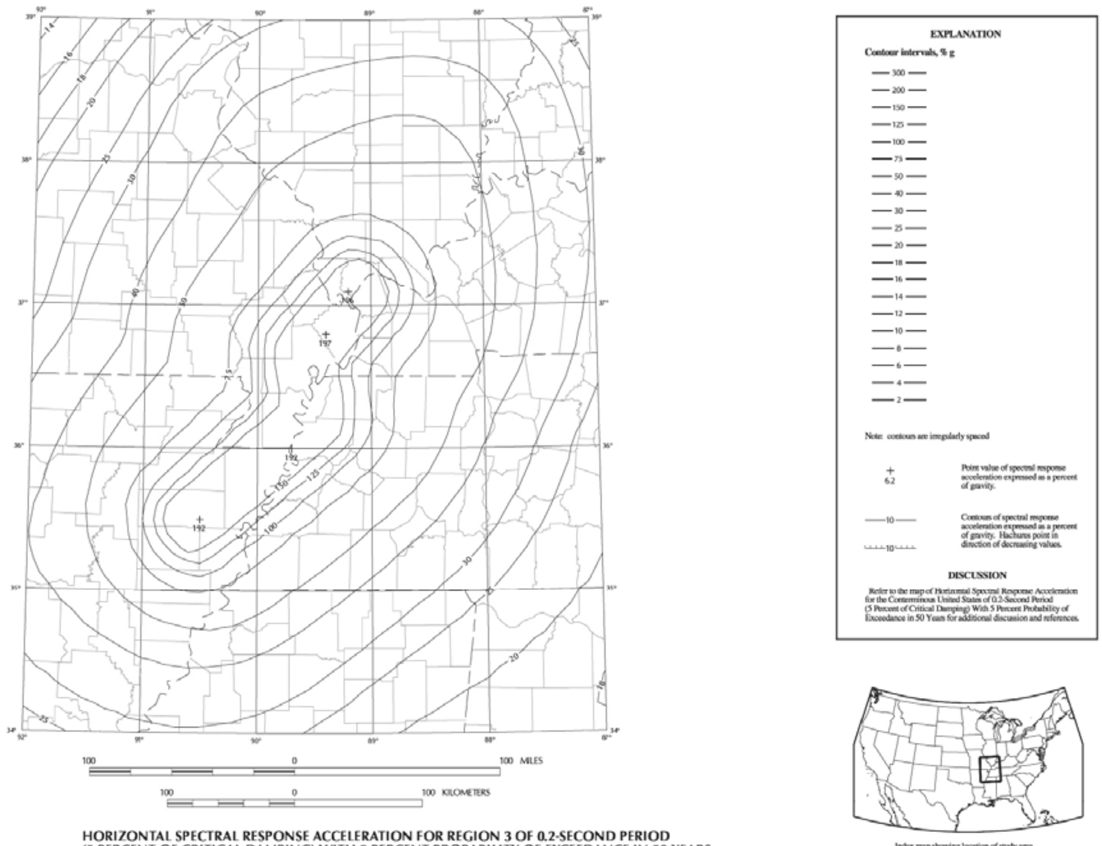


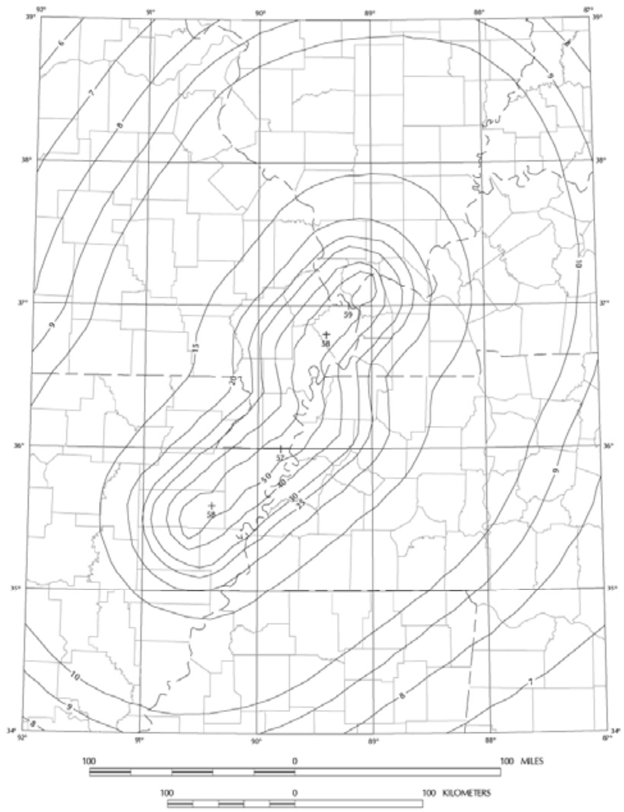
FIGURE 3.4.1-11: Region 3 U.S. PGA



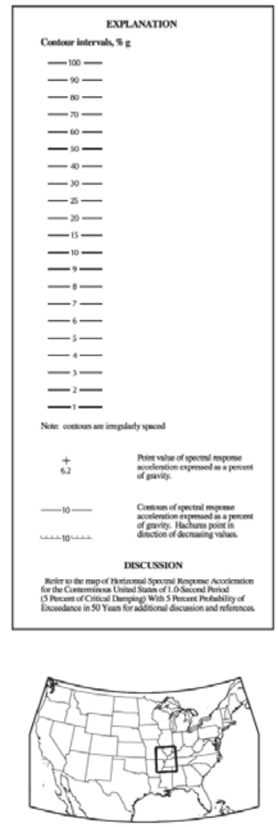
HORIZONTAL SPECTRAL RESPONSE ACCELERATION FOR REGION 3 OF 0.2-SECOND PERIOD (5 PERCENT OF CRITICAL DAMPING) WITH 5 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS

Index map showing location of study area

FIGURE 3.4.1-12: Region 3 U.S. 0.2 Sec.



HORIZONTAL SPECTRAL RESPONSE ACCELERATION FOR REGION 3 OF 1.0-SECOND PERIOD
(5 PERCENT OF CRITICAL DAMPING) WITH 5 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS



Index map showing location of study area

FIGURE 3.4.1-13: Region 3 U.S. 1.0 Sec.

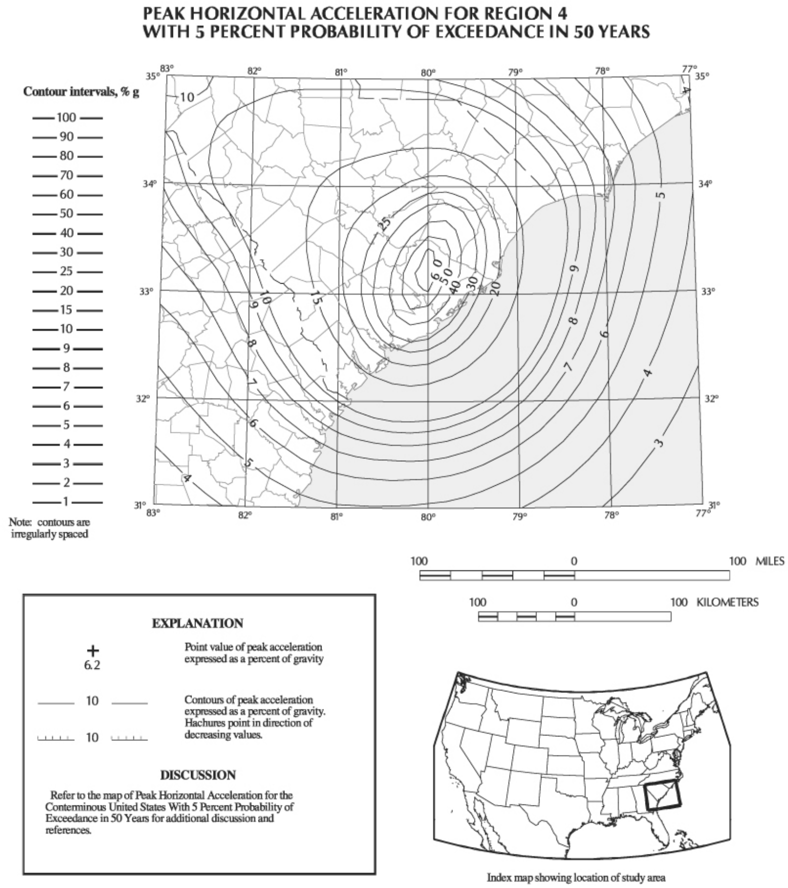


FIGURE 3.4.1-14: Region 4 U.S. PGA

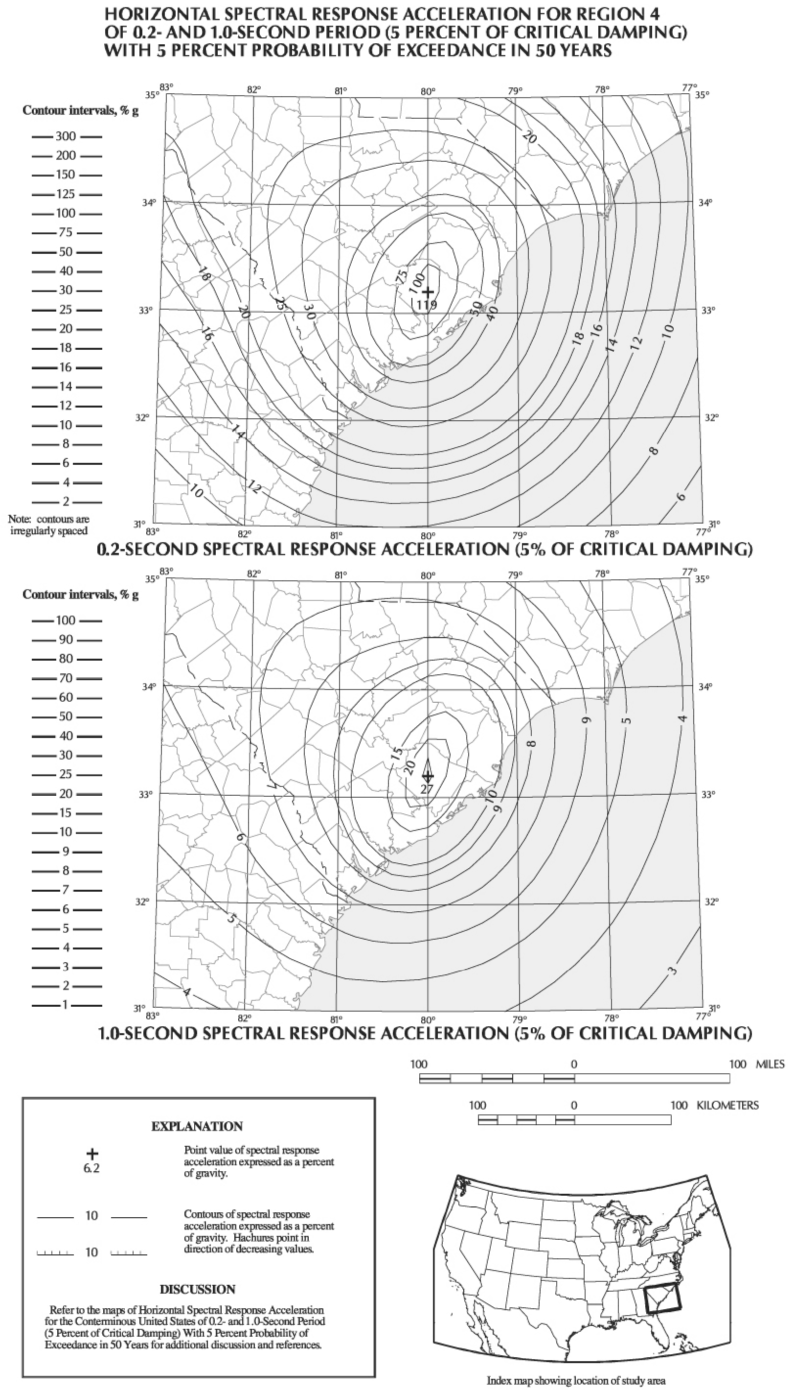


FIGURE 3.4.1-15: Region 4 U.S. 0.2 Sec. and 1.0 Sec.

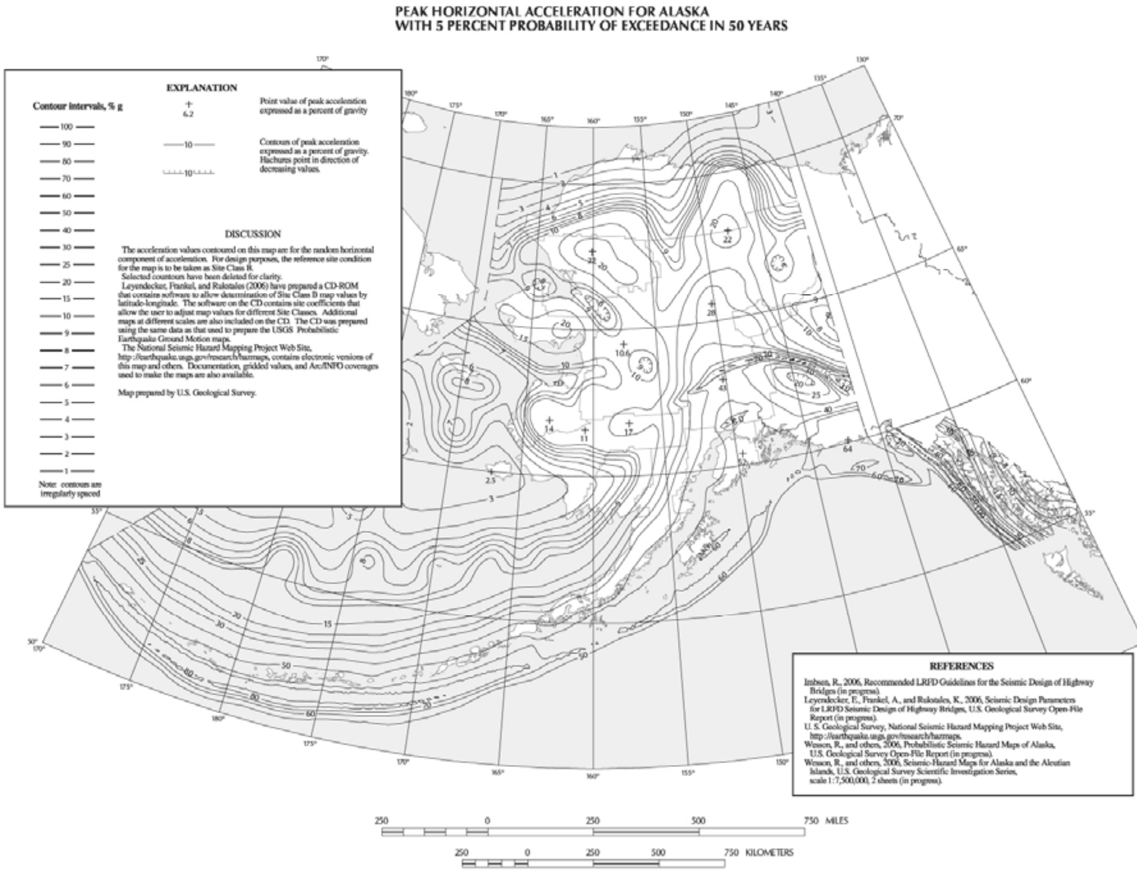


FIGURE 3.4.1-16: Alaska PGA

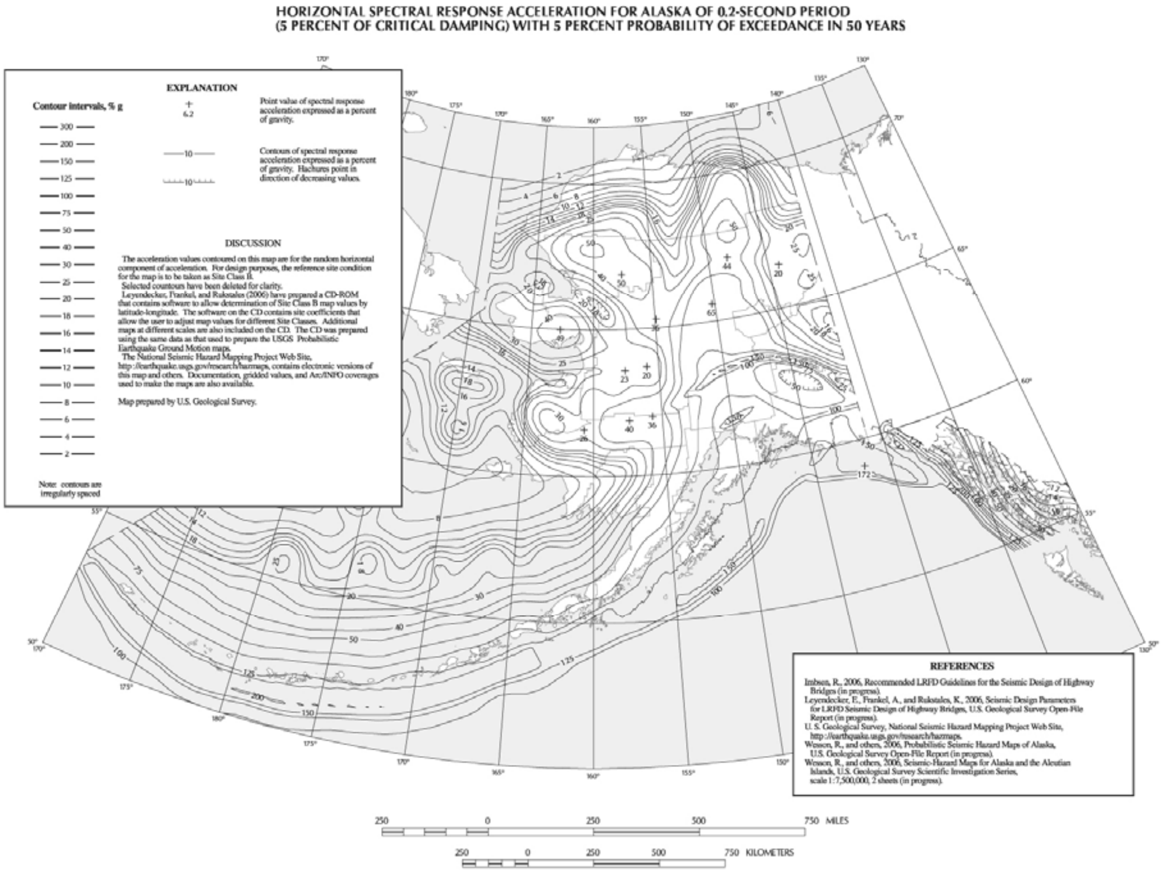


FIGURE 3.4.1-17: Alaska 0.2 Sec.

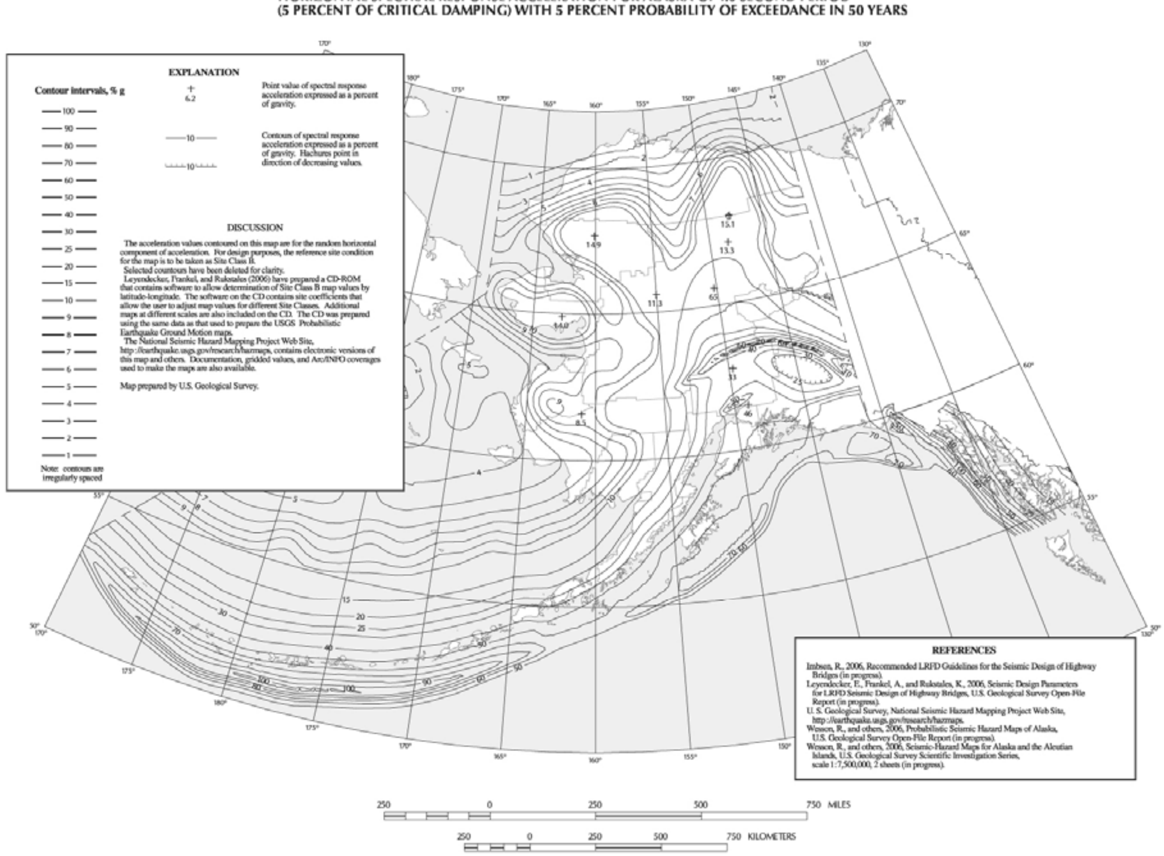


FIGURE 3.4.1-18: Alaska 1.0 Sec.

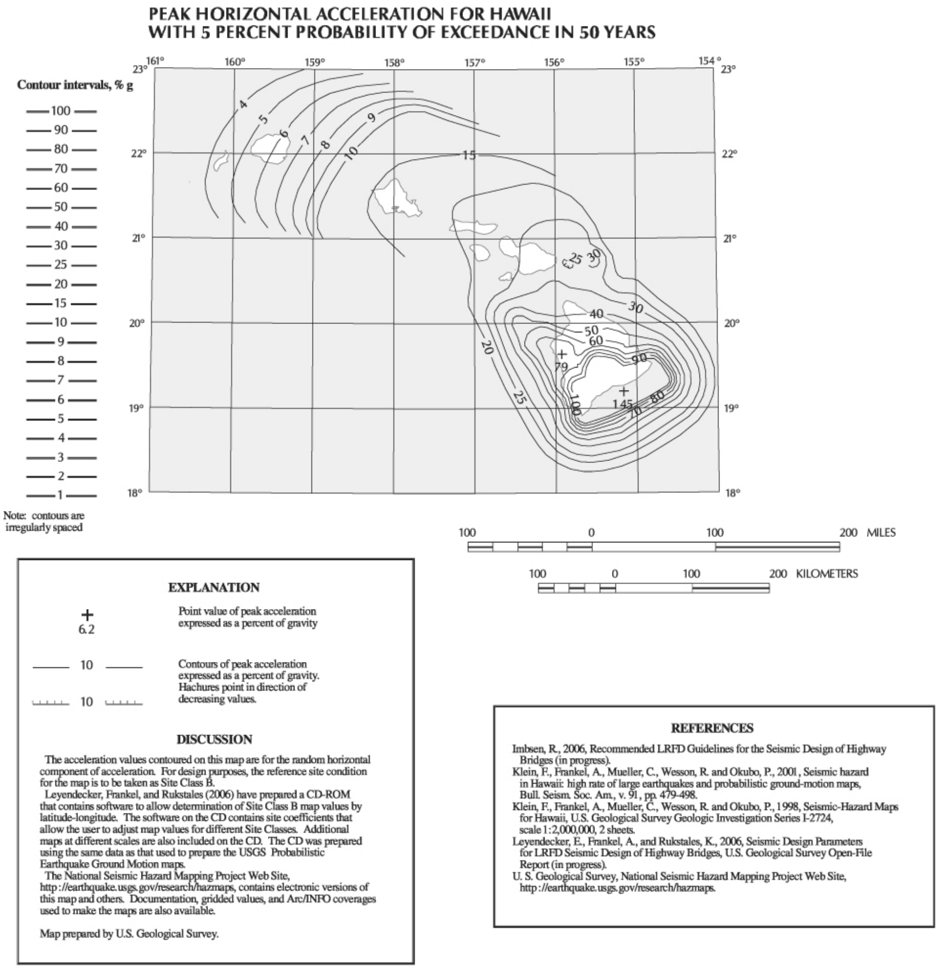
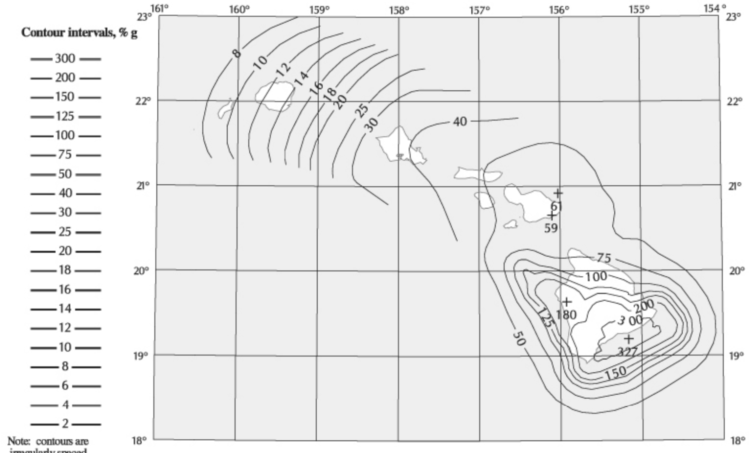
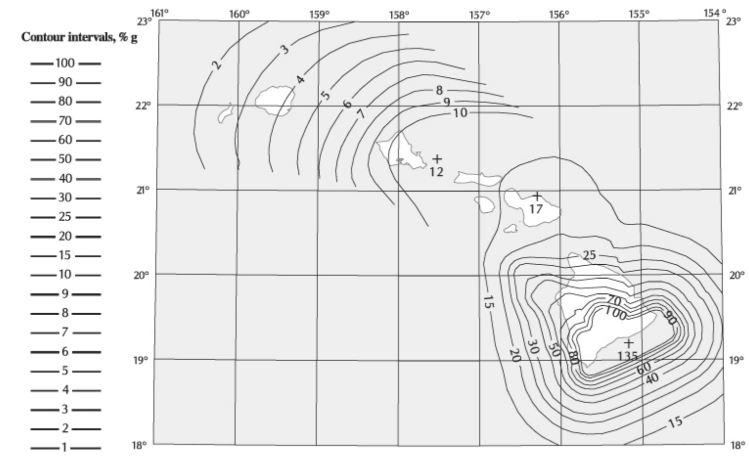


FIGURE 3.4.1-19: Hawaii PGA

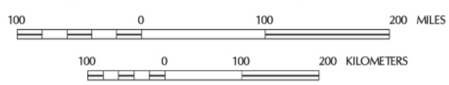
HORIZONTAL SPECTRAL RESPONSE ACCELERATION FOR HAWAII OF 0.2- AND 1.0-SECOND PERIOD (5 PERCENT OF CRITICAL DAMPING) WITH 5 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS



0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)



1.0-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)



EXPLANATION

+ 6.2 — Point value of spectral response acceleration expressed as a percent of gravity.

—— 10 —— Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.

DISCUSSION

The acceleration values contoured on this map are for the random horizontal component of acceleration. For design purposes, the reference site condition for the map is to be taken as Site Class B.

Leyendecker, Frankel, and Rukstales (2006) have prepared a CD-ROM that contains software to allow determination of Site Class B map values by latitude-longitude. The software on the CD contains site coefficients that allow the user to adjust map values for different Site Classes. Additional maps at different scales are also included on the CD. The CD was prepared using the same data as that used to prepare the USGS Probabilistic Earthquake Ground Motion maps.

The National Seismic Hazard Mapping Project Web Site, http://earthquake.usgs.gov/research/hazmaps, contains electronic versions of this map and others. Documentation, gridded values, and Arc/INFO coverages used to make the maps are also available.

Map prepared by U.S. Geological Survey.

REFERENCES

Imbsen, R., 2006, Recommended LRFD Guidelines for the Seismic Design of Highway Bridges (in progress).

Klein, F., Frankel, A., Mueller, C., Wesson, R. and Okubo, P., 2001, Seismic hazard in Hawaii: high rate of large earthquakes and probabilistic ground-motion maps, Bull. Seism. Soc. Am., v. 91, pp. 479-498.

Klein, F., Frankel, A., Mueller, C., Wesson, R. and Okubo, P., 1998, Seismic-Hazard Maps for Hawaii, U.S. Geological Survey Geologic Investigation Series I-2724, scale 1:2,000,000, 2 sheets.

Leyendecker, E., Frankel, A., and Rukstales, K., 2006, Seismic Design Parameters for LRFD Seismic Design of Highway Bridges, U.S. Geological Survey Open-File Report (in progress).

U. S. Geological Survey, National Seismic Hazard Mapping Project Web Site, http://earthquake.usgs.gov/research/hazmaps.

FIGURE 3.4.1-20: Hawaii 0.2 Sec. and 1.0 Sec.

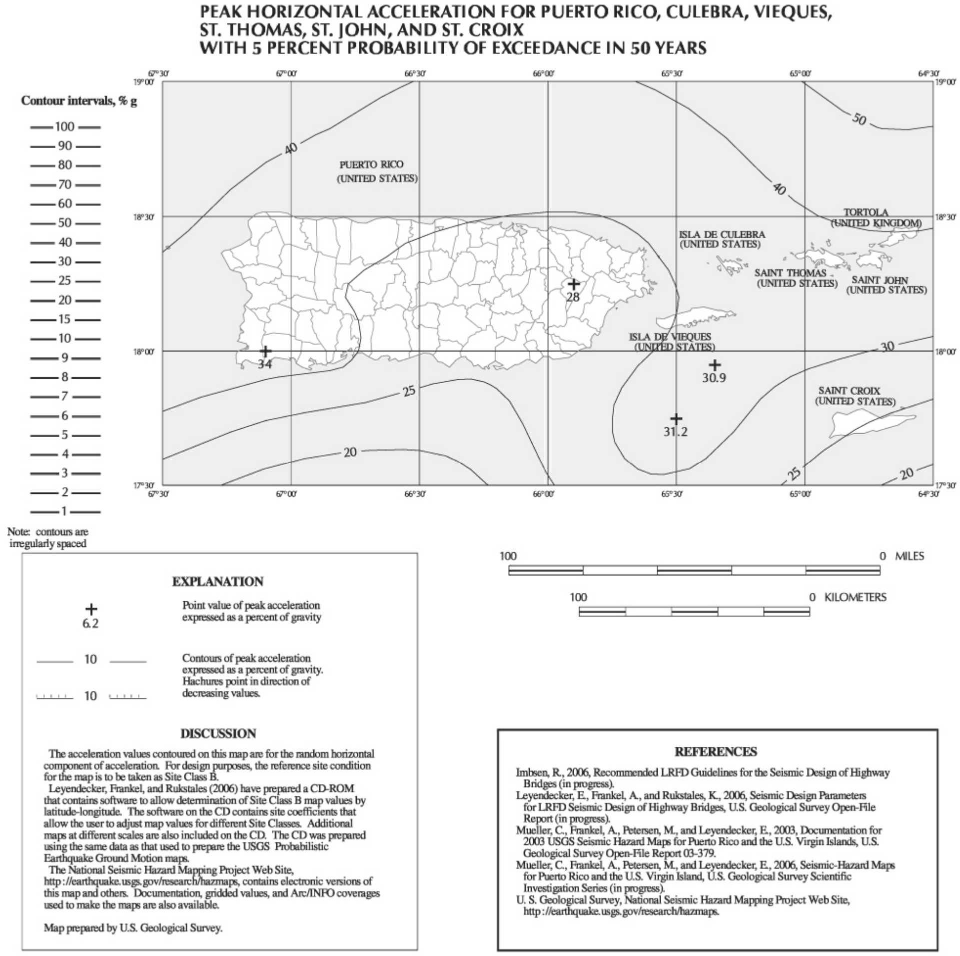
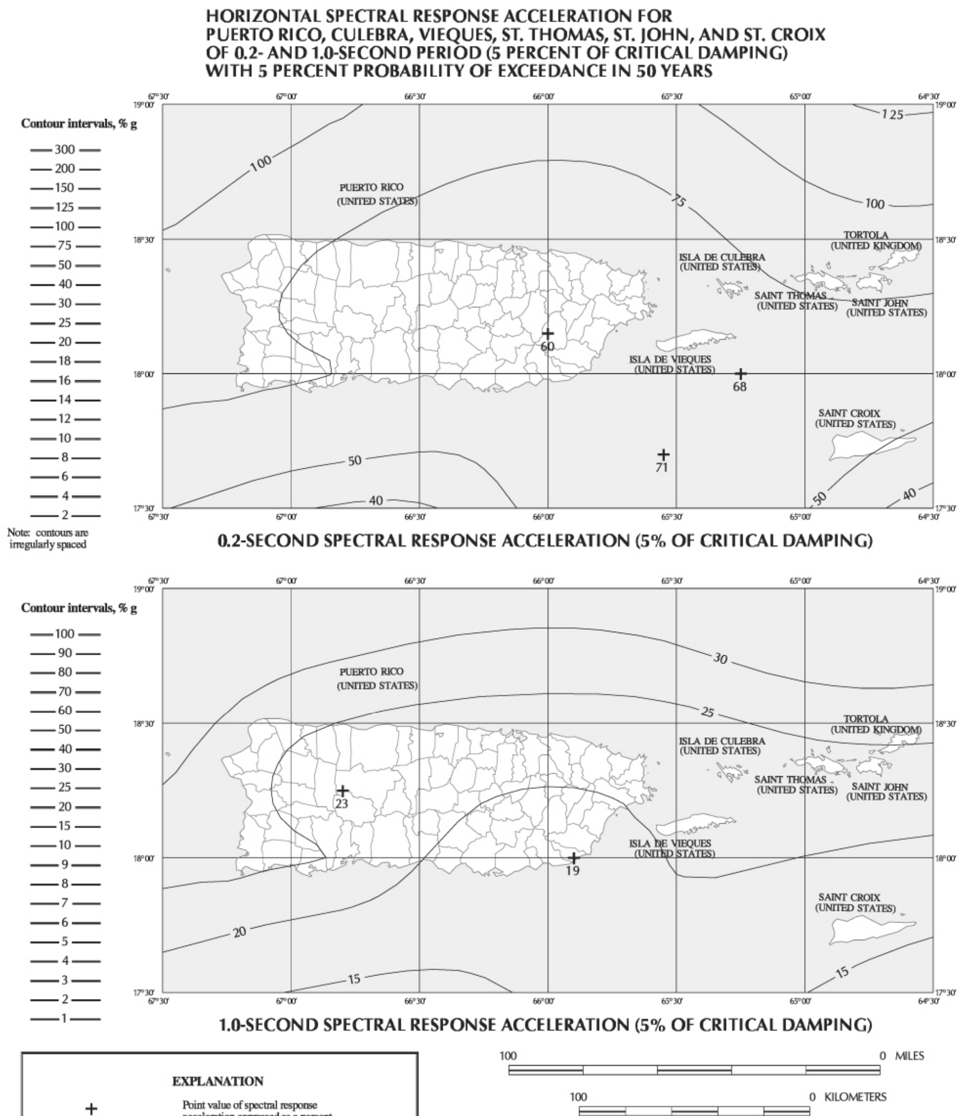


FIGURE 3.4.1-21: Puerto Rico, Culebra, Vieques, St. Thomas, St. John, and St. Croix PGA



EXPLANATION

+ 6.2 — Point value of spectral response acceleration expressed as a percent of gravity.

10 — Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.

DISCUSSION

The acceleration values contoured on this map are for the random horizontal component of acceleration. For design purposes, the reference site condition for the map is to be taken as Site Class B.

Leyendecker, Frankel, and Rukstales (2006) have prepared a CD-ROM that contains software to allow determination of Site Class B map values by latitude-longitude. The software on the CD contains site coefficients that allow the user to adjust map values for different Site Classes. Additional maps at different scales are also included on the CD. The CD was prepared using the same data as that used to prepare the USGS Probabilistic Earthquake Ground Motion maps.

The National Seismic Hazard Mapping Project Web Site, http://earthquake.usgs.gov/research/hazmaps, contains electronic versions of this map and others. Documentation, gridded values, and Arc/INFO coverages used to make the maps are also available.

Map prepared by U.S. Geological Survey.

REFERENCES

Imbsen, R., 2006, Recommended LRFD Guidelines for the Seismic Design of Highway Bridges (in progress).
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Mueller, C., Frankel, A., Petersen, M., and Leyendecker, E., 2006, Seismic-Hazard Maps for Puerto Rico and the U.S. Virgin Island, U.S. Geological Survey Scientific Investigation Series (in progress).
U. S. Geological Survey, National Seismic Hazard Mapping Project Web Site, http://earthquake.usgs.gov/research/hazmaps.

FIGURE 3.4.1-22: Puerto Rico, Culebra, Vieques, St. Thomas, St. John, and St. Croix 0.2 Sec. and 1.0 Sec.

4. ANALYSIS AND DESIGN REQUIREMENTS

4.1 GENERAL

The requirements of this chapter shall control the selection and method of seismic analysis and design of bridges. The seismic design demand displacements shall be determined in accordance with the procedures of Section 5. Material and foundation design requirements are given in Sections 6, 7, and 8.

Seismic design requirements for single span bridges are given in Sections 4.5 and 4.12. Design requirements for bridges classified as SDC A are given in Sections 4.6 and 4.12. Detailed seismic analysis is not required for a single span bridge or for bridges classified as SDC A.

Sections 4.1.1, 4.1.2, 4.1.3 and 4.1.4 include recommendations, which should be considered for SDC D. The recommendations are based on past experience and if satisfied will typically yield preferred seismic performance.

4.1.1 Balanced Stiffness SDC D

It is recommended that the ratio of effective stiffness, as shown in Figure 4.1, between any two bents within a frame or between any two columns within a bent shall satisfy Equation 4.1. It is also strongly recommended that the ratio of effective stiffness between adjacent bents within a frame or between adjacent columns within a bent satisfy Equation 4.2. An increase in mass along the length of the frame should be accompanied by a reasonable increase in stiffness. For variable width frames the tributary mass supported by each bent or column shall be included in the stiffness comparisons as specified in Equations 4.1b and 4.2b.

<u>Constant Width Frames</u> <u>Variable Width Frames</u>

$$\frac{k_i^e}{k_j^e} \geq 0.5 \quad (4.1a) \qquad \frac{k_i^e \times m_j}{k_j^e \times m_i} \geq 0.5 \quad (4.1b)$$

$$\frac{k_i^e}{k_j^e} \geq 0.75 \quad (4.2a) \qquad \frac{k_i^e \times m_j}{k_j^e \times m_i} \geq 0.75 \quad (4.2b)$$

where

k_i^e = The smaller effective bent or column stiffness

C4.1.1 Balanced Stiffness

The distributions of stiffness and mass are included in the model for dynamic analysis. The discretization of the model shall account for geometric and material variation in stiffness and mass. Most of the mass of a bridge is in the superstructure. Four to five elements per span are generally sufficient to represent the mass and stiffness distribution of the superstructure. For spine models of the superstructure, the line of elements shall be located at the locus of the mass centroid. Rigid links can be used to represent the geometric location of mass relative to the spine elements in the model.

For single-column piers, C-bents, or unusual pier configurations, the rotational mass moment of inertia of the superstructure about the longitudinal axis shall be included.

The inertia of live loads need not be included in the seismic analysis. However, the probability of a large live load being on the bridge during an earthquake shall be considered when designing bridges with high live-to-dead-load ratios that are located in metropolitan areas where traffic congestion is likely to occur.

m_i = Tributary mass of column or bent (i)

k_j^e = The larger effective bent or column stiffness

m_j = Tributary mass of column or bent (j)

The following considerations shall be taken into account when calculating effective stiffness of concrete components: framing effects, end conditions, column height, percentage of longitudinal and transverse column steel, column diameter, and foundation flexibility. Some of the consequences of not meeting the relative stiffness recommendations defined above include:

- Increased damage in the stiffer elements
- An unbalanced distribution of inelastic response throughout the structure
- Increased column torsion generated by rigid body rotation of the superstructure

4.1.2 Balanced Frame Geometry SDC D

It is recommended that the ratio of fundamental periods of vibration for adjacent frames in the longitudinal and transverse direction satisfy Equation 4.3.

$$\frac{T_i}{T_j} \geq 0.7 \tag{4.3}$$

where

T_i = Natural period of the less flexible frame

T_j = Natural period of the more flexible frame

The consequences of not meeting the fundamental period requirements of Equation 4.3 include a greater likelihood of out-of-phase response between adjacent frames leading to large relative displacements between the frames that increase the probability of longitudinal unseating and pounding between frames at the expansion joints. The pounding and relative transverse translation of adjacent frames will transfer the seismic demand from one frame to the next, which can be detrimental to the stand-alone capacity of the frame receiving the additional seismic demand.

4.1.3 Adjusting Dynamic Characteristics

The following list of techniques should be considered for adjusting or tuning the fundamental

C4.1.2 Balanced Frame Geometry

For bridges with multiple frames, which are separated by expansion bearings or hinges, it is unnecessary to model and analyze the entire bridge for seismic loads. Each frame shall have sufficient strength to resist inertia loads from the mass of the frame. However, when adjacent frames have large differences in vibration period, the frame with the longer period may increase the seismic load on the frame with the shorter period by impact across the bearing or hinge, or by transverse forces through shear keys. To account for these effects, the number of frames included in a model depends on the ratio of vibration period of the frames. For bridges in which the period ratio of adjacent frames is less than 0.70 (shortest period frame divided by longest period frame), it is recommended to limit a model to five frames. The first and fifth frames in the model are considered to be boundary frames, representing the interaction with the remainder of the structure. The response of the three interior frames can be used for design of those frames. For a bridge with more than five frames, several different models are then used in the design. For bridges with period ratios of frames between 0.70 and 1.0, fewer than five frames may be used in a model.

C4.1.2 Minimum Seat Width

Unseating of girders at abutments and piers must be avoided in all circumstances. The current period of vibration and/or stiffness to satisfy Equations 4.1, 4.2, and 4.3.

- Use of oversized pile shafts
- Adjust effective column lengths (i.e. lower footings, isolation casing)
- Use of modified end fixities
- Reduce and/or redistribute superstructure mass
- Vary the column cross section and longitudinal reinforcement ratios
- Add or relocate columns
- Modify the hinge/expansion joint layout
- Incorporate isolation bearings or dampers (i.e., response modification devices)
- Rearticulation

A careful evaluation of the local ductility demands and capacities is required for SDC D, if project constraints make it impractical to satisfy the stiffness and structure period requirements in Equations 4.1, 4.2, and 4.3.

4.1.4 End Span Considerations

The influence of the superstructure rigidity on the transverse stiffness of single column bents near the abutment shall be considered. This is particularly important when calculating shear demands for single columns where considering single curvature of the column is deemed non-conservative for ensuring adequate shear capacity.

AASHTO Division I-A requirement for minimum seat width is:

$$N = 0.20 + 0.0017L + 0.0067H$$

for seismic performance categories A and B. The seat width is multiplied by 1.5 for SPC C and SPC D. The seat width is further multiplied by 1/cos□ to account for skew effects. The Division I-A expression gives reasonable minimum seat widths, but it is modified herein for higher seismic zones.

The requirement for minimum seat width accounts for (1) relative displacement due to out-of-phase ground motion of the piers, (2) rotation of pier footings, and (3) longitudinal and transverse deformation of the pier.

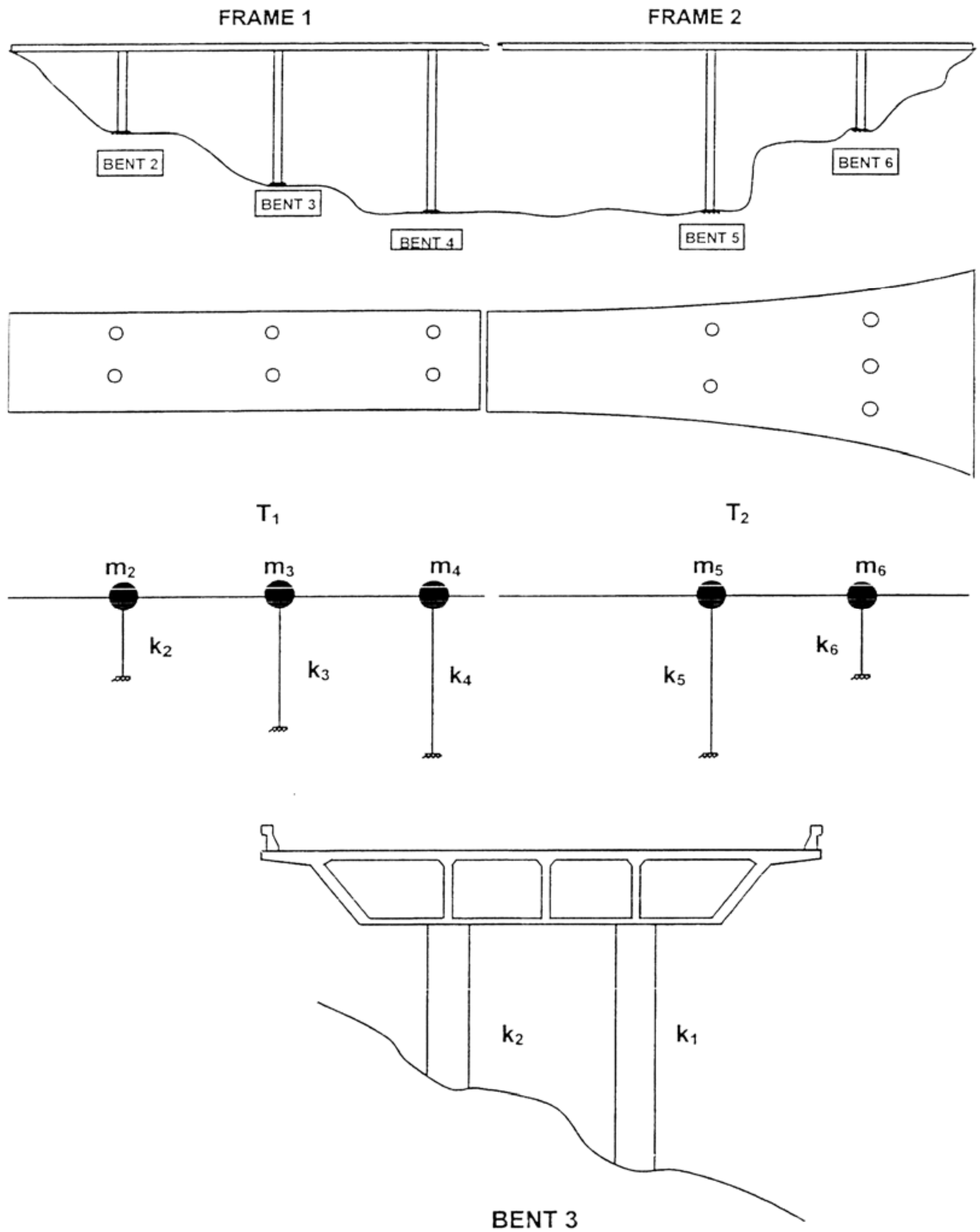


FIGURE 4.1 Balanced Stiffness

4.2 SELECTION OF ANALYSIS PROCEDURE TO DETERMINE SEISMIC DEMANDS

Minimum requirements for the selection of an analysis method to determine seismic demands for a particular bridge type are given in Table 4.1. Applicability is determined by the “regularity” of a bridge which is a function of the number of spans and the distribution of weight and stiffness. Regular bridges are defined as those having less than seven spans, no abrupt or unusual changes in weight, stiffness, or geometry. The changes in these parameters should be within the tolerances given by Equations 4.1 and 4.2 from span-to-span or from support-to-support (abutments excluded). Regular bridge requirements are defined in Table 4.2. Any bridge not satisfying the requirements of Table 4.2 is considered to be “not regular”.

Table 4.1 Analysis Procedures

Seismic Design Category	Regular Bridges with 2 through 6 Spans	Not Regular Bridges with 2 or more Spans
A	Not required	Not required
B, C, or D	Use Procedure 1 or 2	Use Procedure 2

Details of the Analytical model and Procedures mentioned in Table 4.1 are provided in Section 5.

The analysis procedures to be used are as follows:

Procedure Number	Description	Section
1	Equivalent Static	5.4.2
2	Elastic Dynamic Analysis	5.4.3
3	Non-linear Time History	5.4.4

Procedure 3 is generally not repaired unless requested by the Owner under Section 4.2.2.

4.2.1 Special Requirements for Curved Bridges

A curved bridge may be analyzed as if it were straight provided all of the following requirements are satisfied:

(a) the bridge is regular as defined in Table 4.2 except that for a two-span bridge the maximum span length ratio from span-to-span must not exceed 2;

(b) the subtended angle in plan is not greater than 90°, and

(c) the span lengths of the equivalent straight bridge are equal to the arc lengths of the curved bridge.

C4.2 SELECTION OF ANALYSIS PROCEDURE TO DETERMINE SEISMIC DEMANDS

Requirements for single span bridges are not as rigorous as for multi-span bridges because of their favorable response to seismic loads in past earthquakes. Thus, single span bridges need not be analyzed for seismic loads, regardless of the seismic hazard. Design requirements are limited to minimum seat widths and connection forces. Adequate seat widths must be provided in both the transverse and longitudinal directions. Connection forces are based on the premise that the bridge is stiff and that the fundamental period of response is short. This assumption acknowledges the fact that the period of vibration is difficult to calculate because of the significant influence of the soil structure interaction at the abutments.

These reduced requirements are also based on the assumption that there are no vulnerable substructures (i.e., no columns) and that a rigid (or near rigid) superstructure is in place to distribute the in-plane loads to the abutments. If, however, the superstructure is not able to act as a stiff diaphragm and sustain~~s~~ significant in-plane deformation during horizontal loading, it should be analyzed for these loads and designed accordingly. Single span trusses may be sensitive to in-plane loads. In this case, the designer should to take additional precautions to ensure the safety of truss superstructures.

In areas of low seismicity, only minimum seat widths (Article 6.3)~~,~~and minimum connection design forces for bearings are deemed necessary for the life-safety performance level. These default values are used as minimum design forces in lieu of rigorous analysis and are intended to provide adequate functionality for low intensity earthquakes.

This article describes the minimum connection force that must be transferred from the superstructure to its supporting substructures through the bearings. It does not apply if the connection is a monolithic structural joint. Similarly, it does not apply to unrestrained bearings or in the unrestrained directions of bearings that are free to move (slide) in one direction but fixed (restrained) in an orthogonal direction. The minimum force is simply 0.2 times the dead load reaction force in the restrained directions.

It is important that not only the bearing but also the details that fasten the bearing to the sole and masonry plates (including the anchor bolts which engage the supporting members), have sufficient capacity to resist the inertia forces being transferred to the substructure. At a fixed bearing, it is necessary to consider the simultaneous application of the

If these requirements are not satisfied, then curved bridges must be analyzed using the actual curved geometry.

Table 4.2 Regular Bridge Requirements

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle (curved bridge)	90º	90º	90º	90º	90º
Maximum span length ratio from span-to-span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span-to-span (excluding abutments)	-	4	4	3	2

Note: All ratios expressed in terms of the smaller value.

4.2.2 Limitations and Special Requirements

More rigorous methods of analysis are required for certain classes of important bridges which are considered to be critical or essential structures, and/or for those that are geometrically complex or close to active earthquake faults (see Section 3.4.3). Critical and Essential Bridges are not specifically addressed in this specification. Procedure 3, Non-linear Time History Analyses are generally recommended for these bridges as approved by the owner. There are however, some cases, where seismic isolation is used for Normal Bridges, which requires the use of nonlinear time history analysis. Nonlinear time history methods of analysis are described in Section 5 of the specifications.

For a bridge to be classified as an Essential Bridge or a Critical Bridge, one or more of the following items must be present: (1) bridge is required to provide secondary life safety, (2) sufficient time for restoration of functionality after closure creates a major economic impact, and (3) the bridge is formally designated as critical for a defined local emergency plan.

A bridge is classified as Critical, Essential or Normal as follows:

Critical Bridges: Bridges that must be open to all traffic once inspected after the design earthquake and be usable by emergency vehicles and for security/defense purposes immediately after the safety evaluation design earthquake.

Essential Bridges: Bridges that should, as a minimum, be open to emergency vehicles and for security/defense purposes after the design earthquake longitudinal and transverse connection forces when checking these capacities.

The primary purpose of this requirement is to ensure that the connections between the superstructure and its supporting substructures remain intact during a low intensity earthquake. In isolated cases where girders are vulnerable to unseating of high profile bearings, the full design earthquake forces in the bearings shall be used for design in order to protect the girders from unseating. The failure of these connections has been observed in many earthquakes and imposing strength requirements is considered to be a simple but effective strategy to minimize the risk of collapse. However, in areas of low seismic hazard it is not necessary to design the substructures or their foundations for these forces since it is expected that if a column does yield it will have sufficient inherent ductility to survive without collapse.

A common practice is to define the "longitudinal direction" of a curved bridge as that of the chord connecting the ends of the bridge, and the transverse direction as orthogonal to the longitudinal direction.

Essential or Critical Bridges within 6 miles of an active fault require a site-specific study and inclusion of vertical ground motion in the seismic analysis. For normal bridges located within 6 miles from an active fault, the procedures in Article X.X are used to account for the response to vertical ground motion in lieu of including the vertical component in the seismic analysis. For bridges with long, flexible spans, C-bents, or other large eccentricity in the load path for vertical loads, it is recommended to include vertical ground motion in the dynamic analysis.

and open to all traffic within days after that event.

Normal Bridges: Any bridge not classified as a Critical or Essential Bridge

4.3 DETERMINATION OF SEISMIC LATERAL DISPLACEMENTS DEMANDS

The global structure displacement demand, Δ_D, is the total seismic displacement at a particular location within the structure or subsystem. The global displacement demand will include components attributed to foundation flexibility, Δ_f (i.e. foundation rotation or translation), flexibility of essentially elastic components such as bent caps Δ_b, and the flexibility attributed to elastic and inelastic response of ductile members Δ_y and Δ_{pd}, respectively.

Minimum requirements for superstructure, abutment, and foundation modeling are specified in Section 5.

4.3.1 Horizontal Ground Motions

For bridges classified as SDC B, C or D the global seismic displacement demands, Δ_D, shall be determined independently along two perpendicular axes by the use of the analysis procedure specified in Section 4.2 and as modified using Article 4.3.2 and 4.3.3. The resulting displacements shall then be combined as specified in Section 4.4. Typically, the perpendicular axes are the longitudinal and transverse axes of the bridge. The longitudinal axis of a curved bridge may be selected along a chord connecting the two abutments.

4.3.2 Displacement Modification For Other Than 5% Damped Bridges

Damping ratios on the order of 10% can be used for bridges that are substantially influenced by energy dissipation of the soils at the abutments and are expected to respond predominately as a single-degree-of-freedom system. A reduction factor, R_D can be applied to the 5% damped design spectrum coefficient used to calculate the displacement demand.

The following characteristics are typically good indicators that higher damping is justified.

- Total bridge length is less than 300 feet.

C4.3 SDC B

These provisions provide the designers of regular bridges (that comply with certain restrictions) with the ability to design a structure without the need to undertake a complete dynamic analysis. An equivalent static analysis can be performed to establish the overall displacement demands on the bridge structure. The bridge is first designed for all non-seismic requirements and then column displacement capacity (deformability) and shear capacity are checked against displacement and shear demands. The superstructure displacements anticipated in these low seismicity zones are expected to be relatively modest and significant abutment contribution to the response of the bridge is not anticipated but if it occurs it will reduce substructure displacements.

Structures with low axial loads or strong columns (i.e., more steel and large column and pile sizes) have a greater intrinsic strength and are able to resist the design ground motions with less damage. However, deformability of columns and capacity protection against shear vulnerability are provided in accordance with Articles X.X and X.X.

The use of an equivalent static analysis (Procedure 1) does not apply for bridges whose piers have different heights because one or more piers will attract significantly more lateral load. Designers are encouraged to design the portion of piers participating in a seismic mechanism to have similar column lengths.

Variable span lengths can also create uneven loading conditions on the piers resulting from unusual modal behavior and are therefore not permitted.

For highly skewed and curved bridges, biaxial loadings on the piers have problems from a design point of view and hence this method is not applicable. Moreover, extra care needs to be taken in assessing the displacement demands at joints and bearings.

C4.3.2 Displacement Modification for Other than 5% Damped Bridges

Damping may be neglected in the calculation of natural frequencies and associated modal displacements. The effects of damping shall be considered when the dynamic response for seismic loads is considered. The specified ground motion spectra are for 5% viscous damping and this is a reasonably conservative value.

- Abutments are designed for sustained soil mobilization.
- Supports are normal or slight skew (less than 20 degrees).
- The superstructure is continuous without hinges or expansion joints.

$$R_D = \left(\frac{0.05}{\xi}\right)^{0.4} \qquad (4.4)$$

where

ξ = damping ratio (maximum of 0.1)

End diaphragm and rigid frame abutments typically are effective in mobilizing the surrounding soil. However, abutments that are designed to fuse (seat type) or respond in a flexible manner may not develop enough sustained structure-soil interaction to rely on the higher damping ratio. The displacement demands for bridges with abutments designed to fuse shall be based on a 5% damped spectrum curve unless the abutments are specifically designed for sustained soil mobilization.

4.3.3 Displacement Magnification For Short Period Structures

Displacement demand, Δ_D, calculated from elastic analysis shall be multiplied by the factor R_d obtained from Equation 4.5 to obtain the design displacement demand specified in Article 4.3. This magnification applies in cases where the fundamental period of the structure T is less than the characteristic ground motion period T^*, corresponding to the peak energy input spectrum.

Values T^* are given in Table 4.3.

$$R_d = \left(1 - \frac{1}{R}\right)\frac{T^*}{T} + \frac{1}{R} \geq 1 \quad \text{For} \quad \frac{T^*}{T} \geq 1 \qquad (4.5a)$$

$$R_d = 1 \qquad \text{For} \quad \frac{T^*}{T} \leq 1 \qquad (4.5b)$$

The value of R_d used shall be taken based on the maximum value of R expected in the design of the subject bridge. The displacement magnification is applied separately in both orthogonal directions prior to obtaining the orthogonal combination of seismic displacements specified in Article 4.4. For SDC D this value can be obtained by dividing the

Suitable damping values may be obtained from field measurement of induced free vibration or by forced vibration tests. In lieu of measurements, the following values may be used for the equivalent viscous damping ratio of time-history analysis:

- Concrete construction: 5%
- Welded and bolted steel construction: 2%

For single-span bridges or two-span continuous bridges with abutments designed to activate significant passive pressure in the longitudinal direction, a damping ratio of up to 10% may be used.

Equivalent viscous damping may be considered to represent the energy dissipation due to cyclic loading of yielding members. Equivalent damping shall only be used with a secant stiffness estimate for the entire structure. For single-degree-of-freedom models the equivalence can be established within a satisfactory degree of accuracy. For bridges with seismic isolation or other seismic protection components, the equivalence is established in an approximate manner. Equivalent viscous damping shall not be used to represent inelastic energy dissipation for any other model or method of dynamic analysis.

A suitable modification of the 5% response spectrum is to divide the spectrum by:

$$\left(\frac{\beta}{5}\right)^{0.3}$$

for vibration periods greater than T_s, and divide by

$$\left(\frac{\beta}{5}\right)^{0.5}$$

for vibration periods less than or equal to T_s, where β % is the damping ratio, capped at 30%.

Member forces and displacements obtained using the CQC combination method are generally adequate for most bridge systems.

If the CQC method is not readily available, alternative methods include the square root of the sum of the squares method (SRSS), but this method is best suited for combining responses from modes with well-separated frequencies. For closely spaced modes, the absolute sum of the modal responses shall be used.

spectral force corresponding to the orthogonal combination specified in Article 4.4 by the plastic capacity of the bridge component where plastic hinging is expected. For SDC B and C, the value of R may be taken equal to 2 and 3 respectively.

Table 4.3 Values of Characteristic Ground Motion Period, T*

Values of T* (in seconds)												
	M_W=6.5±0.25				M_W=7.25±0.25				M_W=8.0±0.25			
$0.4S_S$ (g)	Class B	Class C	Class D	Class E	Class B	Class C	Class D	Class E	Class B	Class C	Class D	Class E
0.1	0.32	0.45	0.46	0.44	0.41	0.53	0.56	0.56	0.51	0.69	0.71	0.71
0.2	0.37	0.44	0.49	0.64	0.42	0.53	0.55	0.74	0.47	0.61	0.65	0.85
0.3	0.35	0.43	0.50	0.73	0.38	0.51	0.55	0.76	0.48	0.64	0.65	0.98
0.4	0.39	0.47	0.50	0.87	0.42	0.56	0.59	0.93	0.46	0.62	0.66	1.04
0.5	0.37	0.46	0.50	-	0.42	0.53	0.62	-	0.45	0.59	0.70	-
0.6	0.35	0.44	0.50	-	0.43	0.54	0.64	-	0.46	0.60	0.76	-
0.7	-	-	-	-	0.50	0.66	0.76	-	0.54	0.71	0.80	-

Note: M_W is the Mean Earthquake Moment Magnitude. If M_W is not listed, then round up to the next higher M_W value that is listed.

The soil site class should be determined by the final designer's geotechnical engineer. In lieu of more definite information, the soil site class may be determined based on the mean shear wave velocity over the top 100 ft of the ground, as listed in Table 3.4.2-1.

4.4 COMBINATION OF ORTHOGONAL SEISMIC DISPLACEMENT DEMANDS

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

LOAD CASE 1: Seismic demand displacements along each of the principal axes of a member shall be obtained by adding 100% of the absolute value of the member seismic displacements resulting from the analysis in one of the perpendicular (longitudinal) directions to 30% of the absolute value of the corresponding member seismic displacements resulting from the analysis in the second perpendicular direction (transverse).

LOAD CASE 2: Seismic displacements on each of the principal axes of a member shall be obtained by adding 100% of the absolute value of the member

C4.4 COMBINATION OF ORTHOGONAL SEISMIC DISPLACEMENT DEMANDS

The combination of seismic forces computed from a response spectrum analysis has three aspects.

The first is the combination of the vibration modes due to ground motion in one direction (longitudinal, transverse, or vertical). The CQC method ("complete quadratic combination") provides a good estimate of the maximum force, including the correlation of modal responses closely spaced in frequency.

The second is the contribution of two or three orthogonal ground motion components to a single force effect. The SRSS rule ("square root sum of the squares") is the most appropriate rule for combining the contribution of orthogonal, and uncorrelated, ground motion components to a single seismic force. The SRSS method is recommended particularly for seismic analysis including vertical ground motion (Button et. al. 1999). Prior AASHTO seismic provisions were based on a 100% - 30% combination. It was decided to adopt modify this and permit a 100% - 40% combination rule as an seismic displacements resulting from the analysis in the second perpendicular direction (transverse) to 30% of the absolute value of the corresponding member seismic displacements resulting from the analysis in the first perpendicular direction (longitudinal).

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 a horizontal seismic force not less than 0.20 times the dead load reaction force. The lateral force shall be carried into the foundation in accordance with Articles 5.2 and 6.7. The minimum support lengths shall be as specified in Article 4.12.

4.6 DESIGN REQUIREMENTS FOR SEISMIC DESIGN 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 in the directions to be restrained. The minimum support length is specified in Section 4.12.

4.7 DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORIES (SDC) B, C, AND D

4.7.1 Design Methods for Lateral Seismic Displacement Demands

For design purposes, each structure shall be categorized according to its intended structural seismic response in terms of damage level (i.e., ductility demand, μ_D). The following design methods are further defined as follows:

(a) Conventional Ductile Response (i.e. Full-Ductility Structures)

For horizontal loading, a plastic mechanism is intended to develop. The plastic mechanism shall be defined clearly as part of the design strategy. Yielding may occur in areas that are not readily accessible for inspection (i.e., with owner's approval). Inelastic action is intended to be restricted to flexural plastic hinges in columns and pier walls and inelastic soil deformation behind abutment walls

alternative to the SRSS combination rule based on results of literature review considered in the scope of Task 6 of NCHRP Project 20-07.

C4.5 DESIGN REQUIREMENTS FOR SINGLE SPAN BRIDGES

Requirements for single-span bridges are not as rigorous as for multi-span bridges because of their favorable response to seismic loads in past earthquakes. As a result, single-span bridges need not be analyzed for seismic loads regardless of the SDC, and design requirements are limited to minimum seat widths and connection forces. Adequate seat widths must be provided in both the transverse and longitudinal directions. Connection forces are based on the premise that the bridge is very stiff and that the fundamental period of response will be short. This assumption acknowledges the fact that the period of vibration is difficult to calculate because of significant interaction with the abutments.

These reduced requirements are also based on the assumption that there are no vulnerable substructures (i.e., no columns) and that a rigid (or near-rigid) superstructure is in place to distribute the in-plane loads to the abutments. If, however, the superstructure is not able to act as a stiff diaphragm and sustains significant in-plane deformation during horizontal loading, it should be analyzed for these loads and designed accordingly.

Single-span trusses may be sensitive to in-plane loads and the designer may need to take additional precautions to ensure the safety of truss superstructures.

C4.7 DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORIES (SDC) B, C AND D

C4.7.1 Design Methods for Lateral Seismic Displacement Demands

A key element in the design procedure is the flexural capacity of the columns. Philosophically, the lower the flexural capacity of the column the more economic will be the seismic design because the overstrength flexural capacity of a column drives the cost and capacity of both the foundations and connections to the superstructure. For SDC B the capacity of the column designed for nonseismic loads is considered to be acceptable for this lower seismic hazard level.

For SDC C and D, the design procedure provides a trade-off between acceptable design displacements and wing walls. Details and member proportions shall ensure large ductility capacity, μ_C, under load reversals without significant strength loss with ductility demands ($4.0 \le \mu_D \le 8.0$, see Section 4.9). This response is anticipated for a bridge in SDC D designed for the Life Safety Criteria.

(b) Limited-Ductility Response

For horizontal loading, a plastic mechanism as described above for Full-Ductility Structures is intended to develop, but in this case for Limited Ductility Response ductility demands are reduced ($\mu_D \le 4.0$). Intended yielding shall be restricted to locations that are readily accessible for inspection following a design earthquake unless prohibited by the structural configuration. Inelastic action is intended to be restricted to flexural plastic hinges in columns and pier walls, and inelastic soil deformation behind abutment walls and wingwalls. Detailing and proportioning requirements are less than those required for Full-Ductility Structures. This response is anticipated for a bridge in SDC B or C designed for the Life Safety Criteria.

(c) Limited-Ductility Response in Concert with Added Protective Systems

In this case a structure has limited ductility with the additional seismic isolation, passive energy dissipating devices, and/or other mechanical devices to control seismic response. Using this strategy, a plastic mechanism may or may not form. The occurrence of a plastic mechanism shall be verified by analysis. This response may be used for a bridge in SDC C or D designed for an enhanced performance. Non-linear Time History analysis (i.e., Procedure 3) may be required for this design strategy.

4.7.2 Vertical Ground Motions, Design Requirements for SDC D

Bridges, within Seismic Design Category D located within six (6) miles of a fault, shall have at least 15% of the longitudinal top and bottom mild reinforcement continuous over the length of the bridge superstructure to account for the effects of vertical ground motions. In lieu of a more detailed analysis to account for the effects of vertical acceleration, reinforced prestressed and precast prestressed girders shall have a minimum of 15% of the total equivalent mild and prestressing steel in the form of continuous mild reinforcement. Equivalency of prestressing and mild steel is determined in terms of the strength of the reinforcement used. Service

and minimum flexural capacities of columns, which could in turn be governed by $P-\Delta$ effects.

couplers shall be used to splice the required continuous longitudinal reinforcement. Lap splicing is not permitted. The service couplers must be capable of achieving a minimum of 125% of the nominal yield strength of the continuous steel reinforcement. Vertical ground motions design requirements do not apply for steel girders. A case-by-case determination on the effect of vertical ground motions is required for essential and critical bridges.

4.8 STRUCTURE DISPLACEMENT DEMAND/CAPACITY FOR SDC B, C, AND D

For SDC B, C and D, each bridge bent shall satisfy Equation 4.6.

$$\Delta_D^L < \Delta_C^L \tag{4.6}$$

where

Δ_D^L is the displacement demand along the local principal axis of a ductile member resulting from a seismic motion applied to the total structural system according to Article 4.4.

Δ_C^L is the corresponding member displacement capacity available along the same axis as the displacement demand Δ_D^L.

Equation (4.6) shall be satisfied in each of the local axis of every bent. The local axis of a bent typically coincides with the principal axis of the columns in that bent.

The formulas presented below are1 used to obtain Δ_C^L for SDC B and C. These formulas are not intended for use with configuration of bents with struts at mid-height. A more detailed push-over analysis is required to obtain Δ_C^L for SDC D as described in Article 4.8.2. For Pier Walls a displacement demand to capacity check in the transverse direction is not warranted, provided requirements of Article 8.6.9 are satisfied.

4.8.1 Local Displacement Capacity for SDC B and C

For SDC B and C, the displacement capacity, Δ_C^L, of each bent shall be implicitly calculated respectively based on:

For SDC B

C4.8 CAPACITY DESIGN AND STRENGTH REQUIREMENTS OF MEMBERS FRAMING INTO COLUMNS

The principles of capacity design require that the strength of those members that are not part of the primary energy-dissipating system be stronger than the overstrength capacity of the primary energy-dissipating members—that is, the columns with hinges at their member ends.

The geotechnical features of foundations (i.e. soil bearing, and side friction and end bearing on piles) possess inherent ductility. At low to moderate levels of seismic input this manifests itself as minor rocking of the foundation or nominal permanent settlements which do not significantly affect the service level of the bridge.

Full capacity protection of the geotechnical features of the foundation in SDC~~AP~~ B is not required. Should the rare, large, earthquake occur, some limited ductility demand may occur in the piles and some minor rocking and permanent settlement may occur. This trade-off, compared to current practice for SPC B in the existing AASHTO provisions, is considered prudent.

C4.8 STRUCTURAL DISPLACEMENT CAPACITY FOR SDC B, C, AND D

The objective of the displacement capacity verification analysis is to determine the displacement at which the earthquake-resisting elements achieve their inelastic deformation capacity. Damage states are defined by local deformation limits, such as plastic hinge rotation, footing settlement or uplift, or abutment displacement. Displacement may be limited by loss of capacity from either degradation of strength under large inelastic deformations or P-□ effects.

For simple piers or bents, the maximum displacement capacity can be evaluated by hand calculations using the defined mechanism and the maximum allowable deformations of the plastic hinges. If interaction between axial force and moment is significant, iteration is necessary to determine the mechanism.

$$\Delta_C^L(ft) = \frac{H_o}{100} * (-1.27 * \ell n(x) - 0.32) \geq \frac{H_o}{100} \quad (4.7a)$$

For SDC C

$$\Delta_C^L(ft) = \frac{H_o}{100} * (-2.32 * \ell n(x) - 1.22) \geq \frac{H_o}{100} \quad (4.7b)$$

where

$$x = \Lambda \, {B_o}/{H_o} \quad (4.7c)$$

Λ is a fixity factor for the column equal to:

Λ = 1 for fixed-free (pinned on one end).

a. Λ = 2 for fixed top and bottom.

For a partially fixed connection on one end, interpolation between 1 and 2 is permitted.

B_o = Column Width or Diameter (ft.).

H_o = Height from top of footing to top of the column (i.e., column clear height, ft.).

For cases that do not correspond to definitions stated above, H_o is taken as the shortest distance between the point of maximum moment and the contra-flexure point for the purpose of applying Equation 4.7c only in conjunction with Λ equal to 1 in Equation 4.7c.

For bridge bents or frames that do not satisfy Equation 4.7, the designer has the option of either:

- increasing the allowable displacement capacity, Δ_C, by meeting detailing requirements of a higher SDC as described in Article 3.5, or
- increasing the displacement capacity, Δ_C, by means of changing column longitudinal and/or transverse reinforcement and applying pushover analysis for SDC D; or
- adjusting the dynamic characteristics of the bridge as described in Section 4.1 to satisfy Equation 4.7.

4.8.2 Local Displacement Capacity for SDC D (i.e., Pushover Analysis)

Inelastic Quasi-Static Pushover Analysis (IQPA), commonly referred to as "pushover" analysis, shall be used to determine the reliable displacement capacities of a structure or frame as it reaches its limit of structural stability. Displacement Capacity determined for SDC C can be used in lieu of a more elaborate pushover analysis. If the displacement demand is higher than the displacement capacity

For more complicated piers or foundations, displacement capacity can be evaluated using a nonlinear static analysis procedure (pushover analysis).

Displacement capacity verification is required for individual piers or bents. Although it is recognized that force redistribution may occur as the displacement increases, particularly for frames with piers of different stiffness and strength, the objective of the capacity verification is to determine the maximum displacement capacity of each pier. The displacement capacity is to be compared with an elastic demand analysis, which considers the effects of different stiffness. Expected material properties are used for the displacement capacity verification. Generally, the center of mass is at the elevation of the mass centroid of the superstructure.

C4.8.2 Local Displacement Capacity for SDC D

This design procedure is a key element in the philosophic development of these Guidelines. The pushover method of analysis has seen increasing use throughout the 1990s, especially in Caltrans' seismic retrofit program. This analysis method provides additional information on the expected deformation demands of columns and foundations and as such provides the designer with a greater understanding of the expected performance of the bridge. The use of the pushover method of analysis is used in two ways. First, it encouraged designers to be as liberal as possible with assessing ductility capacity. Second, it provides a mechanism to allow EREs that need the owner's approval (Article 3.3.1). The trade-off was the need for a more sophisticated analysis in order that the expected deformations in critical elements could be assessed. Provided the appropriate limits (i.e., plastic rotations for in-ground hinges) are met, the EREs requiring the owner's approval can be used. This method applies to all the EREs shown in Figure C3.3.1.1(b).

determined for SDC C one span a pushover analysis is warranted. IQPA is an incremental linear analysis, which captures the overall nonlinear behavior of the elements, including soil effects, by pushing them laterally to initiate plastic action. Each increment of loading pushes the frame laterally, through all possible stages, until the potential collapse mechanism is achieved.

Because the analytical model used in the pushover analysis accounts for the redistribution of internal actions as components respond inelastically, IQPA is expected to provide a more realistic measure of behavior than can be obtained from elastic analysis procedures.

Where foundation and superstructure flexibility can be ignored as stipulated in Article 5.3.1, the two-dimensional plane frame "pushover" analysis of a bent or a frame can be simplified to a column model (fixed-fixed or fixed-pinned) if it does not cause a significant loss in accuracy in estimating the displacement capacities.

The effect of seismic load path on the column axial load and associated member capacities must be considered in the simplified model.

4.9 MEMBER DUCTILITY REQUIREMENT FOR SDC D

Local member displacements such as column displacements, Δ_{col} are defined as the portion of global displacement attributed to the elastic column idealized displacement Δ_{yi} and plastic displacement demand Δ_{pd} of an equivalent member from the point of maximum moment to the point of contra-flexure. Member section properties are obtained from a Moment-Curvature Analysis and used to calculate Δ_{yi} and the plastic displacement capacity Δ_{pc}.

Local member ductility demand μ_D shall be computed based on the same equivalent member length as follows:

$$\mu_D = 1 + \frac{\Delta_{pd}}{\Delta_{yi}} \tag{4.8}$$

For <u>conventional ductile design</u>, the local member ductility demand shall satisfy the following:

Single Column Bents $\mu_D \leq 6$

Multi Column Bents $\mu_D \leq 8$

Pier Walls Weak Ductile $\mu_D \leq 6$

Pier Walls Strong Ductile $\mu_D \leq 1$

Pile shafts are treated similar to columns.

4.10 COLUMN SHEAR REQUIREMENT FOR SDC B, C, AND D

For SDC B, C, or D, shear design requirements for reinforced concrete columns shall be satisfied according to Article 8.6. Determination of member ductility demand is required for SDC D only as stipulated in Article 8.6.2.

4.11 CAPACITY DESIGN REQUIREMENT FOR SDC C AND D

4.11.1 Capacity Design

Capacity design principles require that those components not participating as part of the primary energy dissipating system (flexural hinging in columns), shall be capacity protected. The components include the superstructure, joints and cap beams, spread footings, pile caps and foundations. This is achieved by ensuring the maximum moment and shear from plastic hinges in the column considering overstrength can be resisted elastically by adjoining elements.

For SDC C or D, exception to capacity design is permitted for the following:

a. The seismic resisting system includes the fusing effects of an isolation device.

b. A ductile end diaphragm is incorporated into the transverse response of a supporting column (See Article 7.4.9).

c. A foundation situated in soft or potentially liquefiable soils.

4.11.2 Plastic Hinging Forces

Plastic hinges shall form before any other failure due to overstress or instability in the overall structure and/or in the foundation. Except for pile bents and drilled shafts, and with owners' approval, plastic hinges shall only be permitted at locations in columns where they can be readily inspected and/or repaired, as described in Section 3.3.

Superstructure and substructure components and their connections to columns that are designed not to yield shall be designed to resist overstrength moments and shears of ductile columns. Except for

C4.11 CAPACITY DESIGN REQUIREMENT

The objective of these Guidelines for conventional design is that inelastic deformation (plastic hinging) occurs at the location in the columns (top or bottom or both) where they can be readily inspected and repaired. To achieve this objective, all members connected to the columns, the shear capacity of the column and all members in the load path from the superstructure to the foundation, shall be capable of transmitting the maximum (overstrength) force effects developed by plastic hinges in the columns. The exceptions to the need for capacity design of connecting elements is when all substructure elements are designed elastically (Article X.X), seismic isolation design (Article X.X) and in the transverse direction of columns when a ductile diaphragm (Article X.X) is used.

C4.11.1 Capacity Design Actions

The objective of these provisions for conventional design is that inelastic deformation (plastic hinging) occurs at the location in the columns (top or bottom or both) where they can be readily inspected and repaired. To achieve this objective, all members connected to the columns, the shear capacity of the column, and all members in the load path from the superstructure to the foundation, shall be capable of transmitting the maximum (overstrength) force effects developed by plastic hinges in the columns. The exceptions to the need for capacity design of connecting elements are (1) when all substructure elements are designed elastically (Article X.X, (2) seismic isolation design (Article X.X) and (3) in the transverse direction of columns when a ductile diaphragm is used (Article X.X)

C4.11.2 Elastically Designed Elements

If all of the supporting substructure elements (columns, piers, pile bents) are designed elastically, there will be no redistribution of lateral loads due to plastic hinges developing in one or more columns. As a consequence the elastic analysis results are appropriate for design. The recommended provisions attempt to prevent any brittle modes of failure from occurring.

If only one or a selected number of supporting substructure elements are designed elastically, there will be a significant redistribution of lateral loads when one or more of the columns develops plastic hinges. Generally, the elastically designed elements will attract more lateral load. Hence the need to use the geotechnical aspects for design of foundations, the moment overstrength capacity (M_{po}) of column/pier/pile members that form part of the primary mechanism resisting seismic loads shall be assessed as follows:

- $M_{po} = \lambda_{mo} M_p$ for reinforced concrete columns where

  $\lambda_{mo} = 1.2$

  The plastic moment capacity, M_p, for reinforced concrete columns is determined using a moment-curvature section analysis; taking into account the expected yield strength of the materials, the confined concrete properties, and the strain hardening effects of the longitudinal reinforcement.

- $M_{po} = \lambda_{mo} M_n$ for steel members where

  $\lambda_{mo} = 1.2$ to 1.5 and M_n is the nominal moment strength in which expected yield strengths are used for steel members (Article 7.3) and λ_{mo} is the overstrength factor.

These overstrength moments and associated shear forces, calculated on the basis of inelastic hinging at overstrength, shall be taken as the extreme seismic forces that the bridge is capable of resisting. Typical methods of applying capacity design at a bent in the longitudinal and transverse directions are shown in Figure 4.2 and illustrated in Article 4.11.3 for single column bents and Article 4.11.4 for multi-column bents.

4.11.3 Single Columns and Piers

Column design shear forces and moments in the superstructure, bent caps, and the foundation structure 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 moment capacities. Use an overstrength factor times the plastic moment capacity or nominal moment as specified in Article 4.11.2. The nominal moment or plastic moment capacity members are calculated using the expected yield strengths and subjected to the applied dead load on the section under consideration. Column overstrength moments should be distributed to the connecting structural elements. (Exception: when calculating the design forces for the geotechnical aspects of foundations such as determining lateral capacity design principles for all elements connected to the elastically designed column. If this is not practical, the complete bridge needs to be reanalyzed using the secant stiffness of any columns in which plastic hinges will form, in order to capture the redistribution of lateral loads that will occur.

C4.11.2 Plastic Hinging Forces

The principles of capacity design require that the strength of those members that are not part of the primary energy-dissipating system should be stronger than the overstrength capacity of the primary energy-dissipating members—that is, the columns with hinges at their member ends.

When assessing overstrength capacity of flexural members using compatibility section analysis (i.e., the moment-curvature method), it is important to differentiate between overstrength resulting from the response of the section to high curvature demands, and overstrength resulting from upper-bound material properties.

For example, for reinforced concrete columns, confined concrete will have enhanced capacity and reinforcing steel will strain-harden at high plastic curvatures. This will result in increased flexural capacity of the column that will be captured by a moment-curvature analysis that considers these factors. In addition, reinforcing steel can have a higher than nominal yield point, and concrete is likely to be stronger than specified and will gain strength with age beyond the 28-day specified strength. It has been recommended that for the purpose of a rigorous calculation that f'_{co} for concrete be assumed to be $1.\cancel{7}5f'_c$ and f_{yo} of steel be $1.3f_y$. In this case the overstrength moment is taken at the design curvature from the moment-curvature analysis (ATC, 1996).

For structural steel, f_{yo} may be taken as $1.2F_{ye}$ where F_{ye} is the expected yield strength, considering the likelihood that higher-than-nominal-strength steel will be used. The plastic section modulus should be used in overstrength moment calculations for steel members.

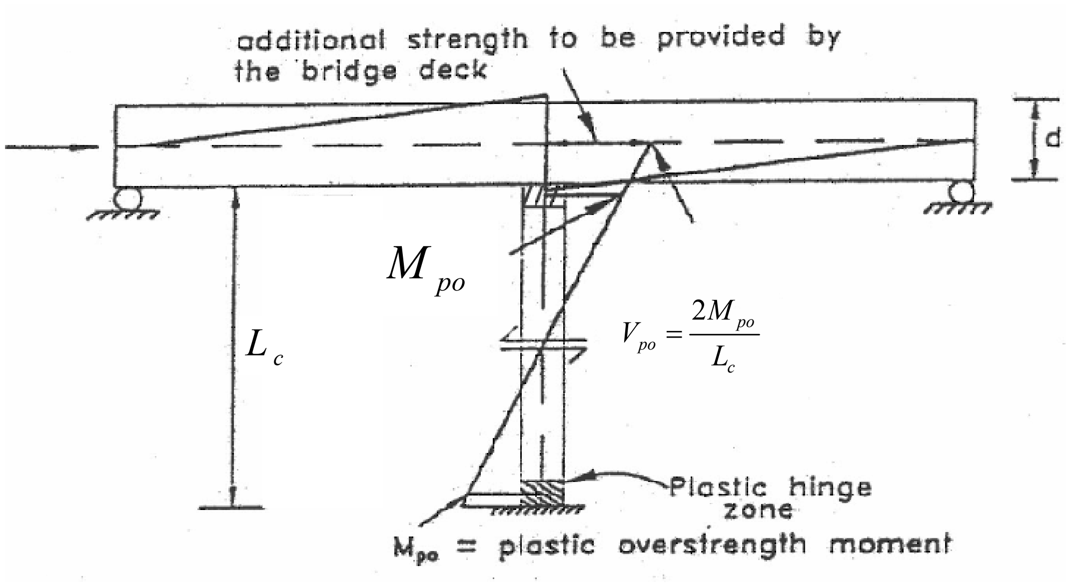
The guidelines require the calculation of capacity design shear forces for columns, pile bents, and drilled shafts at the mud or ground surface. If, however, a concrete traffic barrier or other structural element is added between these members, which effectively shortens them, then only the height above the barrier should be considered in the shear force calculation.

stability or tip elevation, use an overstrength factor of 1.0 on the nominal moment.)

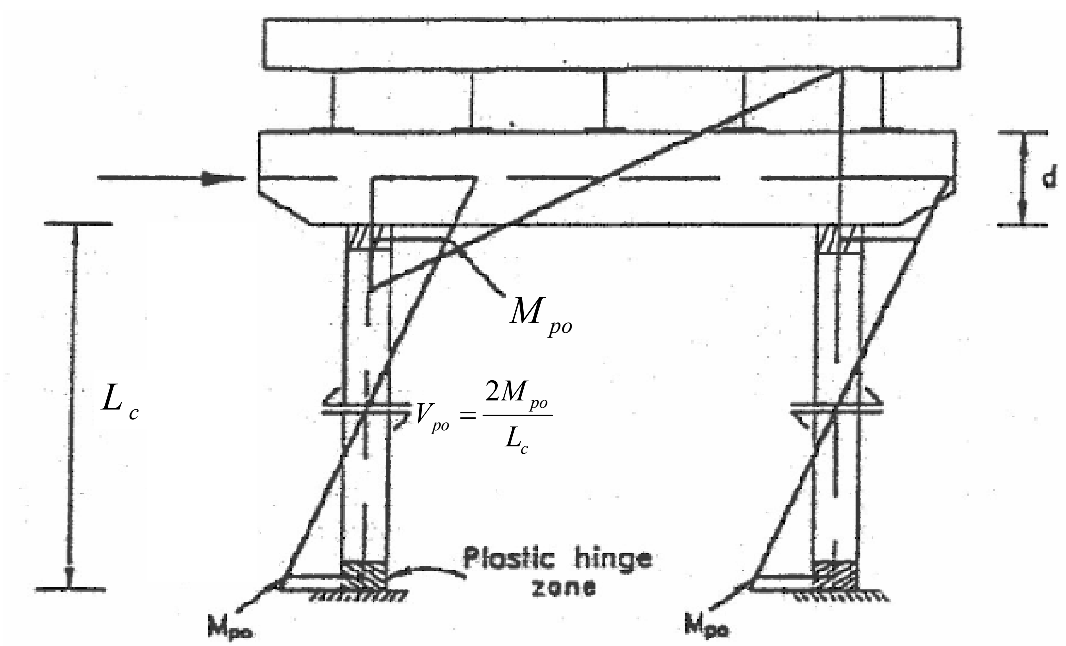
- *Step 2.* Using the column overstrength moments, calculate the corresponding column shear force assuming a quasi-static condition. For flared columns designed to be monolithic with superstructure or with isolation gaps less than required by Article 8.14, the shear shall be calculated as the greatest shear obtained from using:

  a. The overstrength moment at both the top of the flare and the top of the foundation with the appropriate column height.
  b. The overstrength moment at both the bottom of the flare and the top of the foundation with the reduced column height.

- *Step 3.* Calculate forces in the superstructure for longitudinal direction loading and forces in the foundation for both longitudinal and transverse loading.



(a) Longitudinal Response of a Concrete Bridge



(b) Transverse Response of a Concrete Bridge

FIGURE 4.2 Capacity Design of Bridges Using Overstrength Concepts

4.11.4 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 Article 4.11.3. In the plane of the bent the forces shall be calculated as follows:

- *Step 1.* Determine the column overstrength moment capacities. Use an overstrength factor times the plastic moment capacity or nominal moment as specified in Article 4.11.2. The nominal moment or plastic moment capacity for members is calculated using the expected yield strengths and subjected to the applied dead load on the section under consideration.
- *Step 2.* Using the column overstrength moments calculate the corresponding column shear forces. Sum the column shears of the bent to determine the maximum shear force for the bent. 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 Article 4.11.2 - *Step 2.*
- *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 moments are developed.
- *Step 4*. Using these column axial forces combined with the dead load axial forces, determine revised column overstrength moments. With the revised overstrength 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:

- *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*.
- *Moments*—the column overstrength plastic moments or overstrength nominal moment (Article 4.11.2) corresponding to the maximum compressive axial load specified above (in the previously bulleted item).

- *Shear Force*—the shear force corresponding to the final column overstrength moments in *Step 4* above.
- Calculate forces in the superstructure for both longitudinal and transverse direction loading and forces in the foundation for both longitudinal and transverse loading.

4.11.5 P-Δ Capacity Requirement for SDC C & D

The dynamic effects of gravity loads acting through lateral displacements shall be included in the design. The magnitude of displacements associated with P-Δ effects can only be accurately captured with non-linear time history analysis. In lieu of such analysis, P-Δ effects can be ignored if Equation 4.9 is satisfied:

$$P_{dl} \times \Delta_r \leq 0.25 \times M_P \text{ for concrete members} \quad (4.9a)$$

$$P_{dl} \times \Delta_r \leq 0.25 \times M_n \text{ for steel members} \quad (4.9b)$$

where:

Δ_r = The relative lateral offset between the point of contra-flexure and the furthest end of the plastic hinge.

For a single pile shaft where

$$\Delta_r = \Delta_D - \Delta_S \quad (4.10)$$

Δ_S = The pile shaft displacement at the point of maximum moment developed in-ground.

For a pile cap in Site Classification E, or for cases where a modal analysis shows out-of-phase movement of the bottom of the column relative to the top of the column.

$$\Delta_r = \Delta_D + \Delta_F \quad (4.11)$$

where:

Δ_F = pile cap displacement

For bridges or frames that do not satisfy Equation (4.9), the designer has the option of either:

- increasing the column plastic moment capacity M_p by adding longitudinal reinforcement; or

- adjusting the dynamic characteristics of the bridge as discussed in Article 4.1 to satisfy Equation (4.9).
- using non-linear analysis to explicitly consider $P-\Delta$ effects.

4.11.6 Analytical Plastic Hinge Length

The analytical plastic hinge length, L_p, is the equivalent length of column over which the plastic curvature is assumed constant for estimating the plastic rotation. The plastic rotation is then used to calculate the plastic displacement of an equivalent member from the point of maximum moment to the point of contra-flexure. The plastic hinge lengths may be calculated for the two following conditions described below.

(a) Columns framing into a footing, an integral bent cap, an oversized shaft, or cased shaft:

$$L_p = 0.08L + 0.15 f_{ye} d_{bl} \geq 0.3 f_{ye} d_{bl} \quad \text{(in,ksi)} \qquad (4.12)$$

(b) Non-cased Prismatic Pile Shafts:

$$L_p = D^* + 0.08H' \qquad (4.13)$$

D^* = Diameter for circular shafts or the cross section dimension in direction being considered for oblong shafts

H' = Length of pile shaft/column from point of maximum moment to point of contraflexure above ground

(c) Horizontally Isolated Flared Columns

$$L_p = G_f + 0.3 f_{ye} d_{bl} \quad (in, ksi) \qquad (4.14)$$

G_f = The gap between the isolated flare and the soffit of the bent cap

4.11.7 Reinforced Concrete Column Plastic Hinge Region

The plastic hinge region, L_{pr} defines the portion of the column, pier, or shaft that requires enhanced lateral confinement. L_{pr} is defined by the larger of:

1.5 times the gross cross-sectional dimension in the direction of bending

The region of column where the moment demand exceeds 75% of the maximum plastic moment

The analytical plastic hinge length L_p

4.11.8 Steel Column Plastic Hinge Region

In the absence of any experimental or analytical data that support the use of a plastic hinge length for a particular cross-section, the plastic hinge region length for steel column shall be the maximum of the following.

- One eighth of the clear height of a steel column
- 18 inches

4.12 MINIMUM SEAT LENGTH

Minimum bearing support length as determined in this section shall be provided for girders supported on an abutment, bent cap, pier wall, or a hinge seat within a span as shown on Figure 4.3.

4.12.1 Seismic Design Category A

Bridges classified as SDC A shall be designed to provide a minimum support length N (in) measured normal to the face of an abutment, a pier or a hinge seat, not less than specified below:

$$\mathrm{N} = \left(4 + \Delta_{ot} + 0.2H_h\right)\left(1 + \frac{S_k^2}{4000}\right) > 12" \qquad (4.15)$$

where

Δ_{ot} = movement attributed to prestress shortening creep, shrinkage and thermal expansion or contraction to be considered no less than one inch per 100 feet of bridge superstructure length from the point of no movement. (in.)

H_h = Largest column height within the most flexible frame adjacent to the expansion joint, height from top of footing to top of the column (i.e., column clear height, ft.) or equivalent column height for pile extension column (ft.). For single spans seated on abutments, the term H is taken as the abutment height (ft.).

S_k = angle of skew of support in degrees, measured from a line normal to the span.

4.12.2 Seismic Design Category B, C, and D

For seismic categories B, C or D hinge seat or support width, N, shall be available to accommodate the anticipated thermal movement, prestress shortening, creep, shrinkage, and the relative longitudinal earthquake displacement demand at the supports or at the hinge within a span between two frames as follows:

$$N = \left(4 + \Delta_{ot} + 1.65\Delta_{eq}\right)\left(1 + \frac{S_k^2}{4000}\right) \text{ (in.)} \geq 12 \quad (4.16)$$

$\Delta_{eq} =$ seismic displacement demand of the long period frame on one side of the expansion joint (in.). The elastic displacement demand is modified according to Articles 4.3.2 and 4.3.3.

Δ_{ot} and S_k are defined above in Article 4.12.1.

The skew effect multiplier $(1 + \frac{S_k^2}{4000})$ can be set equal to 1 when the global model of the superstructure is modeled to include the full width and the skew effects on the displacement demands at the outer face of the superstructure.

4.13 SUPPORT RESTRAINTS FOR SDC C AND D

Support restraints may be provided for longitudinal linkage at expansion joints within the space and at adjacent sections of simply supported superstructures. Their use is intended to achieve an enhanced performance of the expansion joint and shall be approved and satisfy the Owner requirements. For continuous superstructures spans, restrainers are considered secondary in reducing the out-of-phase motions at the expansion joints between the frames. They are used to minimize displacements (i.e. tune the out-of-phase displacement response between the frames of a multi-frame system. Restrainer units shall be designed and detailed as described in the following sections.

4.13.1 Expansion Joints within a Span

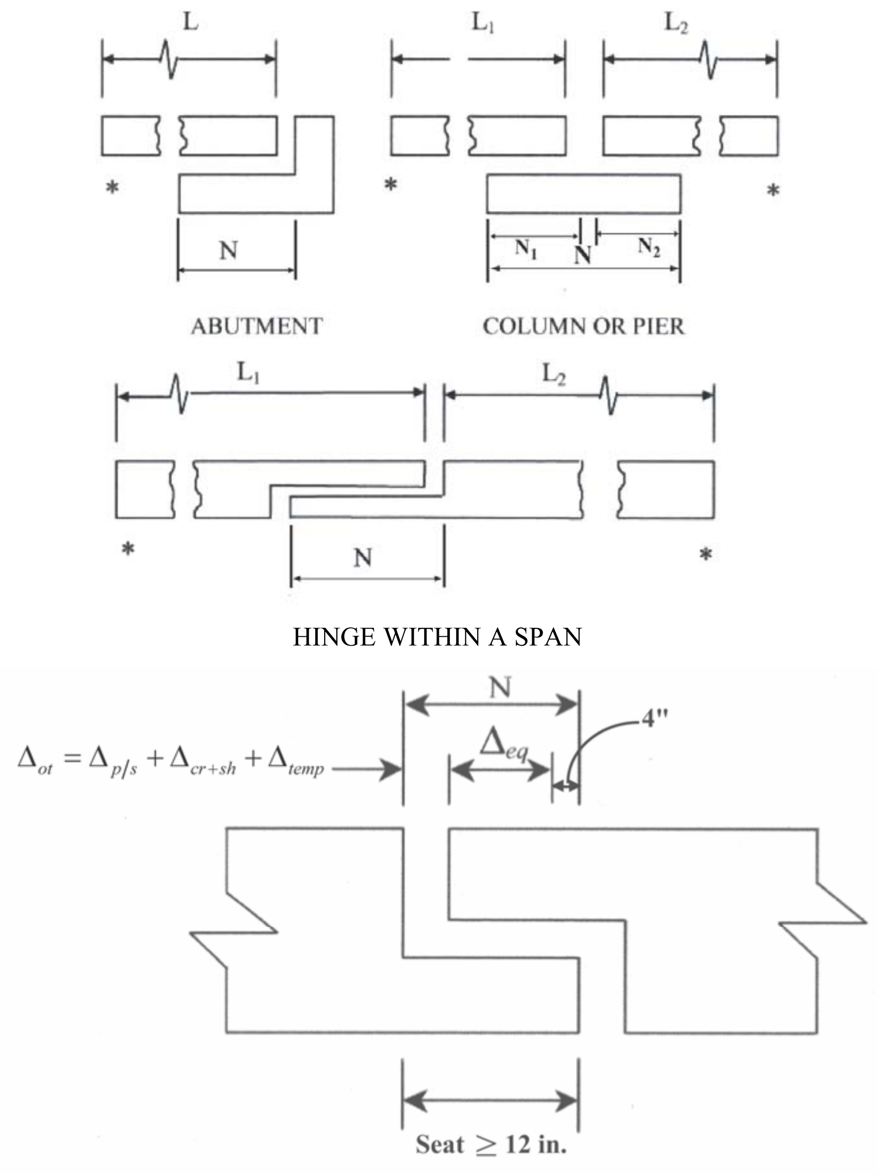
A restrainer unit with a minimum of five cables may be placed in every other cell or bay of a multi girder superstructure. A minimum of two five-cable restrainer units, placed symmetrically about the centerline of the bridge, may be used at each intermediate expansion joint hinge.

4.13.2 Simple Span Superstructures

An elastic response analysis or simple equivalent static analysis is considered adequate and reliable for the design of restrainers for simple spans. An acceleration coefficient of 0.20g shall be used as a minimum.

4.13.3 Detailing Restrainers

- Restrainers shall be detailed to allow for easy inspection and replacement.
- Restrainer layout shall be symmetrical about the centerline of the superstructure.
- Restrainer systems shall incorporate an adequate gap for service conditions.
- Yield indicators may be used on cable restrainers to facilitate post earthquake investigation.



*EXPANSION JOINT OR END OF BRIDGE DECK

FIGURE 4.3 Dimension for Minimum Support Length Requirements

4.14 SUPERSTRUCTURE SHEAR KEYS

Shear keys are typically designed to fuse at the Life Safety Design Event. Minimum requirements in the Guidelines are intended to keep the keys elastic at a lower more frequent earthquake event. The design of the superstructure and the substructure shall take into consideration the possible load path described in Sections 7.1 and 7.2. For slender bents, shear keys on top of the bent cap may function elastically at the Life Safety hazard level. For shear keys at intermediate hinges within a span, the designer shall assess the possibility of a shear key fusing mechanism, which is highly dependent on out-of-phase frame movements.

The nominal shear key capacity V_{nk} shall be determined based on a coefficient of friction : considering concrete placed monolithically or reinforced surface where applicable for the shear key and a cohesion factor, c, equal to zero according to AASHTO LRFD Specifications. The overstrength shear key capacity V_{ok} shall be calculated using:

$$V_{ok} = 2.0V_{nk} \qquad (4.17)$$

The overstrength key capacity should be used in assessing the load path to adjacent members.

For bridges in SDC D where shear keys are needed to achieve a reliable performance at the Life Safety hazard level, (i.e., shear key element is part of the Earthquake Resistant System, ERS, see Section 3.3), non-linear analysis shall be conducted to derive the distribution forces on shear keys affected by out-of-phase motions.

5. ANALYTICAL MODELS AND PROCEDURES

5.1 GENERAL

A complete bridge system may be composed of a single frame or a series of frames separated by expansion joints and/or articulated construction joints. A bridge is composed of a superstructure and a supporting substructure.

Individual frame sections are supported on their respective substructures. Substructures consist of piers, single column or multiple column bents that are supported on their respective foundations.

The seismic response of a bridge includes the development of an analytical model followed by the response analysis of the analytical model to predict the resulting dynamic response for component design. Both the development of the analytical model and the selected analysis procedure are dependent on the seismic hazard, selected seismic design strategy and the complexity of the bridge. There are various levels or degrees of refinement in the analytical model and analytical procedures that are available to the designer.

5.1.1 Analysis of a Bridge ERS

The entire bridge Earthquake Resistant System (ERS) for analysis purposes is referred to as the "global" model, whereas an individual bent or column is referred to as a "local" model. The term "global response" describes the overall behavior of the bridge system including the effects of adjacent components, subsystems, or boundary conditions. The term "local response" referring to the behavior of an individual component or subsystem being analyzed to determine, for example, its capacity using a pushover analysis.

Both global models and local models are included in these Specifications.

Individual bridge components shall have displacement capacities greater than the displacement demands derived from the "global" analysis.

The displacement demands of a bridge system consisting of multiple simple spans can be derived using the equivalent static analysis outlined in Article 5.4.2. Global analysis requirements as

C5.1 GENERAL

Seismic analysis encompasses a demand analysis and a displacement capacity verification. The objective of a demand analysis is to estimate the forces and displacements induced by the seismic excitation. A displacement capacity determination of piers and bents is required for SDC B, C, and D. The objective of a displacement capacity determination is to determine the displacement of an individual pier when its deformation capacity (that of the inelastic earthquake resisting element) is reached. The displacement capacity must be greater than the displacement demand. The accuracy of the demand and capacity analyses depend on the assumption of the model related to the geometry, boundary conditions, material properties, and energy-dissipation incorporated in the model. It is the responsibility of the designer to assess the reasonableness of a model in representing the behavior of the structure at the level of forces and deformations expected for the seismic excitation.

Very flexible bridges shall be analyzed accounting for the nonlinear geometry (i.e., $P-\Delta$ effect). The need for modeling of foundations and abutments depends on the sensitivity of the structure to foundation flexibility and associated displacements. This in turn depends on whether the foundation is a spread footing, pile footing with pile cap, a pile bent, or drilled shaft. Article 5.3 defines the requirements for the foundation modeling in the seismic analysis.

When gross soil movement or liquefaction is determined to be possible, the model shall represent the change in support conditions and additional loads on the substructure associated with soil movement.

For structures whose response is sensitive to the support conditions, such as in a fixed-end arch, the model of the foundation shall account for the conditions present.

given in Article 5.1.2 need not to be applied in this case.

5.1.2 Global Model

A global model should capture the response of the entire bridge system. Bridge systems with irregular geometry, in particular curved bridges and skew bridges, will require a global model with actual geometry defined. Also, multiple transverse expansion joints, massive substructures components, and foundations supported by soft soil can exhibit dynamic response characteristics that should be included in the model. Their effect on the global response is not necessarily intuitively obvious and may not be captured by a separate subsystem analysis.

Linear elastic dynamic analysis shall as a minimum be used for the global response analysis. There are however, some limitations in a linear elastic analysis approach. The nonlinear response of yielding columns, gapped expansion joints, earthquake restrainers and nonlinear soil properties can only be approximated using a linear elastic approach. Piece wise linear analysis can be used to approximate nonlinear response. Sensitivity studies using two bounding conditions may be used to approximate the non-linear effects.

For example, two global dynamic analyses are required to approximate the nonlinear response of a bridge with expansion joints because it possesses different characteristics in tension and compression.

In the tension model, the superstructure joints are permitted to move independently of one another in the longitudinal direction. Truss elements connecting the joints may be used to model the effects of earthquake restrainers. In the compression model, all of the truss (restrainer) elements are inactivated and the superstructure elements are locked longitudinally to capture structural response modes where the joints close up, mobilizing the abutments when applicable.

The structure's geometry will generally dictate if both a tension model and a compression model are required. Structures with appreciable superstructure curvature have a bias response to the outside of the curve and may require additional models, which combine the characteristics identified for the tension and compression models.

Long multi-frame bridges may be analyzed with multiple elastic models. A single multi-frame

C5.1.2 Global Model

Depending on the chosen seismic analysis method, different types of approximations may be used for modeling the strength, stiffness, and energy-dissipation mechanisms. One-dimensional beam-column elements are sufficient for dynamic analysis of structures due to earthquake ground motion (referred to as "spine" models or "stick" models). For seismic analyses, grid or finite-element analyses are generally not necessary. They greatly increase the size of the model and complicate the understanding of the force and deformation distribution through the substructure because of the large number of vibration modes.

The geometry of skew, horizontal curvature, and joint size shall be included in the model. However, two-dimensional models are adequate for bridges with skew angle less than 30 degrees and a subtended angle of horizontal curvature less than 20 degrees. When skew is included in a three-dimensional model, the geometry and boundary conditions at the abutments and bearings shall be represented in order to determine the forces and displacements at these locations. Short columns or piers may be modeled with a single element, but tall columns may have two or more elements, particularly if they have significant mass (in the case of concrete), or are modeled as framed substructures.

The use of compression and tension models is expected to provide a reasonable bound on forces (compression model) and displacements (tension model).

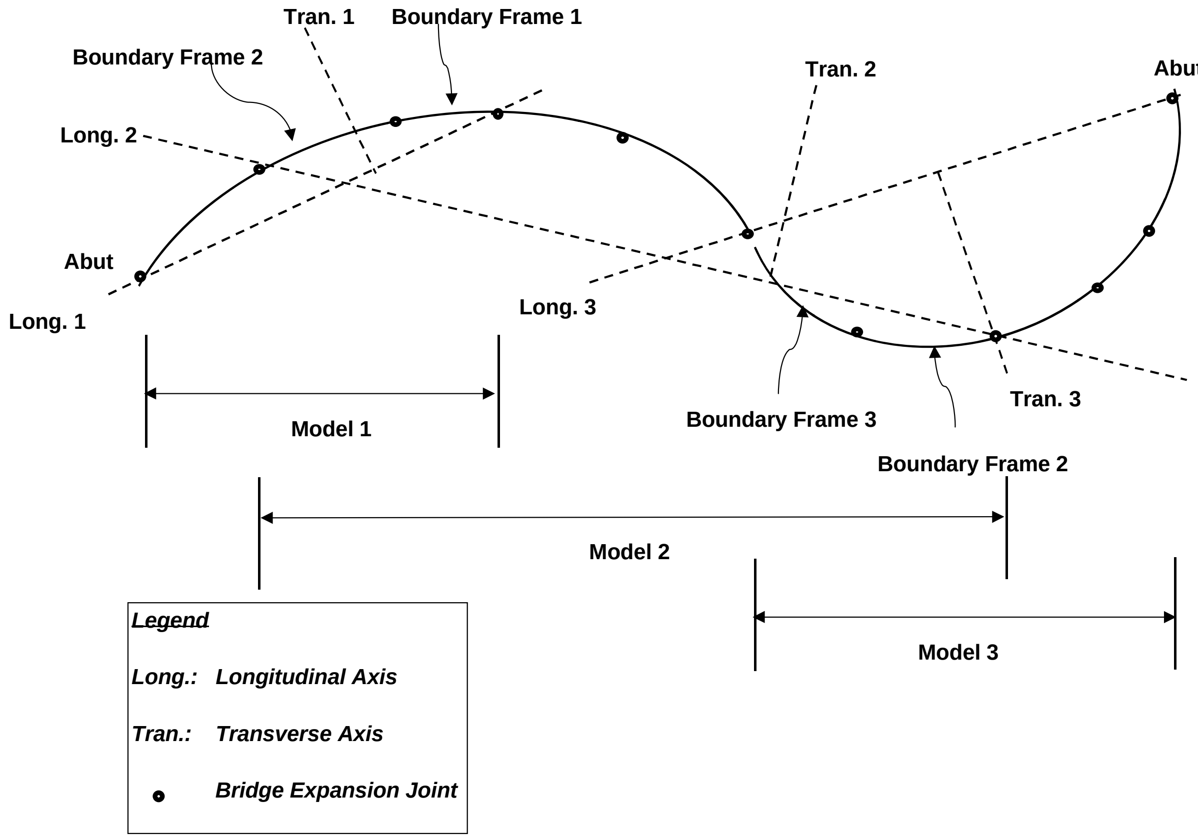


FIGURE 5.1 Elastic Dynamic Analysis Modeling Techniques

model may not be realistic since it cannot account for out-of-phase movement among the frames.

Each multi-frame model may be limited to five frames plus a boundary frame or abutment on each end of the model. Adjacent models shall overlap each other by at least one useable frame, as shown in Figure 5.1. A massless spring should be attached to the dead end of the boundary frames to represent the stiffness of the adjoining structure. The boundary frames provide some continuity between adjacent models but are considered redundant and their analytical results are ignored.

5.2 ABUTMENTS

5.2.1 General

The model of the abutment shall reflect the expected behavior of the abutment with seismic loads applied in each of the two horizontal directions. Resistance of structural components shall be represented by cracked section properties where applicable when conducting an Equivalent Static Analysis or an Elastic Dynamic Analysis.

C5.2.1 General

Article 5.2 provides requirements for the modeling of abutments in the longitudinal and transverse directions. The iterative procedure with secant stiffness coefficients defined in those articles are included in the mathematical model of the bridge to represent the resistance of the abutments in an elastic analysis.

The resistance from passive pressure of the soil embankment at the abutment wall shall be represented by a value for the secant stiffness consistent with the maximum displacement – according to Article 5.2.3. Depending on the bridge configuration, one of two alternatives can be chosen by the designer:

Earthquake Resisting System (ERS) without Abutment Contribution. ERS is designed to resist all seismic loads without any contribution from abutments in either orthogonal direction.

Earthquake Resisting System (ERS) with Abutment Contribution. The ERS is designed with the abutments as a key element of the ERS, in one or both of the orthogonal directions. Abutments are designed and analyzed to sustain the Design Earthquake displacements.

For the Displacement Capacity Verification, the strength of each component in the abutment, including soil, shall be included.

5.2.2 Wingwalls

The participation of abutment walls and wingwalls in the overall dynamic response of bridge systems to earthquake loading and in providing resistance to seismically induced inertial loads shall be considered in the seismic design of bridges. Damage to walls is allowed to occur during earthquakes considering No Collapse criteria. Abutment participation in the overall dynamic response of the bridge system shall reflect the structural configuration, the load-transfer mechanism from the bridge to the abutment system, the effective stiffness and force capacity of the wall-soil system, and the level of expected abutment damage. The capacity of the abutments to resist the bridge inertial load shall be compatible with the structural design of the abutment wall (i.e., whether part of the wall will be damaged by the design earthquake), as well as the soil resistance that can be reliably mobilized. The lateral load capacity of walls shall be evaluated based on an applicable passive earth-pressure theory. A simplistic approach that may be used is to consider one wall 2/3 effective in acting against the abutment soil fill, while the second wall is considered 1/3 effective in acting against the outside sloped berm.

The load-displacement behavior of the abutment may be used in a static nonlinear analysis when the resistance of the abutment is included in the design of the bridge.

C.5.2.1 Abutments

In general the connections between the superstructure and substructure should be designed for the maximum forces that could be developed. In the spirit of capacity design, this implies that the forces corresponding to the full plastic mechanism (with yielding elements at their overstrength condition) should be used to design the connections. In cases where the full plastic mechanism might not develop during the Design Earthquake, the elastic forces for this event are permitted. However, it is still good practice to design the connections to resist the higher forces corresponding to the full plastic mechanism. It is also good practice to design for the best estimate of forces that might develop in cases such as pile bents with battered piles. In such bents the connections should be stronger than the expected forces, and these forces may be large and may have large axial components. In such cases, the plastic mechanism may be governed by the pile geotechnical strengths, rather than the pile structural strengths.

5.2.3 Longitudinal Direction

Under earthquake loading, the earth pressure action on abutment walls changes from a static condition to one of generally two possible conditions; (1) the dynamic active pressure condition as the wall moves away from the backfill, or (2) the passive pressure condition as the inertial load of the bridge pushes the wall to move inward toward the backfill. The governing earth pressure condition depending on the magnitude of seismically induced movement of the abutment walls, the bridge superstructure, and the bridge/abutment configuration. For seat-type abutments where the expansion joint is sufficiently large to accommodate both the cyclic movement between the abutment wall and the bridge superstructure (i.e., superstructure does not push against abutment wall), the seismically induced earth pressure on the abutment wall would be the dynamic active pressure condition. However, when the gap at the expansion joint is not sufficient to accommodate the cyclic wall/bridge movements, a transfer of forces will occur from the superstructure to the abutment wall. As a result, the active earth pressure condition will not be valid and the earth pressure approaches a much larger passive pressure load condition behind the backwall, which is the main cause for abutment damage as witnessed in past earthquakes. For stub or integral abutments, the abutment stiffness and capacity under passive pressure loading are primary design concerns.

5.2.3.1 Abutment Longitudinal Response for SDC B and C

Abutments designed for bridges in SDC B or C are expected to resist earthquake loads with minimal damage. For seat-type abutments, minimal abutment movement could be expected under dynamic passive pressure conditions. However, bridge superstructure displacement demands may be 4 inches or more and could potentially increase the soil mobilization.

Backwall reinforcement of seat-type abutments, or the diaphragm of integral abutments, designed for service load conditions must be checked for the seismic load path and altered if deemed appropriate.

5.2.3.2 Abutment Longitudinal Response for SDC D

For SDC D, passive pressure resistance in soils behind integral abutment walls and back walls for seat abutments will usually be mobilized due to the large longitudinal superstructure displacements associated with the inertial loads. Two alternatives may be considered by the Designer:

Case 1: **Earthquake Resisting System (ERS) without Abutment Contribution.** The bridge ERS is designed to resist all seismic loads without any contribution from abutments. Abutments may contribute to limiting the displacement and providing additional capacity and better performance that are not directly accounted for in the analytical model. To ensure that the columns will always be able to resist the lateral loads, a zero stiffness and capacity at the abutments should be assumed. In this case a check of the abutment displacement capacity to sustain the displacement deamnd and overturning potential should be made.

Case 2: **Earthquake Resisting System (ERS) with Abutment Contribution.** In this case, the bridge is designed with the abutments as a key element of the ERS. Abutments are designed and analyzed to sustain the Design Earthquake displacements. When abutment stiffness and capacity are included in the design, it should be recognized that the passive pressure zone mobilized by abutment displacement extends beyond the active pressure zone normally used for static service load design, as illustrated schematically in Figure 5.2. The approach slab shown in Figure 5.2 is for illustration purposes only. Whether presumptive or computed passive pressures are used for design as stated in Article 5.2.3.3, backfill in this zone should be controlled by specifications, unless the passive pressure considered is less than 70% of the presumptive value.

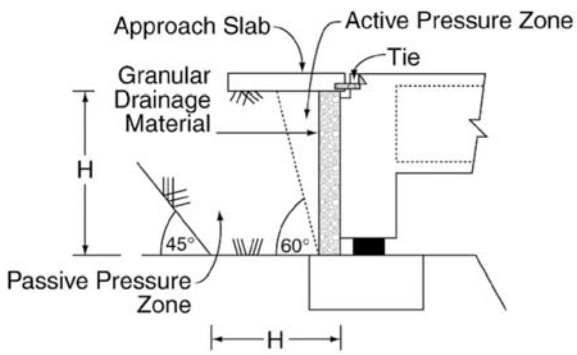


FIGURE 5.2 Design Passive Pressure Zone

5.2.3.3 *Abutment Stiffness and Passive Pressure Estimate*

Abutment stiffness, K_{eff}, and passive capacity, P_p, should be characterized by a bi-linear or other higher-order nonlinear relationship as shown in Figure 5.3. Passive pressures may be assumed uniformly distributed over the height (H_w) of the backwall or diaphragm. Thus the total passive force is:

$$P_p = p_p H_w W_w \tag{5.1}$$

where

H_w = wall height (ft)

p_p = passive pressure behind backwall (ksi)

W_w = wall width (ft)

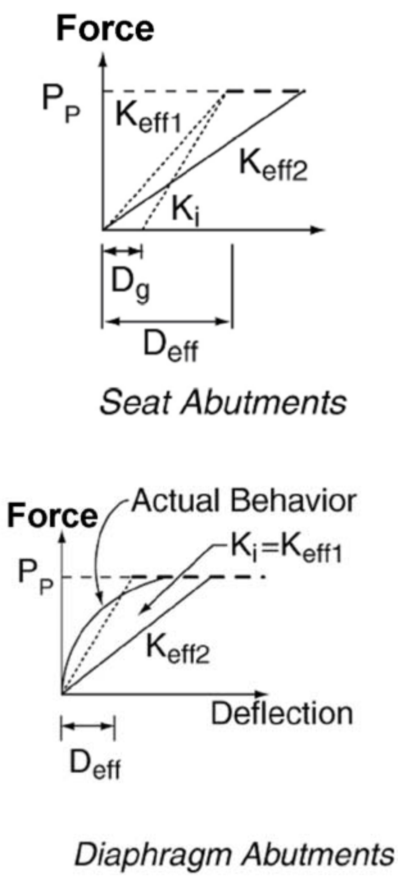


FIGURE 5.3 Characterization of Abutment Capacity and Stiffness

a. Calculation of Best-Estimate Passive Pressure p_p

If the strength characteristics of compacted or natural soils in the "passive pressure zone" (total stress strength parameters c and ϕ) are known, then the passive force for a given height, H, may be computed using accepted analysis procedures. These procedures should account for the interface friction between the wall and the soil. The properties used shall be those indicative of the entire “passive pressure zone” as indicated in Figure 5.2. Therefore the properties of backfill that is only placed adjacent to the wall in the active pressure zone may not be appropriate as a weaker failure surface that can develop in the embankment.

If presumptive passive pressures are to be used for design, then the following criteria shall apply:

- Soil in the "passive pressure zone" should be compacted to a dry density greater than 95% of the maximum per ASTM Standard Method D1557 or equivalent.
- For cohesionless, non-plastic backfill (fines content less than 30%), the passive pressure p_p may be assumed equal to $2H_w/3$ ksf per foot of wall length.
- For cohesive backfill (clay fraction > 15%), the passive pressure p_p may be assumed equal to 5 ksf provided the estimated unconfined compressive strength is greater than 4 ksf.

The presumptive values given above are applicable for use in the “Permissible Earthquake Resisting Elements that Require Owner’s Approval”, as defined in Article 3.3. If the design is based upon presumptive resistances that are not greater than 70% of the values listed above, then the structure may be classified in the “Permissible Earthquake Resisting Elements”.

In all cases granular drainage material must be placed behind the abutment wall to ensure adequate mobilization of wall friction.

b. Calculation of Soil Stiffness

An equivalent linear secant stiffness, K_{eff}, is required for analyses. For integral or diaphragm type abutments, an initial secant stiffness (Figure 5.3) may be calculated as follows:

$$K_{eff1} = P_p / (0.02H_w + .04) \qquad (5.2)$$

If computed abutment forces exceed the soil capacity, the stiffness should be softened iteratively (K_{eff1} to K_{eff2}) until abutment displacements are consistent (within 30%) with the assumed stiffness. For seat type abutments the expansion gap should be included in the initial estimate of the secant stiffness. Thus:

$$K_{eff1} = P_p \big/ (0.02 H_w + D_g) \tag{5.3}$$

where

D_g = gap width

For SDC D, where pushover analyses are conducted, values of P_p and the initial estimate of K_{eff1} should be used to define a bilinear load-displacement behavior of the abutment for the capacity assessment.

5.2.4 Transverse Direction

Two alternatives may be considered by the designer:

Earthquake Resisting System (ERS) without Abutment Contribution. The bridge ERS is designed to resist all seismic loads without any contribution from abutments. Concrete Shear Keys are considered sacrificial when they are designed for lateral loads lower than the Design Earthquake loads. A minimum level of design corresponds to lateral loads not including earthquake loads. If sacrificial concrete shear keys are used to protect the piles, the bridge shall be analyzed and designed according to Sections 5.2.4.1 and 5.2.4.2 as applicable. If a fuse is used, then the effects of internal force redistribution resulting from fusing shall be taken into account in the design of the bridge. Limitations on the use of fusing (hinging or failure of a bridge component along the earthquake load path) for SDC C or D are listed below. Abutment pile foundations are considered adequate to carry the vertical dead loads for satisfying the No Collapse Criteria.

Earthquake Resisting System (ERS) with Abutment Contribution. The bridge is designed with the abutments as a key element of the ERS. Shear keys at the abutment are designed and analyzed to sustain the lesser of the Design Earthquake forces or sliding friction forces of spread footings. Pile supported foundations are designed to sustain the Design Earthquake displacements. Inelastic behavior of piles at the abutment is acceptable.

In the context of these provisions, elastic resistance includes the use of elastomeric, sliding, or isolation bearings designed to accommodate the design displacements, soil frictional resistance acting against the base of a spread footing-supported abutment, pile resistance provided by piles acting in their elastic range, or passive resistance of soil acting at displacements less that 2% of the wall height.

Likewise, fusing includes: breakaway elements, such as isolation bearings with a relatively high yield force; shear keys; yielding elements, such as wingwalls yielding at their junction with the abutment backwall; elastomeric bearings whose connections have failed and upon which the superstructure is sliding; spread footings that are proportioned to slide; or piles that develop a complete plastic mechanism.

The stiffness of the abutment foundation under transverse loading may be calculated based on the procedures given in Article 5.3. Where fusing elements are used, allowance shall be made for the reduced equivalent stiffness of the abutment after fusing occurs.

5.2.4.1 Abutment Transverse Response for SDC B and C

For bridges in these categories, elastic resistance may be achievable provided:

Shear keys shall be designed for a minimum lateral force of 0.2 times the dead load reaction at the abutment.

Shear keys shall be designed for, a lateral force, equal to the difference between the lateral force demand and 0.4 times the dead load reaction at the abutment. The overstrength capacity shall be considered in the design of shear keys according to Article 4.14.

Fusing is not expected for SDC B or C; however, if deemed necessary shall be checked using applicable procedure to SDC D according to Article 5.2.4.2 taking into account the overstrength effects of shear keys according to Article 4.14.

5.2.4.2 Abutment Transverse Response for SDC D

For structures in this category, either elastic resistance or fusing shall be used to accommodate transverse abutment loading. The elastic forces used for transverse abutment design shall be determined from an elastic demand analysis of the structure.

For transverse loading when a fusing mechanism is chosen for pile support foundations, the overstrength capacity of the shear keys shall be less than the combined plastic shear capacity of the piles. For pile-supported abutment foundations, the stiffness contribution of standard size piles (i.e., ≤ 16 inches) shall be ignored if the abutment displacement is greater than 4 inches unless a displacement capacity verification of the pile is performed separately. The capacity provided by the footing-soil friction resistance in addition to the wing walls resistance is considered secondary for ensuring a fusing mechanism.

The design of concrete shear keys should consider the unequal forces that may develop in a skewed abutment, particularly if the intermediate piers are also skewed. (This effect is amplified if intermediate piers also have unequal stiffness, such as wall piers.) The shear key design should also consider unequal loading if multiple shear keys are used. The use of recessed or hidden shear keys should be avoided if possible, since these are difficult to inspect and repair.

5.3 FOUNDATIONS

5.3.1 General

The Foundation Modeling Method (FMM) defined in Table 5.1 is recommended unless deemed otherwise. Articles 5.3.2, 5.3.3 and 5.3.4 provide the requirements for estimating foundation springs for spread footings, pile foundations, and the depth to fixity for drilled shafts. For foundation modeled as rigid, the mass of the foundation may be ignored in the analytical model, which may be important in achieving a total contributory mass of 90%. The Engineer shall assess the merits of including the foundation mass in the analytical model where appropriate taking into account the recommendations in this section.

The required foundation modeling method depends on the Seismic Design Category (SDC).

Foundation Modeling Method I is required as a minimum for SDC B & C provided foundation is located in Site Class A, B, C, or D. Otherwise, Foundation Modeling Method II is required.

Foundation Modeling Method II is required for SDC D.

For SDC D, Foundation Modeling Method II is required in the Displacement Capacity Verification (“pushover”) analysis if it is used in

C5.3.1 General

A wide range of methods for modeling foundations for seismic analysis is available. Generally a refined model is unnecessary for seismic analysis. For many cases the assumption of a rigid foundation is adequate. Flexibility of a pile bent or shaft can be estimated using an assumed point of flexibility associated with the stiffness estimate of the pile (or shaft) and the soil. Spread footings and piles can be modeled with rotational and translational springs.

The requirement for including soil springs for Foundation Modeling Method II depends on the contribution of the foundation to the elastic displacement of the pier. Foundation springs for a pier are required when the foundation flexibility contributes more than 50% of SDC B allowable drift considering column plastic forces. More flexible spread and pile footings should be modeled and included in the seismic analysis.

If foundation springs are included in the multi-mode dynamic analysis, they must be included in the pushover analysis so the two models are consistent for the displacement comparison.

For most spread footings and piles with pile caps a secant stiffness for the soil springs is adequate. Bi-linear soil springs are used for the pushover analysis.

For pile bents and drilled shafts, an estimated depth to fixity is generally adequate for representing the relative flexibility of the soil and pile or shaft. Soil springs with secant stiffness may be used to provide a better representation based on P-y curves for the footing and soil. Bi-linear springs may be used in the pushover analysis if

Table 5.1 Definition of Foundation Modeling Method (FMM)

Foundation Type	Modeling Method I	Modeling Method II
Spread Footing	Rigid	Rigid for Site Classes A and B. For other site classes, foundation springs required if footing flexibility contributes more than 50% of SDC B allowable drift considering column plastic forces.
Pile Footing with Pile Cap	Rigid	Foundation springs required if footing flexibility contributes more than 50% of SDC B allowable drift considering column plastic forces.
Pile Bent/Drilled Shaft	Estimated depth to fixity	Estimated depth to fixity or soil-springs based on P-y curves.

the multi-mode dynamic analysis for displacement demand. The foundation models in the multi-mode dynamic analysis and Displacement Capacity Verification shall be consistent and representative of the footing behavior.

For sites identified as susceptible to liquefaction or lateral spread, the ERS global model shall consider the non-liquefied and liquefied conditions using the procedures specified in Article 6.8.

5.3.2 Spread Footing

When required to represent foundation flexibility, spring constants shall be developed for spread footing using equations given in Tables C1, C2 and Figure C1 of Appendix C. Alternative procedures given in the FEMA 273 *Guidelines for the Seismic Rehabilitation of Buildings* (ATC/BSSC, 1997) are also suitable for estimating spring constants. These computational methods are appropriate for sites that do not liquefy or lose strength during earthquake loading.

The shear modulus (G) used to compute the stiffness values in Appendix C shall be determined by adjusting the low-strain shear modulus (G_{max}) for the level of shearing strain using the following strain adjustment factors, unless other methods are approved by the owner.

For SDC B or C

G/G_{max} = 0.50 for Design Earthquake ground motions

For SDC D

G/G_{max} = 0.25 for Design Earthquake ground motions

Values of G_{max} shall be determined by seismic methods (e.g., crosshole, downhole, or SASW), by laboratory testing methods (e.g., resonant column with adjustments for time), or by empirical

there is particular concern with depth of the plastic hinge and effective depth of fixity.

If bilinear springs are used in a pushover analysis, a secant stiffness, typical of the expected level of soil deformation, is used in the multi-mode dynamic analysis for a valid comparison of displacement demand and capacity.

equations (Kramer, 1996). The uncertainty in determination of G_{max} shall be considered when establishing strain adjustment factors.

No special computations are required to determine the geometric or radiation damping of the foundation system. Five percent system damping shall be used for design, unless special studies are performed and approved by the owner.

Moment-rotation and shear force-displacement shall be represented by a bilinear relationship. The initial slope of the bi-linear curve shall be defined by the rotational spring constant given in Appendix C.

The maximum resisting force (i.e., plastic capacity) on the force-deformation curve shall be defined for the best-estimate case of geotechnical properties. Uplift or rocking shall be allowed for spread footings as stated in Article 6.3.4.

5.3.3 Pile Foundations

The design of pile foundations shall be based on column loads determined by capacity design principles (Article 4.11) or elastic seismic forces, whichever is smaller for SDC B and based on capacity design principles only for SDC C or D. Both the structural and geotechnical elements of the foundation shall be designed accordingly.

Foundation flexibility shall be incorporated into design for SDC D according to Article 5.3.1 and following Appendix C for calculation of spring constants.

The nonlinear properties of the piles shall be considered in evaluating the lateral response of the piles to lateral loads during a seismic event. Group reduction factor established in the geotechnical report shall be included in the analysis.

Liquefaction shall be considered using procedures specified in Article 6.8 for SDC D where applicable during the development of spring constants and capacity values.

5.3.4 Drilled Shafts

The flexibility of the drilled shaft shall be represented using either the estimated depth of fixity or soil springs in a lateral pile analysis. Procedures identified in Article 5.3.3 including those for liquefaction, generally apply except that group reduction factors are typically considered only in the transverse direction of a multi-shaft bent.

5.4 ANALYTICAL PROCEDURES

5.4.1 General

The objective of seismic analysis is to assess displacement demands of a bridge and its individual components. Equivalent static analysis and linear elastic dynamic analysis are the appropriate analytical tools for estimating the displacement demands for normal bridges. Inelastic static analysis “Pushover Analysis” is the appropriate analytical tool used to establish the displacement capacities for normal bridges assigned to SDC D.

Nonlinear Time History analysis should be used for critical or essential bridges as defined in Section 4.2.2 and in some cases for Normal Bridges in SDC D using devices for isolation or energy dissipation. In this type of analysis, component capacities are characterized in the mathematical model used for the seismic response analysis. The procedures mentioned above are described in more detail below in Section 5.5.4.

5.4.2 Procedure 1 Equivalent Static Analysis (ESA)

ESA can be used to estimate displacement demands for structures where a more sophisticated dynamic analysis will not provide additional insight into behavior. ESA is best suited for structures or individual frames with well balanced spans and uniformly distributed stiffness where the response can be captured by a predominant translational mode of vibration.

The seismic load shall be assumed as an equivalent static horizontal force applied to individual frames. The total applied force shall be equal to the product of the Acceleration Response Spectrum value at the calculated period times the tributary weight. The horizontal force shall be applied at the vertical center of mass of the superstructure and distributed horizontally in proportion to the mass distribution. Both the Uniform Load Method and the Single Mode Spectral Analysis Method are considered equivalent static analysis procedures.

5.4.3 Procedure 2 Elastic Dynamic Analysis (EDA)

EDA shall be used to estimate the displacement demands for structures where ESA does not provide an adequate level of

C5.4 Selection of Analysis Procedure

Bridges are expected to remain essentially elastic when subjected to earthquakes with a high probability of occurrence (Expected Earthquake ground motions, which have a 50% probability of being exceeded in 50 years). For low-probability earthquakes (Maximum Considered Earthquake ground motions, which have a 5% probability of being exceeded in 50 years and depending on the desired performance level, bridges are designed to dissipate energy through inelastic deformation in earthquake-resisting elements.

In specifying the Seismic Design Category (SDC), two principles are followed. First, as the seismic hazard increases, improved modeling and analysis for seismic demands is necessary because the behavior may be sensitive to the maximum demands. Second, as the complexity of the bridge increases, more sophisticated models are required for seismic demand and displacement capacity evaluation. For bridges with a regular configuration, a single-degree-of-freedom model is sufficiently accurate to represent the seismic response. For these types of bridges, the equivalent static analysis (Procedure 1) may be used to establish displacement demands.

For structures that do not satisfy the requirements of regularity an elastic response spectrum analysis, Procedure 2, must be used to determine the displacement demands

C5.4.2 Procedure 1 Equivalent Static Analysis (ESA)

The equivalent static analysis is suitable for short to medium span structures with regular configuration. Long bridges, or those with significant skew or horizontal curvature, have dynamic characteristics that should be assessed in a multi-mode dynamic analysis.

C5.4.2 Uniform Load Method

The Uniform Load Method, described in the following steps, may be used for both transverse and longitudinal earthquake motions. It is essentially an equivalent static method of analysis that uses a uniform lateral load to approximate the effect of seismic loads. The method is suitable for regular bridges that respond principally in their fundamental mode of vibration.

Whereas displacements are calculated with reasonable accuracy, the method can overestimate the transverse shears at the abutments by up to 100%. Consequently, the columns may have sophistication to estimate the dynamic behavior. A linear elastic multi-modal spectral analysis utilizing the appropriate response spectrum (i.e., 5% damping) shall be performed. The number of degrees of freedom and the number of modes considered in the analysis shall be sufficient to capture at least 90% mass participation in both the longitudinal and transverse directions. A minimum of three elements per flexible column and four elements per span shall be used in the linear elastic model.

The engineer should recognize that forces generated by linear elastic analysis could vary, depending on the degree of non-linear behavior, from the actual force demands on the structure. Displacements are not as sensitive to the non-linearity's and may be considered good approximations. Sources of nonlinear response that are not captured by EDA include the effects of the surrounding soil, yielding of structural components, opening and closing of expansion joints, and nonlinear restrainer and abutment behavior. EDA modal results shall be combined using the complete quadratic combination (CQC) method.

For multi-frame analysis it is recommended to include a minimum of two boundary frames or one frame and an abutment beyond the frame user consideration. (See Section 5.1.2).

5.4.4 Procedure 3 Nonlinear Time History Method

Any step-by-step, time history method of dynamic analysis that has been validated by experiment and/or comparative performance with similar methods may be used provided the following requirements are also satisfied:

The time histories of input acceleration used to describe the earthquake loads shall be selected in consultation with the Owner or Owner's representative. Time-History Analysis shall be performed with no fewer than three data sets (two horizontal components and one vertical component) of appropriate ground motion time histories selected and called from not less than three recorded events. Appropriate time histories shall represent magnitude, fault distances and source mechanisms that are consistent with those that control the design earthquake ground motion. Each time history shall be modified to be response-

inadequate lateral strength because of the overestimate of abutment forces. A multi-mode dynamic analysis is recommended to avoid unrealistic distributions of seismic forces.

The steps in the uniform load method are as follows:

1. Calculate the static displacements $v_s(x)$ due to an assumed uniform load p_o, as shown in Figure C5.4.2.2-1. The uniform loading p_o is applied over the length of the bridge; it has dimension of force/unit length and may be arbitrarily set equal to 1.0. The static displacement $v_s(x)$ has the dimension of length.

2. Calculate the bridge lateral stiffness, K, and total weight, W, from the following expressions:

$$K = \frac{p_0 L}{V_{s,MAX}} \quad \text{(C5.4.2.2-1)}$$

$$W = \int_0^L w(x)dx \quad \text{(C5.4.2.2-2)}$$

where:

L = total length of the bridge

$v_{s,MAX}$ = maximum value of $v_s(x)$

$w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure.

The weight shall 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.

3. Calculate the period of the bridge, T_m, using the expression:

$$T_m = 2\pi\sqrt{\frac{W}{Kg}} \quad \text{(C5.4.2.2-3)}$$

where:

g = acceleration of gravity

4. Calculate the equivalent static earthquake loading p_e from the expression:

$$p_e = \frac{C_d W}{L} \quad \text{(C5.4.2.2-4)}$$

where:

spectrum compatible using the time-domain procedure.

Where three time history data sets are used in the analysis of a structure, the maximum value of each response parameter (e.g., force in a member, displacement at a specific level) shall be used to determine design acceptability. Where seven or more time history data sets are employed, the average value of each response parameter may be used to determine design acceptability.

The sensitivity of the numerical solution to the size of the time step used for the analysis shall be determined. A sensitivity study shall also be carried out to investigate the effects of variations in assumed material properties.

5.5 MATHEMATICAL MODELING USING EDA (PROCEDURE 2)

5.5.1 General

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 at each node.

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 probability of a large live load being on the bridge during an earthquake should be considered when designing bridges with high live-to-dead load ratios which are located in metropolitan areas where traffic congestion is likely to occur.

5.5.2 Superstructure

The superstructure shall, 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

C_d = the dimensionless elastic seismic response demand coefficient obtained from Article C3.4.1 with the coefficient taken as S_{DS} for short periods.

p_e = equivalent uniform static seismic loading per unit length of bridge applied to represent the primary mode of vibration.

5. Calculate the displacements and member forces for use in design either by applying p_e to the structure and performing a second static analysis or by scaling the results of the first step above by the ratio p_e/p_o.

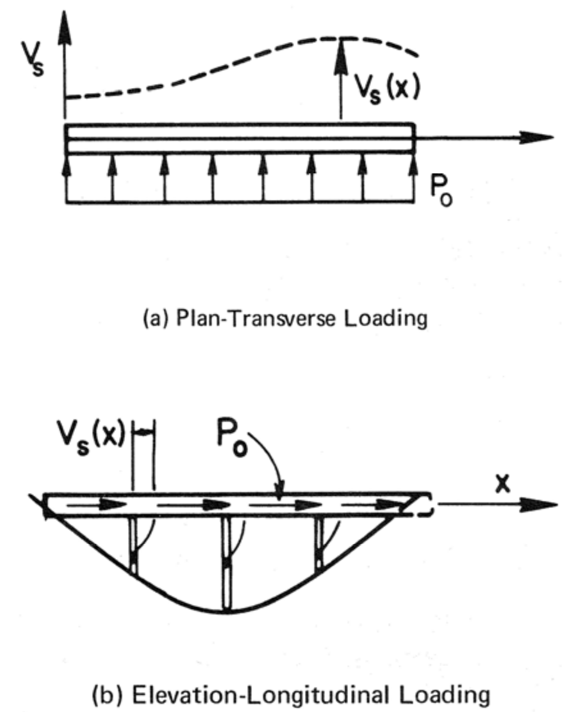


Figure C5.4.2.2-1 Bridge Deck Subjected to Assumed Transverse and Longitudinal Loading

The configuration requirements for Equivalent Static Analysis (Procedure 1) analysis restrict application to individual frames or units that can be reasonably assumed to respond as a single-degree-of-freedom system in the transverse and longitudinal directions. When abutments do not resist significant seismic forces, the superstructure will respond as a rigid-body mass. The lateral-load-resisting piers or bents must be linearly elastic members having the stiffness properties of the engaged restrainer units.

5.5.3 Substructure

The intermediate columns or piers should also be modeled as space frame members. 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.

5.6 EFFECTIVE SECTION PROPERTIES

5.6.1 Effective Reinforced Concrete Section Properties For Seismic Analysis

Elastic analysis assumes a linear relationship between stiffness and strength. Concrete members display nonlinear response before reaching their idealized yield limit state.

Section properties, flexural stiffness, $E_c I_{eff}$, shear stiffness parameter $(GA)_{eff}$, and torsional stiffness $G_c J_{eff}$, shall reflect the cracking that occurs before the yield limit state is reached. The effective moments of inertia, I_{eff} and J_{eff} shall be used to obtain realistic values for the structure's period and the seismic demands generated from ESA and EDA analyses.

5.6.2 $E_c I_{eff}$ and $(GA)_{eff}$ For Reinforced Concrete Ductile Members

The effective moment of inertia I_{eff} should be used when modeling ductile elements. I_{eff} *may* be estimated by Figure 5.4 or the slope of the $M-\phi$ curve between the origin and the point designating the first reinforcing bar yield as defined by Equation 5.4.

$$E_c \times I_{eff} = \frac{M_y}{\phi_y} \qquad (5.4)$$

M_y = Moment capacity of the section at first yield of the reinforcing steel.

uniform in strength and stiffness to justify the assumption of independent transitional response in the longitudinal and transverse directions.

C5.4.3 Procedure 2 Elastic Dynamic Analysis (EDA)

The model for an elastic response spectrum analysis is linear, and as such it does not represent the inelastic behavior of earthquake-resisting elements under strong ground motion. However, with the proper representation of the inelastic elements and interpretation of responses, an elastic analysis provides reasonable estimates of seismic demands. The model must be based on cracked section properties for concrete components, and on secant stiffness coefficients for the foundations, abutments, and seismic isolation components. All must be consistent with the expected levels of deformation of the components. The displacements at the center of mass, generally the superstructure, can be used to estimate the displacement demand of the structure including the effect of inelastic behavior in the earthquake-resisting elements as discussed in Article C3.3.

For SDC D, a displacement capacity evaluation is required. The displacement capacity evaluation involves determining the displacement at which the first component reaches its inelastic deformation capacity. All non-ductile components shall be designed using capacity design principles to avoid brittle failure. For simple piers or bents, the displacement capacity can be evaluated by simple calculations using the geometry of displaced shapes, and forces and deformations at the plastic hinges. For more complicated piers or bents, particularly when foundations and abutments are included in the model, a nonlinear static ("pushover") analysis may be used to evaluate the displacement capacity. It is recommended that the nonlinear static analysis continue beyond the displacement at which the first component reaches its inelastic deformation capacity in order to assess the behavior beyond the displacement capacity and obtain a better understanding of the limit states.

The displacement capacity is compared to the displacement demand determined from an elastic response-spectrum analysis.

ϕ_y = Yield Curvature

The unfactored axial gravity load is typically used when determining the effective properties.

The $M-\phi$ analysis parameters are defined in Section 8.4 and 8.5.

For pier wall in the strong direction, the shear stiffness parameter $(GA)_{eff}$ can be calculated as follows:

$$(GA)_{eff} = G_c A_{ew} \frac{(E_c I_{eff})}{(E_c I_g)} \quad (5.5)$$

When A_{ew} is the cross-sectional area of the pier wall.

C5.4.3 Multi-Mode Dynamic Analysis Method

Vibration modes are convenient representations of dynamic response for response spectrum analysis. Enough modes should be included to provide sufficient participation for bending moments in columns, or other components with inelastic deformation. Dynamic analysis programs, however, usually compute participation factors only for base shear, often expressed as a percentage of total mass. For regular bridges the guideline of including 90% of the modal mass for horizontal components generally provides a sufficient number of modes for accurate estimate of forces in lateral-load-resisting components. For irregular bridges, or large models of multiple-frame bridges, the participating mass may not indicate the accuracy for forces in specific components. It is for this reason that the models of long bridges are limited to five frames.

The response spectrum in Article 3.4.1 is based on 5% damping. For bridges with seismic isolation the additional damping from the seismic isolator units applies only to the isolated vibration modes. Other vibration modes have 5% damping.

C5.4.4 Procedure 3 Nonlinear Time History Method

A nonlinear dynamic analysis is a more comprehensive analysis method because the effect of inelastic behavior is included in the demand analysis. Depending on the mathematical model, the deformation capacity of the inelastic elements may or may not be included in the dynamic response analysis. A nonlinear dynamic response analysis requires a suite of time-histories (Article 3.4.4) of earthquake ground motion that is representative of the hazard and conditions at the site. Because of the complexity involved with nonlinear dynamic analysis, it is best used in conjunction with SDC D or in a case where seismic isolation is included in the design strategy.

Seismically isolated structures with long periods or large damping ratios require a nonlinear dynamic analysis because the analysis procedures using an effective stiffness and damping may not properly represent the effect of isolation units on the response of the structure. The model for nonlinear analysis shall represent the hysteretic relationships for the isolator units.

C5.4.4 Nonlinear Dynamic Analysis Method

The nonlinear dynamic analysis procedure is normally only used for the Maximum Considered Earthquake. The structure is expected to remain essentially elastic for the Expected Earthquake. Hence a multi-mode response spectrum analysis is adequate.

The results of a nonlinear dynamic analysis should be compared with a multi-mode response spectrum analysis to check the nonlinear model is reasonable.

C5.5.1 General

For elastic analysis methods, there is a significant approximation in representing the force-deformation relationship of inelastic structural elements by a single linearized stiffness. For inelastic columns or other inelastic earthquake-resisting elements, the common practice is to use an elastic stiffness for steel elements and a cracked stiffness for reinforced concrete elements. However, the stiffness of seismic isolator units, abutments, and foundation soils are represented by a secant stiffness consistent with the maximum deformation. The designer shall consider the distribution of displacements from an elastic analysis to verify that they are consistent with the inelastic behavior of the earthquake-resisting elements.

Seismic design procedures have been calibrated using stiffness that is representative of deformations close to the yield deformations. At these levels of deformation, reinforced concrete elements will have cracked. The effects of cracking on the stiffness depend on the cross-section,

longitudinal reinforcement ratio, axial load, and amount of bond slip. The cracked flexural stiffness of a reinforced concrete member can be obtained by a moment-curvature analysis of the cross section.

Where the load path depends on torsion of a reinforced concrete column or substructure element, the cracked torsional stiffness may be taken as 20% of the uncracked torsional stiffness.

The objective of the nonlinear displacement capacity verification is to determine the displacement at which the inelastic components reach their deformation capacity. The deformation capacity is the sum of elastic and plastic deformations. The plastic deformation is expressed in terms of the rotation of the plastic hinges. A nonlinear analysis using expected strengths of the components gives larger plastic deformations than an analysis including overstrength. Hence, it is appropriate to use the expected strength of the components when estimating the displacement capacity.

The stiffness of pier caps shall be included in the model. Pile caps and joints in reinforced concrete substructures may be assumed to be rigid.

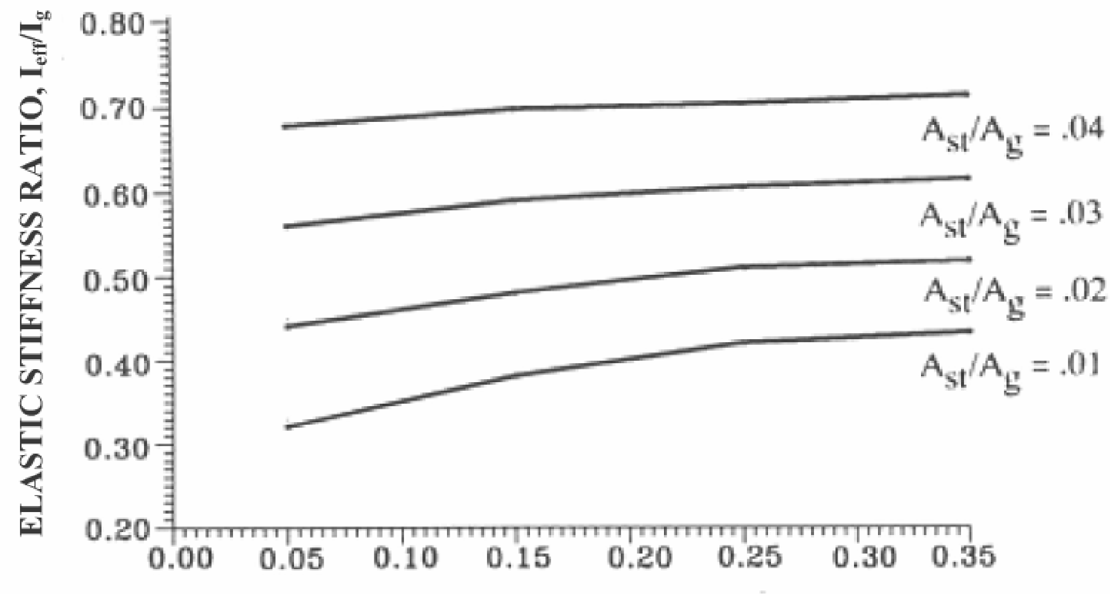
C5.5.2 SUPERSTRUCTURE

For a spine or stick model of the superstructure, the stiffness is represented by equivalent section properties for axial deformation, flexure about two axes, torsion, and possibly shear deformation in two directions. The calculation of the section stiffness shall represent reasonable assumptions about the three-dimensional flow of forces in the superstructure, including composite behavior.

The effects of skew can be neglected in the model of the superstructure. However, for large skew angles, the geometry of the piers with respect to the superstructure and connections between them must be included in the model.

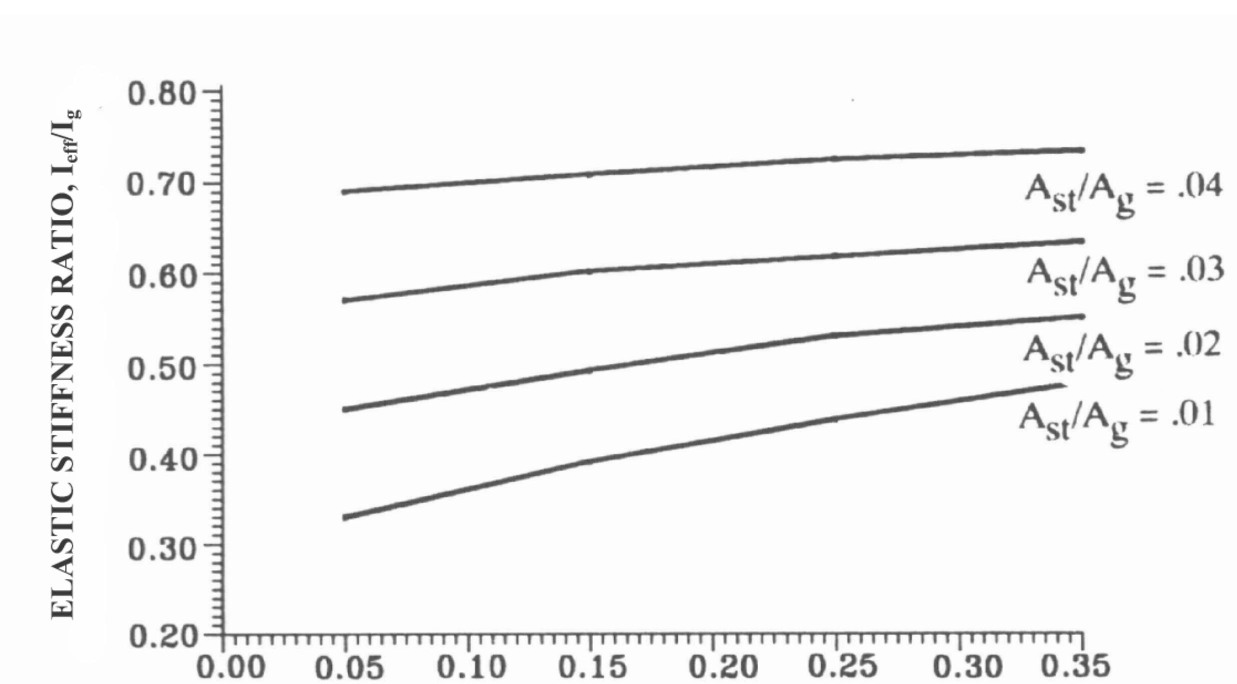
For reinforced box girders the effective stiffness may be based on 75% of the gross stiffness to account for cracking. For prestressed box girders, the full gross stiffness shall be used. The torsional stiffness may be based on a rational shear flow without reduction due to cracking.

The flexural stiffness of the superstructure about a transverse axis is reduced near piers when there is a moment transfer between the superstructure and pier because of shear lag effects. The reduced stiffness shall be represented in the model of the superstructure.



AXIAL LOAD RATIO $P/f'_{ce}A_g$

a) Circular Sections



AXIAL LOAD RATIO $P/f'_{ce}A_g$

b) Rectangular Sections

FIGURE 5.4 Effective Flexural Stiffness of Cracked Reinforced Concrete Sections [x]

5.6.3 I_{eff} For Box Girder Superstructures

I_{eff} in box girder superstructures is dependent on the extent of cracking and the effect of the cracking on the element's stiffness.

I_{eff} for reinforced concrete box girder sections can be estimated between $0.5I_g - 0.75I_g$. The lower bound represents lightly reinforced sections and the upper bound represents heavily reinforced sections.

The location of the prestressing steel's centroid and the direction of bending have a significant impact on how cracking affects the stiffness of prestressed members. Multi-modal elastic analysis is incapable of capturing the variations in stiffness caused by moment reversal. Therefore, no stiffness reduction is recommended for prestressed concrete box girder sections.

5.6.4 I_{eff} For Other Superstructure Types

Reductions to I_g similar to those specified for box girders can be used for other superstructure types and cap beams. A more refined estimate of I_{eff} based on $M-\phi$ analysis may be warranted for lightly reinforced girders and precast elements.

5.6.5 Effective Torsional Moment of Inertia

A reduction of the torsional moment of inertia is not required for bridge superstructures. The torsional stiffness of concrete members can be greatly reduced after the onset of cracking. The torsional moment of inertia for columns shall be reduced according to Equation 5.6.

$$J_{eff} = 0.2 \times J_g \tag{5.6}$$

6. FOUNDATION AND ABUTMENT DESIGN REQUIREMENTS

6.1 GENERAL

This section 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 FOUNDATION INVESTIGATION

6.2.1 Subsurface Investigation

A subsurface investigation, including borings and laboratory soil tests, shall be conducted in accordance with the provisions of Appendix B to provide pertinent and sufficient information for the determination of the Site Class of Article 3.4.2.1. The type and cost of the foundations should be considered in the economic, environmental, and aesthetic studies for location and bridge type selection.

Subsurface explorations shall be made at pier and abutment locations, sufficient in number and depth, to establish a reliable longitudinal and transverse substrata profile. Samples of material encountered shall be taken and preserved for future reference and/or testing. Boring logs shall be prepared in sufficient detail to locate material strata, results of penetration tests, groundwater, any artesian action, and where samples were taken. Special attention shall be paid to the detection of narrow, soft seams that may be located at stratum boundaries.

6.2.2 Laboratory Testing

Laboratory tests shall be performed to determine the strength, deformation, and flow characteristics of soils and/or rocks and their suitability for the foundation selected. In areas of higher seismicity (e.g., SDC D), it may be appropriate to conduct special dynamic or cyclic tests to establish the liquefaction potential or stiffness and material damping properties of the soil at some sites, if unusual soils exist or if the foundation is supporting an essential or critical bridge.

6.2.3 Foundation Investigation for SDC A

There are no special seismic design requirements for this category.

6.2.4 Foundation Investigation for SDC B and C

In addition to the normal site investigation report, the Engineer may require the submission of a report which describes the results of an investigation to determine potential hazards and seismic design requirements related to (1) slope instability, and (2) 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.

6.2.5 Foundation Investigation for SDC D

The Engineer may require the submission of a written report, which shall include in addition to the potential hazard requirements of Section 6.2.4, a determination of the potential for surface rupture due to faulting or differential ground displacement (lurching), a site specific study to investigate the potential hazards of liquefaction and fill settlement in addition to the influence of cyclic loading on the deformation and strength characteristics of foundation soils. 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 could lead to a loss of foundation-bearing capacity and lateral pile support. 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.

6.3 SPREAD FOOTINGS

6.3.1 General

Foundation modeling of spread footings shall meet requirements of Article 5.3 for SDC B, C, and D.

Spread footings are not allowed for SDC D where liquefaction potential is identified as specified in Article 6.8.

6.3.2 SDC B

The design of spread footings for SDC B shall be based on the lesser of:

Forces obtained from an elastic linear analysis.

Forces corresponding to rocking analysis provided footing is in Site Class A, B, C, or D.

6.3.3 SDC C or D

The minimum design requirements of spread footings for SDC C or D shall be based on forces corresponding to rocking analysis, provided footing is in Site Class A, B, C or D and as specified b the Earthquake Resisting System (ERS) requirements of SDC C and D.

6.3.4 Rocking Analysis

Transient foundation uplift or rocking involving separation from the subsoil is permitted under seismic loading, provided that foundation soils are not susceptible to loss of strength under the imposed cyclic loading. The displacement or drift, Δ_T as shown in Figure 6.1, shall be calculated based on the flexibility of the column in addition to the effect of the footing rocking mechanism. For multi-column bents with monolithic connections to the substructure, the effect of rocking shall be examined on the overturning and framing configuration of the subject bent.

For the longitudinal response, multi-column bents that are not monolithic to the superstructure shall be treated similar to a single column bent.

For the case of a single column bent or a multi-column bent without a monolithic connection to the superstructure, the footing is considered to be supported on a rigid perfectly plastic soil with uniform compressive capacity "p_b". The overturning and rocking on the foundation can be

C6.3 SPREAD FOOTINGS

C6.3.1 General

During a seismic event, the inertial response of the bridge deck results in a transient horizontal force at the abutments and central piers. This inertial force is resisted by (1) the abutments, (2) the interior piers, or (3) some combination of the two. Forces imposed on the interior columns or piers result in both horizontal shear force and an overturning moment being imposed on the footing. The footing responds to this load by combined horizontal sliding and rotation. The amount of sliding and rotation depends on the magnitude of imposed load, the size of the footing, and the characteristics of the soil.

For seismic design of spread footings, the responses of the footing to shear forces and moment are normally treated independently, i.e., the problem is de-coupled. The overturning component of the column load results in an increase in pressures on the soil. Since the response to moment occurs as a rotation, pressure is highest at the most distant point of the footing, referred to as the toe. This pressure can temporarily exceed the ultimate bearing capacity of the soil. As the overturning moment continues to increase, soil may yield at the toe and the heel of the footing can separate from the soil, which is referred to as liftoff of the footing. This liftoff is temporary. As the inertial forces from the earthquake change direction, pressures at the opposite toe increase and, if moments are large enough, liftoff occurs at the opposite side. Bearing failure occurs when the force induced by the moment exceeds the total reactive force that the soil can develop within the area of footing contact. Soil is inherently ductile, and therefore, yielding at the toe and liftoff at the heel of the footing are acceptable phenomena, as long as (1) global stability is preserved and (2) settlements induced by the cyclic loading are small.

The shear component of column load is resisted by two mechanisms: (1) the interface friction between the soil and the footing along the side and at the base of the footing, and (2) the passive resistance at the face of the footing. These resistances are mobilized at different deformations. Generally, it takes more displacement to mobilize the passive pressure. However, once mobilized, it normally provides the primary resistance to horizontal loading.

Inertial response of a bridge deck results in a horizontal shear force and a moment at the connection of the column to the footing. The footing should not undergo permanent rotation, sliding, or appreciable settlement under these loads. For essential or critical bridges, any permanent simplified using a linear force/deflection relationship as outlined in the following procedure:

Guess the displacement "Δ" or consider a displacement Δ corresponding to a fixed base analysis.

- Calculate the applied force at the superstructure level "F" based on Rocking Equilibrium shown in Figure 6.1.

From Statics

$$F = W_T * (L_F - a)/2H_r - W_s * \Delta/H_r \quad (6.1)$$

where $a = W_T/(B_r * p_b)$ (6.2)

- Calculate the equivalent system stiffness
$K_r = F/\Delta$ (6.3)

- Calculate the period "T" of the bent system based on "K_r" and W_s.

- Recalculate "Δ" considering 10% damping; this would typically reduce the spectral acceleration ordinates S_a of a 5% damped spectrum by approximately 20%.
$\Delta = (T^2/4\pi^2) * (0.8S_a)$ (ft.) (6.4)

  where

  "Δ" is referred to as the total displacement on top of the column.

  S_a is the spectral acceleration ft/sec^2

- Iterate until convergence, otherwise the bent is shown to be unstable.

Once a converging solution is reached, the local ductility term "μ" can be calculated in order to ensure the column adequacy where rocking mechanism is not mobilized.

$$\mu = \Delta/\Delta_{ycol} \quad (6.5)$$

where,

Δ_{ycol} = the column idealized yield displacement

For soil cover greater than 3 feet, the effect of soil passive resistance needs to be included in the rocking equilibrium of forces.

The design of a column on spread footing system shall follow the steps identified on the flowchart shown on Figure 6.2.

displacement that occurs should be constrained by the limits required to preserve the service level of the bridge as suggested in Table C3.2-1.

C7.4.2.1 Moment and Shear Capacity

The shear component of loading should not be included during the overturning check; i.e., a de-coupled approach should be used in treating the two loads. Experience has shown that combining the horizontal load and moment in simplified bearing capacity equations can result in unreasonably sized footings for seismic loading.

Unfactored resistance is used for the moment capacity check for two reasons: (1) the potential for the design seismic load is very small, and (2) the peak load will occur for only a short duration. The distribution and magnitude of bearing stress, as well as liftoff of the footing, are limited to control settlement of the footing from the cycles of load.

Non-triangular stress distributions or greater than 50% liftoff are allowed if analysis can show that soil settlement from cyclic shakedown does not exceed amounts that result in damage to the bridge or unacceptable movement of the roadway surface. By limiting stress distribution and the liftoff to the specified criteria, the amount of shakedown will normally be small under normal seismic loading conditions.

The restoring moment M_r is calculated as follows:

$$M_r = W_T\left(\frac{L_F - a}{2}\right) \tag{6.6}$$

For the case where, $M_o \geq 1.5M_r$, the column shear capacity shall be determined based on Article 8.6 following SDC B requirements. The column shear demand shall be determined based on $1.5M_r$ moment demand.

For the case where, $M_r \geq M_o$, forces based on column plastic hinging shall be considered; the column shear capacity shall be determined based on Article 8.6 following SDC D requirements. For all other cases, the column shall be designed for $P-\Delta$ requirements based on rocking analysis as well as column plastic hinging shear capacity requirements considering a fixed based analysis and following Article 8.6 SDC C requirements.

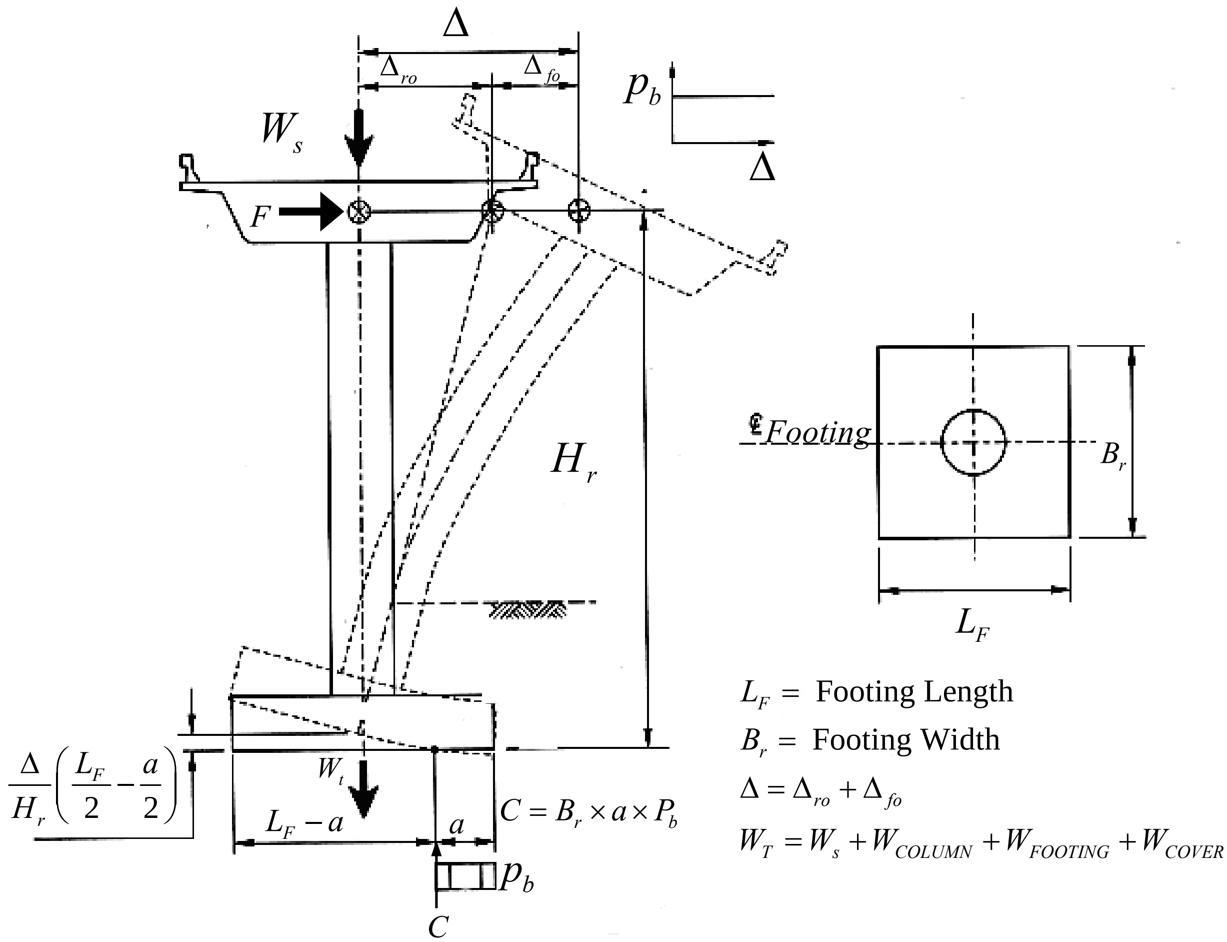


FIGURE 6.1: Rocking Equilibrium of a Single Column Bent.

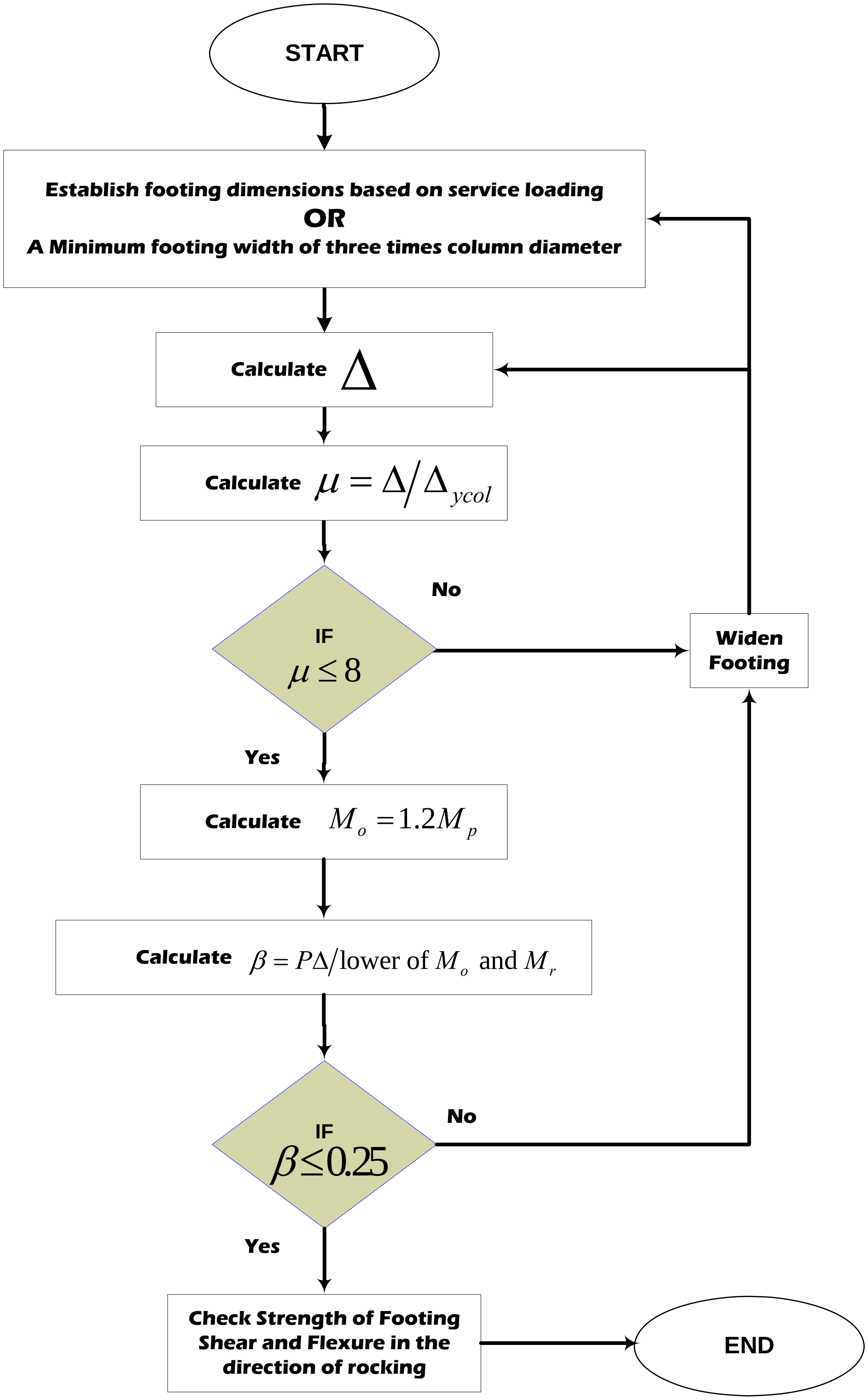


FIGURE 6.2: Flowchart for Design of a New Column on Spread Footing using Rocking Analysis

6.4 PILE CAP FOUNDATION

6.4.1 General

The design of pile foundation for SDC B shall be based on forces determined by capacity design principles or elastic seismic forces, whichever is smaller.

The design of pile foundation for SDC C or D shall be based on forces determined by capacity design principles.

6.4.2 Foundation with Standard Size Piles

Standard size piles are considered to have a nominal dimension less than or equal to 16 inches.

The provisions described below apply for columns with monolithic fixed connections to the footings designed for elastic forces as in SDC B or for column plastic hinge formation at the base as in SDC B, C, or D. For conformance to capacity design principles the foundations shall be designed to resist the overstrength column capacity M_o and the associated plastic shear V_o.

The design of standard size pile foundations in competent soil can be simplified using elastic analysis. For non-standard size piles, the distribution of forces to the piles and the pile cap may be influenced by the fixity of the pile connection to the pile cap in addition to the overall piles/pile cap flexibility. A more refined model that takes into account the pertinent parameters is recommended for establishing a more reliable force distribution.

A linear distribution of forces (see Figure 6.3) at different rows of piles, referred to as a simplified foundation model, is considered adequate provided a rigid footing response can be assumed. The rigid response of a footing can be assumed provided:

$$\frac{L_{ftg}}{D_{ftg}} \leq 2.5 \tag{6.7}$$

where

L_{ftg} = The cantilever length of the pile cap measured from the face of the column to the edge of the footing.

C6.4 PILE CAP FOUNDATION

C6.4.1 General

To meet uplift loading requirements during a seismic event or during ship impact, the depth of penetration may have to be greater than minimum requirements for compressive loading to mobilize sufficient uplift resistance. This uplift requirement can impose difficult installation conditions at locations where very hard bearing layers occur close to the ground surface. In these locations ground anchors, insert piles, and H-pile stingers can be used to provide extra uplift resistance in these situations.

If batter piles are used in SDC D, consideration must be given to (1) downdrag forces caused by dissipation of porewater pressures following liquefaction, (2) the potential for lateral displacement of the soil from liquefaction-induced flow or lateral spreading, (3) the ductility at the connection of the pile to the pile cap, and (4) the buckling of the pile under combined horizontal and vertical loading. These studies will have to be more detailed than those described within Article X.X. As such, use of batter piles should be handled on a case-by-case basis. Close interaction between the geotechnical engineer and the structural engineer will be essential when modeling the response of the batter pile for seismic loading.

For drained loading conditions, the vertical effective stress, σ'_v, is related to the groundwater level and thus affects pile capacity. Seismic design loads have a low probability of occurrence. This low probability normally justifies not using the highest groundwater level during seismic design.

C6.4.2 Design Requirements

Shear forces and overturning moments developing within SDC B will normally be small. Except in special circumstances, the load and resistance factors associated with Strength Limit State will control the number and size of the pile foundation system.

C7.4.3.3 Moment and Shear Design

Capacity Protection for the foundation design is not required for SDC B two reasons: (1) the design seismic load is likely to be small, and (2) the peak load will occur for only a short duration. By allowing uplift in only the most distant row of piles, the remaining piles will be in compression. Normally piles designed for the Strength Limit State will have a capacity reserve of 2.0 or more, resulting in adequate

D_{ftg} = The depth of the footing

Pile groups designed with the simplified foundation model can be sized to resist the plastic moment of the column M_p in lieu of M_{po} defined in Section 8.5.

For conforming to capacity design principles, the distribution of forces on these piles shall be examined about the X and Y axis in addition to the diagonal direction of the foundation cap considering that the principal axes of the column correspond to X and Y axis. For cases where the column principal axes do not correspond to pile cap axes, the number of iterations shall be enough to ensure hinging in the column.

The axial demand on an individual pile is found using Equations 6.8 and 6.9.

$$\left.\begin{matrix} C_{(i)}^{pile} \\ T_{(i)}^{pile} \end{matrix}\right\} = \frac{P_c}{N_p} \pm \frac{M_{p(y)}^{col} \times c_{x(i)}}{I_{p.g.(y)}} \pm \frac{M_{p(x)}^{col} \times c_{y(i)}}{I_{p.g.(x)}} \quad (6.8)$$

$$I_{p.g._{(y)}} = \sum n \times c_{x(i)}^{2} \qquad I_{p.g._{(x)}} = \sum n \times c_{y(i)}^{2} \quad (6.9)$$

where

$I_{p.g.}$ = Moment of inertia of the pile group defined by Equation 6.9

$M_{P(y).(x)}^{col}$ = The component of the column plastic moment capacity about the X or Y axis

N_P = Total number of piles in the pile group

n = The total number of piles at distance $c_{x(i)}$ or $c_{y(i)}$ from the centroid of the pile group

P_c = The total axial load on the pile group including column axial load (dead load+EQ load), footing weight, and overburden soil weight

capacity for vertical loads.

C7.4.3.4 Liquefaction Check

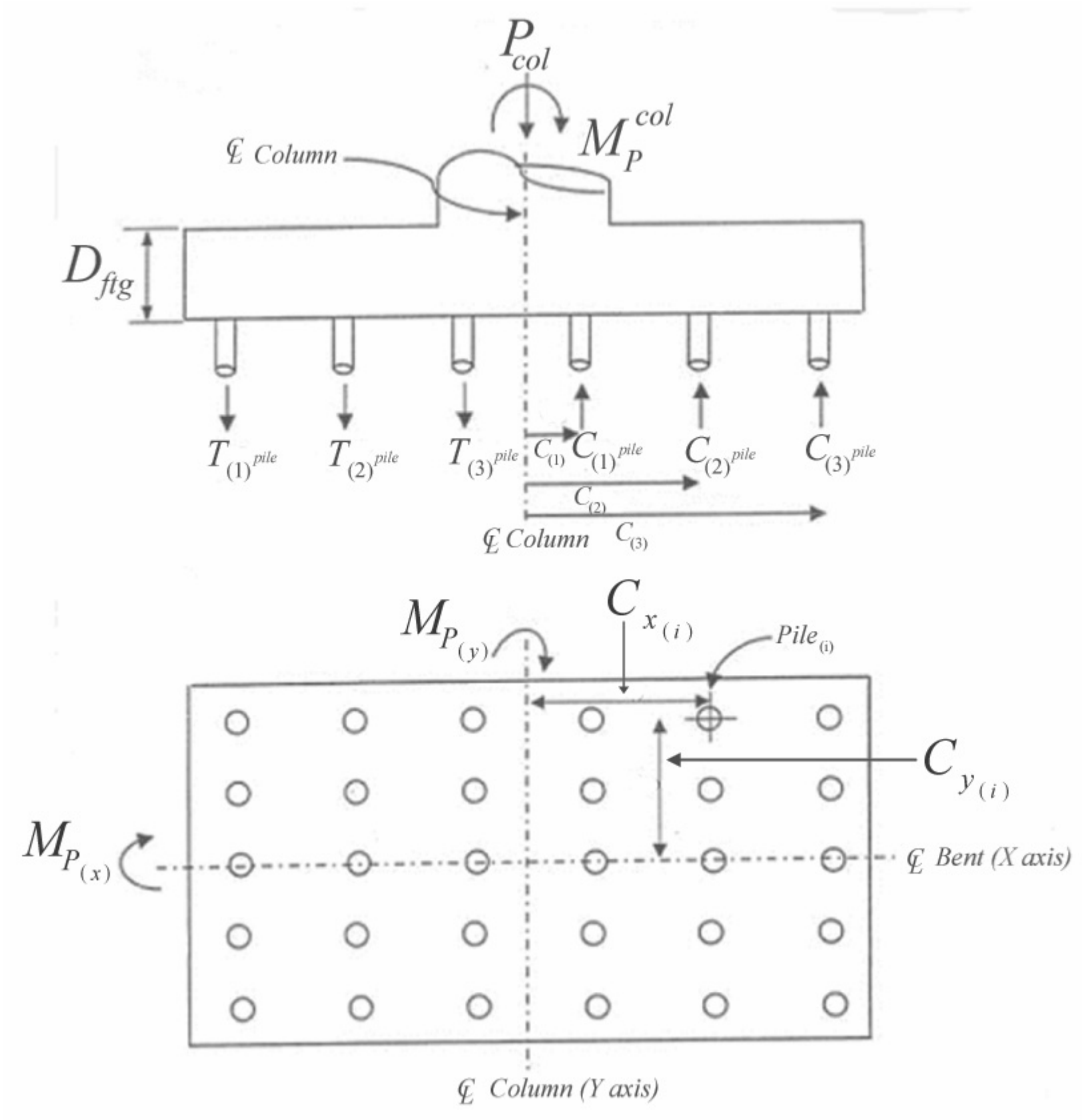


FIGURE 6.3: Simplified Pile Model for Foundations in Competent Soil

6.4.3 Pile Foundations in Soft Soil

In soft soils the pile cap may not dominate the lateral stiffness of the foundation, as is expected in competent soil, possibly leading to significant lateral displacements. The designer shall verify that the pile cap structural capacity exceeds the lateral demand transmitted by the columns, and the piles. In soft soils, piles shall be designed and detailed to accommodate imposed displacements and axial forces based on analytical findings.

6.4.4 Other Pile Requirements

Piles may be used to resist both axial and lateral loads. The minimum depth of embedment, together

C6.4.4 Drilled Shafts

Lam et al. (1998) provides a detailed discussion of the seismic response and design of drilled shaft foundations. Their discussion includes a summary of procedures to determine the stiffness matrix required to represent the shaft foundation in most dynamic analyses.

Drilled shaft foundations will often involve a single shaft, rather than a group of shafts. This is not the case for driven piles. In single shaft configuration the relative importance of axial and lateral response changes. Without the equivalent of a pile cap, lateral-load displacement of the shaft becomes more critical than the (axial) load-displacement relationships discussed above for driven piles.

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. Group reduction factors established in the geotechnical report shall be included in the analysis and design of piles required to resist lateral loads. The ultimate geotechnical capacity of the piles should be used in designing for seismic loads.

When reliable uplift pile capacity from skin-friction is present, the pile/footing connection detail is present, and the pile/footing connection detail and structural capacity of the pile are adequate, uplifting of a pile footing is acceptable, provided that the magnitude of footing rotation will not result in unacceptable performance according to $P-\Delta$ requirements stated in Article 4.11.5. Friction piles may be considered to resist an intermittent but not sustained uplift. For preliminary consideration including seismic loads, tension resistance may be equivalent to 50 percent of the ultimate compressive axial load capacity. In no case shall the uplift exceed the weight of material (buoyancy considered) surrounding the embedded portion of the pile.

Treated or untreated timber piles are not allowed. All concrete piles shall be reinforced to resist the design moments, shears, and axial loads. Minimum reinforcement shall be in accordance with Section 8.16 and seismic details where required.

Footings shall be proportioned to provide the required minimum spacing, clearance and embedment of piles according to current LRFD provisions. The spacing shall be increased when required by subsurface conditions. For SDC D, embedment of pile reinforcement in the footing cap shall be in accordance with Article 8.8.4.

6.4.5 Footing Joint Shear SDC C and D

All footing/column moment resisting joints in SDC C and D shall be proportioned so the principal stresses meet the following criteria:

Principal compression:

$$p_c \leq 0.25 f'_{ce} \tag{6.10}$$

Principal tension:

$$p_t \leq 12\sqrt{f'_{ce}} \text{ (psi)} \tag{6.11}$$

Many drilled-shaft foundation systems consist of a single shaft supporting a column. Compressive and uplift tensile loads on these shafts during seismic loading will normally be within the limits of the load factors used for gravity loading. However, checks should be performed to confirm that any changes in axial load do not exceed ultimate capacities in uplift or compression.

Special design studies can be performed to demonstrate that deformations are within acceptable limits if axial loads approach or exceed the ultimate uplift or compressive capacities if the drilled shaft is part of a group. These studies can be conducted using computer programs, such as APILE Plus (Reese, et al., 1997). Such studies generally will require rigorous soil-structure interaction modeling.

Where:

$$p_t = \frac{f_v}{2} - \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \qquad (6.12)$$

$$p_c = \frac{f_v}{2} + \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \qquad (6.13)$$

and

$$v_{jv} = \frac{T_{jv}}{B_{eff}^{ftg} \times D_{ftg}} \qquad (6.14)$$

$$T_{jv} = T_c - \sum T_{(i)}^{pile} \qquad (6.15)$$

where

T_c = Column tensile force associated with M_o

$\sum T_{(i)}^{pile}$ = Summation of the hold down force in the tension piles.

$$B_{eff}^{ftg} = \begin{cases} \sqrt{2}xD_{cj} & \text{Circular Column} \\ B_c + D_{cj} & \text{Rectangular Column} \end{cases} \qquad (6.16)$$

$$f_v = \frac{P_{col}}{A_{jh}^{ftg}} \qquad (6.17)$$

P_{col} = Column axial force including the effects of overturning

$$A_{jh}^{ftg} = \begin{cases} \left(D_{cj} + D_{ftg}\right)^2 & \text{for Circular Column} \\ \left(B_c + \frac{D_{ftg}}{2}\right) x \left(D_{cj} + \frac{D_{ftg}}{2}\right) & \text{for Rectangular Column} \end{cases} \qquad (6.18)$$

Where: A_{jh}^{ftg} is the effective horizontal area at mid-depth of the footing, assuming a 45° spread away from the boundary of the column in all directions, see Figure 6.4.

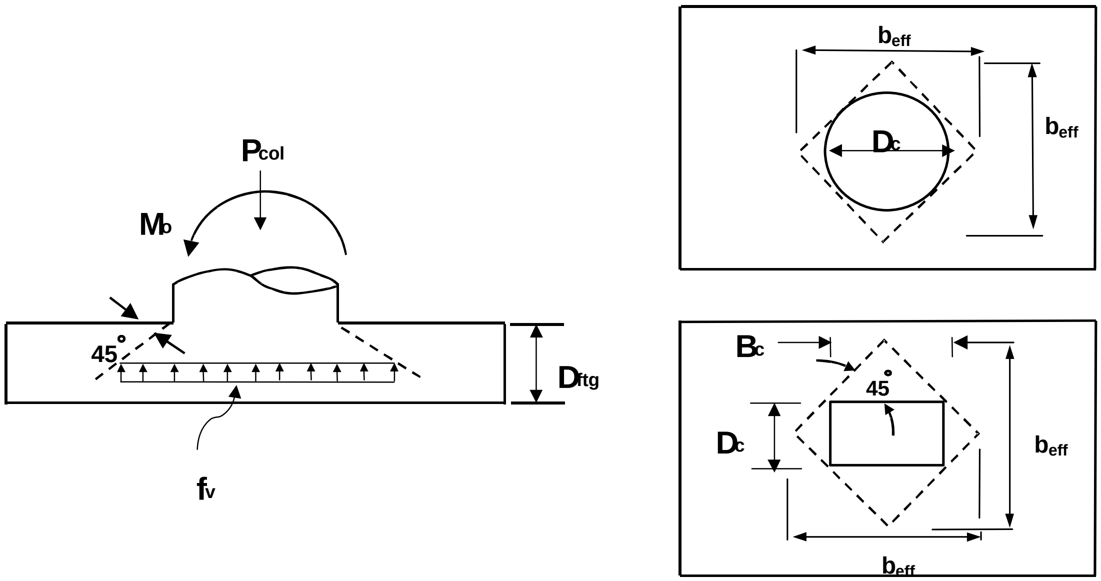


FIGURE 6.4: Effective Joint Width for Footing Joint Stress Calculation

6.4.6 Effective Footing Width For Flexure SDC C and D

For footings exhibiting rigid response and satisfying joint shear criteria the entire width of the footing can be considered effective in resisting the column overstrength flexure and the associated shear.

6.5 DRILLED SHAFTS

Design requirements of drilled shafts shall conform to requirements of columns in SDC B, C, or D as applicable.

The effects of degradation and aggredation in a streambed on fixity and plastic hinges locations shall be considered for SDC B, C, and D.

The effects of liquefaction on loss of $P-y$ strength shall be considered for SDC D.

A stable length shall be ensured for a single column/shaft. The stable length can be determined by using the lesser of 1.5 times the stable length achieved by applying lateral forces based on overstrength properties or applying a 1.5 multiplier factor on the lateral forces based on overstrength principals considered in determining the tip of the shaft required for lateral stability. The ultimate geotechnical capacity of single column/shaft foundation in compression and uplift shall not be exceeded under maximum seismic loads.

C6.5 DRILLED SHAFTS

Various studies (Lam et al., 1998) have found that conventional p-y stiffnesses derived for driven piles are too soft for drilled shafts. This stiffer response is attributed to a combination of (1) higher unit side friction, (2) base shear at the bottom of the shaft, and (3) the rotation of the shaft. The rotation effect is often implicitly included in the interpretation of lateral load tests, as most lateral load tests are conducted in a free-head condition. A scaling factor equal to the ratio of shaft diameter to 2 feet is generally applicable, according to Lam et al. (1998). The scaling factor is applied to either the linear subgrade modulus or the resistance value in the p-y curves. This adjustment is dependent on the construction method.

Base shear can also provide significant resistance to lateral loading for large diameter shafts. The amount of resistance developed in shear will be determined by conditions at the base of the shaft during construction. For dry conditions where the native soil is relatively undisturbed, the contributions for base shear can be significant. However, in many cases the base conditions result in low interface strengths. For this reason the amount of base shear to incorporate in lateral analyses will vary from case to case.

6.6 PILE EXTENSIONS

Design requirements of pile extensions shall conform to requirements of columns in SDC B, C, or D as applicable.

The effects of degradation and aggredation in a streambed on fixity and plastic hinges locations shall be considered in SDC B, C, and D.

The effects of liquefaction on loss of $P-y$ strength shall be considered in SDC D. Group reduction factors shall be included in the analysis and design of pile extensions subjected to lateral loading in the transverse direction.

6.7 ABUTMENT DESIGN REQUIREMENTS

The participation of abutment walls in the overall dynamic response of bridge systems to earthquake loading and in providing resistance to seismically induced inertial loads shall be considered in the seismic design of bridges following Article 5.2.

For no-collapse performance criteria, and assuming conventional cantilever retaining wall construction, horizontal wall translation under dynamic active pressure loading is acceptable. However, rotational instability may lead to collapse and thus must be prevented.

6.7.1 Longitudinal Direction Requirements

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 or retaining walls which may displace horizontally without significant restraint (e.g., superstructure supported by sliding bearings), the design approach is similar to that of a free-standing retaining wall, except that lateral force from the bridge superstructure needs to be included in equilibrium evaluations, as the superstructure moves outwards from the wall. A minimum force of 0.4 times the dead load reaction of the superstructure at the abutment shall be considered. Earthquake-induced active earth pressures should be computed using horizontal accelerations at least equal to 50% of the peak site ground acceleration (i.e., F_aS_s / 5.0). The pseudostatic Mononobe-Okabe method of analysis is recommended for computing lateral active soil pressures during seismic loading.

C6.7 ABUTMENT DESIGN REQUIREMENTS

C6.7.1 General

These LRFD *Guidelines* have been prepared to acknowledge the abutment to be used as an Earthquake Resistant Element (ERE) and be a part of the Earthquake Resistant System (ERS). If designed properly, the reactive capacity of the approach fill can provide significant benefit to the bridge-foundation system.

The effects of vertical acceleration may be omitted. Abutment displacements having a maximum drift of 4% can be tolerated under the No Collapse Performance Criteria. A limiting equilibrium condition should be checked in the horizontal direction. To ensure safety against potential overturning about the toe of a spread footing, a rocking analysis is required to converge in addition to conformance of $P-\Delta$ requirements as specified in Section 6.3.4. If necessary, wall design (initially based on a static service loading condition) should be modified to meet the above condition.

For monolithic abutments where the abutment forms an integral part of the bridge superstructure, the abutment shall be designed using one of the two alternatives depending on the contribution level accounted for in the analytical model:

1. At a minimum, the abutment shall be designed to resist the passive pressure applied by the abutment backfill.
2. If the abutment is part of the ERS and required to mobilize the full active pressure, a reduction factor greater than or equal to 0.5 shall be applied to the design forces provided a brittle failure does not exist in the load path transmitted to the superstructure.

For free-standing abutments which are restrained from horizontal displacement by anchors or concrete batter piles, earthquake-induced active earth pressures should be computed using horizontal accelerations at least equal to the site peak ground acceleration (i.e., $F_a S_s / 2.5$), as a first approximation. The Mononobe-Okabe analysis method is recommended using the above mentioned horizontal acceleration. Up to 50% reduction in the horizontal acceleration can be used provided the various components of the restrained wall can accommodate the increased level of displacement demand.

6.7.2 Transverse Direction Requirements

The provisions outlined in Article 5.2.4 shall be followed depending on the mechanism of transfer of superstructure transverse inertial forces to the bridge abutments and following the abutment contribution to the Earthquake Resisting System (ERS) applicable for SDC C and D.

6.7.3 Other Requirements for Abutments

For SDC D, abutment pile foundation design may be governed by liquefaction design requirements as outlined in Article 6.8.

To minimize potential loss of bridge access arising from abutment damage, monolithic or end diaphragm construction is strongly recommended for short length bridges less than 500 feet.

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

For SDC D, the abutment skew should be minimized. Bridges with skewed abutments above 20^o have a tendency for increased displacements at the acute corner. In the case where a large skew can not be avoided, sufficient seat width in conjunction with an adequate shear key shall be designed to ensure against any possible unseating of the bridge superstructure.

6.8 LIQUEFACTION DESIGN REQUIREMENTS

An evaluation of the potential and consequences of liquefaction within near surface soil shall be made in accordance with the following requirements:

Liquefaction assessment is required for a bridge in SPC D unless one of the following conditions is met:

a. The mean magnitude for the 5% PE in 50-year event is less than 6.5.

b. The mean magnitude for the 5% PE in 50-year event is less than 6.7 and the normalized Standard Penetration Test (SPT) blow count $[(N_1)_{60}]$ is greater than 20.

Procedures given in Appendix D and adopted from California Devision of Mines and Geology (DMG) Special Publication 117 shall be used to evaluate the potential for liquefaction.

If it is determined that liquefaction can occur at a bridge site then the bridge shall be supported on deep foundations or the ground improved so that liquefaction does not occur (See Appendix E). For liquefied sites subject to lateral flow, the Engineer shall consider the use of large diameter shafts in lieu of the conventional pile cap foundation type in order to minimize lateral flow demands on the bridge foundation. If liquefaction occurs then the bridge

C6.8 LIQUEFACTION DESIGN REQUIREMENTS

Liquefaction below a spread footing foundation can result in three conditions that lead to damage or failure of a bridge:

- loss in bearing support which causes large vertical downward movement,
- horizontal forces on the footing from lateral flow or lateral spreading of the soil, and
- settlements of the soil as porewater pressures in the liquefied layers dissipate.

Most liquefaction-related damage during past earthquakes has been related to lateral flow or spreading of the soil. In these cases ground movements could be 3 feet or more. If the spread footing foundation is located above the water table, as is often the case, it will be very difficult to prevent the footing from being displaced with the moving ground. This could result in severe column distortion and eventual loss of supporting capacity.

In some underwater locations, it is possible that the lateral flow could move past the footing without causing excessive loading; however, these cases will be limited.

Additional discussion of the consequences of liquefaction is provided in Appendix D to these *Guidelines*.

shall be designed and analyzed in two configurations as follows:

1. Nonliquefied Configuration: The structure shall be analyzed and designed, assuming no liquefaction occurs using the ground response spectrum appropriate for the site soil conditions.

2. Liquefaction Configuration: The structure as designed in Nonliquefied Configuration above shall be reanalyzed and redesigned, if necessary, assuming that the layer has liquefied and the liquefied soil provides the appropriate residual resistance (i.e., "p-y curves" or modulus of sub-grade reaction values for lateral pile response analyses consistent with liquefied soil conditions). The design spectra shall be the same as that used in Nonliquefied Configuration unless a site-specific response spectra has been developed using nonlinear, effective stress methods (e.g., computer program DESRA or equivalent) that properly account for the buildup in pore-water pressure and stiffness degradation in liquefiable layers. The reduced response spectra resulting from the site-specific nonlinear, effective stress analyses shall not be less than 2/3's of that used in Nonliquefied Configuration.

The Designer shall cover explicit detailing of plastic hinging zones for both cases mentioned above since it is likely that locations of plastic hinges for the Liquefied Configuration are different than locations of plastic hinges for the Non-Liquefied Configuration. Design requirements of SDC "D" including shear reinforcement shall be met for the Liquefied and Non-Liquefied Configuration.

C6.8 DESIGN REQUIREMENTS IF LIQUEFACTION AND GROUND MOVEMENT OCCURS

If liquefaction with no lateral flow occurs for SDC D bridges, then the only additional design requirements are those reinforcement requirements specified for the piles. Additional analyses are not required, although for essential or critical bridges additional analyses may be considered in order to assess the impact on the substructures above the foundation.

If liquefaction and lateral flow are predicted to occur for SDC D, a detailed evaluation of the effects of lateral flow on the foundation should be performed. Lateral flow is one of the more difficult issues to address because of the uncertainty in the movements that may occur. The design steps to address lateral flow are given in Appendix. Ultimate plastic rotation of the piles is permitted. This plastic rotation does imply that the piles and possibly other parts of the bridge will need to be replaced if these levels of deformation do occur. Design options range from (a) an acceptance of the movements with significant damage to the piles and columns if the movements are large, to (b) designing the piles to resist the forces generated by lateral spreading. Between these options are a range of mitigation measures to limit the amount of movement to tolerable levels for the desired performance objective. Pile group effects are not significant for liquefied soil.

7. STRUCTURAL STEEL COMPONENTS

7.1 GENERAL

The Engineer shall demonstrate that a clear, straight-forward load path (see Figure 7.1) within the superstructure, through the bearings or connections to the substructure, within the substructure, and ultimately to the foundation exists. All components and connections shall be capable of resisting the imposed seismic load effects consistent with the chosen load path.

The flow of forces in the prescribed load path must be accommodated through all affected components and their connections including, but not limited to, flanges and webs of main beams or girders, cross-frames, steel-to-steel connections, slab-to-steel interfaces, and all components of the bearing assembly from bottom flange interface through the anchorage of anchor bolts or similar devices in the substructure. The substructure shall also be designed to transmit the imposed force effects into the soils beneath the foundations.

The analysis and design of end diaphragms and cross-frames shall include the horizontal supports at an appropriate number of bearings, consistent with Articles 7.8 & 7.9.

The following requirements apply to bridges with either:

- a concrete deck that can provide horizontal diaphragm action or
- a horizontal bracing system in the plane of the top flange, which in effect provides diaphragm action.

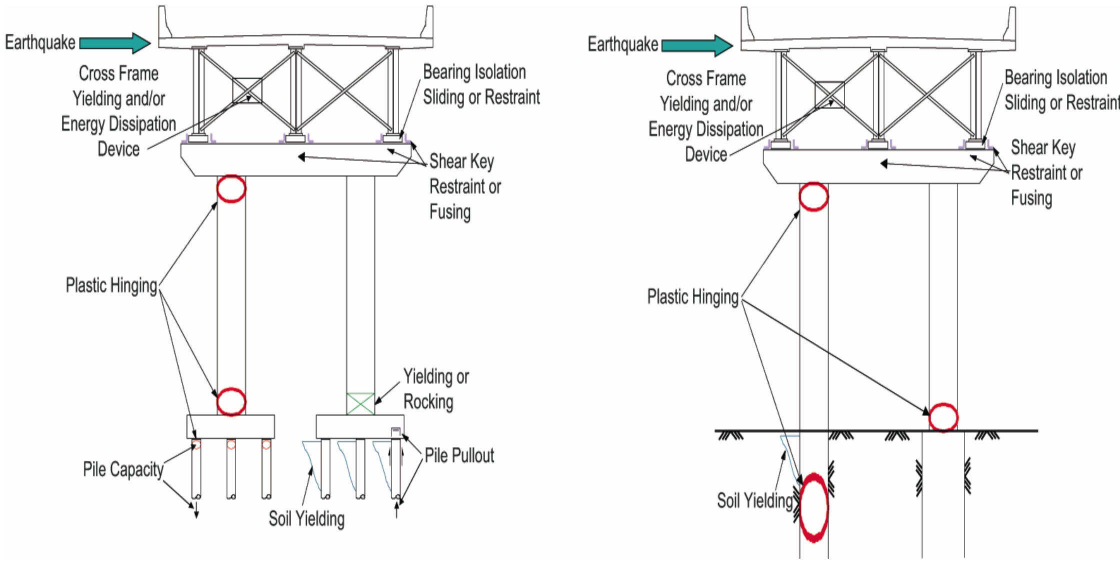
A viable load path (see Figure 7.1) shall be established to transmit the inertial loads to the foundation based on the stiffness characteristics of the deck, diaphragms, cross-frames, and lateral bracing. Unless a more refined analysis is made, an approximate load path shall be assumed as follows:

C7.1 General

Most components of steel bridges are not expected to behave in a cyclic inelastic manner during an earthquake. The provisions of Article 7.1 are only applicable to the limited number of components (such as specially detailed ductile substructures or ductile diaphragms) whose stable hysteretic behavior is relied upon to ensure satisfactory bridge seismic performance. The seismic provisions of Article 8.7 are not applicable to the other steel members expected to remain elastic during seismic response. In most steel bridges, the steel superstructure is expected (or can be designed) to remain elastic.

Recently, the number of steel bridges seriously damaged in earthquakes has risen dramatically. One span of the San Francisco-Oakland Bay Bridge collapsed due to loss of support at its bearings during the 1989 Loma Prieta earthquake, and another bridge suffered severe bearing damage (EERI, 1990). The end diaphragms of some steel bridges suffered damage in a subsequent earthquake in northern California (Roberts, 1992). During the 1994 Northridge earthquake some steel bridges, located close to the epicenter, sustained damage to either their reinforced concrete abutments, connections between concrete substructures and steel superstructures, steel diaphragms or structural components near the diaphragms (Astaneh-Asl et al., 1994). Furthermore, a large number of steel bridges were damaged by the 1995 Hyogoken-Nanbu (Kobe) earthquake. The concentration of steel bridges in the area of severe ground motion was considerably larger than for any previous earthquake and some steel bridges collapsed. Many steel piers, bearings, seismic restrainers and superstructure components suffered significant damage (Bruneau, Wilson and Tremblay, 1996). This experience emphasizes the importance of ductile detailing in the critical elements of steel bridges.

Research on the seismic behavior of steel bridges (e.g. Astaneh-Asl, Shen and Cho, 1993; Dicleli and Bruneau, 1995a, 1995b; Dietrich and Itani, 1999; Itani et al., 1998a; McCallen and Astaneh-Asl, 1996; Seim, Ingham and Rodriguez, 1993; Uang et al., 2000; Uang et al., 2001; Zahrai and Bruneau 1998) and findings from recent seismic evaluation and



Note: Affected components shown are inclusive to Type 1, 2 and 3 and do reflect specific components that may fuse under Type 1, 2 or 3 specified in Section 7.2.

FIGURE 7.1: Seismic Load Path and Affected Components

- The seismic inertia loads in the deck shall be assumed to be transmitted directly to the bearings through end diaphragms or cross-frames.
- The development and analysis of the load path through the deck or through the top lateral bracing, if present, shall utilize assumed structural actions analogous to those used for the analysis of wind loadings.

Reference to AASHTO LRFD Provisions is based on the 2004 Third Edition with subsequent updates pertinent to the section article(s) mentioned in this document.

7.2 PERFORMANCE CRITERIA

This section is intended for design of superstructure steel components. Those components are classified into two categories: Ductile and Essentially Elastic. Based on the characteristics of the bridge structure, the designer has one of three options for a seismic design strategy:

Type 1 – Design a ductile substructure with an essentially elastic superstructure.

rehabilitation projects (e.g. Astaneh and Roberts, 1993, 1996; Ballard et al., 1996; Billings et al, 1996; Dameron et al., 1995; Donikian et al., 1996; Gates et al., 1995; Imbsen et al., 1997; Ingham et al., 1996; Jones et al., 1997; Kompfner et al., 1996; Maroney 1996; Prucz et al., 1997; Rodriguez and Inghma, 1996; Schamber et al., 1997; Shirolé and Malik, 1993; Vincent et al., 1997) further confirm that seismically induced damage is likely in steel bridges subjected to large earthquakes and that appropriate measures must be taken to ensure satisfactory seismic performance.

The intent of Article 7.2 is to ensure the ductile response of steel bridges during earthquakes. First, effective load paths must be provided for the entire structure. Following the concept of capacity design, the load effect arising from the inelastic deformations of part of the structure must be properly considered in the design of other elements that are within its load path.

Second, steel substructures must be detailed to ensure stable ductile behavior. Note that the term “substructure” here refers to structural systems exclusive of bearings (Article 8.9) and articulations, which are considered in other sections. Steel substructures, although few, need ductile detailing to provide satisfactory seismic performance.

Type 2– Design an essentially elastic substructure with a ductile superstructure.

Type 3 – Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.

In this section, reference to an essentially elastic component is used where the force demand to the nominal capacity ratio of any member in the superstructure is less than 1.5.

Seismic design forces for individual members and connections of bridges identified as Type 2 are determined by dividing the unreduced elastic forces by the appropriate Response Modification Factor (R) as specified in Article 7.2.2. These factors shall only be used when all of the design requirements of Section 7 are satisfied. A combination of orthogonal seismic forces equivalent to the orthogonal seismic displacement combination specified in Section 4.4 shall be used to obtain the unreduced elastic forces.

7.2.1 Type 1

For Type 1 choice, the designer shall refer to Section 8 or Article 7.5 of this document on designing for a ductile substructure as applicable to SDC C and D.

7.2.2 Type 2

For Type 2 choice, the design of the superstructure is accomplished using a force based approach with an appropriate reduction for ductility. Those factors are used for the design of transverse bracing members, top laterals and bottom laterals. For SDC B, C, or D a reduction factor, R, equal to 3 is used for ordinary bracing that is a part of the Earthquake Resistant System (ERS) not having ductile end-diaphragms as defined in Section 7.4.6. The reduction factor, R, may be increased to 4 for SDC D, if the provisions in Article 7.4.6, Ductile End-Diaphragm in Slab-on-Girder Bridge, are satisfied.

For simply supported spans with special end-diaphragms in compliance with Article 7.4.6 are used, the location of the diaphragms must as a minimum be placed at the ends of each span.

For continuous spans where these special diaphragms are used, the location of diaphragms must as a minimum be placed over each bent and one cross-frame spacing adjacent to the opposite faces of

Special consideration may be given to slip-critical connections that may be subjected to cyclic loading. Some researchers have expressed concern that the Poisson effect may cause steel plate thickness to reduce, when yielding on a component's net section occurs during seismic response, which may translate into a reduced clamping action on the faying surfaces after the earthquake. This has not been experimentally observed, nor noted in post-earthquake inspections, but the impact of such a phenomenon would be to reduce the slip-resistance of the connection, which may have an impact on fatigue resistance. This impact is believed to be negligible for a Category C detail for finite life, and a Category D detail for infinite life. Design to prevent slip for the Expected Earthquake should be also considered.

If the forces from the substructure corresponding to the overstrength condition are used to design the superstructure, the distribution of these forces maynot be the same as that of the elastic demand analysis forces. The Engineer may calculate a more refined distribution of the inertial forces present when a full inelastic mechanism has developed in the EREs. However, in lieu of such a calculation, the simpler linear distribution may be used, as long as the applied forces are in equilibrium with the substructure's plastic-moment forces. The vertical spatial relationship between location of the substructure plastic resistance and the location of the superstructure inertia force application shall also be considered in this analysis.

Diaphragms, cross-frames, lateral bracing, bearings, and substructure elements are part of an earthquake-resisting system in which the lateral loads and performance of each element are affected by the strength and stiffness characteristics of the other elements. Past earthquakes have shown that when one of these elements responded in a ductile manner or allowed some movement, damage was limited. In the strategy followed herein, it is assumed that ductile plastic hinging in substructure or seismic isolator units are the primary source of energy dissipation.

Even if a component does not participate in the load path for seismic forces it must deform under the seismic loads. Such components must be checked that they have deformation capacity sufficient to maintain their load resistance under seismic-induced deformations.

A continuous path is necessary for the transmission of the superstructure inertia forces to the substructure. Concrete decks have significant rigidity in their horizontal plane, and in short-to-medium slab-on-girder spans, their response approaches rigid the bent. The use of special diaphragms at opposite faces of an in-span hinge should be carefully assessed to ensure adequate vertical load capacity of the in-span hinge when subjected to deformations in the inelastic range.

For SDC B, C, or D a single angle bracing maybe used for the diagonal member of the end-cross-frame. As this practice is typical and favored for ease of construction, the design process for a single angle bracing shall follow AISC stand alone document on "LRFD Design Specification for Single-Angle Members". This document is included in Appendix F.

For SDC D, double angles with stitches may be used as members of the end diaphragm ERS. Members with stitches shall follow the design process included in the AISC LRFD Specifications Chapter E on compact and non-compact prismatic members subject to axial compression through the centroidal axis.

7.2.3 Type 3

For Type 3 choice, the designer shall assess the overstrength capacity for the fusing interface including shear keys and bearings, then design for an essentially elastic superstructure and substructure. The minimum lateral design force shall be calculated using an acceleration of 0.4 g or the elastic seismic force whichever is smaller. If isolation devices are used, the superstructure shall be designed as essentially elastic (see Section 7.8).

Other framing systems and frames that incorporate special bracing, active control, or other energy absorbing devices, or other types of special ductile superstructure elements shall be designed on the basis of published research results, observed performance in past earthquakes, or special investigation, and provide a level of safety comparable to those in the AASHTO LRFD provisions.

7.3 MATERIALS

For SDC C and D ductile substructure elements and ductile end-diaphragms, as defined in Articles 7.4.6 through 7.5, shall be made of either:

a. ASTM A709 Grade 50 and Grade 50W steels

b. ASTM A992 steel, or

c. A500 Grade B or A501 steels (if structural tubing or pipe).

body motion. Therefore, the lateral loading of the intermediate diaphragms is minimal, consisting primarily of local tributary inertia forces from the girders themselves.

All bearings in a bridge do not usually resist load simultaneously, and damage to only some of the bearings at one end of a span is not uncommon. When this occurs, high load concentrations can result at the location of the other bearings, and this effect shall be taken into account in the design of the end diaphragms and pier diaphragms. Also, a significant change in the load distribution between end diaphragm members and the pier may occur.

C7.3 Materials

To ensure that the objective of capacity design is achieved, Grade 36 steel is not permitted for the components expected to respond in a ductile manner. Grade 36 is difficult to obtain and contractors often substitute it with a Grade 50 steel. Furthermore it has a wide range in its expected yield and ultimate strength and large overstrength factors to cover the anticipated range of property variations. The common practice of dual-certification for rolled shapes, recognized as a problem from the perspective of capacity design following the Northridge earthquake, is now becoming progressively more common also for steel plates. As a result, only Grade 50 steels are allowed within the scope of Article X.X, with a R_y of 1.1.

In those instances when Grade 36 must be used, capacity design must be accomplished assuming a Grade 50 steel (i.e., with a R_y of 1.5 applied to the F_y of 36 ksi)

The use of A992 steel is explicitly permitted. Even though this ASTM grade is currently designated for "shapes for buildings", there is work currently being done to expand applicability to any shapes. ASTM 992 steel, recently developed to ensure good ductile seismic performance, is specified to have

For Grade 50 steel, an overstrength factor equal to 1.2 shall be used for design of adjacent members and connections.

For SDC B ASTM A709 Grade 36 may be used provided an overstrength factor, R_y, of 1.5 is used for design of adjacent members and connections.

In Article 7.2, the nominal capacity is defined as the resistance of a member, connection or structure based on the expected yield strength (F_{ye}) and the nominal dimensions and details of the final section(s) chosen, calculated with all material resistance factors taken as 1.0.

Overstrength capacity is defined as the resistance of a member, connection or structure based on the nominal dimensions and details of the final section(s) chosen, calculated accounting for the expected development of large strains and associated stresses larger than the minimum specified yield values.

The expected yield strength shall be used in the calculation of nominal resistances, where expected yield strength is defined as $F_{ye} = R_y F_y$ where R_y shall be taken as 1.1 for the permitted steels listed above.

Welding requirements shall be compatible with AWS/AASHTO D1.5-96 *Structural Bridge Welding Code*. However, under-matched welds are not permitted for special seismic hysteretic energy dissipating systems (such as ductile substructures and ductile diaphragms).

Steel members expected to undergo significant plastic deformations during a seismic event shall meet the toughness requirements of ASTM Standard A709/A709M, Supplementary Requirement S84 (Fracture Critical). Welds metal connecting these members shall meet the toughness requirements specified in the AWS D1.5 Bridge Specification for Zone III (ANSI/AASHTO/AWS, 1995).

7.4 MEMBER REQUIREMENTS FOR SDC C AND D

7.4.1 Limiting Slenderness Ratios

Bracing members shall have a slenderness ratio KL/r less than 120. The length of a member shall be taken between the points of intersection of members. An effective length factor K of 0.85 of compression members in braced structures shall be used unless a lower value can be justified by an appropriate analysis. The slenderness parameter λ_c for axial

both a minimum and maximum guaranteed yield strength, and may be worthy of consideration for ductile energy-dissipating systems in steel bridges.

Since other steels may be used, provided that they are comparable to the approved Grade 345 steels, High Performance Steel (HPS) Grade 345 would be admissible, but not HPS Grade 485 (or higher). This is not a detrimental restriction for HPS steel, as the scope of Article 8.7 encompasses only a few steel members in a typical steel bridge. (Based on limited experimental data available, it appears that HPS Grade 485 has a lower rotational ductility capacity and may not be suitable for “ductile fuses” in seismic applications).

When other steels are used for energy dissipation purposes, it is the responsibility of the designer to assess the adequacy of material properties available and design accordingly.

Other steel members expected to remain elastic during earthquake shall be made of steels conforming to Article 6.4 of the AASHTO LRFD provisions.

Steel members and weld materials shall have adequate notch toughness to perform in a ductile manner over the range of expected service temperatures. The A709/A709M S84 "Fracture-Critical Material Toughness Testing and Marking" requirement, typically specified when the material is to be utilized in a fracture-critical application as defined by AASHTO, is deemed to be appropriate to provide the level of toughness sought for seismic resistance. For weld metals, the AASHTO/AWS D1.5 Bridge Welding Code requirement for Zone III, familiar to the bridge engineering community, is similar to the 20 ft-lbs at -20F requirement proposed by the SAC Joint Venture for weld metal in welded moment frame connections in building frames.

The capacity design philosophy and the concept of capacity-protected element are defined in Article 4.8.

compressive load dominant members, and λ_b for flexural dominant members shall not exceed the limiting values, λ_{cp} and λ_{bp} respectively as specified in Table 7.1.

7.4.2 Limiting Width-Thickness Ratios

For essentially elastic components, the width-thickness ratios shall not exceed the limiting value λ_r as specified in Table 7.2. For ductile components, width-thickness ratios shall not exceed the value λ_p as specified in Table 7.2.

7.4.3 Flexural Ductility for Members with Combined Flexural and Axial Load

Ductility in bending may be utilized only if axial loads are less than 60% of the nominal yield strength of member. Demand-to-capacity ratios or displacement ductilities shall be kept less than unity if the axial load coinciding with the moment is greater than 60% of the nominal yield strength of the member.

7.4.4 Combined Axial and Bending

Members under combined axial and bending interaction shall be checked using interaction equations following current AASHTO-LRFD Provisions.

Table 7.1: Limiting Slenderness Parameters

Member Classification		Limiting Slenderness Parameters (λ_c or λ_b)	
Ductile	Axial Load Dominant $P/P_n \geq M/M_{ns}$	λ_{cp}	0.75
	Flexural Moment Dominant $M/M_{ns} \geq P/P_n$	λ_{bp}	$2500/F_y$
Essentially Elastic	Axial Load Dominant $P/P_n \geq M/M_{ns}$	λ_{cp}	1.5
	Flexural Moment Dominant $M/M_{ns} \geq P/P_n$	λ_{bp}	$750/\sqrt{F_y}$

The following symbols are used in Table 7.1

M = flexural moment of a member due to seismic and permanent loads (kips-in.)

M_{ns} = nominal flexural moment strength of a member (kips-in.)

P = axial load of a member due to seismic and permanent loads (kips)

P_n = nominal axial strength of a member (kips)

λ_c = $\left(\dfrac{KL}{r\pi}\right)\sqrt{\dfrac{F_y}{E}}$ (slenderness parameter of axial load dominant members)

λ_b = $\dfrac{KL}{r_y}$ (slenderness parameter of flexural moment dominant members)

λ_{cp} = limiting slenderness parameter for axial load dominant members

λ_{bp} = limiting slenderness parameter for flexural moment dominant members

K = effective length factor of a member

L = unsupported length of a member (in.)

r = radius of gyration (in.)

r_y = radius of gyration about minor axis (in.)

F_y = specified minimum yield strength of steel (ksi)

E = modulus of elasticity of steel (29,000 ksi)

Table 7.2: Limiting Width-Thickness Ratios

Description of Elements	Width-Thickness Ratios	λ_r	λ_p
UNSTIFFENED ELEMENTS			
Flanges of I-shaped rolled beams and channels in flexure.	*b/t*	$\frac{141}{\sqrt{F_y - 10}}$	$\frac{52}{\sqrt{F_y}}$
Outstanding legs of pairs of angles in continuous contact; flanges of channels in axial compression; angles and plates projecting from beams or compression members.	*b/t*	$\frac{95}{\sqrt{F_y}}$	$\frac{52}{\sqrt{F_y}}$
STIFFENED ELEMENTS			
Flanges of square and rectangular box and hollow structural section of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds.	*b/t*	$\frac{238}{\sqrt{F_y}}$	$110/\sqrt{F_y}\,^{(tubes)}$ $150/\sqrt{F_y}\,^{(others)}$
Unsupported width of cover plates perforated with a succession of access holes.	*b/t*	$\frac{317}{\sqrt{F_y}}$	$\frac{152}{\sqrt{F_y}}$
All other uniformly compressed stiffened elements, i.e., supported along two edges.	*b/t* *h/t$_w$*	$\frac{253}{\sqrt{F_y}}$	$110/\sqrt{F_y}\,^{(w/lacing)}$ $150/\sqrt{F_y}\,^{(others)}$
Webs in flexural compression.	*h/t$_w$*	$\frac{970}{\sqrt{F_y}}$	$\frac{520}{\sqrt{F_y}}$
Webs in combined flexural and axial compression.	*h/t$_w$*	$\frac{970}{\sqrt{F_y}}\,x\left(1-\frac{0.74P}{\phi_b P_y}\right)$	$P_u \le 0.125\phi_b P_y$ $\frac{520}{\sqrt{F_y}}\left(1-\frac{1.54P_u}{\phi_b P_y}\right)$ *For* $P_u > 0.125\phi_b P_y$ $\frac{191}{\sqrt{F_y}}\left(2.33-\frac{P_u}{\phi_b P_y}\right)$ $\ge \frac{253}{\sqrt{F_y}}$
Longitudinally stiffened plates in Compression.	*b/t*	$\frac{113\sqrt{k}}{\sqrt{F_y}}$	$\frac{75\sqrt{k}}{\sqrt{F_y}}$
Round HSS in axial compression or Flexure.	*D/t*	$\frac{2600}{F_y}$	$\frac{1300}{F_y}$

Notes:

1. Width-Thickness Ratios shown with a ★ are from AISC-LRFD (1993) and AISC-Seismic Provisions (1997).
2. k = buckling coefficient specified by Article 6.11.2.1.3a of the current AASHTO-LRFD (AASHTO, 2002)
   for n=1, $\boldsymbol{k} = (8\boldsymbol{I}_s/\boldsymbol{bt}^3)^{1/3} \le 4.0$ for n=2,3,4 and 5, $\boldsymbol{k} = (14.3\boldsymbol{I}_s/\boldsymbol{bt}^3\boldsymbol{n}^4)^{1/3} \le 4.0$
   n = number of equally spaced longitudinal compression flange stiffeners
   I_s = moment of inertia of a longitudinal stiffener about an axis parallel to the bottom flange and taken at the base of the stiffener.
3. Resistance factor ϕ_b =0.9

7.4.5 Weld Locations

Welds located in the expected inelastic region of ductile components shall be made complete penetration welds. Partial penetration groove welds are not permitted in these regions. Splices are not permitted in the inelastic region of ductile components.

7.4.6 Ductile End-Diaphragm in Slab-on-Girder Bridge

Ductile end-diaphragms in slab-on-girder bridges can be designed to be the ductile energy dissipating elements for seismic excitations in the transverse directions of straight bridges provided that:

a. Specially detailed diaphragms, which are capable of dissipating energy in a stable manner without strength degradation, may be used. The diaphragm behavior must be verified by cyclic testing;

b. Only ductile energy dissipating systems with adequate seismic performance that has been proven through cyclic inelastic testing are used;

c. Design considers the combined relative stiffness and strength of end-diaphragms and girders (including bearing stiffeners) in establishing the diaphragms strength and design forces to consider for the capacity protected elements;

d. The response modification factor, R, to be considered in design of the ductile diaphragm is given by:

$$R = \left(\frac{\mu + \dfrac{K_{DED}}{K_{SUB}}}{1 + \dfrac{K_{DED}}{K_{SUB}}} \right) \qquad (7.1)$$

where μ is the ductility capacity of the end-diaphragm itself, and K_{DED}/K_{SUB} is the ratio of the stiffness of the ductile end-diaphragms and substructure; unless the engineer can demonstrated otherwise, μ should not be taken greater than 4;

e. All details/connections of the ductile end-diaphragms are welded;

f. The bridge does not have horizontal wind-bracing connecting the bottom flanges of girders. However, if the last wind bracing panel before each support is designed as a ductile panel equivalent and in parallel to its adjacent vertical end-diaphragm;

g. An effective mechanism is present to ensure transfer of the inertia-induced transverse horizontal seismic forces from the slab to the diaphragm.

Special design provisions for a Concentrically Braced Frame (CBF) or an Eccentrically Braced Frame (EBF), following the LRFD AISC Seismic Provisions for Structural Steel Buildings 1997, shall be used in addition to requirements stated in this document.

Overstrength factors to be used to design the capacity-protected elements depend on the type of ductile diaphragm used, and shall be supported by available experimental research results.

7.5 DUCTILE MOMENT RESISTING FRAMES AND SINGLE COLUMN STRUCTURES FOR SDC C AND D

This section applies to ductile moment-resisting frames and bents, constructed with steel I-shape beams and columns connected with their webs in a common plane. For SDC C or D, complying with a Type 1 design, the columns shall be designed as ductile structural elements using a force reduction factor of 4. The beams, the panel zone at column-beam intersections and the connections shall be designed as Essentially Elastic Elements.

7.5.1 Columns

Width-to-thickness ratios of compression elements of columns shall be in compliance with Table 7.3. Full penetration flange and web welds are required at column-to-beam (or beam-to-column) connections.

The resistance of columns to combined axial load and flexure shall be determined in accordance with Article 6.9.2.2 of the AASHTO LRFD provisions. The factored axial compression due to seismic load and permanent loads shall not exceed $0.20A_gF_y$.

The shear resistance of the column web shall be determined in accordance with Article 6.10.9 of the AASHTO LRFD provisions.

The potential plastic hinge regions (Article 4.11.8), near the top and base of each column, shall be laterally supported and the unsupported distance (i.e., between the plastic hinges) from these locations

C.7.5 DUCTILE MOMENT-RESISTING FRAMES AND SINGLE COLUMN STRUCTURES

It is believed that properly detailed fully welded column-to-beam or beam-to-column connections in the moment-resisting frames that would typically be used in bridges (See Figure C8.7.4-1) can exhibit highly ductile behavior and perform adequately during earthquakes (contrary to what was observed in buildings following Northridge). As a result, strategies to move plastic hinges away from the joints are not required in the *Specifications*.

However, the designer may still elect to provide measures (such as haunches at the end of yielding members) to locate plastic hinges some distance away from the welded beam-to-column or column-to-beam joint (SAC, 1995, 1997, 2000).

Although beams, columns and panel zones can all be designed, detailed and braced to undergo severe inelastic straining and absorb energy, the detailing requirements of Article 8.7 address common bridge structures with deep non-compact beams much stiffer in flexure than their supporting steel columns, and favor systems proportioned so that plastic hinges form in the columns. This is consistent with the philosophy adopted for concrete bridges.

Even though some bridges could be configured and designed to develop stable plastic hinging in beams without loss of structural integrity, the large gravity loads that must simultaneously be resisted by those beams also make plastic hinging shall not exceed $17250 r_y / F_y$. These lateral supports shall be provided either directly to the flanges or indirectly through a column web stiffener or a continuity plate. Each column flange lateral support shall resist a force of not less than 2% of the nominal column flange strength (btF_y) at the support location. The possibility of complete load reversal shall be considered.

When lateral support can not be provided, the column maximum slenderness shall not exceed 60 and transverse moments produced by the forces otherwise resisted by the lateral bracing (including the second order moment due to the resulting column displacement) shall be included in the seismic load combinations.

Splices that incorporate partial joint penetration groove welds shall be located away from the plastic hinge regions as defined in Article 4.11.8 at a minimum distance equal to the greater of:

a. one-fourth the clear height of column;

b. twice the column depth; and

c. 39 inches.

7.5.2 Beams

The factored resistance of the beams shall be determined in accordance with Article 6.10.1.2 of the AASHTO LRFD provisions. At a joint between beams and columns the sum of the factored resistances of the beams shall not be less than the sum of the probable resistances of the column(s) framing into the joint. The probable flexural resistance of columns shall be taken as the product of the overstrength factor times the columns nominal flexural resistance determined either in accordance to Article 6.9.2.2 of the AASHTO LRFD provisions, or by

$$M_{nx} = 1.18 M_{px}\left[1 - \frac{P_u}{AF_{ye}}\right] \le M_{px} \qquad (7.2)$$

unless demonstrated otherwise by rational analysis, and where M_{px} is the column plastic moment under pure bending calculated using F_{ye} .

at mid-span likely as part of the plastic collapse mechanism. The resulting deformations can damage the superstructure (for example, the diaphragms or deck).

The special case of multi-tier frames is addressed in Article 8.7.4.4.

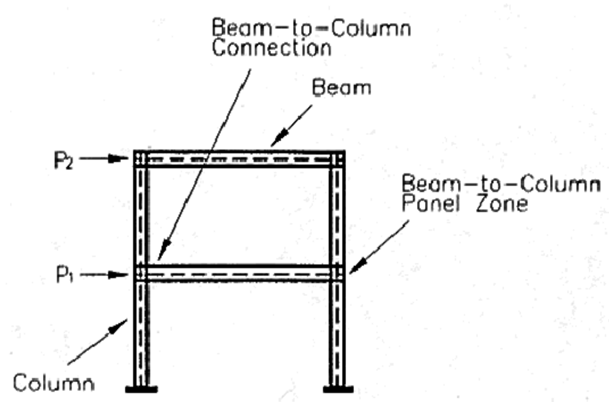


Figure C8.7.4-1 Example of Moment Frame/Bent

C7.5.1 Columns

At plastic hinge locations, members absorb energy by undergoing inelastic cyclic bending while maintaining their resistance. Therefore, plastic design rules apply, namely, limitations on width-to-thickness ratios, web-to-flange weld capacity, web shear resistance, and lateral support.

Axial load in columns is also restricted to avoid early deterioration of beam-column flexural strengths and ductility when subject to high axial loads. Tests by Popov et al. (1975) showed that W-shaped columns subjected to inelastic cyclic loading suffered sudden failure due to excessive local buckling and strength degradation when the maximum axial compressive load exceeded $0.50A_gF_y$. Tests by Schneider et al. (1992) showed that moment-resisting steel frames with hinging columns suffer rapid strength and stiffness deterioration when the columns are subjected to compressive load equal to approximately $0.25A_gF_y$. Most building codes set this limit at $0.30A_gF_y$.

The requirement for lateral support is identical to Equation 6.10.4.1.7-1 of the AASHTO LRFD provisions with a moment M_l of zero at one end of the member, but modified to ensure inelastic rotation capacities of at least four times the elastic rotation corresponding to the plastic moment (resulting in a coefficient of 17250 instead of the approximately 25000 that would be obtained for Equation

7.5.3 Panel Zones and Connections

Column-beam intersection panel zones, moment resisting connections and column base connections shall be designed as Essentially Elastic Elements.

Panel zones shall be designed such that the vertical shearing resistance is determined in accordance with Article 6.10.9.3 of the AASHTO LRFD provisions.

Beam-to-column connections shall have resistance not less than the resistance of the beam stipulated in Article 7.5.2.

Continuity plates shall be provided on both sides of the panel zone web and shall finish with total width of at least 0.8 times the flange width of the opposing flanges. Their *b/t* shall meet the limits for projecting elements of Article 6.9.4.2 of the AASHTO LRFD provisions. These continuity plates shall be proportioned to meet the stiffener requirements stipulated in Article 6.10.8.2 of the AASHTO LRFD provisions and shall be connected to both flanges and the web.

Flanges and connection plates in bolted connections shall have a factored net section ultimate resistance calculated by Equation 6.8.2.1-2, at least equal to the factored gross area yield resistance given by Equation 6.8.2.1-1, with A_g and A_n in Article 6.8.2.1 taken here as the area of the flanges and connection plates in tension. These referenced equations and article are from the AASHTO LRFD provisions.

7.5.4 Multi-Tier Frame Bents

For multi-tier frame bents, capacity design principles as well as the requirements of Articles 7.5.1, 7.5.2, and 7.5.3 may be modified by the engineer to achieve column plastic hinging only at the top of the column. Column plastic hinging at the base where fixity to the foundation is needed shall be assessed where applicable.

7.6 CONCRETE FILLED STEEL PIPES FOR SDC C AND D

Concrete-filled steel pipes used as columns, piers, or piles expected to develop full plastic hinging of the composite section as a result of seismic response shall be designed in accordance with Articles 6.9.2.2, 6.9.5, 6.12.3.2.2, of the AASHTO LRFD provisions as well as the requirements in this article.

6.10.4.1.7-1 of the AASHTO LRFD provisions). Consideration of a null moment at one end of the column accounts for changes in location of the inflexion point of the column moment diagram during earthquake response. Figure 10.27 in Bruneau et al. (1997) could be used to develop other unsupported lengths limits.

Built-up columns made of fastened components (e.g., bolted or riveted) are beyond the scope of these Guidelines.

C7.5.2 Beams

Since plastic hinges are not expected to form in beams, beams need not conform to plastic design requirements.

The requirement for beam resistance is consistent with the outlined capacity-design philosophy. The beams should either resist the full elastic loads or be capacity-protected. In the extreme load situation, the capacity-protected beams are required to have nominal resistances of not less than the combined effects corresponding to the plastic hinges in the columns attaining their probable capacity and the probable companion permanent load acting directly on the beams. The columns' probable capacity should account for the overstrength due to higher yield than specified yield and strain hardening effects. The value specified in Article 6.9.2.2 of the AASHTO LRFD provisions, used in conjunction with the resistance factor ϕ_f for steel beams in flexure of 1.00, (Article 6.5.4.2 of the AASHTO LRFD provisions) is compatible with the AISC (1997) $1.1R_y$ used with a resistance factor ϕ of 0.9 (here R_y is embedded in F_{ye}).

C7.5.3 Panel Zones and Connections

The panel zone should either resist the full elastic load (i.e., R=1.0) or be capacity-protected.

Column base connections should also resist the full elastic loads (R=1.0) or be capacity-protected, unless they are designed and detailed to dissipate energy.

Panel zone yielding is not permitted.

There is a concern that doubler plates in panel zones can be an undesirable fatigue detail. For plate-girder sections, it is preferable to specify a thicker web plate, if necessary, rather than use panel zone doubler plates.

7.6.1 Combined Axial Compression and Flexure

Concrete-filled steel pipe members required to resist both axial compression and flexure and intended to be ductile substructure elements shall be proportioned so that:

$$\frac{P_u}{P_r} + \frac{BM_u}{M_{rc}} \leq 1.0 \tag{7.3}$$

and

$$\frac{M_u}{M_{rc}} \leq 1.0 \tag{7.4}$$

where P_r is defined in Articles 6.9.2.1 and 6.9.5.1 of the AASHTO LRFD provisions, and M_{rc} is defined in Article 7.6.2,

$$B = \frac{P_{ro} - P_{rc}}{P_{rc}} \tag{7.5}$$

P_{ro} = factored compressive resistance (Articles 6.9.2.1 and 6.9.5.1 of the AASHTO LRFD provisions) with $\lambda = 0$,

$$P_{rc} = \phi_c A_c f'_c \tag{7.6}$$

and M_u is the maximum resultant moment applied to the member in any direction, calculated as specified in Article 4.5.3.2.2 of the AASHTO LRFD provisions.

7.6.2 Flexural Strength

The factored moment resistance of a concrete filled steel pipe for Article 7.6.1 shall be calculated using either of the following two methods:

a. Method 1 – Using Exact Geometry

$$M_{rc} = \phi_f \ [C_r e + C'_r e'] \tag{7.7}$$

where

$$C_r = F_y \beta \frac{Dt}{2} \tag{7.8}$$

$$C'_r = f'_c \left[\frac{\beta D^2}{8} - \frac{b_c}{2}\left(\frac{D}{2} - a\right)\right] \tag{7.9}$$

$$e = b_c \left[\frac{1}{(2\pi - \beta)} + \frac{1}{\beta} \right] \tag{7.10}$$

$$e' = b_c \left[\frac{1}{(2\pi - \beta)} + \frac{b_c^2}{1.5\beta D^2 - 6b_c(0.5D - a)} \right] \tag{7.11}$$

C7.5.3 Design Requirements for Ductile Bracing Members

In the ductile design of concentrically braced frames in buildings, the slenderness ratio limits for braces, up until the late 1990s, were approximately 75% of the value specified here. The philosophy was to design braces to contribute significantly to the total energy dissipation when in compression. Member slenderness ratio was restricted because the energy absorbed by plastic bending of braces in compression diminishes with increased slenderness. To achieve these more stringent *KL*/*r* limits, particularly for long braces, designers have almost exclusively used tubes or pipes for the braces. This is unfortunate as these tubular members are most sensitive to rapid local buckling and fracture when subjected to inelastic cyclic loading (in spite of the low width-to-thickness limits prescribed). Recent reviews of this requirement revealed that it may be unnecessary, provided that connections are capable of developing at least the member capacity in tension. This is partly because larger tension brace capacity is obtained when design is governed by the compression brace capacity, and partly because low-cycle fatigue life increases for members having greater *KL*/*r*. As a result, seismic provisions for buildings (AISC, 1997; CSA, 2001) have been revised to permit members having greater *KL*/*r* values. The proposed relaxed limits used here are consistent with the new recently adopted philosophy for buildings. The limit for back-to-back legs of double-angle bracing members is increased from the value of Table 8.7.4-1 to $200/\sqrt{F_y}$.

Early local buckling of braces prohibits the braced frames from sustaining many cycles of load reversal. Both laboratory tests and real earthquake observations have confirmed that premature local buckling significantly shortens the fracture life of high-strength steel (HSS) braces. The more stringent requirement on the *b*/*t* ratio for rectangular tubular sections subjected to cyclic loading is based on tests (Tang and Goel, 1987; Uang and Bertero, 1986). The *D*/*t* limit for circular sections is identical to that in the AISC plastic design specifications (AISC, 1993; Sherman, 1976).

C7.5.4 Multi-Tier Frame Bents

Multi-tier frame bents are sometimes used, mostly because they are more rigid transversely than single-tier frame bents. In such multi-tier bents, the intermediate beams are significantly smaller than the top beam as they are not supporting the gravity loads from the superstructure.

$$a = \frac{b_c}{2}\tan\left(\frac{\beta}{4}\right) \tag{7.12}$$

$$b_c = D\sin\left(\frac{\beta}{2}\right) \tag{7.13}$$

where β is in radians and found by the recursive equation:

$$\beta = \frac{A_s F_y + 0.25 D^2 f'_c\left[\sin(\beta/2) - \sin^2(\beta/2)\tan(\beta/4)\right]}{\left(0.125\ D^2 f'_c + \ DtF_y\right)} \tag{7.14}$$

b. Method 2 – Using Approximate Geometry

A conservative value of M_{rc} is given by

$$M_{rc} = \phi_f\ \left[(Z - 2th_n^2)Fy + \left[\frac{2}{3}(0.5D - t)^3 - (0.5D - t)h_n^2\right] f'_c\right] \tag{7.15}$$

where

$$h_n = \frac{A_c f'_c}{2Df'_c + 4t(2F_y - f'_c)} \tag{7.16}$$

and Z is the plastic modulus of the steel section alone.

For capacity design purposes, in determining the force to consider for the design of capacity protected elements, the moment calculated by this approximate method shall be increased according to Section 4.11.2.

7.6.3 Beams and Connections

Capacity-protected members must be designed to resist the forces resulting from hinging in the concrete-filled pipes calculated from Article 7.6.2.

7.7 CONNECTIONS FOR SDC C AND D

7.7.1 Minimum Strength for Connections to Ductile Members

Connections and splices between or within members having a ductility demand greater than unity shall be designed to have a nominal capacity at least 10% greater than the nominal capacity of the member they connect based on expected material properties.

As a result, in a multi-tier frame, plastic hinging in the beams may be unavoidable, and desirable, in all but the top beam. In fact, trying to ensure strong-beam weak-column design at all joints in multi-tier bents may have the undesirable effect of concentrating all column plastic hinging in one tier, with greater local ductility demands than otherwise expected in design.

Using capacity design principles, the equations and intent of Article 8.7.4 may be modified by the designer to achieve column plastic hinging only at the top and base of the column, and plastic hinging at the ends of all intermediate beams, as shown in Figure C7.5.4-1.

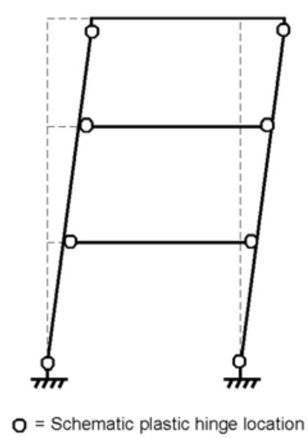


Figure C7.5.4-1 Acceptable Plastic Mechanism for Multi-Tier Bent

C8.7.6 Concentrically Braced Frames with Nominal Ductility

Detailing requirements are relaxed for concentrically braced frames having nominal ductility (a steel substructure having less stringent detailing requirements). They are consequently being designed to a greater force level.

C8.7.6.1 Bracing Systems

This requirement ensures some redundancy. It also ensures similarity between the load-deflection characteristics in two opposite directions. A significant proportion of the horizontal shear is carried by tension braces so that compression-brace buckling will not cause a catastrophic loss in overall horizontal shear capacity.

7.7.2 Yielding of Gross Section for Connectors to Ductile Members

Yielding of the gross section shall be checked (see Section 7.7.6). Fracture in the net section and the block shear rupture failure shall be prevented.

7.7.3 Welded Connections

Partial penetration fillet weld shall not be used in regions of members subject to inelastic deformations. Outside of those regions, partial penetration welds shall provide at least 150% of the strength required by calculation, and not less than 50% of the strength of the connected parts (regardless of the action of the weld).

7.7.4 Gusset Plate Strength

Gusset plates shall be designed to resist shear, flexure and axial forces generated by overstrength capacities of connected ductile members and force demands of connected essentially elastic members. The design strength shall be based on the effective width in accordance with Whitmore's method.

7.7.5 Limiting Unsupported Edge Length to Thickness Ratio for a Gusset Plate

The unsupported edge length to thickness ratio of a gusset plate shall satisfy:

$$\frac{L_g}{t} \le 2.06\sqrt{\frac{E}{F_y}} \qquad (7.17)$$

where,

L_g = unsupported edge length of a gusset plate (in.)

t = thickness of a gusset plate (in.)

7.7.6 Gusset Plate Tension Strength

The tension strength of the gusset plates shall be:

$$\phi P_n = \phi A_g F_y \le \phi_{tf} A_n F_u \, or \, \phi_{bs} P_{bs} \qquad (7.18)$$

where

$$P_{bs} = 0.58 F_y A_{vg} + F_u A_{tn} \qquad (7.19)$$

$$\text{for } A_{tn} \ge 0.58 A_{vn}$$

or

Tension-only systems are bracing systems in which braces are connected at beam-to-column intersections and are designed to resist in tension 100% of the seismic loads.

Systems in which all braces are oriented in the same direction and may be subjected to compression simultaneously shall be avoided.

K-braced frames, in which pairs of braces meet a column near its mid-height, and knee-braced frames shall not be considered in this section.

Analytical and experimental research, as well as observations following past earthquakes, have demonstrated that K-bracing systems are poor dissipaters of seismic energy. The members to which such braces are connected can also be adversely affected by the lateral force introduced at the connection point of both braces on that member due to the unequal compression buckling and tension yielding capacities of the braces.

Knee-braced systems in which the columns are subjected to significant bending moments are beyond the scope of this article.

C8.7.6.2 Design Requirements for Nominally Ductile Bracing Members

Nominally ductile braced frames are expected to undergo limited inelastic deformations during earthquakes. Braces yielding in tension are relied upon to provide seismic energy dissipation. While frames with very slender braces (i.e. tension-only designs) are generally undesirable for multistoried frames in buildings, this is mostly because energy dissipation in such frames tend to concentrate in only a few stories, which may result in excessive ductility demands on those braces. However, non-linear inelastic analyses show that satisfactory seismic performance is possible for structures up to 4 stories with tension-only braces, provided that connections are capable of developing at least the member capacity in tension and that columns are continuous over the frame height (CSA, 2001). The width-to-thickness ratios for the compression elements of columns can be relaxed for braces having KL/r approaching 200, as members in compression do not yield at that slenderness.

C8.7.6.3 Brace Connections

The additional factor of 1.10 for tension-only bracing systems is to ensure, for the slender members used in this case, that the impact resulting when slack is taken up, does not cause connection failure. Details leading to limited zones of yielding, such as

$$P_{bs} = 0.58F_u A_{vn} + F_y A_{tg} \quad (7.20)$$

$$\text{for } A_{tn} < 0.58A_{vn}$$

where

A_{vg} = gross area along the plane resisting shear (in.^{2})

A_{vn} = net area along the plane resisting shear (in.^{2})

A_{tg} = gross area along the plane resisting tension (in.^{2})

A_{tn} = net area along the plane resisting tension (in.^{2})

F_y= specified minimum yield strength of the connected material (ksi)

F_u = specified minimum tensile strength of the connected material (ksi)

A_n = net area (in.^{2})

ϕ_{tf} = 0.8 for fracture in net section.

ϕ_{bs} = 0.8 for block shear failure.

7.7.7 Compression Strength of a Gusset Plate

The nominal compression strength of the gusset plates, P_{ng}, shall be calculated according to AASHTO-LRFD Provisions.

7.7.8 In-Plate Moment (Strong Axis)

The nominal moment strength of a gusset plate, M_{ng}, shall be:

$$M_{ng} = S_{sm} F_y \quad (7.21)$$

where

S_{sm} = elastic section modulus about strong axis (in.^{3})

7.7.9 In-Plate Shear Strength

The nominal moment strength of a gusset plate, V_n, shall be:

$$V_n = 0.58 F_y A_{gg} \quad (7.22)$$

where

A_{gg} = gross area of a gusset plate (in.^{2})

occur at partial joint penetration groove welds should be avoided.

C7.7.1 Brace Connections

Eccentricities that are normally considered negligible (for example at the ends of bolted or welded angle members) may influence the failure mode of connections subjected to cyclic load (Astaneh, Goel and Hanson, 1986).

A brace which buckles out-of-plane will form a plastic hinge at mid-length and hinges in the gusset plate at each end. When braces attached to a single gusset plate buckle out-of-plane, there is a tendency for the plate to tear if it is restrained by its attachment to the adjacent frame members (Astaneh, Goel and Hanson, 1982). Provision of a clear distance, approximately twice the plate thickness, between the end of the brace and the adjacent members allows the plastic hinge to form in the plat and eliminates the restraint. When in-plane buckling of the brace may occur, ductile rotational behavior should be possible either in the brace or in the joint. Alternatively, the system could be designed to develop hinging in the brace, and the connections shall then be designed to have a flexural strength equal to or greater than the expected flexural strength $1.2R_yM_p$ of the brace about the critical buckling axis.

Buckling of double-angle braces (legs back-to-back) about the axis of symmetry leads to transfer of load from one angle to the other, thus imposing significant loading on the stitch fastener (Astaneh, Goel and Hanson, 1986).

C8.7.5.4 Columns, Beams and Other Connections

Columns and beams that participate in the lateral-load-resisting system must also be designed to ensure that a continuous load path can be maintained. A reduced compressive resistance must be considered for this purpose. This takes into account the fact that, under cyclic loading, the compressive resistance of a bracing member rapidly diminishes. This reduction stabilizes after a few cycles to approximately 30% of the nominal compression capacity.

The unreduced brace compressive resistance must be used if it leads to a more critical condition, as it will be attained in the first cycle. However, redistributed loads resulting from the reduced buckled compressive brace loads must be considered in beams and columns as well as in connections, if it leads to a more critical condition.Other connections that participate in the lateral-load-resisting system must also be designed to ensure that a continuous load path can be maintained. Therefore, they should

7.7.10 Combined Moment, Shear and Axial Force

The initial yielding strength of a gusset plate subjected to a combination of in-plane moment, shear and axial force shall be determined by the following equations:

$$\frac{M_g}{M_{ng}} + \frac{P_g}{P_{yg}} \leq 1 \qquad (7.23)$$

and

$$\left(\frac{V_g}{V_{ng}}\right)^2 + \left(\frac{P_g}{P_{yg}}\right)^2 \leq 1 \qquad (7.24)$$

where

V_g = shear force (kips)

M_g = moment (kips-in.)

P_g = axial load (kips)

M_{ng} = nominal moment strength

V_{ng} = nominal shear strength

P_{yg} = yield axial strength

Full yielding of shear-moment-axial load interaction for a plate shall be:

$$\frac{M_g}{M_{Pg}} + \left(\frac{P_g}{P_{yg}}\right)^2 + \frac{\left(\frac{V_g}{V_{pg}}\right)^4}{\left[1 - \left(\frac{P_g}{P_{yg}}\right)^2\right]} = 1 \qquad (7.25)$$

where

M_{pg} = plastic moment of plate under pure bending (kips-in.)

V_{pg} = plastic shear capacity of gusset plate ($0.58A_{gg}F_y$) (kips)

7.7.11 Fastener Capacity

Fastener capacity shall be determined using AASHTO-LRFD Provisions under combined shear and tension.

a. resist the combined load effect corresponding to the bracing connection loads and the permanent loads that they must also transfer; and

b. resist load effect due to load redistribution following brace yielding or buckling.

C7.6 CONCRETE-FILLED STEEL PIPES

This article is only applicable to concrete-filled steel pipes without internal reinforcement, and connected in a way that allows development of their full composite strength. It is not applicable to design a concrete-filled steel pipe that relies on internal reinforcement to provide continuity with another structural element, or for which the steel pipe is not continuous or connected in a way that enables it to develop its full yield strength. When used in pile bent, the full composite strength of the plastic hinge located below ground can only be developed if it can be ensured that the concrete fill is present at that location.

Recent research (e.g., Alfawahkiri, 1998; Bruneau and Marson, 1999) demonstrates that the AASHTO equations for the design of concrete-filled steel pipes in combined axial compression and flexure (Articles 6.9.2.2, 6.9.5, and 6.12.2.3.2 of the AASHTO LRFD provisions), provide a conservative assessment of beam-column strength. Consequently, the calculated strength of concrete-filled steel pipes that could be used as columns in ductile moment-resisting frames or pile-bents, could be significantly underestimated. This is not surprising given that these equations together are deemed applicable to a broad range of composite member types and shapes, including concrete-encased steel shapes. While these equations may be perceived as conservative in a non-seismic perspective, an equation that more realistically captures the plastic moment of such columns is essential in a capacity design perspective. Capacity-protected elements must be designed with adequate strength to withstand elastically the plastic hinging in the columns. Underestimates of this hinging force translate into under-design of the capacity-protected elements; a column unknowingly stronger than expected will not yield before damage develops in the foundations or at other undesirable locations in the structure. This can have severe consequences, as the capacity-protected elements are not detailed to withstand large inelastic deformations. The provisions of Article 8.7.7 are added to prevent this behavior.

Note that for analysis, as implied by Article 6.9.5 of the AASHTO LRFD provisions, flexural stiffness of the composite section can be taken as E_sI_s + 0.4

7.8 ISOLATION DEVICES

Design and detailing of seismic isolation devices shall be designed in accordance with the provisions of the AASHTO Guide Specifications for Seismic Isolation Design.

7.9 FIXED AND EXPANSION BEARINGS

7.9.1 Applicability

The provisions shall apply to pin bearings, roller bearings, rocker bearings, bronze or copper-alloy sliding bearings, elastomeric bearings, spherical bearings, and pot disc bearings in common slab-on-steel girder bridges. Curved bridges, seismic isolation-type bearings, and structural fuse bearings are not covered by this section.

7.9.2 Design Criteria

The selection of seismic design of bearings shall be related to the strength and stiffness characteristics of both the superstructure and the substructure.

Bearing design shall be consistent with the intended seismic design strategy and the response of the whole bridge system.

Rigid-type bearings are assumed not to move in restrained directions, and therefore the seismic forces from the superstructure shall be assumed to be transmitted through diaphragms or cross frames and their connections to the bearings, and then to the substructure without reduction due to local inelastic action along that load path.

Deformable-type bearings having less than full rigidity in the restrained directions, but not specifically designed as base isolators or fuses have demonstrated a reduction in force transmission, and may be used under any circumstances. The reduced force transmitted thru the bearing shall as a minimum not be less than 0.4g times the bearing dead load reaction.

7.9.3 Design and Detail Requirements

The Engineer shall assess the impact on the lateral load path due to unequal participation of bearings considering connection tolerances, unintended misalignments, the capacity of individual bearings, and skew effects.

Roller bearings or rocker bearings shall not be used in new bridge construction.

E_cI_c, where I_c is the gross inertia of the concrete ($\pi D^4/16$), I_s is the inertia of the steel pipe, and E_s and E_c are respectively the steel and concrete moduli of elasticity.

C7.6.1 Combined Axial Compression and Flexure

This equation is known to be reliable up to a maximum slenderness limit D/t of $28000/F_y$, underestimating the flexural moment capacity by 1.25, on average (see Figure C7.6.1-1). It may significantly overestimate columns strength having greater D/t ratios.

This new equation is only applicable to concrete-filled steel pipes. Other equations may similarly be needed to replace those of Article 6.9.2.2 of the AASHTO LRFD provisions for other types of composite columns (such as concrete-encased columns).

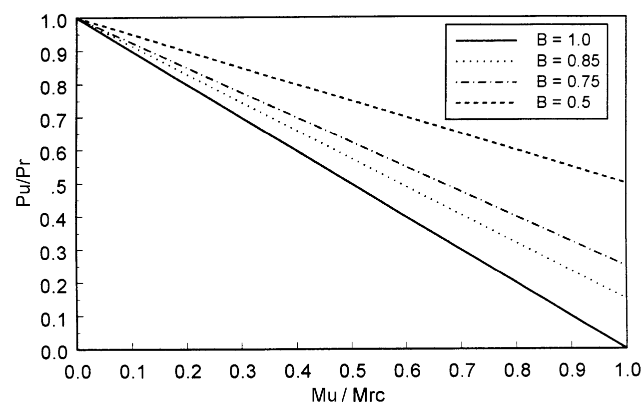


Figure C7.6.1-1 Interaction Curves for Concrete-Filled Pip

C7.6.2 Flexural Strength

When using these equations to calculate the forces acting on capacity-protected members as a result of plastic hinging of the concrete-filled pipes, F_y should be replaced by F_{ye}, for consistency with the capacity design philosophy.

Figure C7.6.2-1 illustrates the geometric parameters used in this Article.

Expansion bearings and their supports shall be designed in such a manner that the structure can undergo movements in the unrestrained direction not less than the seismic displacements determined from analysis without collapse. Adequate seat width shall also be provided for fixed bearings.

In their restrained directions, bearings shall be designed and detailed to engage at essentially the same movement.

The frictional resistance of bearing interface sliding-surfaces shall be neglected when it contributes to the resisting seismic loads. Conversely this shall be conservatively estimated (i.e., overestimated) when friction results in the application of force effects to the structural components.

Elastomeric expansion bearings shall be provided with anchorage to adequately resist the seismically induced horizontal forces in excess of those accommodated by shear in the pad. The sole plate and base plate shall be made wider to accommodate the anchor bolts. Inserts through the elastomer shall not be allowed. The anchor bolts shall be designed for the combined effect of bending and shear for seismic loads. Elastomeric fixed bearings shall be provided with horizontal restraint adequate for the full horizontal load.

Spherical bearings shall be evaluated for component and connection strength and bearing stability.

Pot and disc bearings shall not be used for seismic applications where significant vertical acceleration must be considered and, where their use is unavoidable, they shall be provided with independent seismically resistant anchorage systems.

7.9.4 Bearing Anchorage

Sufficient reinforcement shall be provided around the anchor bolts to develop the horizontal forces and anchor them into the mass of the substructure unit. Potential concrete crack surfaces next to the bearing anchorage shall have sufficient shear friction capacity to prevent failure.

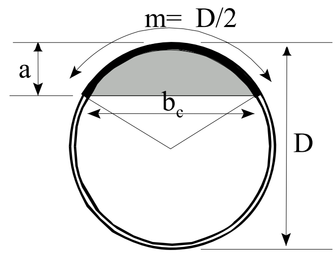
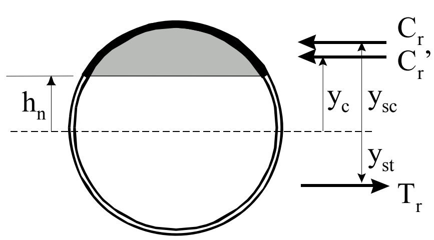


Figure C7.6.2-1 Flexure of Concrete-Filled Pipe; Shaded Area is Concrete in Compression above the Neutral Axis

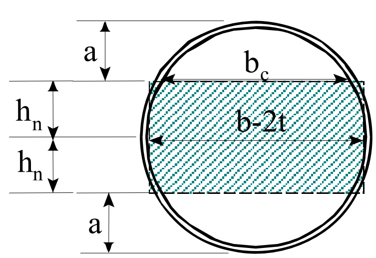
Moment resistance is calculated assuming the concrete in compression at f'_c, and the steel in tension and compression at F_y. The resulting free-body diagram is shown in Figure C8.7.7.2-2, where e is equal to $y_{sc}+y_{st}$, e' is equal to y_c+y_{st}, and y_c is the distance of the concrete compressive force (C_r') from the center of gravity, and y_{st} and y_{sc} are the respective distances of the steel tensile (T_r) and compressive forces (C_r) from the center of gravity.

In Method 2, a geometric approximation is made in calculating the area of concrete in compression by subtracting the rectangular shaded area shown in Figure C8.7.7.2-3 from the total area enclosed by the pipe (and dividing the result by 2). Neutral axis is at height h_n.



$$M_{rc} = C_r'(y_c+y_{st}) + C_r\,(y_{sc}+y_{st})$$

Figure C7.6.2-2 Free-Body Diagram Used to Calculate Moment Resistance of Concrete-Filled Pipe



7.10 STRUCTURAL STEEL DESIGN REQUIREMENTS FOR ENERGY DISIPATION COMPONENTS IN SDC C AND D

7.10.1 General

The provisions of this article shall apply only to a limited number of specially detailed steel components designed to dissipate hysteretic energy during earthquakes. This article does not apply to steel members that are designed to remain elastic during earthquakes.

For the few specially designed steel members that are within the scope of this article, the other requirements of Section 6 of the AASHTO LRFD provisions are also applicable (unless superseded by more stringent requirements in this article).

Continuous and clear load path or load paths shall be assured. Proper load transfer shall be considered in designing foundations, substructures, superstructures and connections.

Welds shall be designed as capacity protected elements. Partial penetration groove welds shall not be used in ductile substructures.

Abrupt changes in cross sections of members in ductile substructures are not permitted within the plastic hinge zones defined in Article 4.9 unless demonstrated acceptable by analysis and supported by research results.

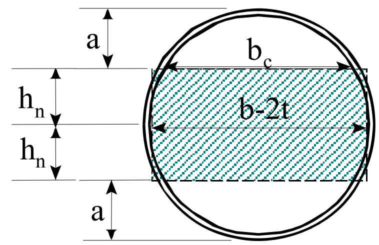


Figure C7.6.2-3 Flexure of Concrete-Filled Pipe – Illustrates Approximation Made in Method 2

Method 2 (using approximate geometry) gives smaller moments than Method 1 (exact geometry). The requirement to increase the calculated moment by 10% for capacity design when using the approximate method was established from the ratio of the moment calculated by both methods for a *D*/*t* of 10. That ratio decreases as *D*/*t* increases.

C7.6.3 Beams and Connections

Recent experimental work by Bruneau and Marson (1999), Shama et al. (2001), Azizinamini et al. (1999), provide examples of full fixity connection details. In some instances, full fixity may not be needed at both ends of columns. Concrete-filled steel pipes, when used in pile bents, only require full moment connection at the pile-cap.

C7.8 SEISMIC ISOLATOR UNITS

The requirements for analysis of bridges with seismic isolation systems are specified in Article X.X and are based on the 1999 AASHTO *Guide Specifications for Seismic Isolation Design*, which provide requirements for modeling seismic isolator units, including the use of property modification factors as given in Article X.X.

The force-deformation characteristics can be idealized as a bilinear relationship with two key variables: second slope stiffness and characteristic strength. The area under the bilinear curve is equal to the energy dissipated by hysteretic work during cyclic loading. For design, the force-deformation relationship can be represented by an effective stiffness based on the secant stiffness, and a damping coefficient.

Bridges that have elastomeric or sliding bearings at each pier shall be designed as isolated structures using all of the provisions of Article X.X because it is essential that the columns remain essentially elastic (i.e., $\mu \leq 2$).

C7.9 FIXED AND EXPANSION BEARINGS

Bearings are important elements of the overall Earthquake Resistant System of a bridge structure. The 1995 Kobe earthquake, and others that preceded it or have occurred since, clearly showed poor performance of some recent bearing types and the disastrous consequences that a bearing failure can have on the overall performance of a bridge. A consensus was developed that some testing of bearings would be desirable provided a designer had the option of providing restraints or permitting the bearing to fail if an adequate surface for subsequent movement is provided. An example occurred in Kobe where a bearing failed. The steel diaphragm and steel girder were subsequently damaged because the girder became jammed on the failed bearing and could not move.

There have been a number of studies performed when girders slide either on specially designed bearings or concrete surfaces. A good summary of the range of the results that can be anticipated from these types of analyses can be found in Dicleli and Bruneau (1995).

C7.9.1 Prototype and Quality Control Tests

The types of tests that are required by these *Guidelines* are similar to but significantly less extensive than those required for seismically isolated bridges. Each manufacturer is required to conduct a prototype qualification test to qualify a particular bearing type and size for its design forces or displacements. This series of tests only needs to be performed once to qualify the bearing type and size, whereas for seismically isolated bridges, prototype tests are required on every project. The quality control tests required on 1 out of every 10 bearings is the same as that required for every isolator on seismic isolation bridge projects. The cost of the much more extensive prototype and quality control testing of isolation bearings is approximately 10 to 15% of the total bearing cost, which is of the order of 2% of the total bridge cost. The testing proposed herein is much less stringent than that required for isolation bearings and is expected to be less than 0.1% of the total bridge cost. However, the benefits of testing are considered to be significant since owners would have a much higher degree of confidence that each new bearing will perform as designed during an earthquake. The testing capability exists to do these tests on full-size bearings. The Owner has the final determination on the extent of the testing requirements as deemed appropriate for the type of bridge considered.

C7.10 SEISMIC ISOLATION DESIGN REQUIREMENTS

The commentary on this subject is given in C15, which will become a new section in the AASHTO LRFD provisions.

8. REINFORCED CONCRETE COMPONENTS

8.1 GENERAL

Design and construction of concrete components that include superstructures columns, piers, footings and their connections shall conform to the requirements of this section.

For the purpose of this article, a vertical support shall be considered to be a column if the ratio of the clear height to the maximum plan dimensions of the support is not less than 2.5. For a flared column, the maximum plan dimension shall be taken at the minimum section of the flare. For supports with a ratio less than 2.5, the provisions for piers of Article 8.6 shall apply.

A pier may be designed as a pier member in its strong direction and a column in its weak direction.

The pile extension of pile bents as well as drilled shafts and caissons shall be regarded as columns for design and detailing purposes.

If architectural flares or other treatments are provided to columns adjacent to potential plastic hinge zones, they shall be either "structurally isolated" in such a way that they do not add to the flexural strength capacity of the columns. If "structural isolation" is not used the column and adjacent structural elements shall be designed to resist the forces generated by increased flexural strength capacity according to Article 8.14.

C8.1 GENERAL

The 1989 Loma Prieta and 1994 Northridge earthquakes confirmed the vulnerability of columns with inadequate transverse reinforcement and inadequate anchorage of longitudinal reinforcement. Also of concern are:

- lack of adequate reinforcement for positive moments that may occur in the superstructure over monolithic supports when the structure is subjected to longitudinal dynamic loads;
- lack of adequate shear strength in joints between columns and bent caps under transverse dynamic loads;
- inadequate reinforcement for torsion, particularly in outrigger-type bent caps; and
- inadequate transverse reinforcement for shear and for restraint against global buckling of longitudinal bars ("bird caging").

The purpose of the design is to ensure that a column is provided with reasonable ductility and is forced to yield in flexure and that the potential for a shear, compression failure due to longitudinal bar buckling, or loss of anchorage mode of failure is minimized.

The actual ductility demand on a column or pier is a complex function of a number of variables, including:

- Earthquake characteristics, including duration, frequency content and near-field (or pulse) effects,
- Design force level,
- Periods of vibration of the bridge,
- Shape of the inelastic hysteresis loop of the columns, and hence effective hysteretic damping,
- Elastic damping coefficient,
- Contributions of foundation and soil conditions to structural flexibility, and
- Spread of plasticity (plastic hinge length) in the column.

The damage potential of a column is also related to the ratio of the duration of strong ground shaking to the natural period of vibration of the bridge.

8.2 SEISMIC DESIGN CATEGORY A

No consideration of seismic forces is required for the design of structural components except for the design of the connection for the superstructure to the substructure as specified in Section 4.6 and the minimum bearing support length as specified in Section 4.12.

8.3 SEISMIC DESIGN CATEGORIES B, C, D

8.3.1 General

Initial sizing of columns can be performed using service load combinations. Columns may be governed by Strength Level Cases. For columns on spread footings where rocking analysis is used, refer to Article 6.3.4.

8.3.2 Force Demands SDC B

The design forces shall be the lesser of forces resulting from plastic hinging or unreduced elastic seismic forces in columns or pier walls. Force demands established based on Article 8.3.2 shall be less than capacities established in Articles 8.5 and 8.6.

8.3.3 Force Demands SDC C & D

The design forces shall be based on forces resulting from plastic hinging or maximum connection capacity following capacity design principles as specified in Article 4.11. For SDC D where liquefaction is identified, plastic hinging in the foundation is acceptable as specified in Article 3.3.

8.3.4 Local Ductility Demands SDC D

The local displacement ductility demands of Equivalent Members shall be determined based on the analysis adopted in Section 5. The ductility demands shall be less than the ductility capacities determined based on parameters established in Section 8.5 or the maximum allowable ductilities established in Section 4.9.

The definition of a column in this article is provided as a guideline to differentiate between the additional design requirements for a wall-type pier and the requirements for a column.

Certain oversize columns exist for architectural or aesthetic reasons. These columns, if fully reinforced, place excessive demands of moment, shear, or both, on adjoining elements. The designer should strive to “isolate structurally” those architectural elements that do not form part of the primary energy dissipation system that are located either within or in close proximity to plastic hinge zones. Nevertheless, the architectural elements must remain serviceable throughout the life of the structure. For this reason, minimum steel for temperature and shrinkage should be provided. When architectural flares are not isolated, Article X.X requires that the design shear force for a flared column be the worst case calculated using the overstrength moment of the oversized flare or the shear generated by a plastic hinge at the bottom of the flare.

8.4 PROPERTIES AND APPLICATIONS OF REINFORCING STEEL, PRESTRESSING STEEL AND CONCRETE FOR SDC B, C, D

Expected Material Properties shall be used to determine section stiffness and strength properties as well as establishing displacement capacity of the bridge system and the ductility capacities of the various components. For SDC B and C the expected material properties are used for calculating only the section stiffness and strength properties.

8.4.1 Reinforcing Steel

Reinforcing bars, deformed wire, cold-drawn wire, welded plain wire fabric, and welded deformed wire fabric shall conform to the material standards as specified in the current AASHTO *LRFD Specifications*.

High strength high alloy bars, with an ultimate tensile strength of up to 250 ksi, may be used for longitudinal column reinforcement for seismic

C8.4.1 Reinforcing Steel

High-strength reinforcement reduces congestion and cost as demonstrated by Mander and Cheng (1999), and Dutta, Mander and Kokorina, (1999). However it is important to ensure that the cyclic fatigue life is not inferior when compared to ordinary mild steel reinforcing bars. Mander, Panthaki, and Kasalanati, (1994) have shown that modern high-alloy prestressing threadbar steels can have sufficient ductility to justify their use in seismic design.

The Modulus of Toughness is defined as the area beneath the monotonic tensile stress-strain curve from initial loading (zero stress) to fracture.

loading providing it can be demonstrated through tests that the low cycle fatigue properties is not inferior to normal reinforcing steels with yield strengths of 75 ksi or less.

Wire rope or strand may be used for spirals in columns if it can be shown through tests that the modulus of toughness exceeds 14 ksi.

In compression members, all longitudinal bars shall be enclosed by hoops or spirals. Ties shall be used to provide lateral restraint to intermediate longitudinal bars within the reinforced concrete cross section.

The minimum size of transverse hoops and ties shall be equivalent to or greater than:

- #4 bars for #9 or smaller longitudinal bars,
- #5 bars for #10 or larger longitudinal bars, and
- #5 bars for bundled longitudinal bars.

The spacing of transverse hoops and ties shall not exceed the least dimension of the compression member or 12 inches. Where two or more bars larger than #11 are bundled together, the spacing shall not exceed half the least dimension of the member or 6 inches.

Deformed wire**,** wire rope or welded wire fabric of equivalent area may be used instead of bars for the ties, hoops or spirals.

For SDC D, A706 reinforcing steel shall be used in members where plastic hinging is expected under the Design Earthquake. For columns in SDC C and D, the following detailing provisions shall apply:

- Hoops and ties shall be arranged so that every corner and alternate longitudinal bar has lateral support provided by the corner of a tie having an included angle of not more than 135°.
- No longitudinal bar shall be farther than 6 inches clear on each side along the tie from such a laterally supported bar.
- Ties shall be located vertically not more than half a tie spacing above the footing or other support and not more than half a tie spacing below the lowest horizontal reinforcement in the supported member.

8.4.2 Reinforcing Steel Modeling

Reinforcing steel shall be modeled with a stress-strain relationship (see Figure 8.1) that exhibits an initial elastic portion, a yield plateau, and a strain hardening range in which the stress increases with strain. Within the elastic region the modules of elasticity, E_s, shall be 29,000 ksi. A706 reinforcing steel shall be used with the following expected properties:

$$f_{ye} = 1.1 f_y \qquad (8.1)$$

where

f_{ye} = the expected yield strength

f_y = the specified minimum yield strength

$$f_{ue} = 1.4\ f_{ye} \qquad (8.2)$$

where

f_{ue} is the expected tensile strength

The ultimate tensile strain ε_{su} shall be:

$$\varepsilon_{su} = \begin{cases} 0.120 & \text{\#10 bars or smaller} \\ 0.090 & \text{\#11 bars and larger} \end{cases}$$

The onset of strain hardening ε_{sh} shall be:

$$\varepsilon_{sh} = \begin{cases} 0.0150 & \text{\#8 bars} \\ 0.0125 & \text{\#9 bars} \\ 0.0115 & \text{\#10 \& \#11 bars} \\ 0.0075 & \text{\#14 bars} \\ 0.0050 & \text{\#18 bars} \end{cases}$$

A reduced ε^{R}_{su} equal to 0.06 shall be used for column longitudinal reinforcement.

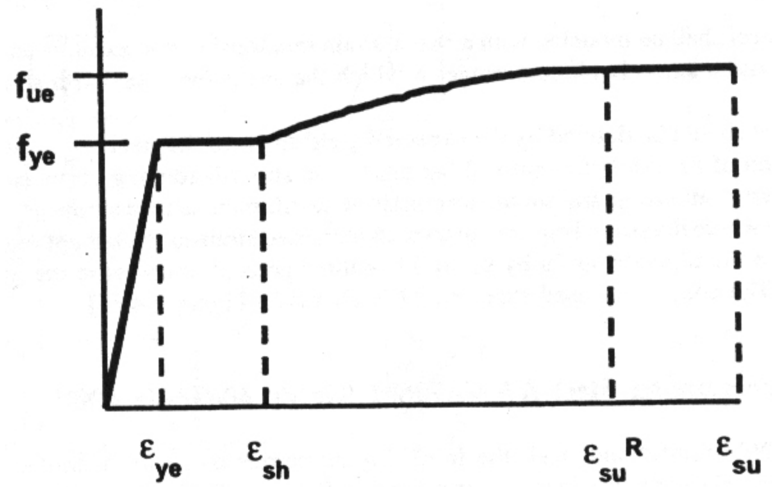


FIGURE 8.1 Steel Stress-Strain Model

8.4.3 Prestressing Steel

Prestressing steel shall be modeled with an idealized nonlinear stress-strain model. The ultimate prestress steel strain $\varepsilon_{ps,u}$ shall not exceed 0.04. Figure 8.2 shows an idealized stress-strain model for 7-wire low-relaxation prestressing strand.

Essentially elastic prestress steel strain

$$\varepsilon_{ps,EE} = \begin{cases} 0.0076 \text{ for } f_u = 250 \text{ ksi} \\ \\ 0.0086 \text{ for } f_u = 270 \text{ ksi} \end{cases}$$

Reduced ultimate prestress steel strain

$$\varepsilon_{ps,u}^{R} = 0.04$$

250 ksi Strand:

$$\varepsilon_{ps} \le 0.0076 : f_{ps} = 28{,}500 \times \varepsilon_{ps} \quad \text{(ksi)} \qquad (8.3)$$

$$\varepsilon_{ps} > 0.0076 : f_{ps} = 250 - \frac{0.25}{\varepsilon_{ps}} \quad \text{(ksi)} \qquad (8.4)$$

270 ksi Strand:

$$\varepsilon_{ps} \le 0.0086 : f_{ps} = 28{,}500 \times \varepsilon_{ps} \quad \text{(ksi)} \qquad (8.5)$$

$$\varepsilon_{ps} > 0.0086 : f_{ps} = 270 - \frac{0.04}{\varepsilon_{ps} - 0.007}$$

(ksi) (8.6)

where

ε_{ps} = Prestressing steel strain

f_{ps} = Prestressing steel stress

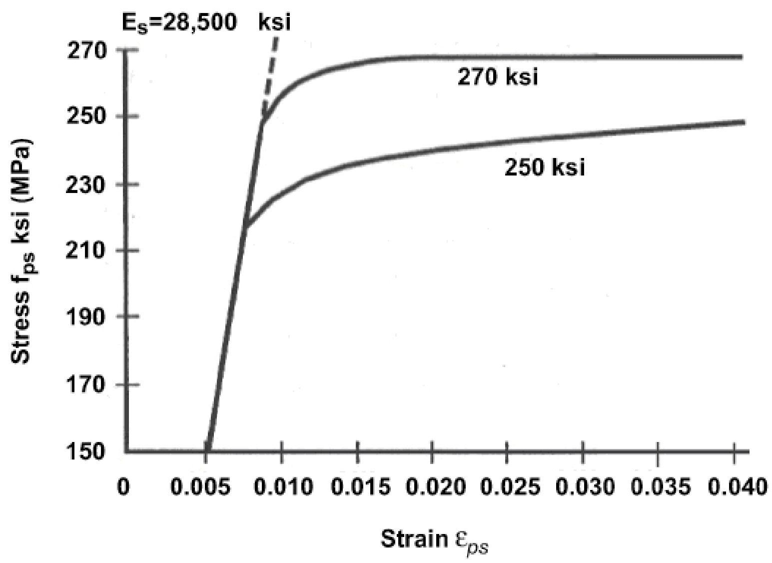


FIGURE 8.2 Prestressing Strand Stress-Strain Model

8.4.4 Concrete

A stress-strain model for confined and unconfined concrete shall be used. Mander's stress strain model for confined concrete is commonly used for determining section properties (see Figure 8.3).

The expected concrete compressive strength f'_{ce} shall be the greater of:

$$f'_{ce} = \begin{cases} 1.3 \times f_c' \\ \text{or} \\ 5000 \text{ (psi)} \end{cases} \tag{8.7}$$

The unconfined concrete compressive strain at the maximum compressive stress ε_{co} is equal to 0.002. And the ultimate unconfined compression (spalling) strain ε_{sp} is equal to 0.005.

The confined compressive strain ε_{cc} and the ultimate compressive strain for confined concrete ε_{cu} are computed using Mander's model.

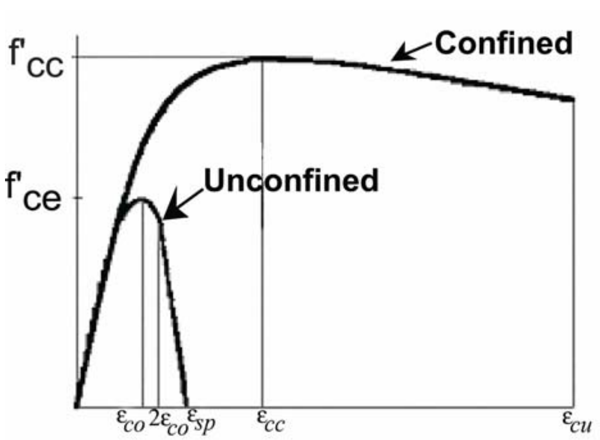


FIGURE 8.3 Concrete Stress-Strain Model

8.5 PLASTIC MOMENT CAPACITY FOR DUCTILE CONCRETE MEMBERS SDC B, C, AND D

The plastic moment capacity of all ductile concrete members shall be calculated by moment-curvature $(M - \phi)$ analysis based on the expected material properties. Moment curvature analysis derives the curvatures associated with a range of moments for a cross section based on the principles of strain compatibility and equilibrium of forces. The axial forces considered in the section shall be based on dead load or the net axial load derived based on capacity design principles included in Section 4.11. The $M\text{–}\phi$ curve can be idealized with an elastic perfectly plastic response to estimate the plastic moment capacity of a member's cross section. The elastic portion of the idealized curve should pass through the point marking the first reinforcing bar yield. The idealized plastic moment capacity is obtained by equating the areas between the actual and the idealized $M\text{–}\phi$ curves beyond the first reinforcing bar yield point. See Figure 8.4.

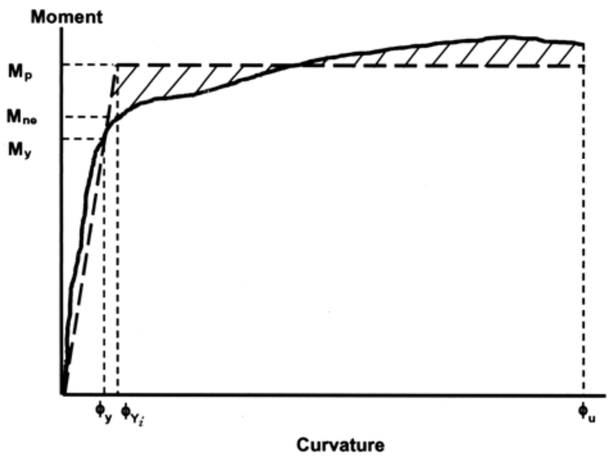


FIGURE 8.4 Moment-Curvature Model

In order to determine force demands on Capacity Protected Members connected to a yielding member, a 20% overstrength magnifier shall be applied to the plastic moment capacity of the column to account for:

- Material strength variations between the column and adjacent members (e.g. superstructure, bent cap, footings, oversized pile shafts)
- Column moment capacities greater than the idealized plastic moment capacity

$$M_{po} = 1.2 \times M_p \qquad (8.8)$$

C8.6 SHEAR DEMAND AND CAPACITY FOR DUCTILE CONCRETE MEMBERS SDC B, C AND D

The requirements of this article are intended to avoid column shear failure by using the principles of "capacity protection". The design shear force is specified as a result of the actual longitudinal steel provided, regardless of the design forces. This requirement is necessary because of the potential for superstructure collapse if a column fails in shear.

A column may yield in either the longitudinal or transverse direction. The shear force corresponding to the maximum shear developed in either direction for noncircular columns should be used for the determination of the transverse reinforcement.

For a noncircular pile, this provision may be applied by substituting the larger cross-sectional dimension for the diameter.

8.6 SHEAR DEMAND AND CAPACITY FOR DUCTILE CONCRETE MEMBERS SDC B, C AND D

8.6.1 Shear Demand and Capacity

The shear demand for a column, V_d, in SDC B shall be determined based on the lesser of:

- The force obtained from an elastic linear analysis
- The force, V_o, corresponding to plastic hinging of the column including an overstrength factor

The shear demand for a column, V_d, in SDC C or D shall be determined based on the force, V_o, associated with the overstrength moment, M_{po}, defined in Article 8.5.

The column shear strength capacity shall be calculated based on the nominal material strength properties.

$$\phi V_n \geq V_d \qquad \phi = 0.85 \tag{8.9}$$

$$V_n = V_c + V_s \tag{8.10}$$

8.6.2 Concrete Shear Capacity SDC B, C and D

The concrete shear capacity of members designed for SDC B, C and D shall be determined as specified in Equation 8.11 through 8.24.

$$V_c = v_c A_e \tag{8.11}$$

$$A_e = 0.8 A_g \tag{8.12}$$

For members whose net axial load is in tension, $v_c = 0$

For shear stress capacity inside the plastic hinge zone

$$v_c = \alpha' \left(1 + \frac{P}{2000 A_g} \right) \sqrt{f'_{ce}} \leq 3.5 \sqrt{f'_{ce}} \tag{8.13}$$

where,

P = axial compressive force (kips)

For columns with hoops or spirals

$$\alpha' = 0.015 \rho_s f_{yt} \qquad \text{for SDC B} \tag{8.14}$$

$$\alpha' = 0.010\rho_s f_{yt} \qquad \text{for SDC C} \qquad (8.15)$$

$$\alpha' = \frac{0.03}{\mu_D}\rho_s f_{yt} \qquad \text{for SDC D} \qquad (8.16)$$

For tied reinforced columns

$$\alpha' = 0.030\rho_w f_{yt} \qquad \text{for SDC B} \qquad (8.17)$$

$$\alpha' = 0.020\rho_w f_{yt} \qquad \text{for SDC C} \qquad (8.18)$$

$$\alpha' = \frac{0.06}{\mu_D}\rho_w f_{yt} \qquad \text{for SDC D} \qquad (8.19)$$

For SDC D the displacement ductility μ_D used to derive α' shall be based on the maximum local ductility demand in either of the principal local member axes.

For shear stress capacity outside the plastic hinge zone

$$v_c = \alpha''\left(1+\frac{P}{2A_g}\right)\sqrt{f'_{ce}} \leq 3.5\sqrt{f'_{ce}} \qquad (8.20)$$

For columns with spirals or hoops

$$\alpha'' = 0.03\rho_s f_{yt} \qquad (8.21)$$

For tied reinforced columns

$$\alpha'' = 0.06\rho_w f_{yt} \qquad (8.22)$$

Where the reinforcement ratio of spirals or hoops,

$$\rho_s = \frac{4A_{sp}}{Ds} \qquad (8.23)$$

and the web reinforcement ratio
(in the direction of bending)

$$\rho_w = \frac{A_v}{bs} \qquad (8.24)$$

8.6.3 Shear Reinforcement Capacity

For confined circular or interlocking core sections, as described in Article 8.6.4, the shear reinforcement strength capacity is calculated using Equation 8.25.

C8.6.3 Transverse Reinforcement for Confinement at Plastic Hinges

Plastic hinge regions are generally located at the top and bottom of columns and pile bents.

These requirements and equations govern the transverse reinforcement for confinement at plastic hinges.

These equations ensure that the concrete is adequately confined so that the transverse hoops will not prematurely fracture as a result of the plastic work done on the critical column section. For typical bridge columns with low levels of axial load, these equations rarely govern, but must be checked.

$$V_s = \frac{\pi}{2}\left(\frac{nA_{sp} f_{yh} D}{s}\right), \tag{8.25}$$

where

n = number of individual interlocking spiral or hoop core sections.

For tied columns or pier walls (in the weak direction).

$$V_s = \left(\frac{A_v f_{yh} D}{s}\right) \tag{8.27}$$

$A_v =$ Total area of the shear reinforcement

8.6.4 Shear Reinforcement Capacity of Interlocking Spirals

The shear reinforcement strength provided by interlocking spirals or hoops shall be taken as the sum of all individual spiral or hoop shear strengths calculated in accordance with Equation 8.25.

8.6.5 Maximum Shear Reinforcement

The shear strength provided by the reinforcing steel, $V_{s,}$ shall not be taken greater than:

$$8\sqrt{f'_{ce}}\ A_e \text{ (psi)} \tag{8.28}$$

8.6.6 Minimum Shear Reinforcement

The area of column spiral reinforcement, A_{sp}, or column web reinforcement A_v shall be determined based on Equations 8.23 and 8.24. The minimum spiral reinforcement ratio, ρ_s for each individual core of a column and the minimum web reinforcement ratio ρ_w shall be as follows:

For SDC B

ρ_s = .2%

ρ_w = .3%

For SDC C or D

ρ_s = .4%

Preventing the 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 to simply lap the spiral reinforcement. If the concrete cover spalls, the spiral will be able to unwind. Similarly, rectangular hoops should be anchored by bending ends back into the core.

Examples of transverse column reinforcement are shown in Figures C8.6.3-1 to C8.6.3-4. Figures C8.6.3-1 through C8.6.3-4 also illustrate the use of Equations 8.6.3-1 and 8.6.3-2. The required total area of hoop reinforcement should be determined for both principal axes of a rectangular or oblong column, and the greater value should be used.

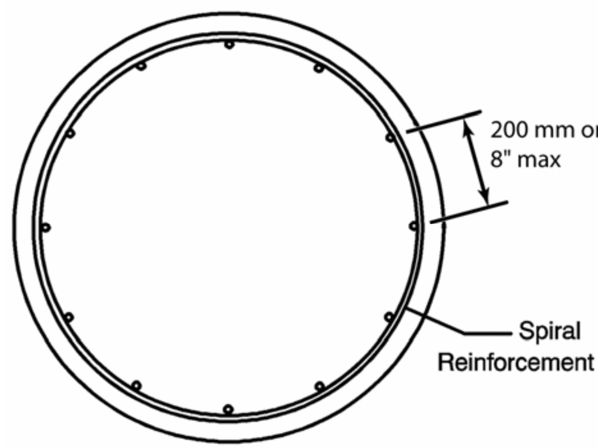


Figure C.8.6.3-1 Single Spiral

While these Guidelines allow the use of spirals, hoops or ties for transverse column reinforcement, the use of spirals is recommended as the most effective and economical solution. Where more than one spiral cage is used to confine an oblong column core, the spirals should be interlocked with longitudinal bars as shown in Figure C8.8.2.4-3. Spacing of longitudinal bars of a maximum of 200 mm center-to-center is also recommended to help confine the column core.

ρ_w = .5%

8.6.7 Pier Wall Shear Capacity in the Weak Direction

The shear capacity for pier walls in the weak direction shall be designed according to Articles 8.6.2 & 8.6.3.

8.6.8 Minimum Vertical Reinforcement in Interlocking Portion

The longitudinal rebars in the interlocking portion of the column shall have a maximum spacing of 8 inches and need not be anchored in the footing or the bent cap unless deemed necessary for the flexural capacity of the column. The longitudinal rebar size in the interlocking portion of the column shall be chosen correspondingly to the rebars outside the interlocking portion as shown in Table 8.1.

Table 8.1: Reinforcement Size for Interlocking Portion of Columns

Minimum Size of rebars required inside the interlocking portion	Size of rebars used outside the interlocking portion
#6	#10
#8	#11
#9	#14
#11	#18

8.6.9 Pier Wall Shear Capacity in the Strong Direction

The shear capacity of pier walls in the strong direction shall resist the maximum shear demand V_u specified in Article 8.3 including superstructure to substructure connection capacity or foundation capacity whichever is smaller.

$$\phi V_n > V_u \tag{8.29}$$

where

$\phi = 0.85$

The nominal shear resistance, V_n, in the pier shall satisfy Equations 8.30a and 8.30b.

$$V_n = \left[4\sqrt{f'_{ce}} + \rho_h f_{ye}\right] bd \tag{8.30a}$$

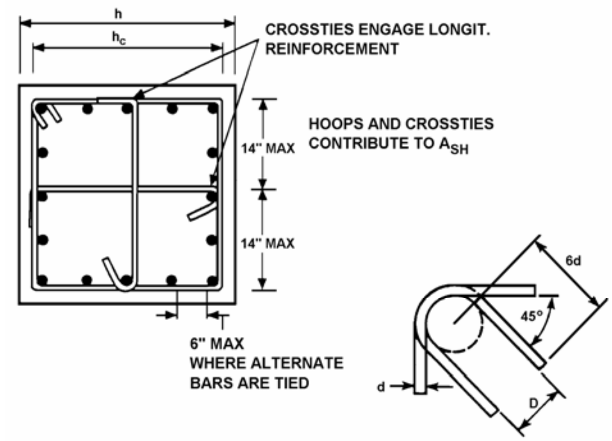


Figure C8.6.3-2 Column Tie Details

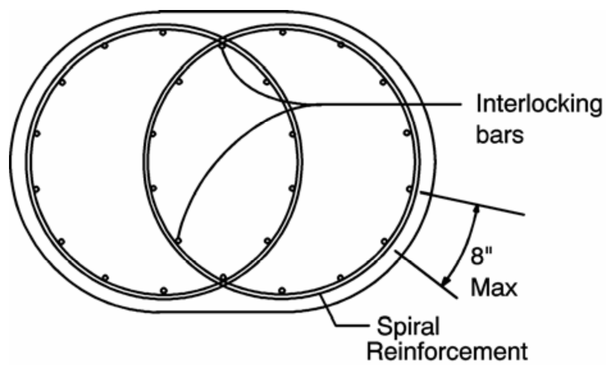


Figure C8.6.3-3 Column Interlocking Spiral Details

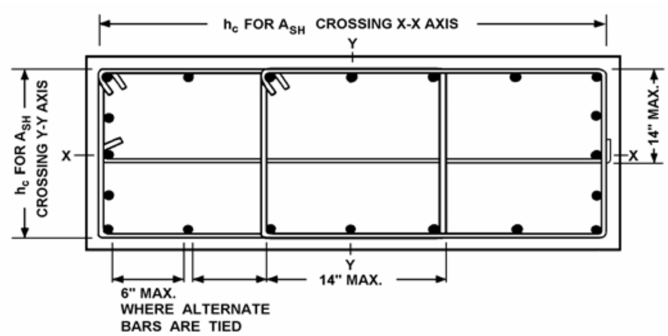


Figure C8.6.3-4 Column Tie Details

Longitudinal reinforcing bars in potential plastic hinge zones may be highly strained in compression to the extent that they may buckle. Buckling may either be

a. local between two successive hoop sets or spirals, or

b. global and extend over several hoop sets or spirals.

Studies of squat shear walls have demonstrated that the large shear stresses associated with the moment capacity of the wall may lead to a sliding failure brought about by crushing of the concrete at the base of the wall. The thickness of pier walls shall be selected so the shear stress satisfies Equation 8.30b.

$$\frac{V_n}{0.8A_g} < 8\sqrt{f'_{ce}} \quad \text{(psi)} \tag{8.30b}$$

8.6.10 Pier Wall Minimum Reinforcement

The minimum reinforcement ratio, both horizontally, ρ_h, and vertically, ρ_v, in any pier shall not be less than 0.0025. The vertical reinforcement ratio shall not be less than the horizontal reinforcement ratio.

Reinforcement spacing, either horizontally or vertically, shall not exceed 18 inches. The reinforcement required for shear shall be continuous and shall be distributed uniformly. Horizontal and vertical layers of reinforcement should be provided on each face of a pier. Splices in horizontal pier reinforcement shall be staggered.

8.7 REQUIREMENTS FOR DUCTILE MEMBERS DESIGN

8.7.1 Minimum Lateral Strength

Each column shall have a minimum lateral flexural capacity (based on expected material properties) to resist a lateral force of $0.1P_{dl}$. Where P_{dl} is the axial dead load effects corresponding to the lateral inertia lumped on top of the column.

The requirement for pier wall flexural capacity in the weak direction is similar to a column. Piles extension where ductility demand is greater than one shall have the same requirement.

8.7.2 Maximum Axial Load In A Ductile Member in SDC C and D

The maximum axial load in a column, a pier wall, or a pile where ductility demand is greater than two shall not be greater than $0.2f'_{ce}A_g$ where f'_{ce} is expected concrete strength and A_g is the gross cross-sectional area. A higher axial load value may be used provided a moment-curvature pushover analysis is performed to compute the maximum

Condition (a) is prevented by using the maximum vertical spacing of transverse reinforcement given by Equation 7.8.2.5-1 or 8.8.2.5-1 of the *Specifications*.

Although research has been conducted to determine the amount of transverse reinforcement required to prevent condition (b), this research has not been fully peer reviewed, and thus has not been included as part of the *Specifications*. However, designers should not ignore the possibility of condition (b) and should take steps to prevent it.

The following tentative criteria for transverse reinforcement to prevent condition (b) have been proposed:

i. For circular sections confined by spirals or circular hoops:

$$\rho_s = 0.016\left(\frac{D}{s}\right)\left(\frac{s}{d_b}\right)\rho_t\frac{f_y}{f_{yh}}$$

ii. For rectangular sections confined by transverse hoops and/or cross ties the area of the cross tie or hoop legs (A_{bh}) shall be:

$$A_{bh} = 0.09A_b\frac{f_y}{f_{yh}}$$

where

ρ_s = ratio of transverse reinforcement

$$\rho_s = \frac{4A_{bh}}{sD'}$$

D = diameter of circular column

d_b = diameter of longitudinal reinforcing bars being restrained by circular hoop or spiral

A_b = area of longitudinal reinforcing bars being restrained by rectilinear hoops and/or cross ties

A_{bh} = bar area of the transverse hoops or ties restraining the longitudinal steel

ρ_t = volumetric ratio of longitudinal reinforcement

f_y = yield stress of the longitudinal reinforcement

f_{yh} = yield stress of the transverse reinforcing bars

Trial applications have shown that the above equations result in excessive transverse reinforcement in some cases. This is usually associated with high amounts of column longitudinal reinforcement, and so it may be prudent for a designer to limit the volumetric ratio of longitudinal reinforcement.

ductility capacity of the member. The nominal shear capacity of the wall needs not to be greater than 40% of the elastic spectral forces obtained using analysis procedure 1 or 2.

8.8 LONGITUDINAL AND LATERAL REINFORCEMENT REQUIREMENTS

8.8.1 Maximum Longitudinal Reinforcement

The area of longitudinal reinforcement for compression members shall not exceed the value specified in Equation 8.31.

$$0.04 \times A_g \qquad (8.31)$$

8.8.2 Minimum Longitudinal Reinforcement

The minimum area of longitudinal reinforcement for compression members shall not be less than the value specified in Equations 8.32 thru 8.34.

$0.007 \times A_g$ for Columns in SDC B, C (8.32a)

$0.01 \times A_g$ for Columns in SDC D (8.32b)

$0.0025 \times A_g$ for Pier Walls in SDC B, C (8.33)

$0.005 \times A_g$ for Pier Walls in SDC D (8.34)

8.8.3 Splicing of Longitudinal Reinforcement in Columns Subject to Ductility Demands for SDC C or D

Splicing of longitudinal column reinforcement in SDC C or D shall be outside the plastic hinging region as defined in Article 4.11.7. For SDC D ultimate strength splicing of reinforcement shall be used by means of mechanical couplers as Approved by Owner. For a shaft in SDC D where mechanical or lap splicing can not be avoided due to the extent of zone comprising the location of hinging in the liquefied and non-liquefied cases ultimate strength splicing of reinforcement by means of mechanical couplers shall be used outside the above mentioned zone.

8.8.4 Minimum Development Length of Reinforcing Steel for SDC C or D

Column longitudinal reinforcement shall be extended into footings and cap beams as close as practically possible to the opposite face of the footing or cap beam.

C8.6.7 Limited Ductility Requirements for Wall-Type Piers

The requirement that $\rho_v \geq \rho_h$ is intended to avoid the possibility of having inadequate web reinforcement in piers which are short in comparison to their height. Splices should be staggered to avoid weak sections.

C8.6.7.1 Reinforcement for Joint Force Transfer

C8.6.7.1.1 Acceptable Reinforcement Details

A "rational" design is required for joint reinforcement when principal tension stress levels exceed $0.29\sqrt{f_c'}$ MPa. The amounts of reinforcement required are based on the mechanism shown in Figure C8.8.4.3-1 which primarily uses external reinforcement for joint resistance to reduce joint congestion.

C8.8 LONGITUDINAL REINFORCEMENT

This requirement is intended to apply to the full section of the columns. The 1.0% 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 4% maximum ratio is to avoid congestion and extensive shrinkage cracking and to permit anchorage of the longitudinal steel, but most importantly, the smaller the amount of longitudinal reinforcement, the greater the ductility of the column.

The anchorage length for longitudinal column bars l_{ac} developed into the cap beam for seismic loads shall not be less than $24d_{bl}$ (in).

For SDC D, the anchorage length shall not be reduced by means of adding hooks or mechanical anchorage devices.

8.8.5 Anchorage of Bundled Bars in Ductile Components for SDC C or D

The anchorage length of individual column bars within a bundle anchored into a cap beam shall be increased by twenty percent for a two-bar bundle and fifty percent for a three-bar bundle. Four-bar bundles are not permitted in ductile elements.

8.8.6 Maximum Bar Diameter for SDC C, or D

In order to ensure adequate bond to concrete, the nominal diameter of longitudinal reinforcement, $d_{b\ell}$, in columns shall satisfy Equation (8.35):

$$d_{b\ell} \leq \frac{25 \times \sqrt{f_c^{'}}(L - 0.5D_C)}{f_{ye}} \text{ (in, psi)}, \quad (8.35)$$

where L is the length of column from the point of maximum moments to the point of contra-flexure established based on Capacity Design principles specified in Article 4.11. Where longitudinal bars in columns are bundled, this requirement of adequate bond shall be checked for the effective bar diameter, assumed as$1.2\times d_{bl}$, for two-bar bundles, and $1.5\times d_{bl}$ for three-bar bundles.

8.8.7 Lateral Reinforcement Inside The Plastic Hinge Region for SDC D

The volume of lateral reinforcement typically defined by the volumetric ratio, ρ_s or ρ_w provided inside the plastic hinge length shall be sufficient to ensure the column or pier wall has adequate shear capacity and confinement level to achieve the required ductility capacity.

For columns designed to achieve a ductility greater than 4, the lateral reinforcement shall be either butt-welded hoops or continuous spiral. Combination of hoops and spiral is not permitted except in the footing or the bent cap.

Hoops can be placed around the column cage (i.e., extended longitudinal reinforcing steel) in lieu of continuous spiral reinforcement in the cap and footing.

C8.8.7 Spacing for Transverse Reinforcement for Confinement and Longitudinal Bar Restraint

This requirement ensures all inelastic portions of the column are protected by confining steel.

C8.8.2.7 Splices

It is often desirable to lap longitudinal reinforcement with dowels at the column base. This is undesirable for seismic performance because:

- The splice occurs in a potential plastic hinge region where requirements for bond are critical, and
- 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 severe local curvature demand.

The simplified method for calculating an overstrength moment-axial-load interaction diagram (Mander, et. al, 1998) involves a parabolic curve.

8.8.8 Lateral Column Reinforcement Outside The Plastic Hinge Region for SDC C or D

The volume of lateral reinforcement required outside of the plastic hinge region, shall not be less than 50% of the determined in accordance with Articles 8.8.7 or 8.6.

The lateral reinforcement type outside the plastic hinge region shall be the same as inside the plastic hinge region. At spiral or hoop to spiral discontinuities, the spiral shall terminate with one extra turn plus a tail equal to the cage diameter.

8.8.9 Maximum Spacing for Lateral Reinforcement for SDC C or D

The maximum spacing for lateral reinforcement in the plastic end regions shall not exceed the smallest of the following:

- One fifth of the least dimension of the cross-section for columns and one-half of the least cross-section dimension of piers.
- Six times the nominal diameter of the longitudinal reinforcement.
- 6 inches for single hoop or 8 inches for bundled hoops.

The lateral reinforcement shall extend in to the footing to the beginning of the longitudinal bar bend above the bottom mat. For the bent cap the longitudinal steel shall extend a distance to ensure adequate development length for the plastic hinge mechanism.

8.8.10 Development Length for Column Bars Extended into Shafts for SDC C or D

Column longitudinal reinforcement shall be extended into enlarged shafts in a staggered manner with the minimum recommended embedment lengths of $2 \times D_{c,\max}$ and $3 \times D_{c,\max}$, where $D_{c,\max}$ is the larger cross section dimension of the column.

8.8.11 Lateral Reinforcement Requirements For Columns Supported On Oversized Pile Shafts for SDC C or D

The volumetric ratio of lateral reinforcement for columns supported on oversized pile shafts shall meet the requirements specified in Articles 8.8.7 and 8.8.8. At least 50% of the confinement reinforcement required at the base of the column shall extend over the entire embedded length of the column cage.

8.8.12 Lateral Confinement For Oversized Pile Shafts for SDC C or D

The lateral confinement in an oversized shaft shall be 50% of the confinement at the base of the column provided the shaft is designed for a flexural expected nominal capacity equal to 1.25 times the moment demand generated by the overstrength moment of the embedded column. The lateral confinement shall extend along the shaft until the embedded column cage is terminated. The spacing of the oversized shaft confinement can be doubled beyond the column cage termination length.

8.8.13 Lateral Confinement for Non Oversized Strengthened Pile Shafts for SDC C or D

The volumetric ratio of lateral confinement in the top segment $4 \times D_{C,max}$ of the shaft shall be at least 75% of the confinement reinforcement required at the base of the column provided the shaft is designed for a flexural expected nominal capacity equal to 1.25 times the moment demand generated by the overstrength moment of the embedded column. The lateral confinement shall extend along the shaft until the embedded column cage is terminated. The spacing of the shaft confinement can be doubled beyond the column cage termination length.

8.9 REQUIREMENTS FOR CAPACITY PROTECTED MEMBERS

Members, adjacent to plastic hinging locations, such as footings, oversized pile shafts, bent caps, joints, and girders shall be designed to remain essentially elastic when the column reaches its overstrength capacity. The expected nominal moment capacity M_{ne} for essentially elastic members shall be determined based on stress-strain compatibility analysis using a $(M-\phi)$ diagram. The expected nominal capacity M_{ne} is used in establishing the capacity of essentially elastic members.

Expected nominal moment capacity for essentially elastic concrete components shall be based on the expected concrete and steel strengths when the concrete strain reaches a magnitude of 0.005.

8.10 SUPERSTRUCTURE CAPACITY DESIGN FOR LONGITUDINAL DIRECTION SDC C & D

The superstructure shall be designed as a capacity protected member. Any moment demand caused by dead load or secondary prestress effects shall be distributed to the entire width of the superstructure. The column overstrength moment M_{po} in addition to the moment induced due to the eccentricity between the plastic hinge location and the center of gravity of the superstructure shall be distributed to the left and right spans of the bent based on their stiffness distribution factors. This moment demand shall be considered within the effective width of the superstructure.

The effective width of superstructure resisting longitudinal seismic moments is defined by Equation 8.36. The effective width for open soffit structures (i.e. T-Beams & I Girders) is reduced because they offer less resistance to the torsional rotation of the bent cap. The effective superstructure width can be increased at a 45° angle away from the bent cap until the full section becomes effective. On skewed bridges, the effective width shall be projected normal to the girders where the centerline of girder intersects the face of the bent cap. (see Figure 8.5).

$$B_{eff} = \begin{cases} D_c + 2\mathrm{x}\mathrm{D_s} & \text{Box Girders \& Solid Superstructures} \\ D_c + D_s & \text{Open Soffit Superstructure} \end{cases} \tag{8.36}$$

Additional superstructure width can be considered effective if the designer verifies that the torsional stiffness of the cap can distribute the rotational demands beyond the effective width stated in Equation 8.36.

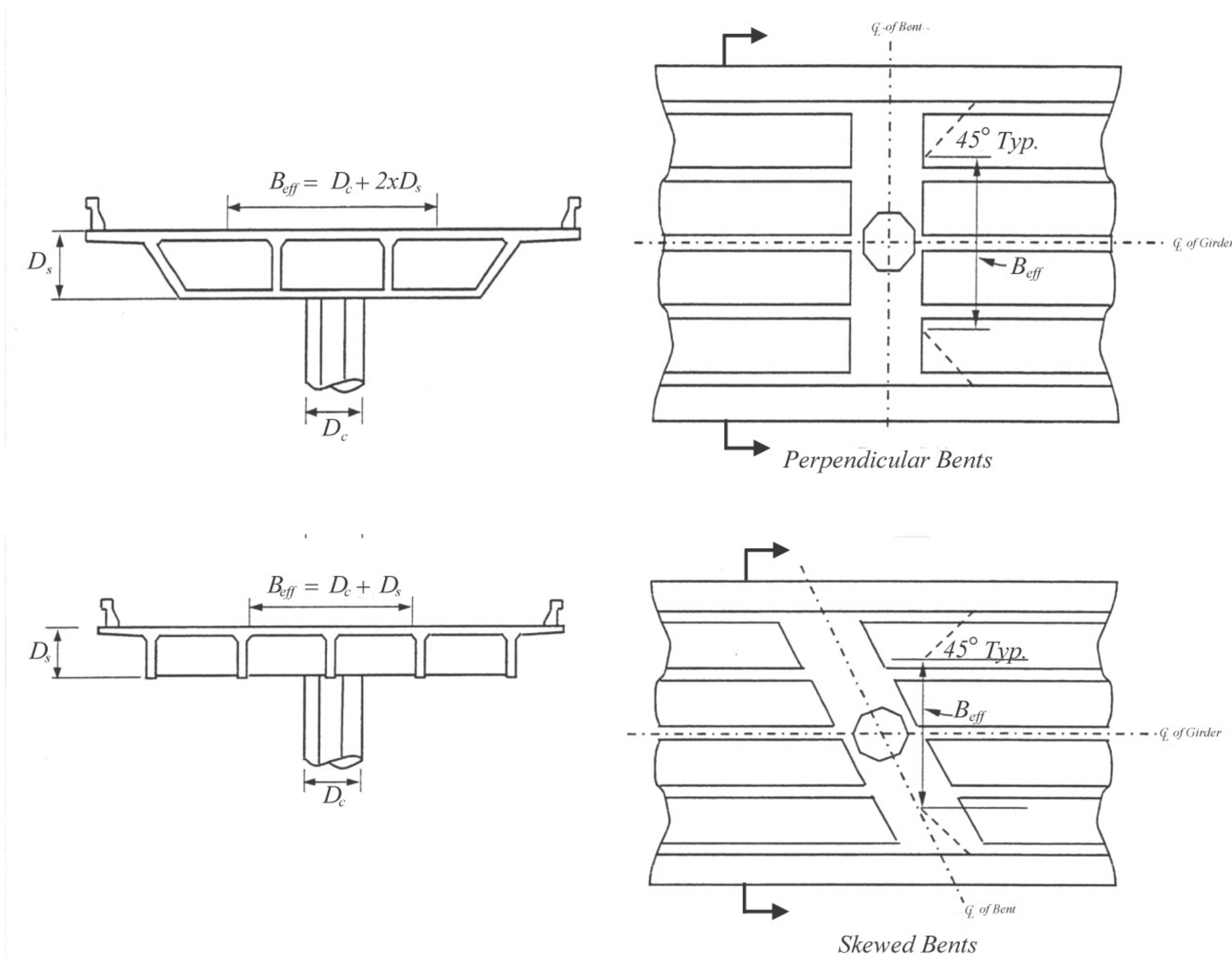


FIGURE 8.5 Effective Superstructure Width

8.11 SUPERSTRUCTURE CAPACITY DESIGN FOR TRANSVERSE DIRECTION (INTEGRAL BENT CAP) SDC C & D

Bent caps are considered integral if they terminate at the outside of the exterior girder and respond monolithically with the girder system during dynamic excitation.

The bent cap shall be designed as an essentially elastic member. Any moment demand caused by dead load or secondary prestress effects shall be distributed to the effective width of the bent cap, as shown in Figure 8.6. The column overstrength moment M_o in addition to the moment induced due to the eccentricity between the plastic hinge location and the center of gravity of the bent cap shall be distributed based on the effective stiffness characteristics of the frame. This moment shall be considered within the effective width of the bent cap. The effective widths shall be determined using Equation 8.37 (see Figure 8.6).

$$B_{eff} = B_{cap} + (12t) \tag{8.37}$$

t = thickness of the top or bottom slab

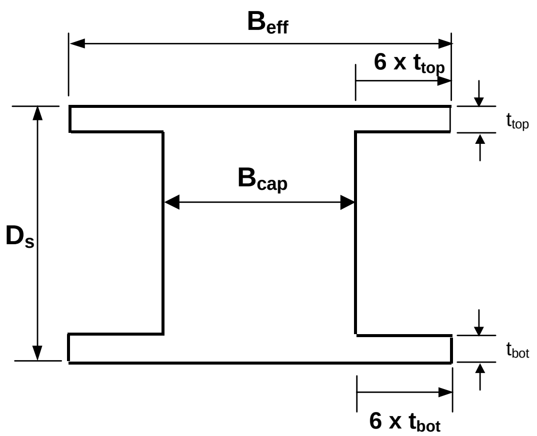


FIGURE 8.6 Effective Bent Cap Width

For SDC C or D, cutting off bent cap reinforcement shall be avoided. Splicing of reinforcement shall be done using service couplers at a minimum for SDC C or D.

8.12 SUPERSTRUCTURE DESIGN FOR NONINTEGRAL BENT CAP SDC C AND D

Nonintegral bent caps shall satisfy all requirements stated for frames with integral bent cap in the transverse direction. The minimum lateral transfer mechanism at the superstructure/substructure interface shall be established using the smaller of the elastic seismic force or the sum of an acceleration of 0.4g times the Dead Load reaction and the overstrength capacity of shear keys.

Superstructure members supported on non-integral bent caps shall be simply supported at the bent cap or span continuously with a separation detail such as an elastomeric pad or isolation bearing between the bent cap and the superstructure. Refer to Type 3 choice of Article 7.2.

Drop caps supporting superstructures with expansion joints at the cap shall have sufficient width

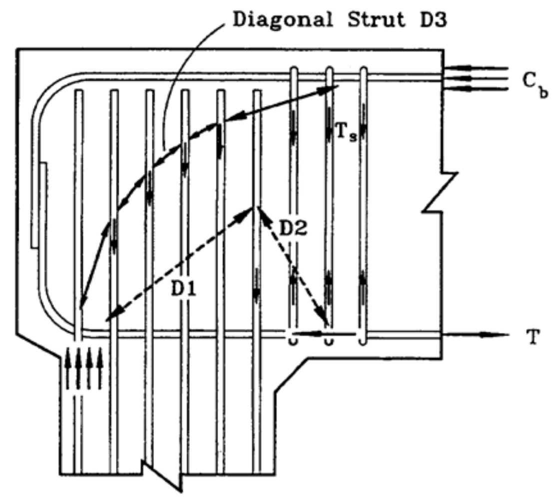


Figure C8.8.4.3-1 External Vertical Joint Reinforcement for Joint Force Transfer

C8.8.4.3.2 Vertical Reinforcement

Stirrups

Figure C8.8.4.3-1 is intended to clarify this clause. A_{ST} is the total area of column reinforcement anchored in the joint. Reinforcement A_{jv} is required to provide the tie force T_s resisting the vertical component of strut D2 in Figure C8.8.4.3-1. This reinforcement should be placed close to the column cage for maximum efficiency.

Clamping Reinforcement

In addition, it will be recognized that the cap beam top reinforcement or footing bottom reinforcement may have severe bond demands, since stress levels may change from close to tensile yield on one side of the joint to significant levels of compression stress on the other side. The required 0.08 A_{ST} vertical ties inside the joint are intended to help provide this bond transfer by clamping the cap-beam rebar across possible splitting cracks. Similar restraint may be required for superstructure top longitudinal rebar. Cap beam widths one foot greater than column diameter are encouraged so that the joint shear reinforcement is effective.

When the cap-beam, superstructure, or both, are prestressed, the bond demands will be much less severe and the clamping requirement can be relaxed. It can also be shown theoretically (Priestley, Seible and Calvi, 1996) that the volumetric ratio of hoop to prevent unseating. The minimum seat width for non-integral bent caps shall be determined based on Article 4.12. Continuity devices such as rigid restrainers or web plates may be used to ensure unseating does not occur but shall not be used in lieu of adequate bent cap width.

8.13 SUPERSTRUCTURE JOINT DESIGN SDC C OR D

8.13.1 Joint Performance

Moment resisting connections between the superstructure and the column shall be designed to transmit the maximum forces produced when the column has reached its overstrength capacity, M_{po} including the effects of overstrength shear, V_{po}.

8.13.2 Joint Proportioning

All superstructure/column moment resisting joints shall be proportioned so the principal stresses satisfy Equations 8.38 and 8.39

Principal compression:

$$p_c \leq 0.25 f'_{ce} \tag{8.38}$$

Principal tension:

$$p_t \leq 12\sqrt{f'_{ce}}\,(psi) \tag{8.39}$$

8.13.3 Joint Description

The following types of joints are considered "T" joints for joint shear analysis:

- Integral interior joints of multi-column bents in the transverse direction
- All column/superstructure joints in the longitudinal direction
- Exterior column joints for box girder superstructures if the cap beam extends beyond the joint far enough to develop the longitudinal cap reinforcement. All other exterior joints are considered knee joints in the transverse direction and require special analysis and detailing.

reinforcement can be proportionately reduced to zero as the prestress force approaches $0.25T_c$.

C8.8.4.4 Structural Strength of Footings

Under extreme seismic loading, it is common for the footing to be subjected to positive moments on one side of the column and negative moments on the other. In this case, shear lag considerations show that it is unrealistic to expect footing reinforcement at lateral distances greater than the footing effective depth to participate effectively in footing flexural strength. Tests on footings (Xiao et al., 1994) have shown that a footing effective width complying with this clause will produce a good prediction of maximum footing reinforcement stress. If a larger effective width is adopted in design, shear lag effects will result in large inelastic strains developing in the footing reinforcement adjacent to the column. This may reduce the shear strength of the footing and jeopardize the footing joint force transfer mechanisms. Since the reinforcement outside the effective width is considered ineffective for flexural resistance, it is permissible to reduce the reinforcement ratio in such regions to 50% of that within the effective width unless more reinforcement is required to transfer pile reactions to the effective sections.

Arguments similar to those for moment apply to the effective width for shear strength estimation.

C8.8.5.3 Cast-in-Place and Precast Concrete Piles

No commentary is provided for Article 8.8.5.3.

C8.8.6 Plastic Rotation Capacities

A moment-curvature analysis based on strain compatibility and nonlinear stress-strain relations can be used to determine plastic limit states. From this a rational analysis is used to establish the rotational capacity of plastic hinges.

C8.8.6.1 Life Safety Performance

If a section has been detailed in accordance with the transverse reinforcement requirement of these provisions, then the section is said to be 'capacity protected' against undesirable modes of failure such as shear, buckling of longitudinal bars, and concrete crushing due to lack of confinement.

8.13.4 T Joint Shear Design

8.13.4.1 Principal Stress Definition

The principal tension and compression stresses in a joint are defined as follows:

$$p_t = \left| \frac{(f_h + f_v)}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} \right| \quad (8.40)$$

$$p_c = \frac{(f_h + f_v)}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} \quad (8.41)$$

$$v_{jv} = T_c / A_{jv} \quad (8.42)$$

$$A_{jv} = l_{ac} \times B_{cap} \quad (8.43)$$

$$f_v = \frac{P_c}{A_{jh}} \quad (8.44)$$

$$A_{jh} = (D_c + D_s) \times B_{cap} \quad (8.45)$$

$$f_h = \frac{P_b}{B_{cap} \times D_s} \quad (8.46)$$

Where:

A_{jh} = The effective horizontal joint area

A_{jh} = The effective vertical joint area

B_{cap} = Bent cap width

$D_c =$ Cross-sectional dimension of column in the direction of bending

$D_s =$ Depth of superstructure at the bent cap

$l_{ac} =$ Length of column reinforcement embedded into the bent cap

$P_c =$ The column axial force including the effects of overturning

$P_b =$ The beam axial force at the center of the joint including prestressing

$T_c =$ The column tensile force defined as M_o^{col}/h, where h is the distance from c.g. of tensile force to c.g. of compressive force on the section, or alternatively, T_c may be obtained from the moment-curvature analysis of the cross section

Note: Unless the prestressing is specifically designed to provide horizontal joint compression, f_h can typically be ignored without significantly effecting the principle stress calculation.

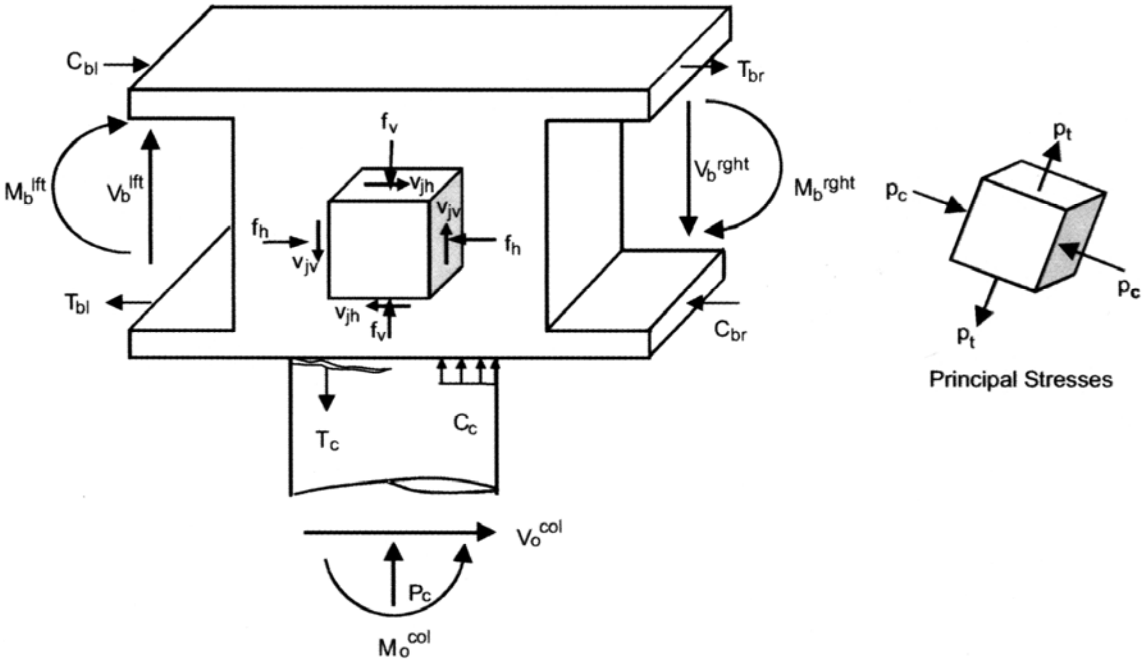


FIGURE 8.7 Joint Shear Stresses in T Joints

8.13.4.2 Minimum Joint Shear Reinforcement SDC C or D

If the principal tension stress p_t does not exceed $3.5\sqrt{f'_{ce}}$ psi the minimum joint shear reinforcement, as specified in Equation 8.47, shall be provided. This joint shear reinforcement may be provided in the form of column transverse steel or exterior transverse reinforcement continued into the bent cap. No additional joint reinforcement as prescribed in Section 8.13.4.3 is required for SDC C. The volumetric ratio of transverse column reinforcement ρ_s continued into the cap shall not be less than the value specified by equation 8.47.

$$\rho_{s,\min} = \frac{3.5\sqrt{f'_{ce}}}{f_{yh}} \text{ (psi)} \tag{8.47}$$

The reinforcement shall be in the form of tied column reinforcement, spirals, hoops, or intersecting spirals or hoops.

If the principal tension stress p_t exceeds $3.5\sqrt{f'_{ce}}$ psi for SDC D, the following shall apply:

- The joint shear reinforcement specified in Article 8.13.4.3 is required.

- The bent cap width shall extend 12 inches on opposite sides of the column as shown in Figure 8.9.

8.13.4.3 Joint Shear Reinforcement SDC D

A) Vertical Stirrups:

$$A_s^{jv} = 0.2 \times A_{st} \tag{8.48}$$

A_{st} = Total area of column reinforcement anchored in the joint

Vertical stirrups or ties shall be placed transversely within a distance D_c extending from either side of the column centerline. The vertical stirrup area, A_s^{jv} is required on each side of the column or pier wall, see Figures 8.8, 8.9 and 8.10. The stirrups provided in the overlapping areas shown in Figure 8.8 shall count towards meeting the requirements of both areas creating the overlap. These stirrups can be used to meet other requirements documented elsewhere including the shear in the bent cap.

B) Horizontal Stirrups:

Horizontal stirrups or ties shall be placed transversely around the vertical stirrups or ties in two or more intermediate layers spaced vertically at not more than 18 inches. This horizontal reinforcement A_s^{jh} shall be placed within a distance D_c extending from either side of the column centerline, see Figure 8.10.

$$A_s^{jh} = 0.1 \times A_{st} \tag{8.49}$$

C) Horizontal Side Reinforcement:

The total longitudinal side face reinforcement in the bent cap shall be at least equal to the greater of the areas specified in Equation 8.50 and shall be placed near the side faces of the bent cap with a maximum spacing of 12 inches, see Figure 8.9. Any side reinforcement placed to meet other requirements shall count towards meeting the requirement in this section.

$$
A_s^{sf} \geq \begin{cases} 0.1 \times A_{cap}^{top} \\ \text{or} \\ 0.1 \times A_{cap}^{bot} \end{cases} \tag{8.50}
$$

where,

$A_{cap}^{top} =$ Area of bent cap top flexural steel

$A_{cap}^{bot} =$ Area of bent cap bottom flexural steel

D) J-Dowels:

For integral cap of bents skewed greater than 20, J-dowels hooked around the longitudinal top deck steel extending alternatively 24 inches and 30 inches into the bent cap are required. The J-dowel reinforcement shall be equal or greater than the area specified in Equation 8.51.

$$
A_s^{j-bar} = 0.08 \times A_{st} \tag{8.51}
$$

The J-dowels shall be placed within a rectangular region defined by the width of the bent cap and the distance D_c on either side of the centerline of the column, see Figure 8.11.

E) Transverse Reinforcement:

Transverse reinforcement in the joint region shall consist of hoops with a minimum reinforcement ratio specified by Equation 8.52. The column confinement reinforcement extended into the bent cap may be used to meet this requirement.

$$
\rho_s = 0.4 \times \frac{A_{st}}{l_{ac}^2} \quad \text{(in)} \tag{8.52}
$$

For interlocking cores, ρ_s shall be based on area of reinforcement $\left(A_{st}\right)$ of each core.

F) Main Column Reinforcement

The main column reinforcement shall extend into the cap to the top bent cap reinforcement to fully develop the compression strut mechanism in the joint.

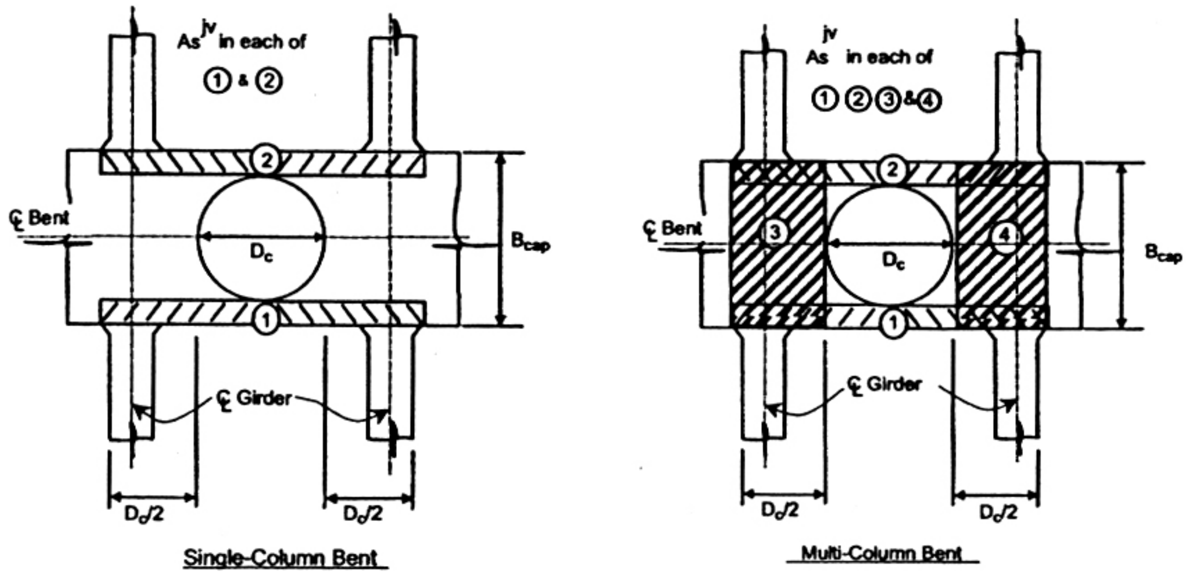


FIGURE 8.8 Location of Vertical Joint Reinforcement

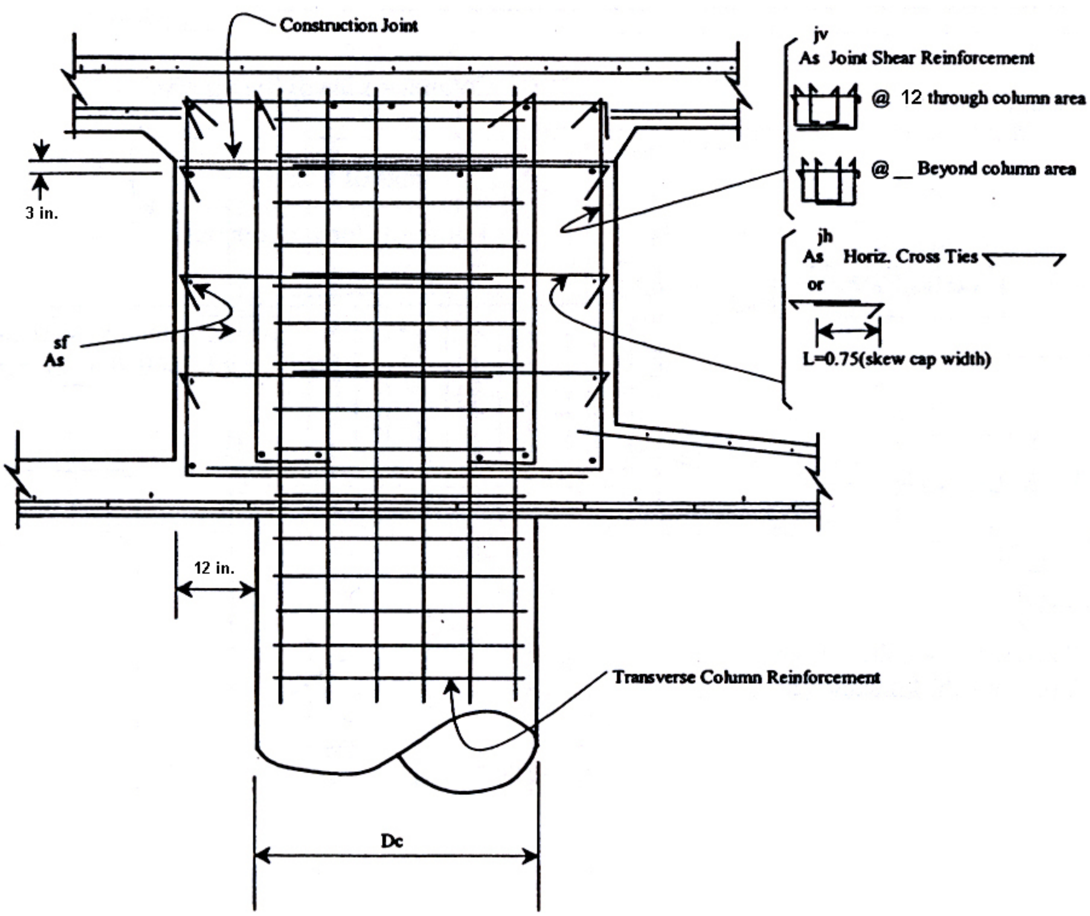


FIGURE 8.9 Joint Shear Reinforcement Details

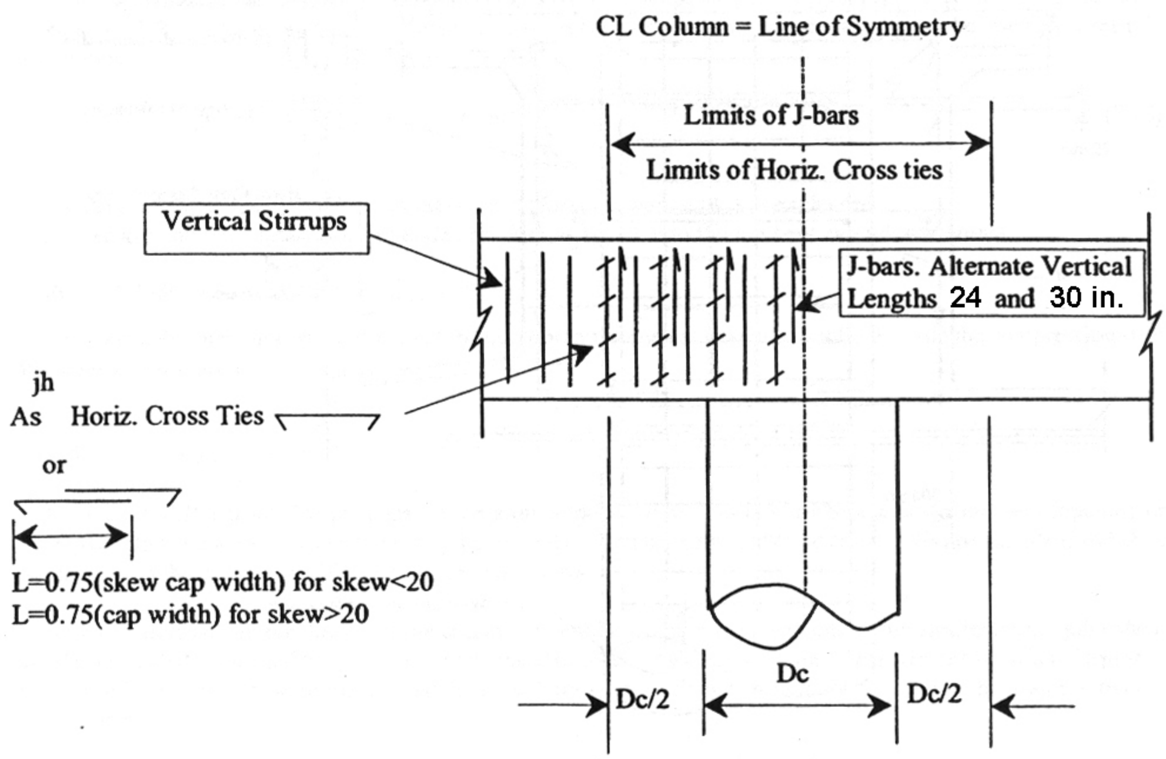


FIGURE 8.10 Location of Horizontal Joint Shear Steel

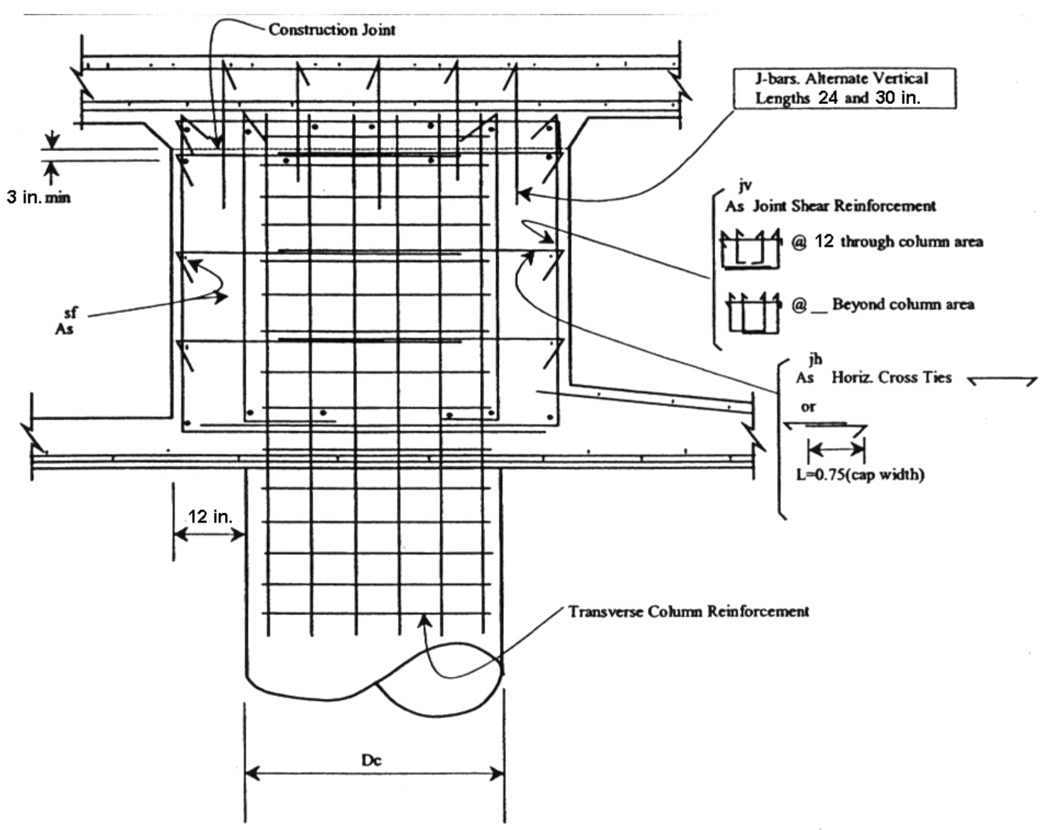


FIGURE 8.11 Additional Joint Shear Steel For Skewed Bridges

8.14 COLUMN FLARES SDC C & D

8.14.1 Horizontally Isolated Flares

The preferred method for detailing flares is to horizontally isolate the top of flared sections from the soffit of the cap beam. Isolating the flare allows the flexural hinge to form at the top of the column, thus minimizing the seismic shear demand on the column. The added mass and stiffness of the isolated flare typically can be ignored in the dynamic analysis.

A horizontal gap isolating the flare from the cap beam shall extend over the entire cross section of the flare excluding a core region equivalent to the prismatic column cross section. For SDC C, a minimum gap thickness of 2 inches shall be used.

For SDC D the gap shall be large enough so that it will not close during a seismic event. The gap thickness, *G* shall be the largest of:

a) 1.5 times the calculated plastic rotation demand from the pushover analysis times the distance from the center of the column to the extreme edge of the flare,

b) a minimum gap thickness of 2 inches.

8.14.2 Integral Column Flares

Column Flares that are integrally connected to the bent cap soffit should be avoided whenever possible. Lightly reinforced integral flares shall only be used when required for service load design or aesthetic considerations and are permitted for Seismic Design Categories A & B. The flare geometry shall be kept as slender as possible. Test results have shown that slender lightly reinforced flares perform adequately after cracking has developed in the flare concrete essentially separating the flare from the confined column core. However, integral flares require higher shear forces and moments to form the plastic hinge at the top of the column compared to isolated flares. The higher plastic hinging forces must be considered in the design of the column, superstructure and footing.

8.14.3 Flare Reinforcement

Column flares shall be nominally reinforced outside the confined column core to prevent the flare concrete from completely separating from the column at high ductility levels. The reinforcement ratio for the transverse reinforcement, outside of the column core, that confines the flared region ρ_{fs} shall be 0.45% for the upper third of the flare and 0.075% for the bottom two-thirds of the flare. The minimum longitudinal reinforcement within the flare shall be equivalent to #5 bars @ 12 inch spacing.

8.15 COLUMN SHEAR KEY DESIGN SDC C & D

Column shear keys shall be designed for the axial and shear forces associated with the column's overstrength moment M_o including the effects of overturning. The key reinforcement shall be located as close to the center of the column as possible to minimize developing a force couple within the key reinforcement. Steel pipe sections may be used in lieu of reinforcing steel to relieve congestion and reduce the moment generated within the key. Any appreciable moment generated by the key reinforcing steel should be considered in applying capacity design principles.

8.16 CONCRETE PILES

8.16.1 Transverse Reinforcement Requirements

For SDC C or D where piles are not designed as capacity protected members (i.e., piles, pile shafts, pile extensions where plastic hinging is allowed in soft soil E or F, liquefaction case), the upper end of every pile shall be reinforced and confined as a potential plastic hinge region as specified in Article 4.11. The shear reinforcement requirements specified in Article 8.6 shall apply. If an analysis of the bridge and pile system indicates that a plastic hinge can form at a lower level, the plastic hinge region shall extend 3*D* below the point of maximum moment, and the requirements mentioned above shall apply.

8.16.2 Cast-in-Place and Precast Concrete Piles

For cast-in-place and precast concrete piles, longitudinal steel shall be provided for the full length of the pile. In the upper two-thirds of the pile, the longitudinal steel ratio, provided by not less than four bars, shall not be less than 0.007**.** For special cases where a permanent casing is used, the extent of longitudinal reinforcement can be reduced to only the upper portion of the pile required to develop ultimate tension and compression forces.

References

Aschheim, Mark, and Jack P. Moehle, *Shear Strength and Deformability of RC Bridge Columns Subjected to Inelastic Cyclic Displacements,* Berkeley, Calif.: Earthquake Engineering Research Center, University of California. March 1992. (UCB/EERC 92/04).

Berry, Michael and Eberhand, Marc, Pacific Earthquake Engineering Research Center (PEER), "Estimating Flexural Damage in Reinforced Concrete Columns," University of California, Berkeley, August 2003.

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