PREFACE
to
SCDOT – SEISMIC DESIGN SPECIFICATION
for
HIGHWAY BRIDGES

October 2001

Division I-A Seismic Design of the AASHTO Standard Specifications for Highway Bridges has not been revised to incorporate recent developments in defining seismic ground motion hazard maps, site response and bridge performance levels. Recognizing the availability of improved seismic design methodologies and the high seismicity in South Carolina, the South Carolina Department of Transportation has developed the SCDOT – Seismic Design Specification for Highway Bridges. Some of the new methodologies that have been incorporated into the new specification for the SCDOT have also been incorporated into the "Recommended LRFD Guidelines for Seismic Design of Highway Bridges" (NCHRP Project 12-49), and the Caltrans "Seismic Design Criteria, July 1999. The revisions incorporate a new generation of probabilistic ground motion hazard maps produced by the U.S. Geological Survey under the National Earthquake Hazard Reduction Program (NEHRP), which provide uniform hazard spectra for the large earthquake. Basically, the revised standards specify that the design of new bridges in South Carolina directly account for the effects of the large earthquake as done by the State of California. Additionally, performance levels have been increased for critical lifeline bridges. The performance levels are established by the expected post earthquake usage of the bridge. Using various combinations of seismic hazard levels and performance levels the post-earthquake condition of the bridges are established in accordance with their importance and expected post-earthquake usage. Several earthquakes have occurred internationally, which have caused serious damage to bridges. The lessons learned from these recent earthquakes have resulted in some new enhancements in the Seismic Design of bridges. Many of these enhancements have been incorporated into these standards and reflect the latest trends in seismic bridge design practice.

As Amended by

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SECTION 1

INTRODUCTION

1.1 PURPOSE AND PHILOSOPHY

This manual establishes design and construction provisions for bridges in South Carolina to minimize their susceptibility to damage from earthquakes. This manual is intended to be used in conjunction with Division I, Design of the *AASHTO Standard Specifications for Highway Bridges, sixteenth Edition, 1996* and as a replacement to Division I-A, Seismic Design, of the same specifications. Except for those portions replaced by this manual, the Commentary to AASHTO Division I-A, Seismic Design still applies and is specifically referenced in this manual.

The principles used for the development of the provisions are:

1. Small to moderate earthquakes should be resisted within the essentially elastic range of the structural components without significant damage. The Functional Evaluation Earthquake (FEE) defined in Section 3.4.2 is adopted to represent seismic ground motion level produced by small to moderate earthquakes.

2. State of the Practice seismic ground motion intensities and forces are used in the design procedures.

3. Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge. Where possible, damage that does occur should be readily detectible and accessible for inspection and repair unless prohibited by the structural configuration. The Safety Evaluation Earthquake (SEE) defined in Section 3.4.3 is adopted to represent seismic ground motion level produced by large earthquakes.

The seismic hazard varies form very small to high across the State of South Carolina. Therefore, for purposes of design, four Seismic Performance Categories (SPC) are defined on the basis of the spectral acceleration $S_{D1\text{-}SEE}$ for the site defined in Section 3.4.3 and the Importance Classification (IC). Different degrees of complexity and sophistication of seismic analysis and design are specified for each of the four Seismic Performance Categories.

1.2 BACKGROUND

The South Carolina Department of Transportation (SCDOT) has initiated the development and implementation of a bridge seismic design and retrofit program. A central feature of the new SCDOT bridge design and retrofit program is the development of new seismic bridge design criteria and standards that incorporate new generation U.S. Geological Survey seismic ground shaking hazard maps and treat certain inadequacies of existing bridge design codes to adequately address the large earthquake and associated bridge collapse and life safety issues in the central and eastern United States. This manual presents the upgraded bridge seismic design provisions and describes variations in national seismicity that motivated the development of the provisions. Basically, the revised standards specify that the design of new bridges in South Carolina directly account for the effects of the large earthquake as done by the State of California. This is to ensure conformance with the guiding principle used in the development of AASHTO provisions that the "exposure to shaking from the large earthquake should not cause collapse of all or part of the bridge." Many of the revisions were adopted from bridge design provisions of the California Department of Transportation (Caltrans), because of similar high intensity seismic hazard at the SEE level and the state of the practice progress gained due to recent earthquakes.

1.3 UPGRADED SEISMIC DESIGN REQUIREMENT

At least two developments of the U.S. Geological Survey during the past several years have been a major contribution to bridge earthquake engineering. One development was an assessment of the nature of the seismic ground shaking hazard as it varies nationally that revealed apparent inequalities in safety result when a single level of probability common to bridge code design is used. The second development was a new generation of probabilistic ground-motion hazard maps that provide uniform hazard spectra for exposure times of 50 to 250 years and make possible the treatment of the inequality in safety of bridge code design using existing earthquake engineering design and evaluation provisions and methodology.

The new generation of probabilistic ground motion maps were produced by the USGS under the National Earthquake Hazard Reduction Program (NEHRP) with significant input from the committee on Seismic Hazard Maps of the Building Seismic Safety Council (BSSC) and the Structural Engineers Association of California (SEAOC). They allow development of uniform hazard spectra and permit direct definition of the design spectra by mapping the response spectral ordinates at different periods.

This manual presents such upgraded technical standards that have been developed incorporating the new generation ground motion maps.

The recommended seismic design procedures were developed to meet current bridge code objectives, including both serviceability and life safety in the event of the large earthquake. The primary function of these provisions is to provide minimum standards for use in bridge design to maintain public safety in the extreme earthquake likely to occur within the state of South Carolina. They are intended to safeguard against major failures and loss of life, to minimize damage, maintain functions, or to provide for easy repair.

For normal or essential bridges defined in Section 3.2.1, the **Single Level Design Method** is adopted by this code. This method consists of applying seismic design loading calculated based upon the value of the spectral accelerations of the 2%/50-year earthquake (i.e.,, the Safety Evaluation Earthquake defined in Section 3.4.3).

For critical bridges defined in Section 3.2.1, and with the approval of the SCDOT, the seismic performance goals are to be achieved by a two-level design approach (i.e., a direct design for each of the two earthquakes). **(Two-Level Design Method).** In addition to the 2%/50-year earthquake (Safety Evaluation Earthquake), critical bridges shall also be designed to provide adequate functionality after the 10%/50-year earthquake (Functional Evaluation Earthquake). The minimum performance levels for the design and evaluation of bridges shall be in accordance with the level of service and damage defined in Section 3.2.1 for the two design earthquakes. The SCDOT may specify project-specific or structure-specific performance requirements different from those defined in Section 3.2.1. For example, for a Critical or Essential bridge it may be desirable to have serviceability following a 2%/50-year earthquake. The SCDOT may require a site specific design spectrum as part of the analysis.

1.4 PROJECT ORGANIZATION

The edition of the SCDOT Seismic Design Specifications for Highway Bridges is an enhancement of the sixteenth edition of the AASHTO Standard Specifications for Highway Bridges, Division I-A. The enhancements include selected portions of the Caltrans Seismic Design Criteria, July 1999 and other enhancements that reflect the most recent developments in seismic design of bridges. This edition of the SCDOT Seismic Design Specifications was completed as part of Phase 1 of a project to enhance the seismic design criteria for bridges in South Carolina. This work was completed by Imbsen & Associates, Inc. (IAI).

1.5 DOCUMENT RESOURCES

This document is an enhancement of Division I-A, Seismic Design of the AASHTO Standard Specifications for Highway Bridges, Sixteenth Edition, 1996. Many of the enhancements are taken from the California Department of Transportation (Caltrans) document entitled “Seismic Design Criteria” Version 1.1 dated July 1999.

1.6 FLOW CHARTS

Flow charts outlining the steps in the seismic design procedures implicit in these specifications are given in Figures 1.6A to 1.6E.

The flow chart in Figure 1.6A 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 Performance Category A versus a bridge in Seismic Performance Category B, C, or D.

The flow chart in Figure 1.6B outlines the design procedure for bridges in SPC B, C, and D. Since the displacement approach is the main thrust of this criteria, the flow chart in Figure 1.6B directs the designer to Figure 1.6D in order to establish the displacement demands on the subject bridge and Figure 1.6E in order to establish member ductility requirements based on the type of the structure chosen for seismic resistance.

1.7 REFERENCES

AASHTO (1998), *LRFD Bridge Design Specifications*, 2nd Ed., American Association of State Highway and Transportation Officials, Washington, DC.

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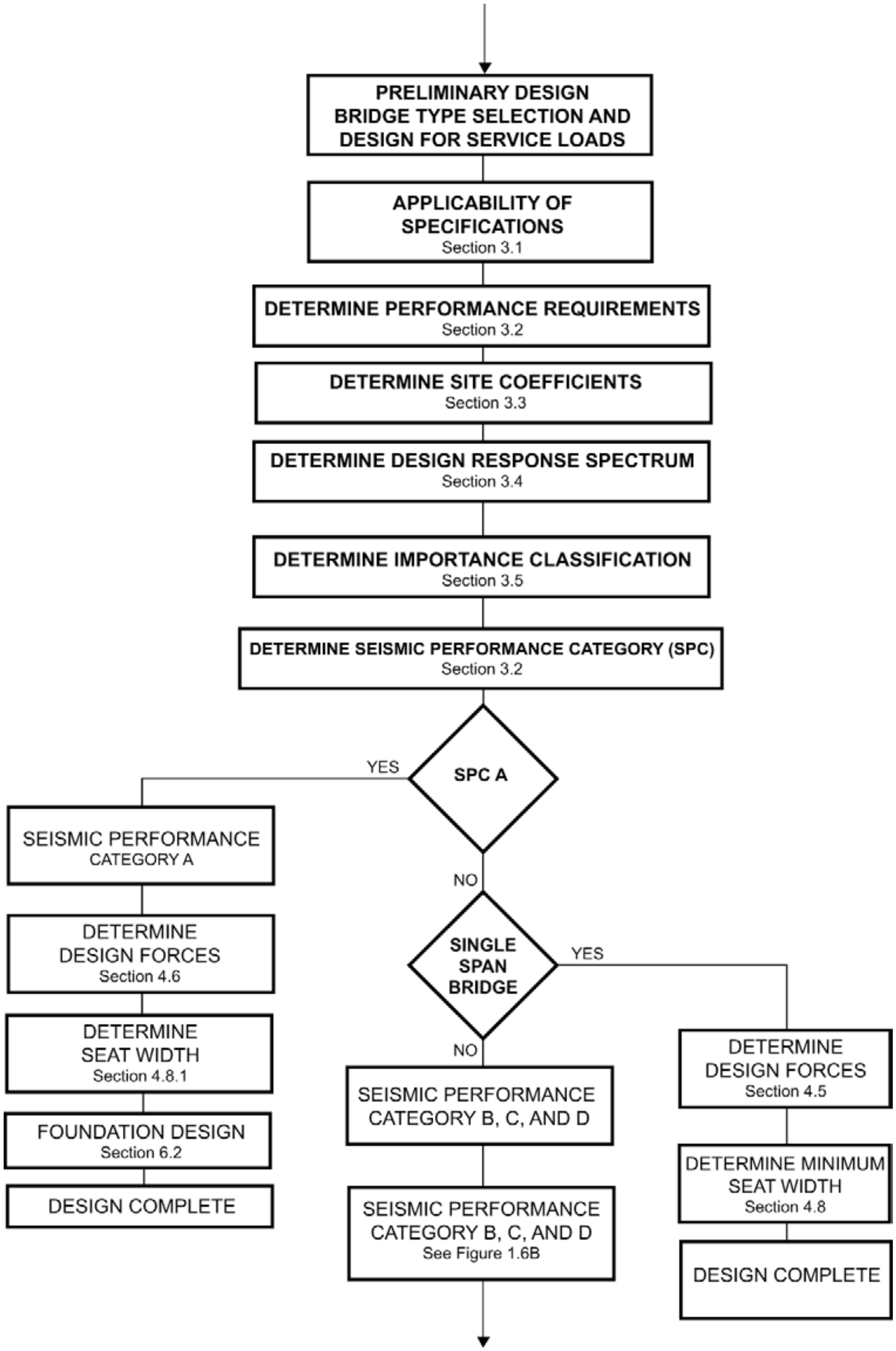


FIGURE 1.6A Design Procedure Flow Chart

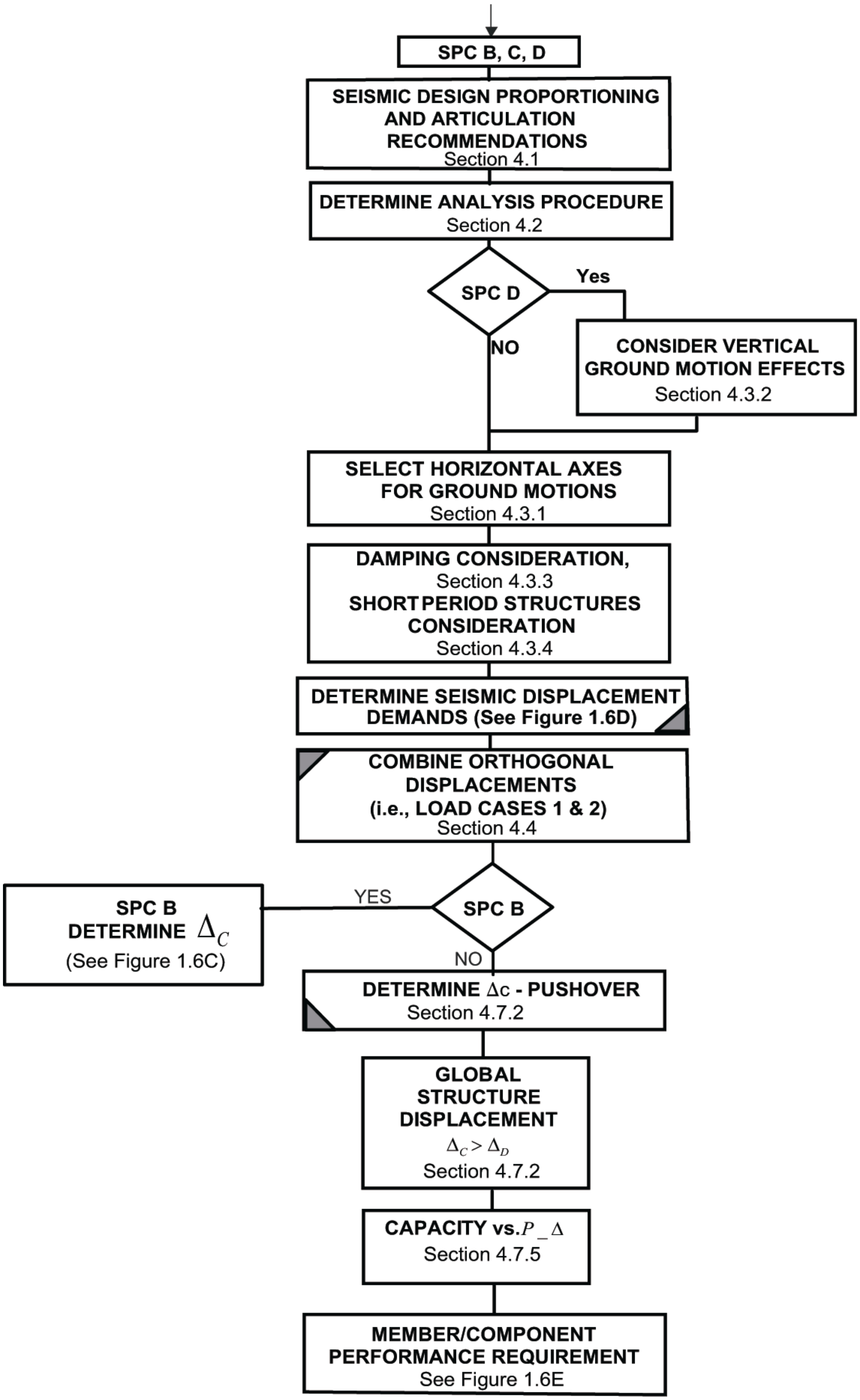


FIGURE 1.6B Sub Flow Chart for Seismic Performance Categories B, C, and D

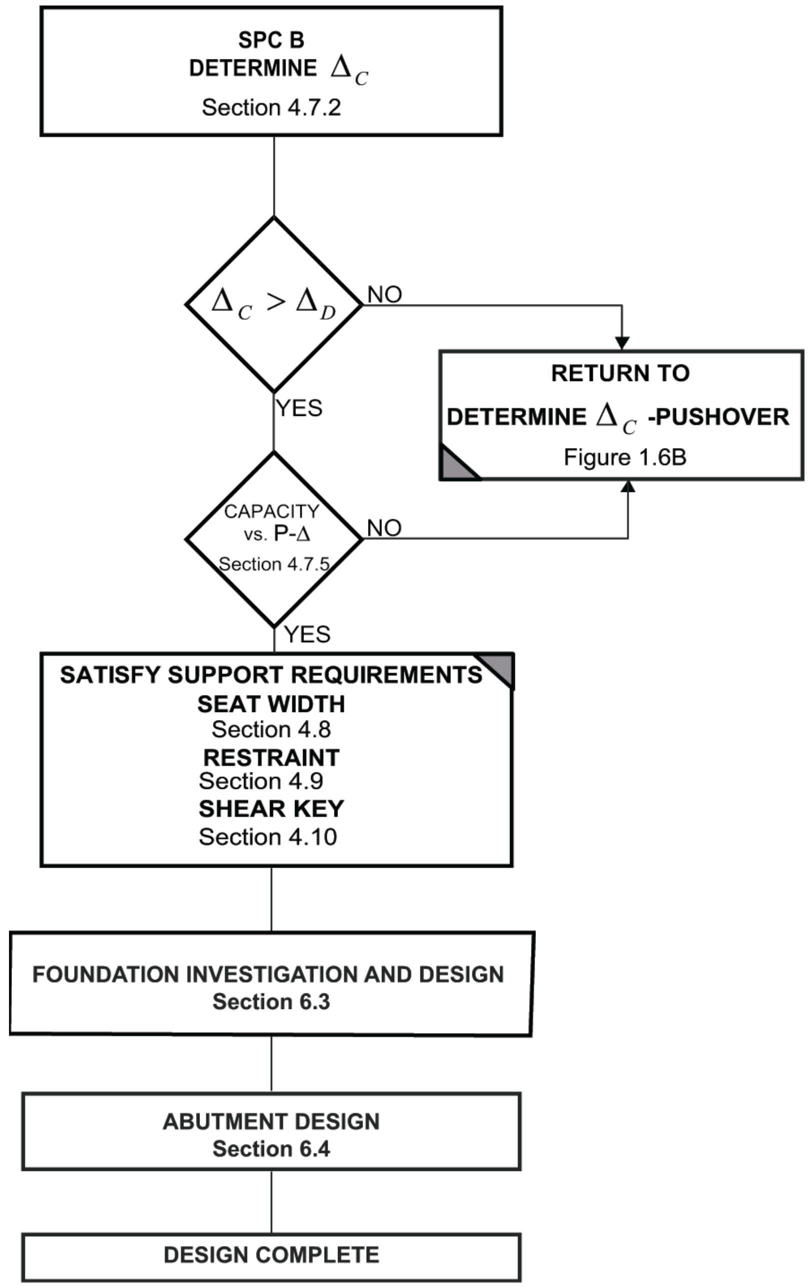


FIGURE 1.6C Sub Flow Chart for Seismic Performance Categories B, C and D (continued)

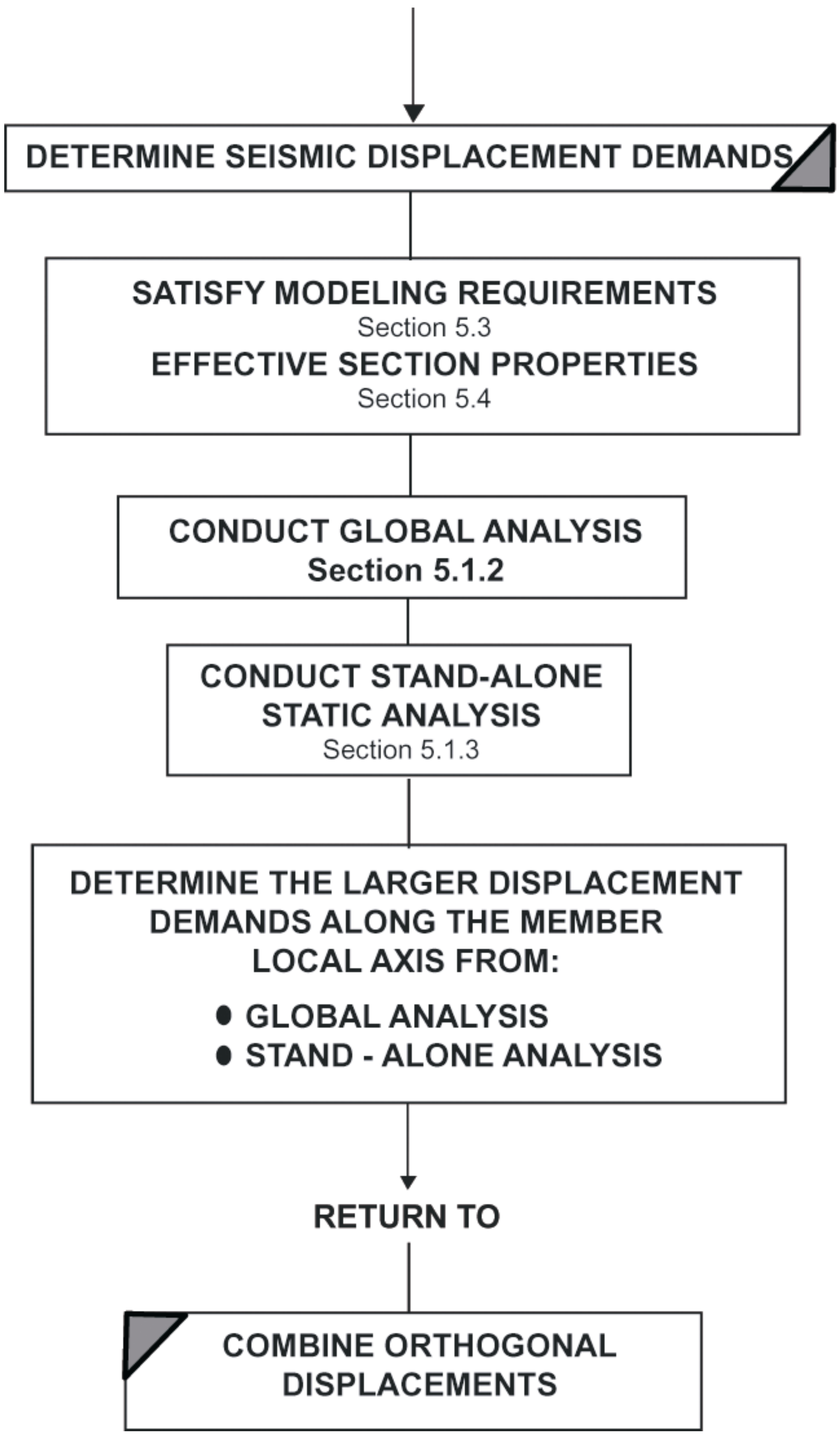


FIGURE 1.6D Displacement Demands Flow Chart

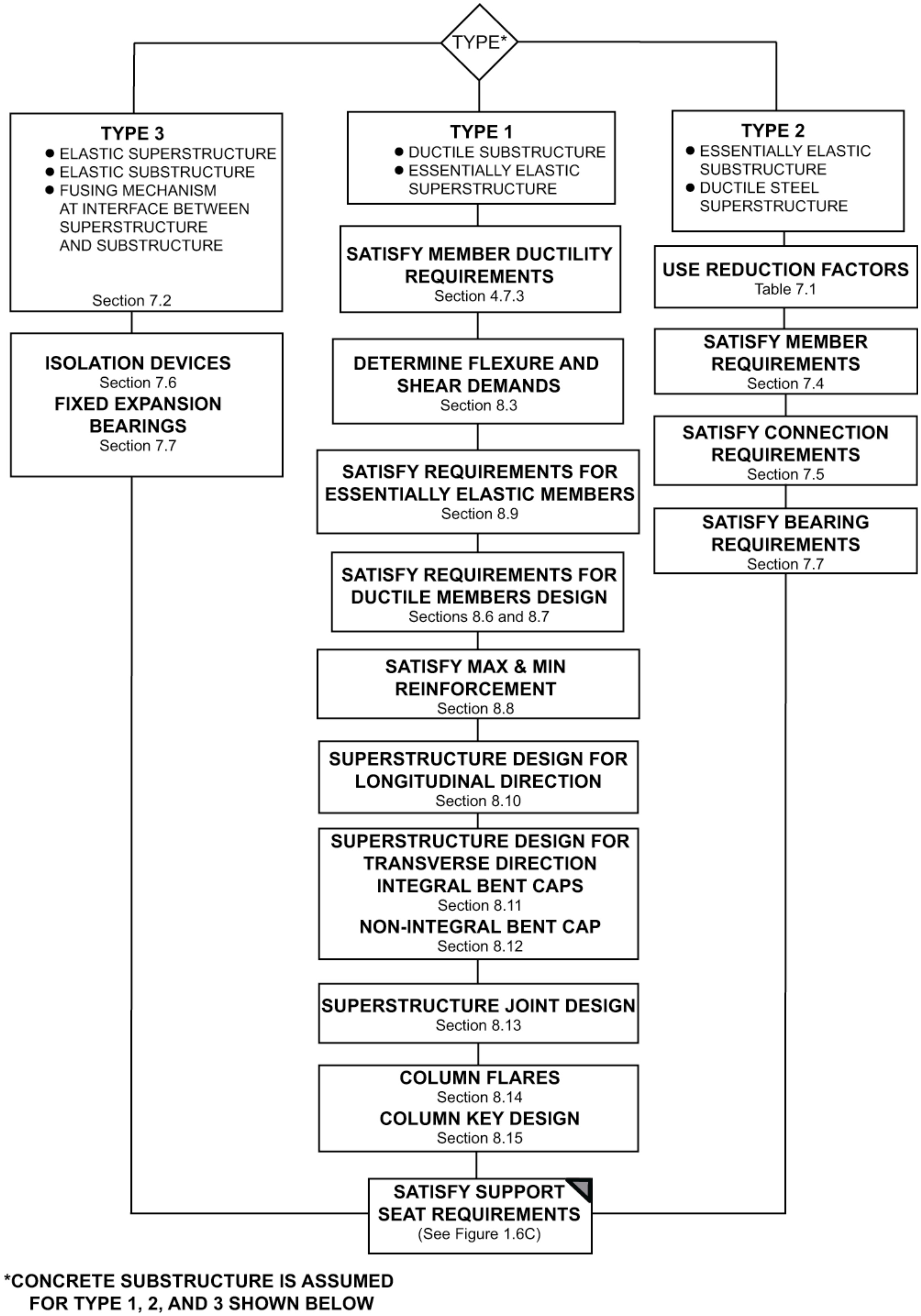


FIGURE 1.6E Member/Component Ductility Requirement Flow Chart

SECTION 2

SYMBOLS AND DEFINITIONS

2.1 NOTATIONS

The following symbols and definitions apply to these Standards:

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_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_{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 a hoop or spiral bar (in^2)

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)

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

D' = Core diameter of a column (in)

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)

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

D_{ftg} = Footing depth (in)

D_s = Superstructure depth (in)

E_cI = Flexural rigidity (kips-in^2)

E_s = Steel elastic modulus (ksi)

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 parameter S_1

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

G_cJ = Torsional rigidity

H_h = Average height used to calculate minimum support length N (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)

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

L_l = Length of bridge deck defined in Article 4.8 (ft)

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_{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_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)

$M_{P(y),(x)}^{col}$ = The component of the column plastic moment capacity about the X or Y axis

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

N = Minimum support length (in)

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)

P_n = Nominal axial strength of a member (kip)

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

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

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

R = Force reduction factor is obtained by dividing th 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

S = Site coefficient specified in Article 3.5.1 (dimensionless)

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

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

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

T_j = Natural 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_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_{po} = Overstrength plastic shear demand (kip)

V_s = Transverse steel shear contribution (kip)

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)

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

f'_{ce} = Expected compressive strength of concrete

f_h = Horizontal effective compressive stress in a joint (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_c = Principal compressive stress (psi)

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)

t = Thickness of a gusset plate (in)

v_c = Concrete shear stress (psi)

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

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_{psu} = Ultimate prestress steel strain

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

ε_{su} = Ultimate strain of steel reinforcement

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

Δ_b = 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 generated by seismic design applied to the structural system

Δ_{Di} = The larger earthquake displacement demand for each frame calculated by the global analysis for the 2% in 50 years safety evaluation

Δ_{eq} = Design displacement at the expansion joint due to earthquake

Δ_f = Foundation flexibility

Δ_p = Plastic displacement

$\Delta_{p/s}$ = 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_S = The pile shaft displacement at the point of maximum moment

Δ_{temp} = Displacement due to temperature variation

Δ_y = Elastic displacement

ϕ = Shear strength reduction factor (dimensionless)

ϕ_b = Resilience 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_y = Yield Curvature

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

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

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_p = Limiting width-thickness ratio for ductile component

λ_r = Limiting width-thickness ratio for essentially elastic component

μ_D = Local member ductility demand

SECTION 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 (150 m). For other types of construction (suspension bridges, cable-stayed bridges, arch type and movable bridges) and spans exceeding 500 ft, the SCDOT 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 essential or critical bridges. Those provisions are site/project specific and are tailored based on structure type.

No detailed seismic analysis is required for any single span bridge or for any bridge in Seismic Performance Category A. For both single span bridges and bridges classified as SPC A the connections must be designed for specified forces, in Section 4.5 and Section 4.6 respectively, and must also meet minimum support length requirements of Section 4.8.

3.2 PERFORMANCE CRITERIA

Unless specified by SCDOT on a project specific or structure specific basis, all bridges shall meet the seismic performance criteria given in Table 3.2.1. Definitions of the terms in Table 3.2.1 are given in Sections 3.2.1 through 3.2.3. The SCDOT will classify all bridges on the State Highway System.

3.2.1 Bridge Category

Bridge structures on the state highway system have been classified as “normal bridges”, “essential bridges” or “critical bridges”. 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) time for restoration of functionality after closure creates a major economic impact, and (3) the bridge is formally designated as critical by a local emergency plan.

Each bridge shall be classified as either Critical, Essential or Normal as follows:

(a) Critical bridges: Bridges that must be open to all traffic once inspected after the safety evaluation design earthquake and be usable by emergency vehicles and for security/defense purposes immediately after the safety evaluation design earthquake, i.e., a 2,500-year return period event.

(b) Essential bridges: Bridges that should, as a minimum, be open to emergency vehicles and for security/defense purposes after the safety evaluation design earthquake, i.e., a 2,500-year return period event and open to all traffic within days after the SEE event.

(c) Normal Bridges: Any bridge not classified as a Critical or Essential Bridge.

TABLE 3.2.1 Seismic Performance Criteria

Ground Motion Level	Performance Level	Normal Bridges	Essential Bridges	Critical Bridges
Functional-Evaluation	Service	NR*	NR	Immediate
	Damage	NR	NR	Minimal
Safety-Evaluation	Service	Impaired	Recoverable	Maintained
	Damage	Significant	Repairable	Repairable

*Functional Evaluation Not Required.

3.2.2 Ground Motion Levels

The following two ground motion levels are used for the hazard:

Functional Evaluation Earthquake (FEE). The ground shaking having a 10% probability of exceedance in 50 years (10%/50 year). The FEE earthquake is similar to the design earthquake specified in the 1996 AASHTO (with interims through 2000) seismic design provisions. It has a return period of 474 years. The FEE spectra are used for the functional evaluation of critical bridges only. The FEE spectra shall be determined according to Section 3.4.2. The mapped values are to be adjusted for Site Class effects using the site coefficients of Section 3.3.3.

Safety Evaluation Earthquake (SEE). The ground shaking having a 2% probability of exceedance in 50 years (2%/50 year). The SEE earthquake has a return period of 2,500 years. It represents the rare, but possible, large earthquake. The SEE spectra are used for safety evaluation of all bridges. The SEE spectra shall be determined in accordance with Section 3.4.3. The mapped values are to be adjusted for Site Class effects using the site coefficients of Section 3.3.3.

3.2.3 Service Levels and Damage Levels

The following performance levels, expressed in terms of service levels and damage levels, are defined as follows:

(a) Service Levels

- *Immediate:* Full access to normal traffic is available immediately following the earthquake.
- *Maintained:* Short period of closure to Public. Immediately open to emergency traffic.
- *Recoverable:* Limited period of closure to Public.
- *Impaired:* Extended closure to Public.

(b) Damage Levels

- *Minimal Damage:* No collapse, essentially elastic performance.
- *Repairable Damage:* No collapse. Concrete cracking, spalling of concrete cover, and minor yielding of structural steel will occur. However, the extent of damage should be sufficiently limited that the structure can be restored essentially to its pre-earthquake condition without replacement of reinforcement or replacement of structural members (i.e., ductility demands less than 2). Damage can be repaired with a minimum risk of losing functionality.
- *Significant Damage:* Although there is minimum risk of collapse, permanent offsets may occur in elements other than foundations. Damage consisting of concrete cracking, reinforcement yielding, major spalling of concrete, and deformations in minor bridge components may require closure to repair. Partial or complete demolition and replacement may be required in some cases.

3.3 SEISMIC GROUND SHAKING HAZARD

The ground shaking hazard prescribed in these Specifications is defined in terms of ground motion accelerations, represented by response spectra and coefficients derived from these spectra. They shall be determined in accordance with the general procedure of Section 3.4.4 or the site-specific procedure of Section 3.4.5.

In the general procedure, the spectral response parameters are defined using the 1996 seismic hazard maps produced by the U.S. Geological Survey depicting probabilistic ground motion and spectral response with 10% and 2% probabilities of exceedance in 50 years. The site-specific procedure of Section 3.4.5 shall be used for bridges on sites classified as Site Class F in accordance with Section 3.3.2, or as specified by SCDOT on a project-specific or structure-specific basis.

3.3.1 Seismic Loading

The design earthquake ground motions are represented by the spectral response acceleration at short period (i.e. the 0.2-second period), S_s, and at 1-second period, S_1 obtained directly from the U.S. Geological Survey maps. These maps (for Site Class B: Rock Site) are presented in Sections 3.4.2 and 3.4.3 for the Functional Evaluation Earthquake (Figures 3.4.2A and 3.4.2B) and Safety Evaluation Earthquake (Figures 3.4.3A and 3.4.3B), respectively. In using the maps, the parameters S_s and S_1 shall be obtained by interpolating between the values shown on the response acceleration contour lines on either side of the site, on the appropriate map, or by using the value shown on the map for the higher contour adjacent to the site.

3.3.2 Site Class Definitions

The site shall be classified as one of the site classes defined in Table 3.3.2A. Where the soil shear wave velocity, $\overline{v}_s$, is not known, site class shall be determined, as permitted in Table 3.3.2A, from standard penetration resistance, $\overline{N}$ or $\overline{N}_{ch}$, or from soil undrained shear strength, $\overline{s}_u$. When the soil properties are not known in sufficient detail to determine the Site Class, Site Class D shall be used. Site Classes E or F need not be assumed unless the SCDOT determines that Types E or F may be present at the site or in the event that Types E or F are established by geotechnical data.

The steps for classifying a site shall be as follows:

Step 1: Check for the four categories of Site Class F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Soil Profile Type F and conduct a site-specific evaluation.

Step 2: Check for the existence of a total thickness of soft clay > 10 ft (3 m) where a soft clay layer is defined by: $\overline{s}_u$ < 500 psf (25 kPa), w ≥ 40 percent, and PI > 20. If these criteria are satisfied, classify the site as Site Class E. The plasticity index, PI, is determined according to ASTM D4318-93. Moisture content, w, is determined according to ASTM D2216-92.

TABLE 3.3.2A Site Class Classification

SOIL CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 FT (30 M)		
		SOIL SHEAR WAVE VELOCITY $\overline{v}_s$	STANDARD PENERTRATION RESISTANCE $\overline{N}$ or $\overline{N}_{ch}$	UNDRAINED SHEAR STRENGTH $\overline{s}_u$
A	Hard Rock	>5,000 ft/sec (>1500 m/s)	Not applicable	Not applicable
B	Rock	2,500 to 5,000 ft/sec (760 to 1500 m/s)	Not applicable	Not applicable
C	Very dense soil and soft rock	1,200 to 2,500 ft/sec (360 to 760 m/s)	>50	≥ 2,000 psf (≥ 100kPa)
D	Stiff soil	600 to 1,200 ft/sec (180 to 360 m/s)	15 to 50	1,000 to 2,000 psf (50 to 100kPa)
E	Soft soil	< 600 fps (< 180 m/s)	<15	< 1,000 psf (<50 kPa)
E		Any profile with more than 10 ft (3m) of soft clay defined with: PI(1)>20; w(2) ≥ 40 percent; and $\overline{s}_u$ < 500 psf (25 kPa)		
F		Any soil profile containing one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays (H>10 ft [3 m] of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays (H>25 ft [8 m] with PI(1) > 75) 4. Very thick soft/medium stiff clays (H> 120 ft [36 m])		

(1) The plasticity index, PI, is determined according to ASTM D4318-93.
(2) Moisture content, w, is determined according to ASTM D2216-92.

Step 3: 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 below.

Method 1: $\bar{v}_s$ for the top 100 ft (30 m) ($\bar{v}_s$ method)

$\bar{v}_s$ is the generalized shear wave velocity for the upper 100 feet of the soil profile defined as

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \qquad (3\text{-}1)$$

where

$\sum_{i=1}^{n} d_i = d_s$ is equal to 100 ft (30 meters)

v_{si} is the shear wave velocity of layer i in feet per second (meters per second)

d_i is the thickness of any layer i between 0 and 100 feet (30 meters)

Method 2: $\bar{N}$ for the top 100 ft (30 m) ($\bar{N}$ method)

$\bar{N}$ is the generalized standard penetration resistance of all soils in the upper 100 feet (30 meters) of the soil profile defined as

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad (3\text{-}2)$$

Where $\sum_{i=1}^{n} d_i$ is equal to 100 feet (30 meters)

N_i is the standard penetration resistance of layer i (ASTM D1586-84), not to exceed 100 blows/ft, as directly measured in the field without corrections.

Method 3: $\bar{N}_{ch}$ for cohesionless soil layers (PI<20) in the top 100 ft (30 m) and average, $\bar{s}_u$, for cohesive soil layers (PI>20) in the top 100 ft (30 m) ($\bar{s}_u$ method)

$\bar{N}_{ch}$ is the generalized standard penetration resistance for only the cohesionless soil layers of the upper 100 feet of the soil profile defined as

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad (3\text{-}3)$$

where

d_s is the total thickness of cohesionless soil layers in the top 100 feet (30 meters).

$\sum_{i=1}^{n} \frac{d_i}{N_i}$ includes cohesionless soil layers only when calculating N_{ch}

$\bar{s}_u$ is the generalized undrained shear strength for only the cohesive soil layers of the upper 100 feet of the soil profile defined as

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{n} \frac{d_i}{s_{ui}}} \qquad (3\text{-}4)$$

where

d_c is the total thickness (100-d_s) of cohesive soil layers in the top 100 feet (30 meters).

s_{ui} is the undrained shear strength in psf (kPa), not to exceed 5,000 psf (250 kPa), as determined by ASTM D2166-91 or D2850-87.

$\sum_{i=1}^{n} \frac{d_i}{s_{ui}}$ includes cohesive soil layers only

If Method 3 is used (i.e., the $\bar{s}_u$ method) and the $\bar{N}_{ch}$ and $\bar{s}_u$ criteria differ, select the category with the softer soils (for example, use Site Class D instead of C).

The shear wave velocity for rock, Site Class B, shall be either measured on site or estimated by a geotechnical engineer or engineering geologist/ seismologist for competent rock with moderate fracturing and weathering.

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 amount of weathering and fracturing.

The rock categories, Site Classes A and B, shall not be used if there is more than 10 ft (3 m) of soil between the rock surfaces and the bottom of the spread footing or mat foundation.

3.3.3 Site Coefficients and Adjusted Spectral Response Acceleration Parameters

The design earthquake spectral response acceleration for short periods (S_S) and at 1 second (S_l), shall be adjusted for site class effects using Equation 3-5 and Equation 3-6, respectively:

$$S_{DS} = F_a S_S \qquad (3\text{-}5)$$

and

$$S_{D1} = F_v S_l \qquad (3\text{-}6)$$

where:

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

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

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_v = site coefficient defined in Table 3.3.3B, based on the site class and the values of the response acceleration parameter S_l

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_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)

TABLE 3.3.3A. Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Response Acceleration S_S

Site Class	Design Spectral Acceleration at Short Periods				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
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	a
F	a	a	A	a	a

Note: Use straight line interpolation for intermediate values of S_S. [a]Site specific geotechnical investigation and dynamic site response analysis shall be performed.

TABLE 3.3.3B. Values of Site-Coefficient Fv as Function of Site Class and Mapped Spectral Response Acceleration at One-Second Period, S_l

Site Class	Mapped Spectral Response Acceleration at One-Second Period				
	$S_l \le 0.1$	$S_l = 0.2$	$S_l = 0.3$	$S_l = 0.4$	$S_l \ge 0.5$
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	a
F	a	a	A	a	a

Note: Use straight line interpolation for intermediate values of S_l. [a]Site specific geotechnical investigation and dynamic site response analysis shall be performed.

3.4 DESIGN SPECTRAL RESPONSE ACCELERATION PARAMETERS

3.4.1 General

The bridge design provisions of these Specifications require safety and/or functional evaluations as described in Section 3.2. The evaluations are to be conducted using a two-level approach for Critical Bridges or, in the case of Essential and Normal Bridges, a one-level design approach. A safety evaluation shall be conducted for all bridges. A functional evaluation is required for all Critical Bridges. These requirements involve two different sets of design response spectra as described in Sections 3.4.2 and 3.4.3.

3.4.2 Design Spectral Response Acceleration Parameters – FEE (10%/50 years)

For the Functional Evaluation Earthquake, the five percent (5%) damped design spectral response acceleration at the short period, $S_{DS\text{-}FEE}$, and at 1 second period, $S_{D1\text{-}FEE}$, shall be determined from Equations 3-7 and 3-8, respectively.

$$S_{DS\text{-}FEE} = F_a S_{S\text{-FEE}} \quad (3\text{-}7)$$
$$S_{D1\text{-}FEE} = F_v S_{1\text{-FEE}} \quad (3\text{-}8)$$

where:

$S_{S\text{-FEE}}$ = the mapped spectral acceleration at short period determined from Figure 3.4.2A (with 10% probability of exceedance in 50 years).

$S_{1\text{-FEE}}$ = the mapped spectral acceleration at 1 second period from Figure 3.4.2B (with 10% probability of exceedance in 50 years).

F_a = site coefficient defined in Table 3.3.3A.

F_V = site coefficient defined in Table 3.3.3B.

3.4.3 Design Spectral Response Acceleration Parameters – SEE (2%/50 years)

For the Safety Evaluation Earthquake, the five percent (5%) damped design spectral response acceleration at short periods, $S_{DS\text{-SEE}}$, and at 1 second period, $S_{D1\text{-SEE}}$, shall be determined from Equations 3-9 and 3-10, respectively.

$$S_{DS\text{-}SEE} = F_a S_{S\text{-}SEE} \quad (3\text{-}9)$$
$$S_{D1\text{-}SEE} = F_v S_{1\text{-}SEE} \quad (3\text{-}10)$$

where:

$S_{S\text{-SEE}}$ = the mapped spectral acceleration at short periods determined from Figure 3.4.3A (with 2% probability of exceedance in 50 years).

$S_{1\text{-SEE}}$ = the mapped spectral acceleration at 1 second period determined from Figure 3.4.3B (with 2% probability of exceedance in 50 years).

F_a = site coefficient defined in Table 3.3.3A.

F_V = site coefficient defined in Table 3.3.3B.

3.4.4 General Procedure Response Spectrum.

Where a design response spectrum is required by these Specifications and site specific procedures are not used, the general design response spectrum curve shall be developed as indicated in Figure 3.4.1 and as follows.

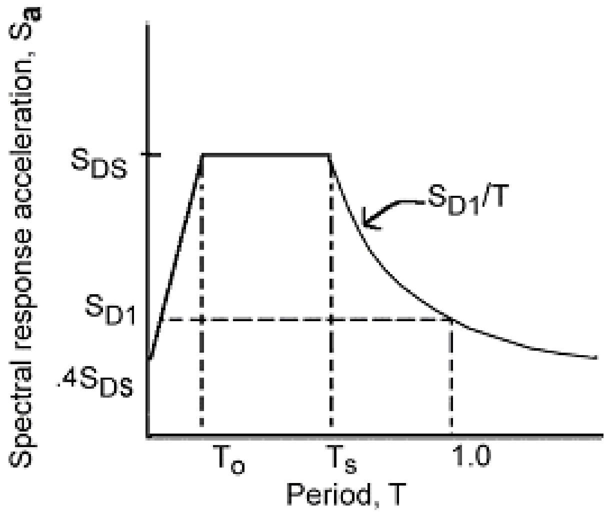


FIGURE 3.4.1 Design Response Spectrum

1. For periods less than or equal to T_o, the design spectral response acceleration S_a shall be given by Equation 3-11.

2. For periods greater than or equal to T_o and less than or equal to T_s, the design spectral response acceleration, S_a, shall be taken equal to S_{DS}.

3. For periods greater than T_s the design spectral response acceleration, S_a, shall be given by Equation 3-12.

$$S_a = 0.6 \frac{S_{DS}}{T_o} T + 0.4 S_{DS} \quad (3\text{-}11)$$

$$S_a = \frac{S_{D1}}{T} \quad (3\text{-}12)$$

where:

S_{DS} = the design response spectral acceleration at short periods (equal to $S_{DS\text{-}FEE}$ as determined in Section 3.4.2 for the Functional Evaluation Earthquake, or $S_{DS\text{-}SEE}$ as determined in Section 3.4.3 for the Safety Evaluation Earthquake)

S_{D1} = the design spectral response acceleration at 1 second period (equal to $S_{D1\text{-}FEE}$ as determined in Section 3.4.2 for the Functional Evaluation Earthquake, or $S_{D1\text{-}SEE}$ as determined in Section 3.4.3 for the Safety Evaluation Earthquake)

T = Fundamental period (in seconds) of the structure)

$$T_o = 0.2\left(\frac{S_{D1}}{S_{DS}}\right) \quad (3\text{-}13)$$

$$T_S = \frac{S_{D1}}{S_{DS}} \quad (3\text{-}14)$$

The design spectrums for the FEE (10%/50 Years) and the SEE (2%/50 Years) were developed using the procedure described above and are included in Figures 3.4.4 and 3.4.5 respectively. Project site specific studies are required for bridges with a fundamental period greater than 3 seconds.

3.4.5 Site-Specific Ground Shaking Hazard

Where site-specific ground shaking characterization is used as the basis of the design, the characterization shall be developed in accordance with this section.

3.4.5.1 Site-Specific Response Spectrum

Development of site-specific response spectra shall be based on the geologic, seismologic, and soil characteristics associated with the specific site. Response spectra should be developed for an equivalent viscous damping ratio of 5%. Additional spectra should be developed for other damping ratios appropriate to the indicated structural behavior. When the 5% damped site-specific spectrum has spectral amplitudes in the period range of greatest significance to the structural response that are less than 70 percent of the spectral amplitudes of the General Response Spectrum defined in accordance with Section 3.4.4, an independent third-party review of the spectrum should be made by an individual with expertise in the evaluation of ground motion.

When a site-specific response spectrum has been developed and other sections of this manual require values for the design spectral response parameters, S_{DS}, S_{D1}, or T_o, they may be obtained in accordance with this section. The value of design spectral response acceleration at short periods, S_{DS}, shall be taken as the response acceleration obtained from the site-specific spectrum at a period of 0.2 seconds, except that it should be taken as not less than 90 % of the peak response acceleration at any period. In order to obtain a value for the design spectral response acceleration parameter S_{D1}, a curve of the form $S_a = S_{D1}/T$ should be graphically overlaid on the site-specific spectrum such that at any period, the value of S_a obtained from the curve is not less than 90% of that which would be obtained directly from the spectrum. The value of T_o shall be determined in accordance with Equation 3-13.

3.4.5.2 Acceleration Time Histories

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 that shall be selected and scaled from no fewer than three recorded events. Appropriate time histories shall have magnitude, fault distances, and source mechanisms that are consistent with those that control the design earthquake ground motion. Where three appropriate recorded ground motion time history data sets are not available, appropriate simulated time history data sets may be used to make up the total number required. For each data set, the square root of the sum of the squares (SRSS) of the 5%-damped site-specific spectrum of the scaled horizontal components shall be constructed. The data sets shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5%-damped spectrum for the design earthquake for periods between 0.2T seconds and 1.5T seconds (where T is the fundamental period of the structure). Each set of time histories (i.e., all three components) shall be applied simultaneously to the model.

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.

3.5 IMPORTANCE CLASSIFICATION

An Importance Classification (IC) shall be assigned for all bridges for the purpose of determining the Seismic Performance Category (SPC) in Section 3.6 as follows:

1. Critical bridges IC=I
2. Essential bridges IC=II
3. Normal bridges IC=III

Bridges shall be classified as described in Section 3.2.

3.6 SEISMIC PERFORMANCE CATEGORIES

Each bridge shall be designed to one of four Seismic Performance Categories (SPC), A through D, based on the Importance Classification (IC) and the one-second period design spectral acceleration for the Safety Evaluation Earthquake (S_{D1-SEE}, refer to Section 3.4.3) as shown in Table 3.6

TABLE 3.6 Seismic Performance Category (SPC)

Value of S_{D1-SEE}	Importance Classification (IC)		
	I	II	III
S_{D1-SEE}<0.30g	B	B	A
0.3g≤S_{D1-SEE}<0.45g	C	C	B
0.45g≤ S_{D1-SEE}<0.6g	D	C	C
0.6g≤ S_{D1-SEE}	D	D	C

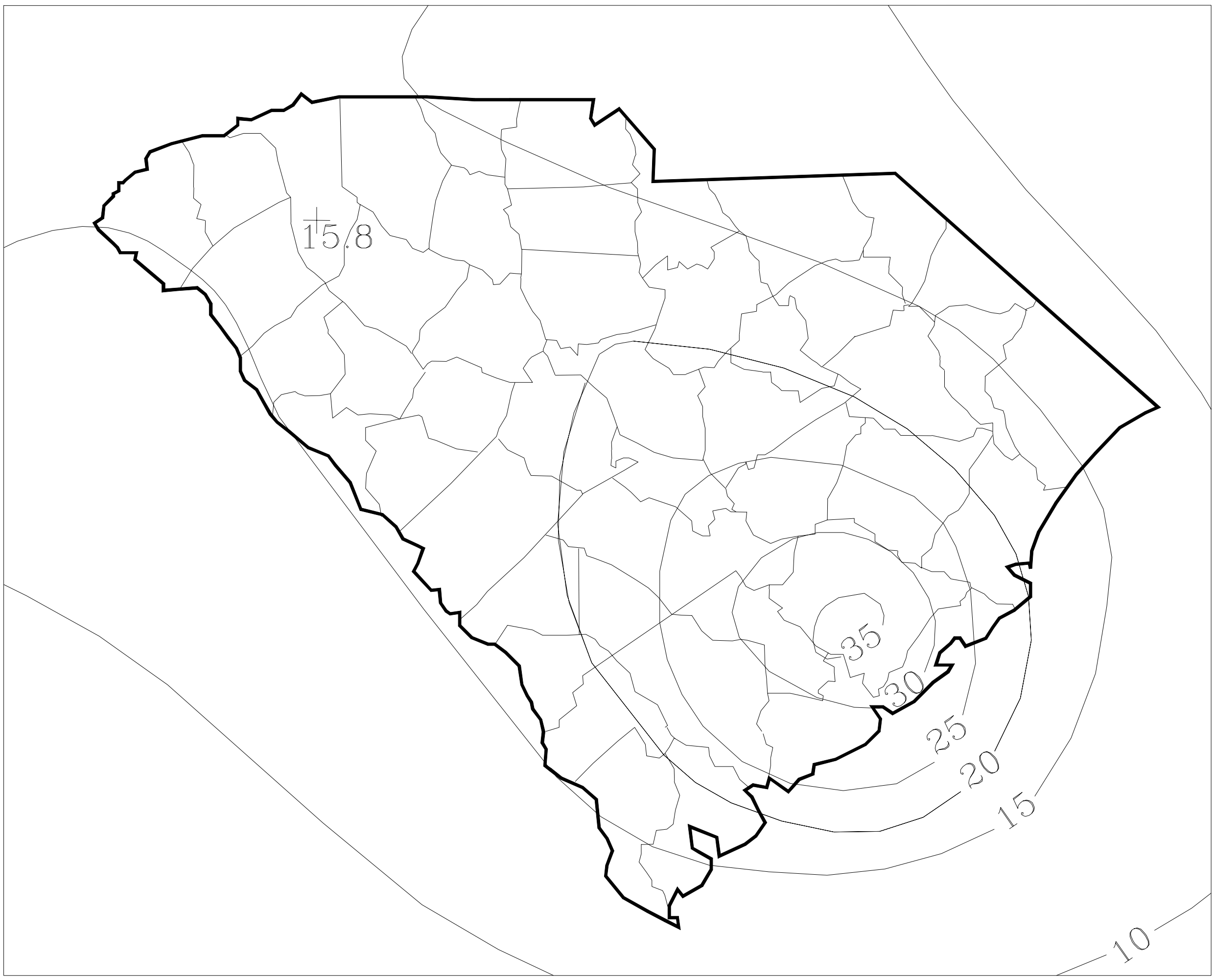


FIGURE 3.4.2A

Short Period (0.2-Sec) $S_{S\text{-}FEE}$ for Site Class B

(Response Spectral Acceleration: 10% Probability of Exceedance in 50 years, 5% Critical Damping)

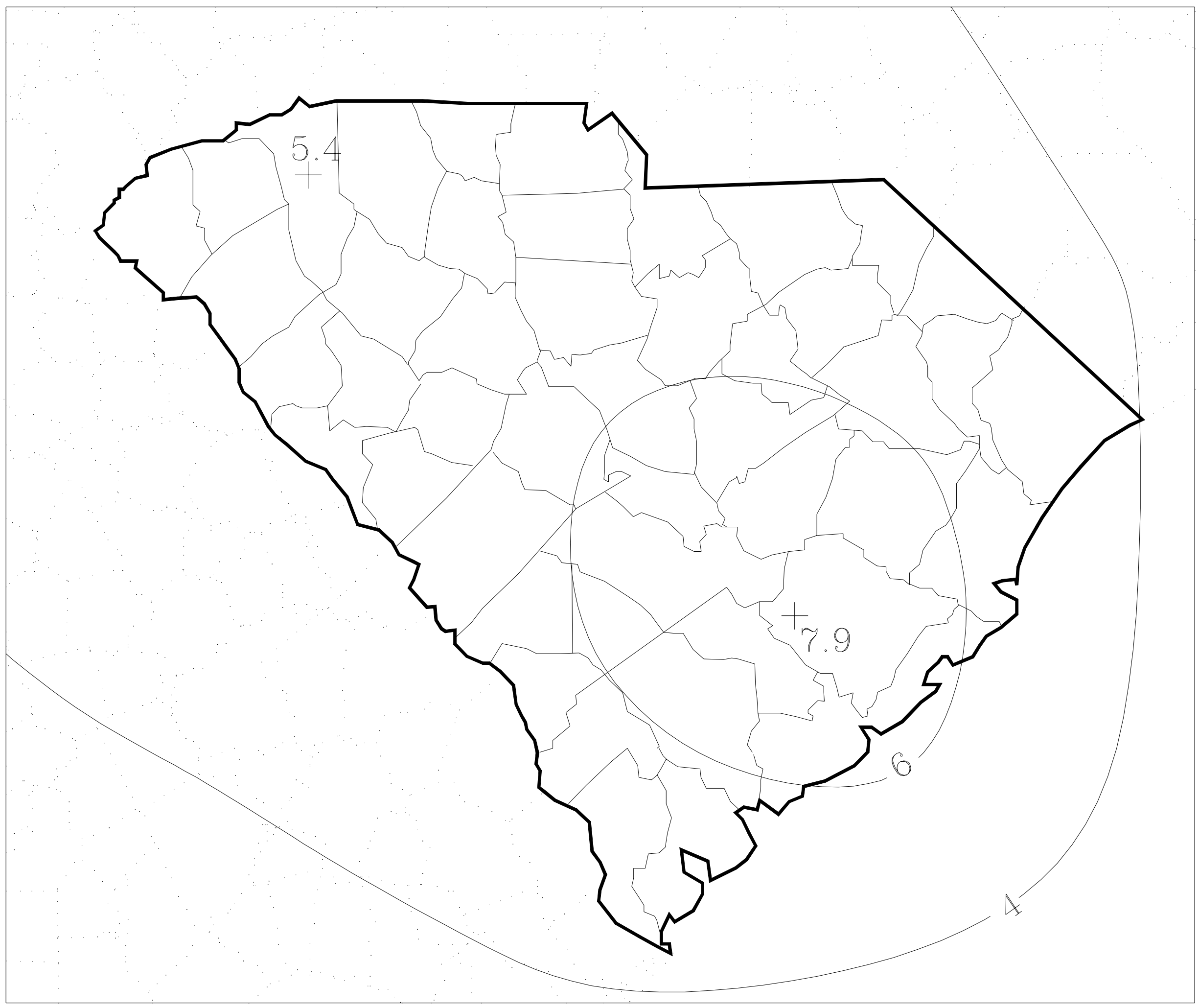


FIGURE 3.4.2B
1-Second Period $S_{1\text{-}FEE}$ for Site Class B
(Response Spectral Acceleration: 10% Probability of Exceedance in 50 years, 5% Critical Damping)

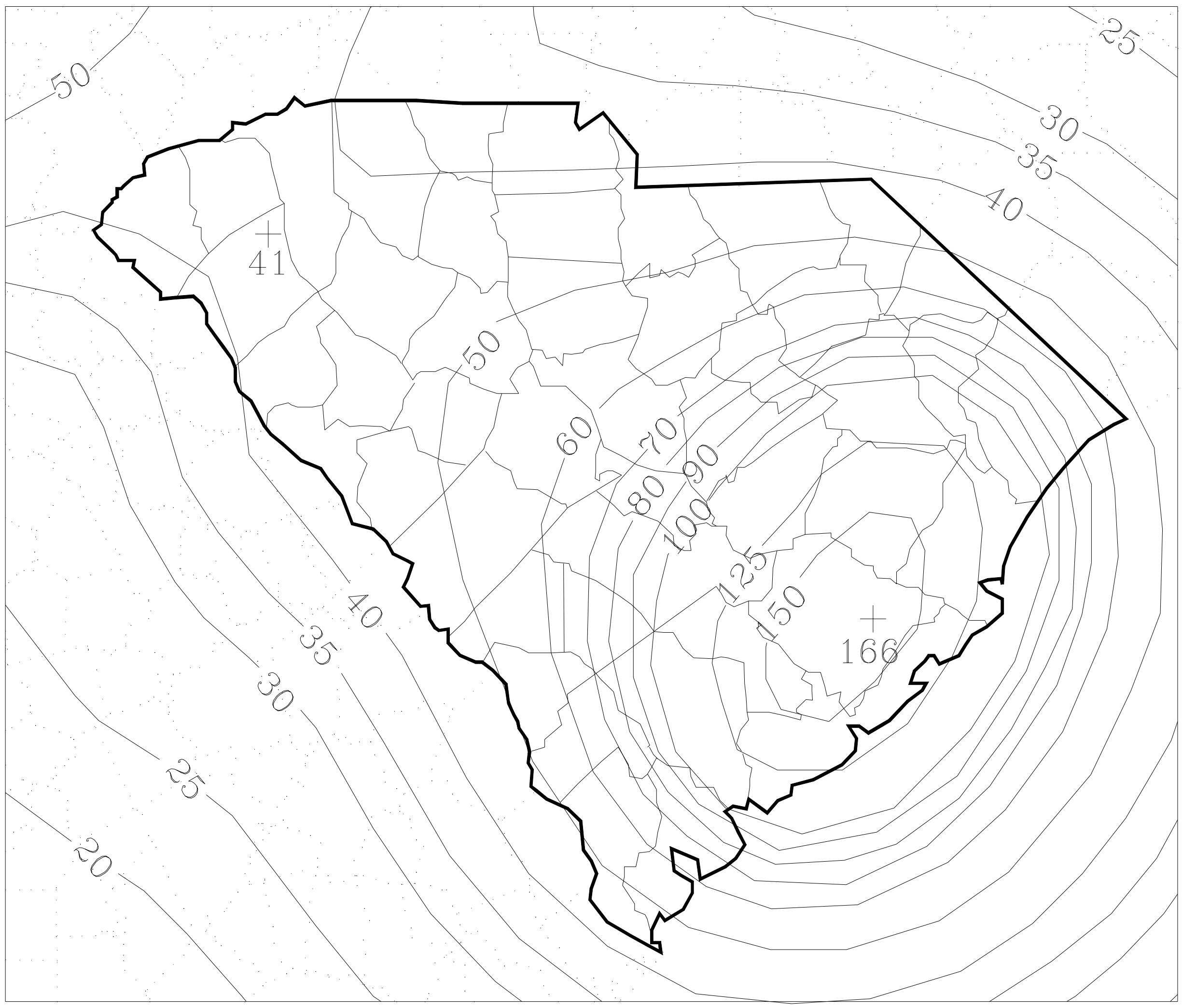
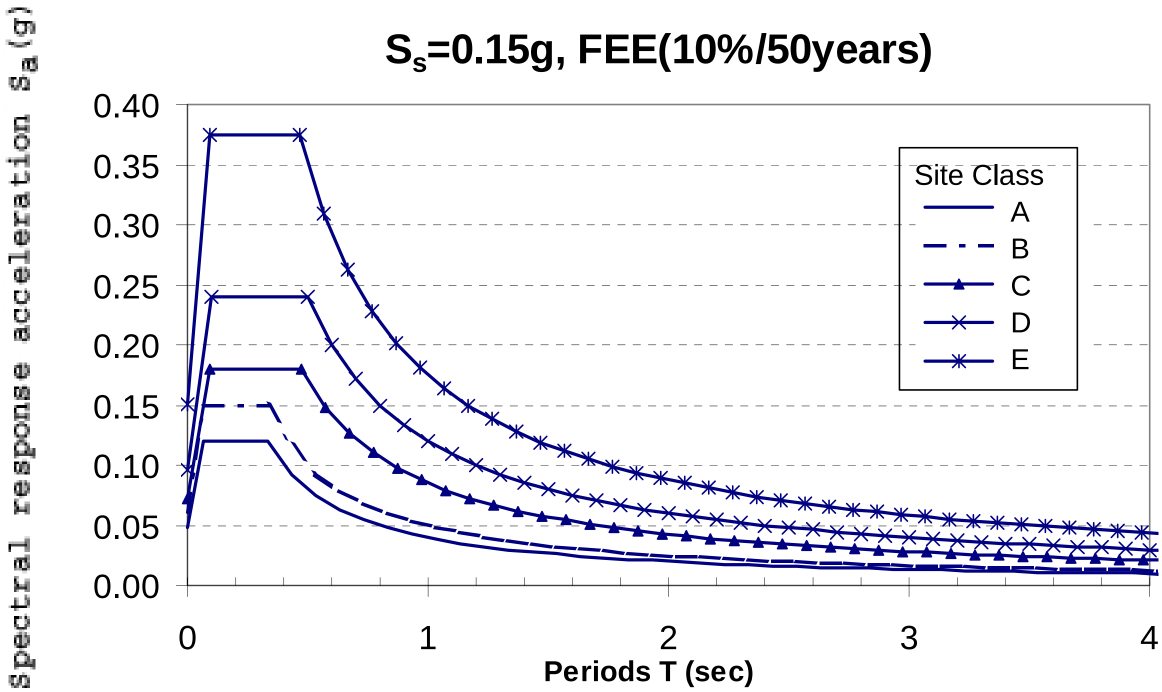


FIGURE 3.4.3A
Short Period (0.2-Sec) $S_{S\text{-}SEE}$ for Site Class B
(Response Spectral Acceleration: 2% Probability of Exceedance in 50 years, 5% Critical Damping)

FIGURE 3.4.3B
1-Second Period $S_{1\text{-}SEE}$ for Site Class B
(Response Spectral Acceleration: 2% Probability of Exceedance in 50 years, 5% Critical Damping)



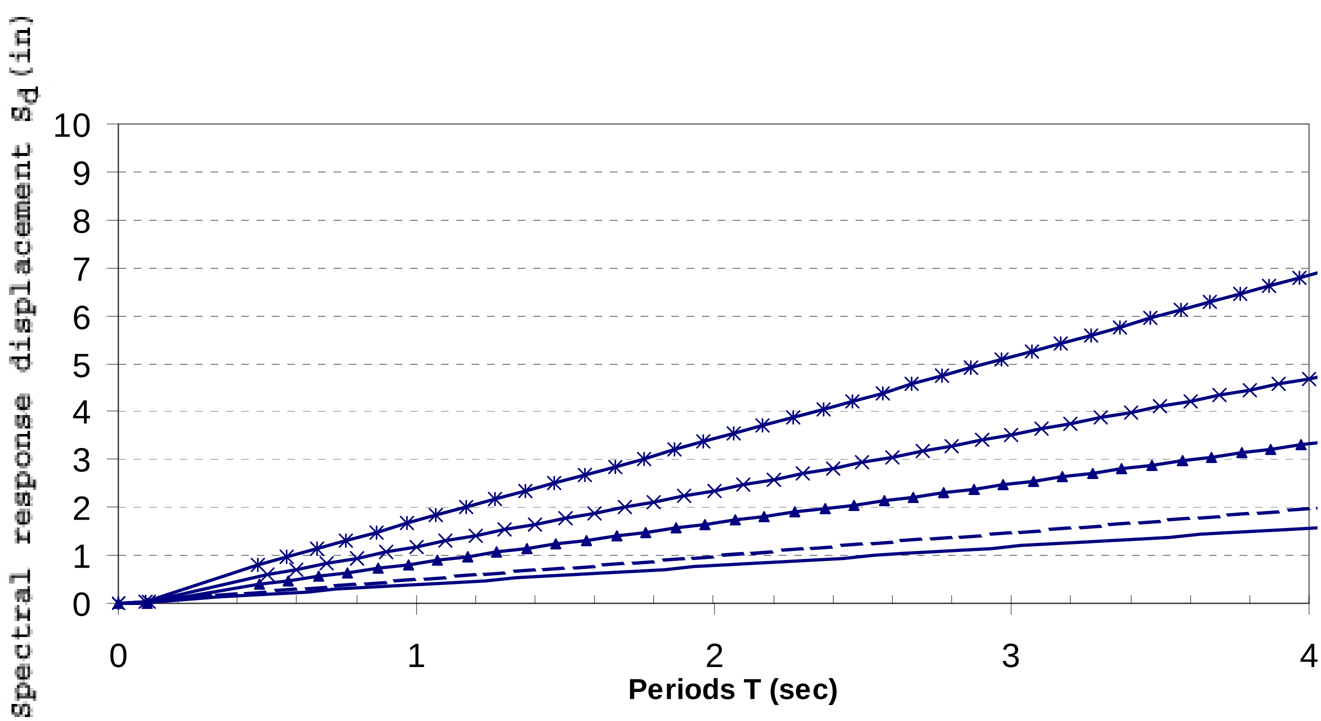
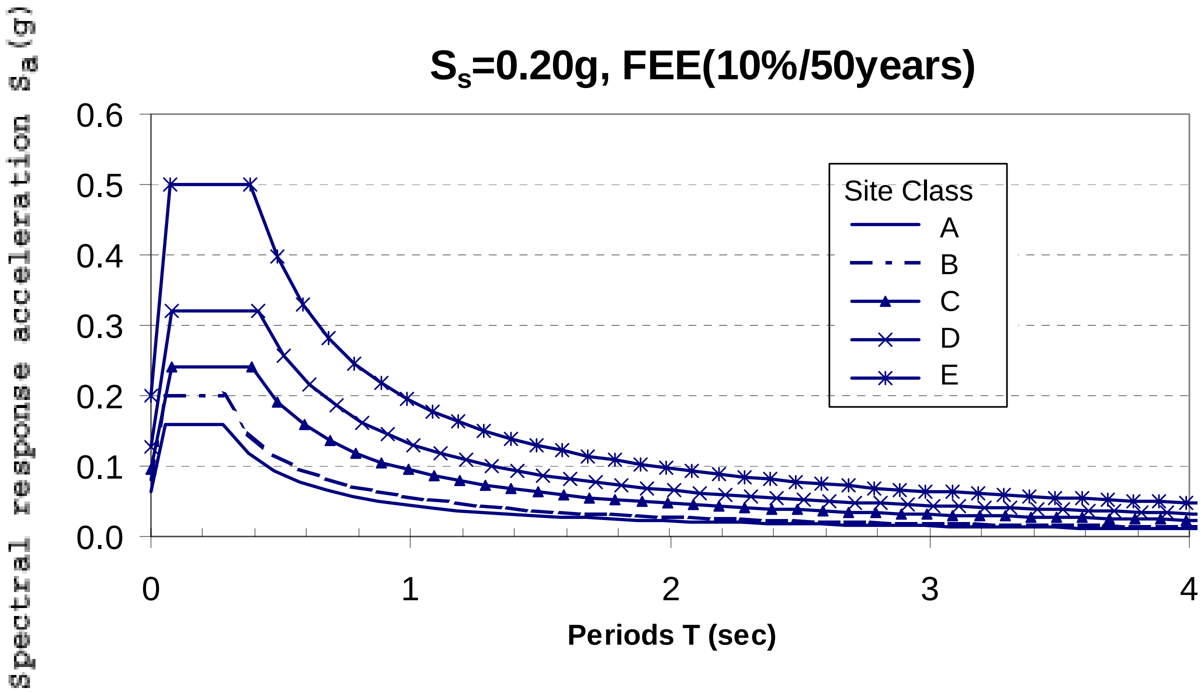


FIGURE 3.4.4A Design Spectra, S_a and S_d, for Site Class A, B, C, D and E, 5% Damping



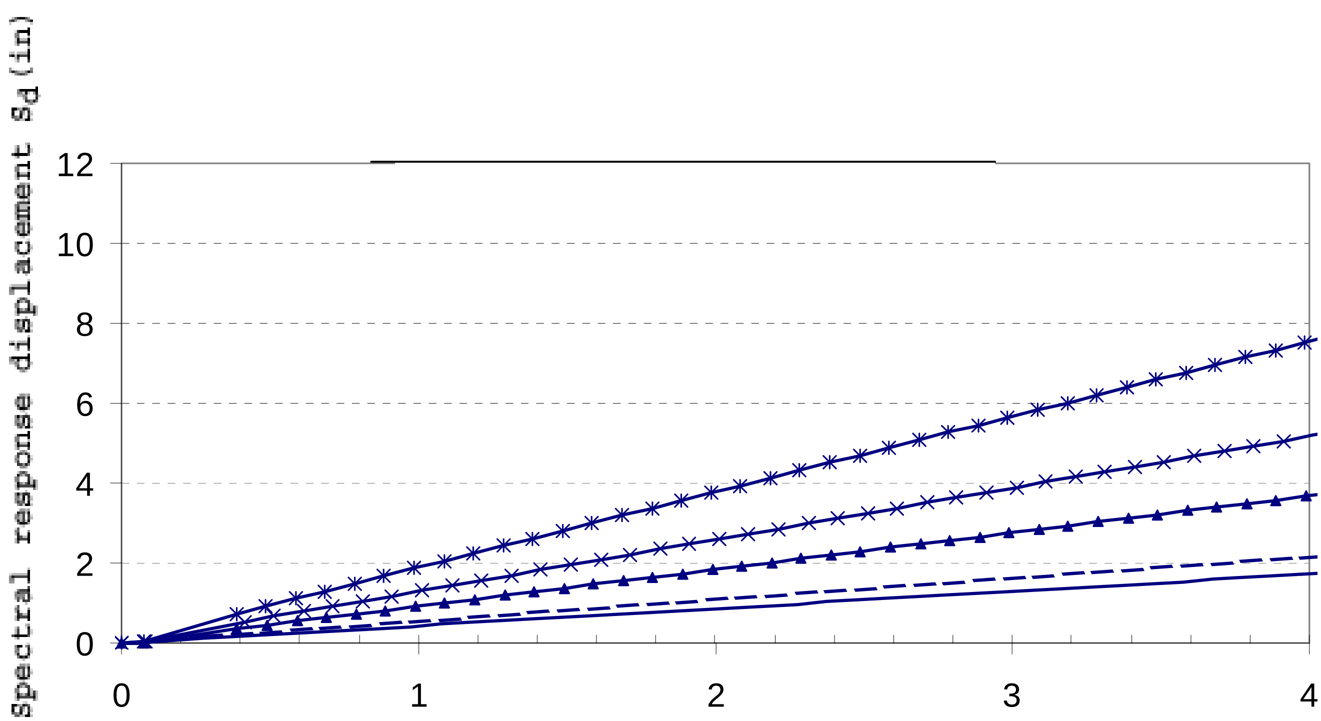


FIGURE 3.4.4B Design Spectra, S_a and S_d, for Site Class A, B, C, D and E, 5% Damping

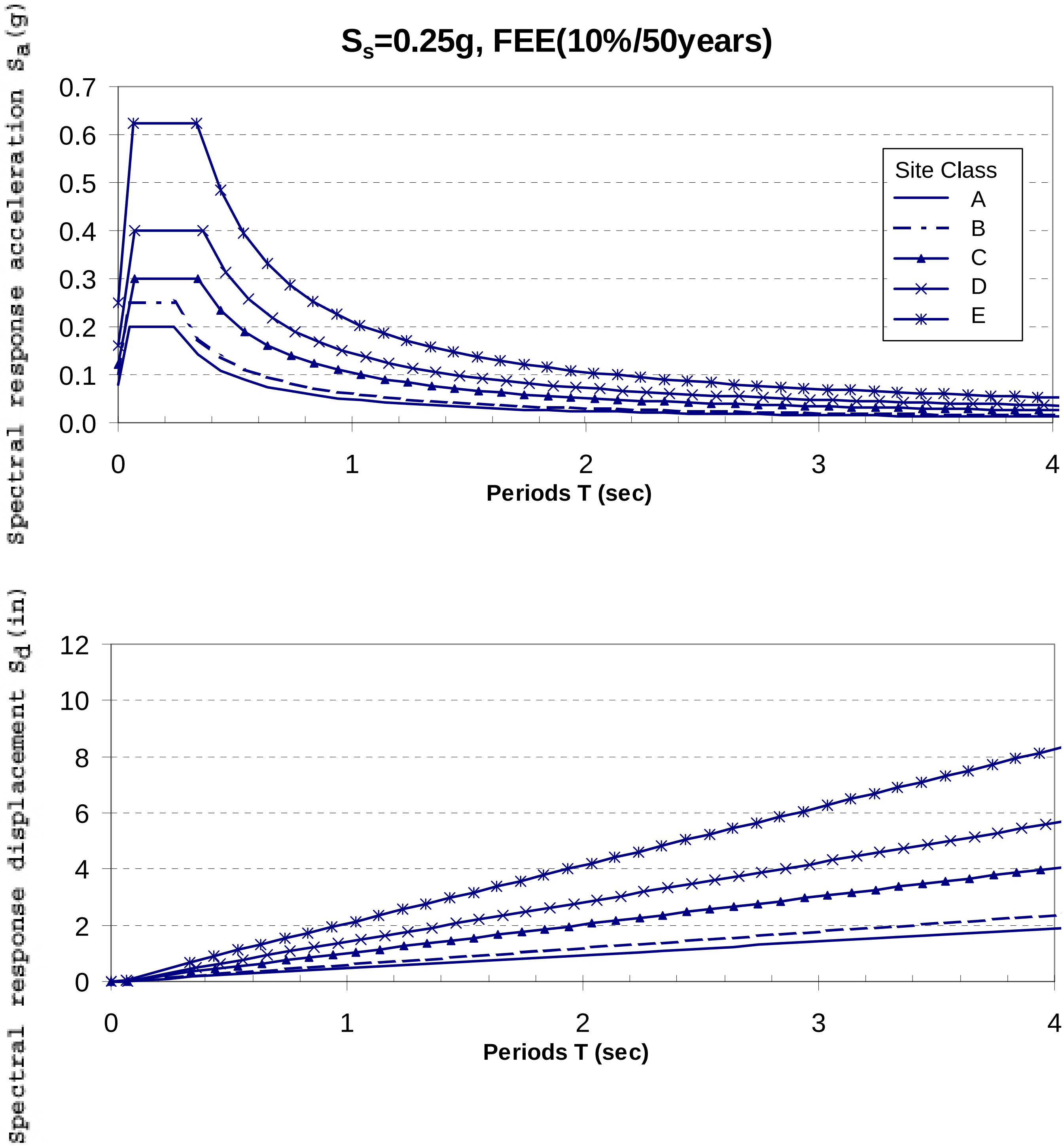
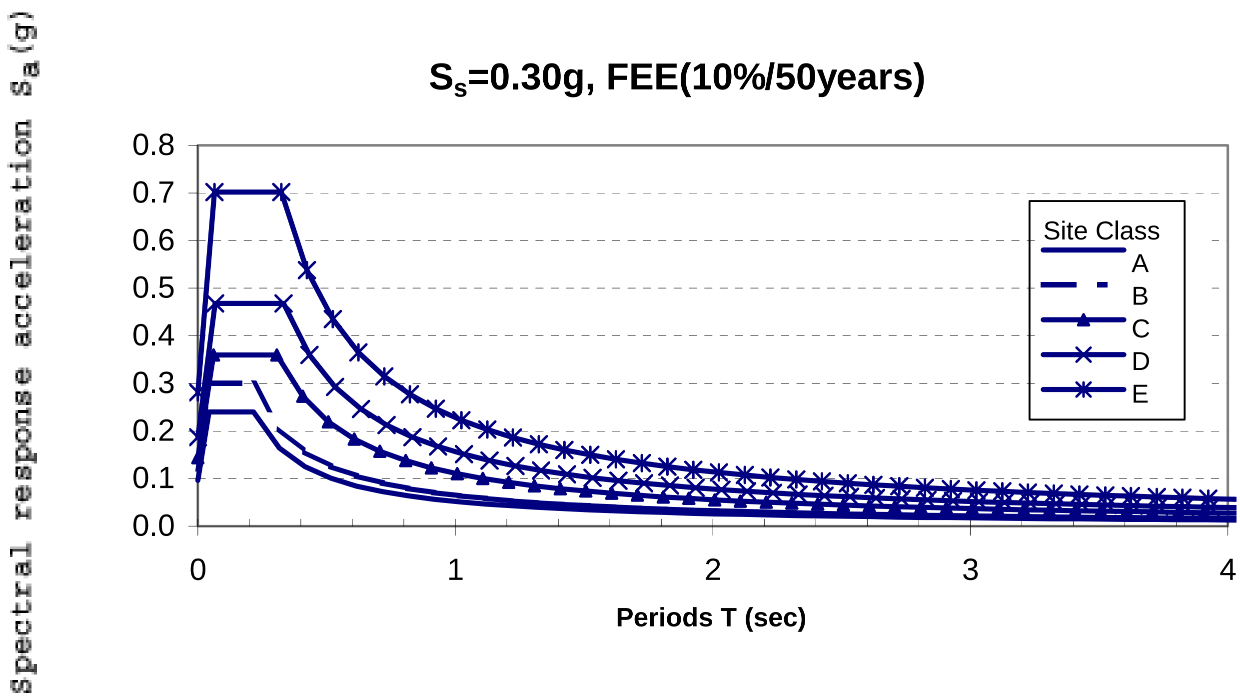


FIGURE 3.4.4C Design Spectra, S_a and S_d, for Site Class A, B, C, D and E, 5% Damping



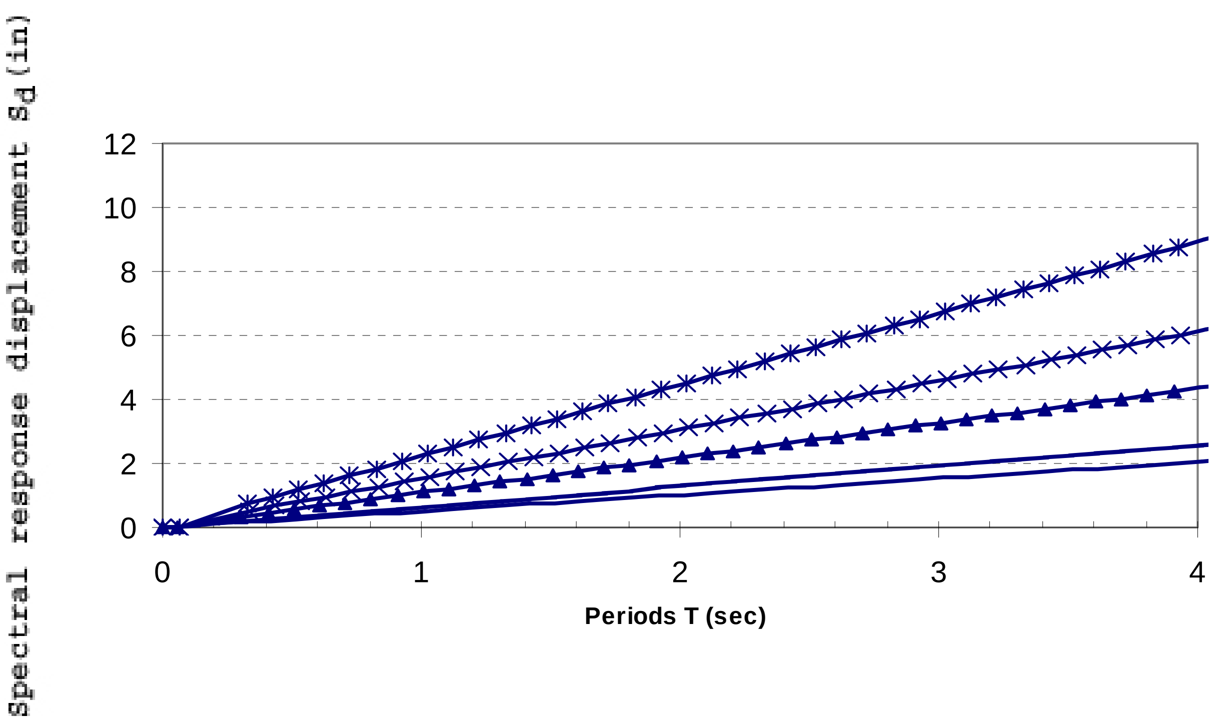


FIGURE 3.4.4D Design Spectra, S_a and S_d, for Site Class A, B, C, D and E, 5% Damping

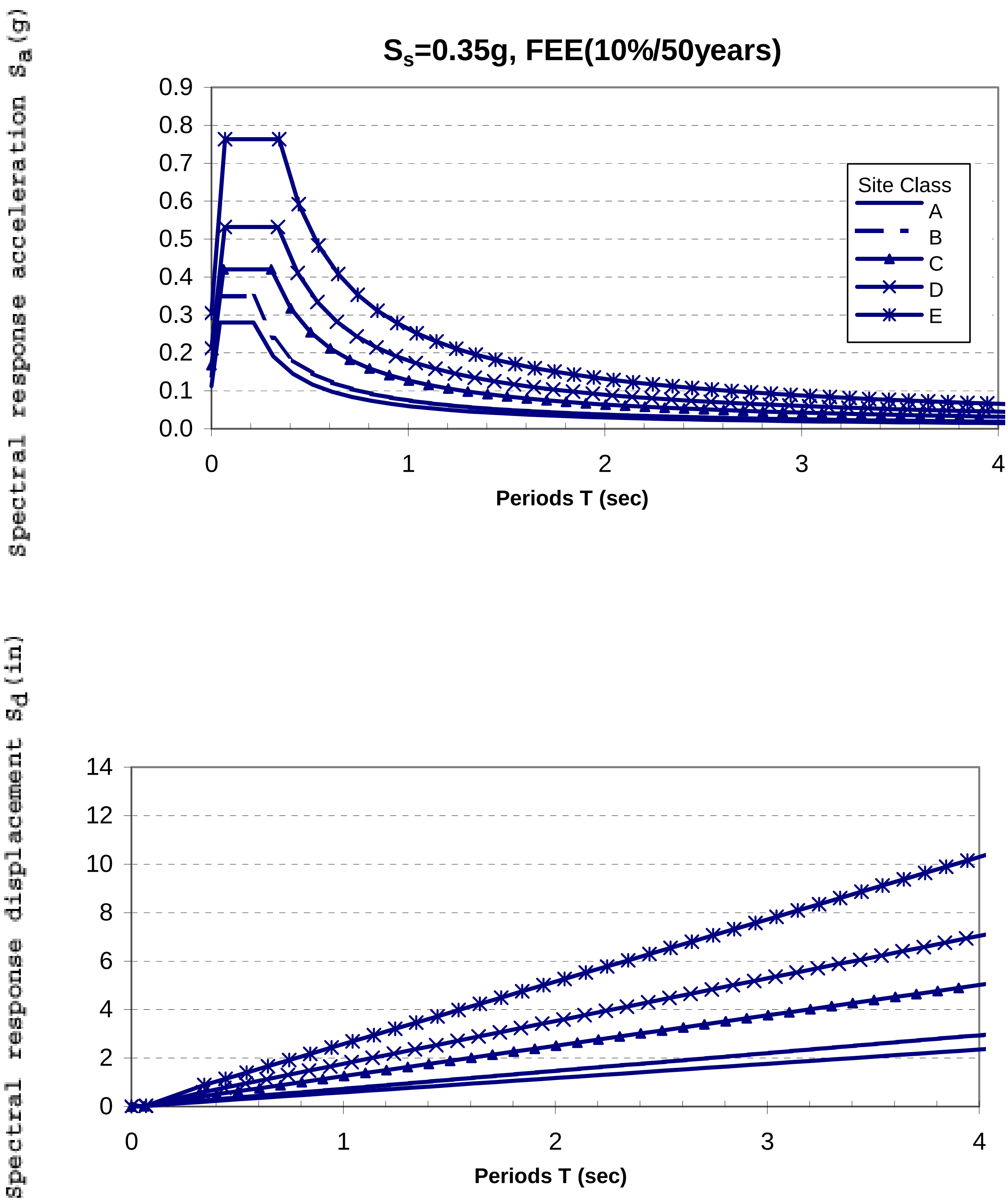
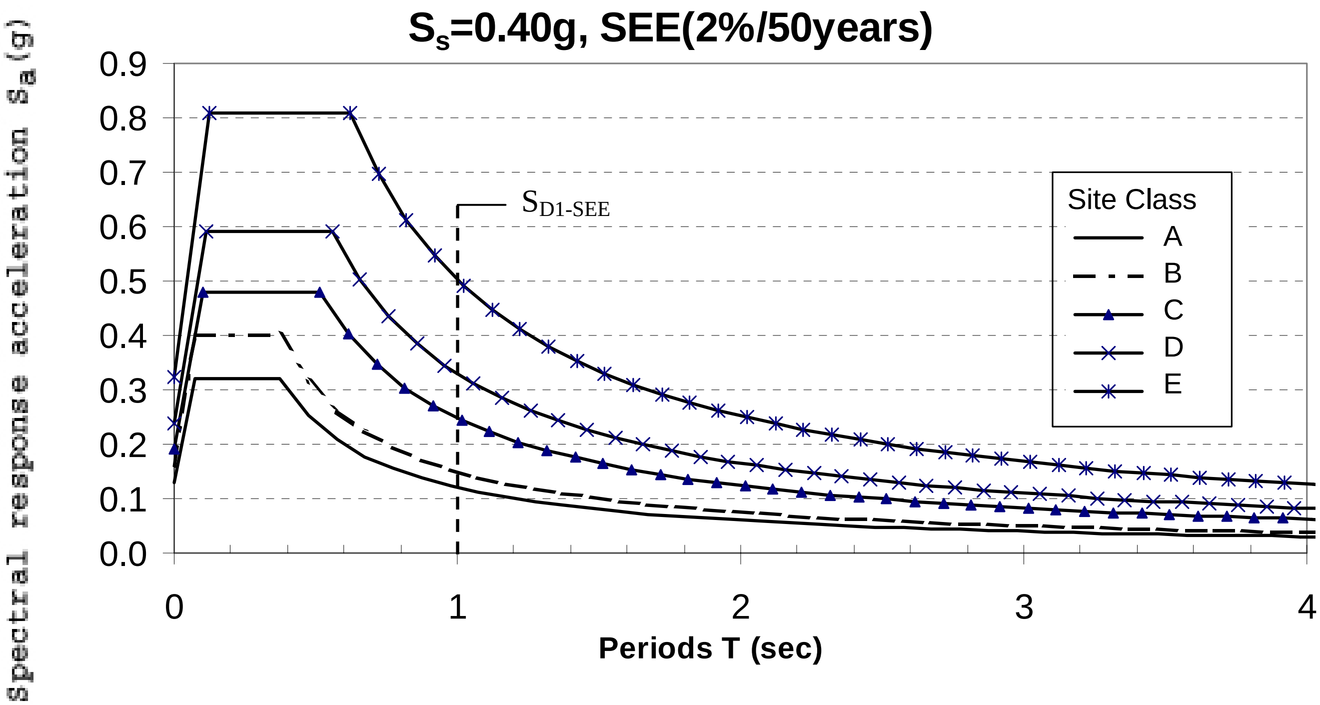


FIGURE 3.4.4E Design Spectra, S_a and S_d, for Site Class A, B, C, D and E, 5% Damping



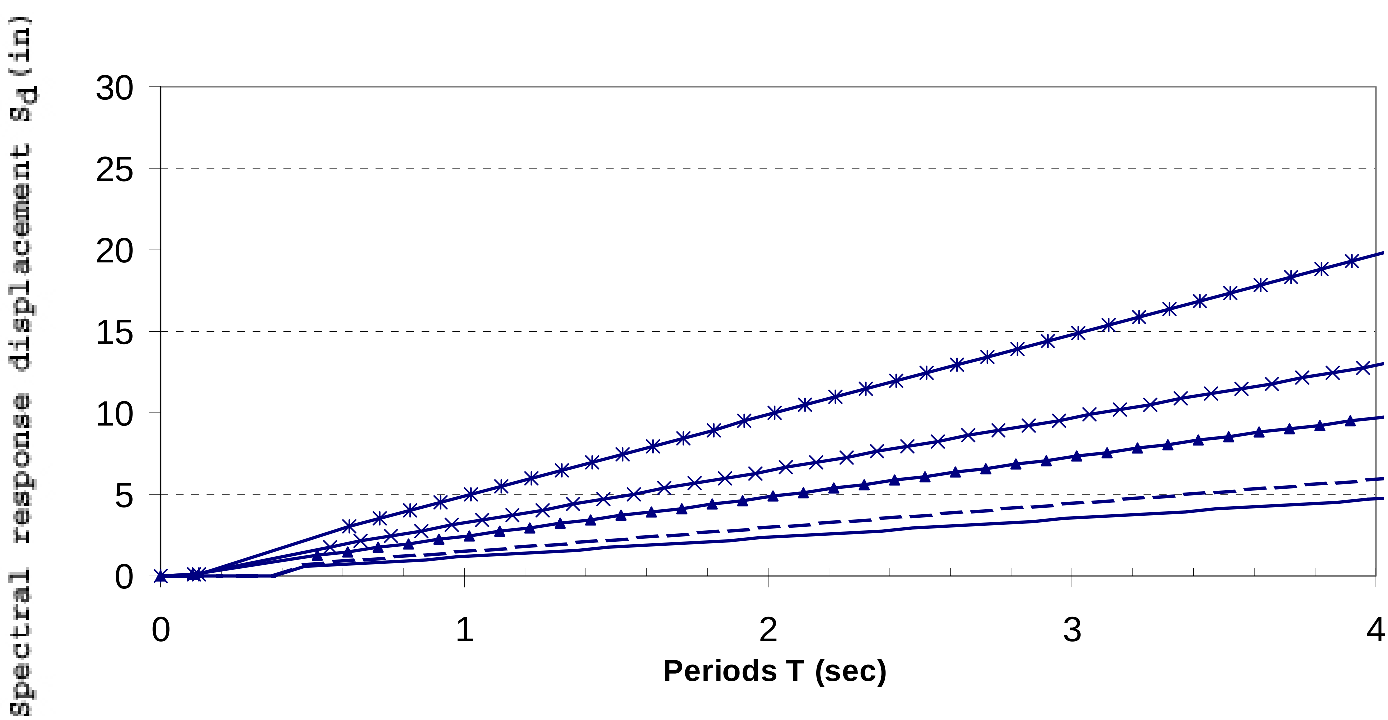


FIGURE 3.4.5A Design Spectra, S_a and S_d, for Site Class A, B, C, D and E, 5% Damping

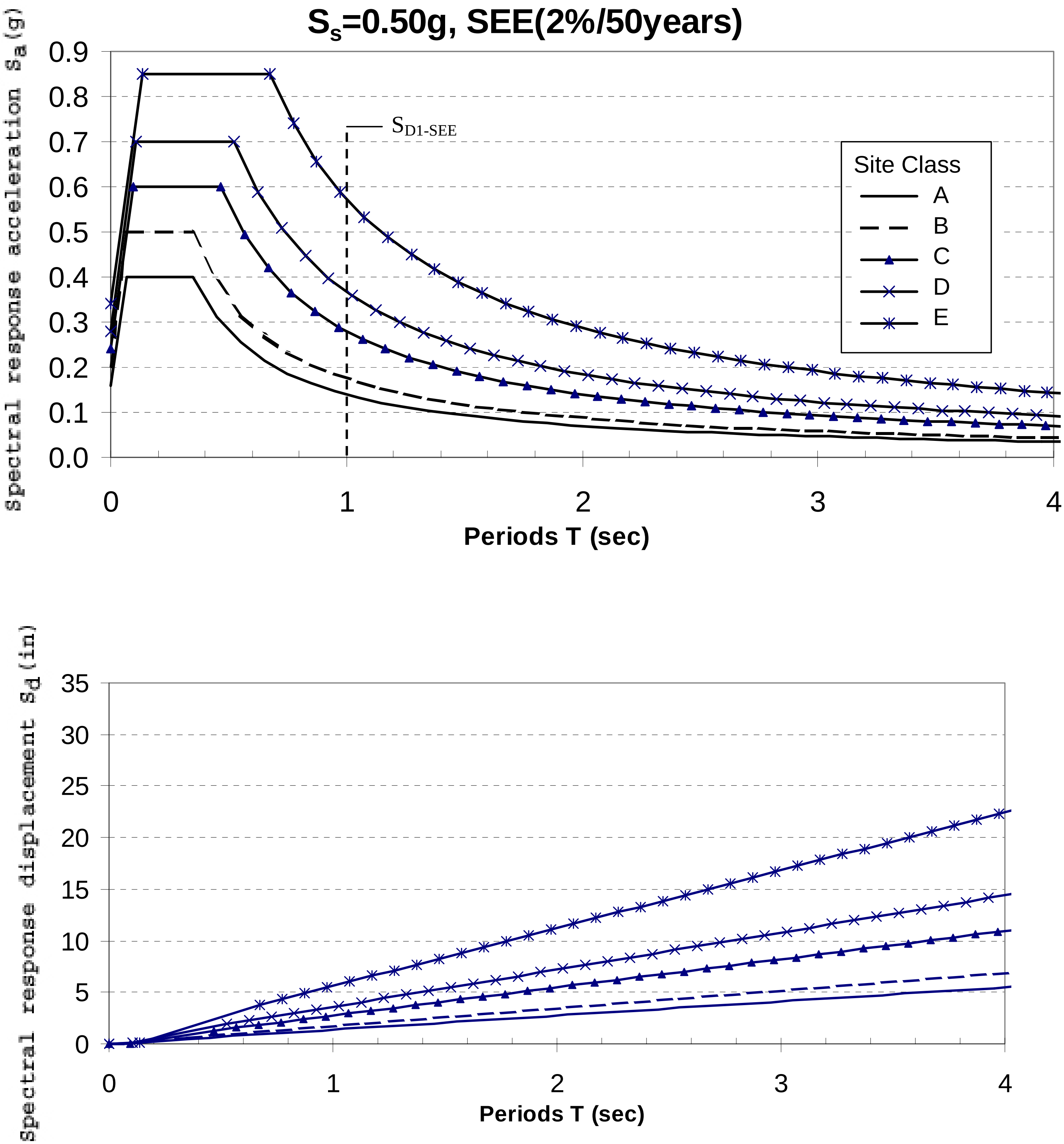
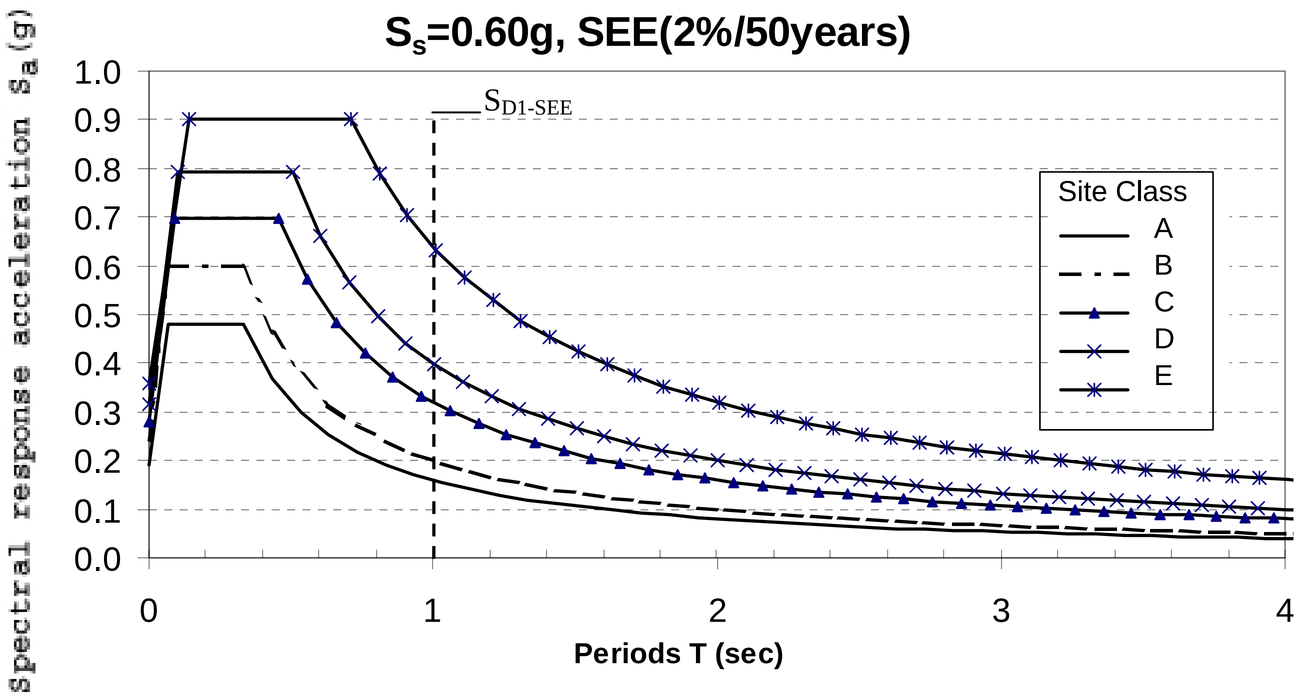


FIGURE 3.4.5B Design Spectra, S_a and S_d, for Site Class A, B, C, D and E, 5% Damping



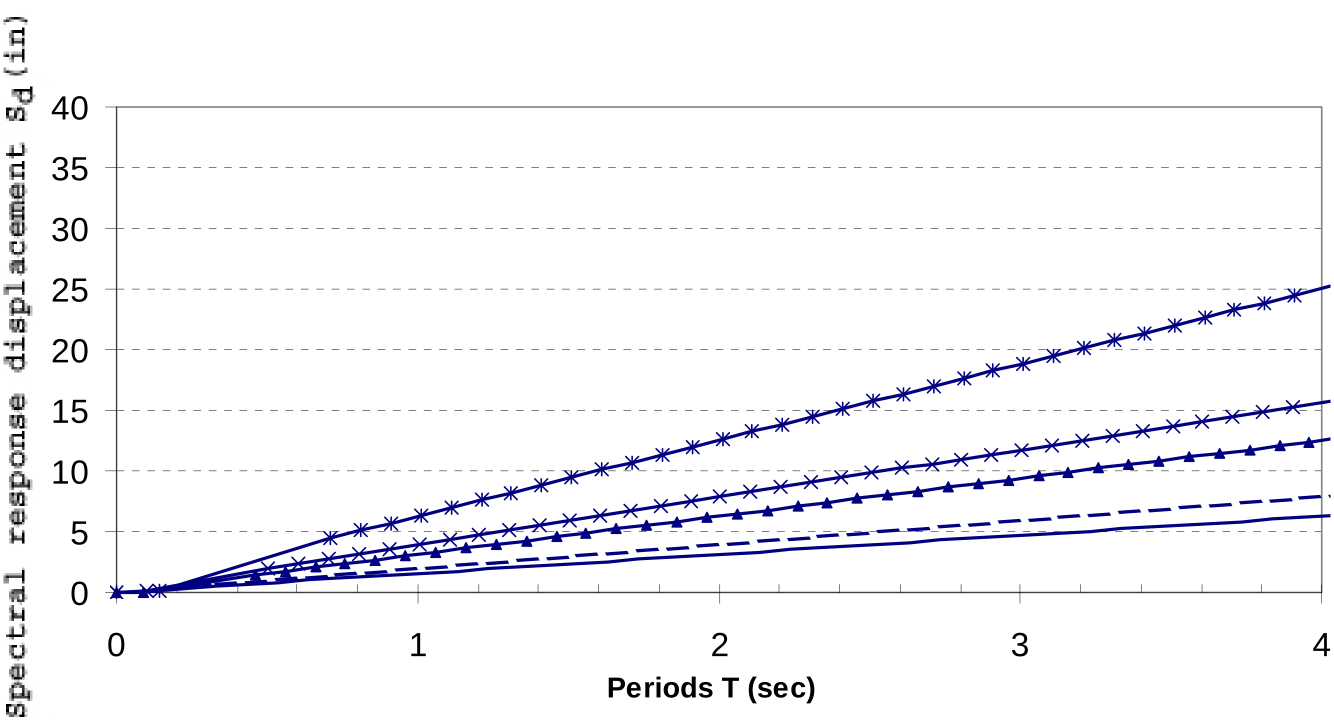
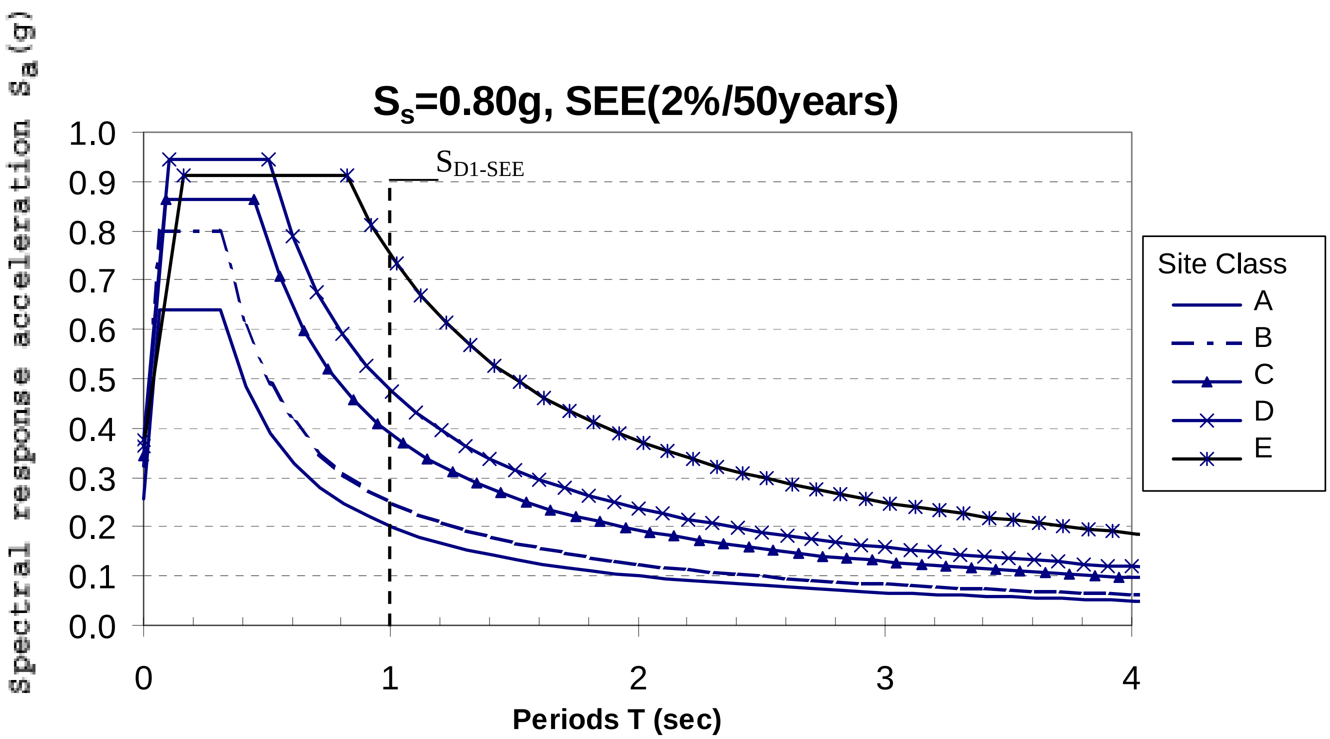


FIGURE 3.4.5C Design Spectra, S_a and S_d, for Site Class A, B, C, D and E, 5% Damping



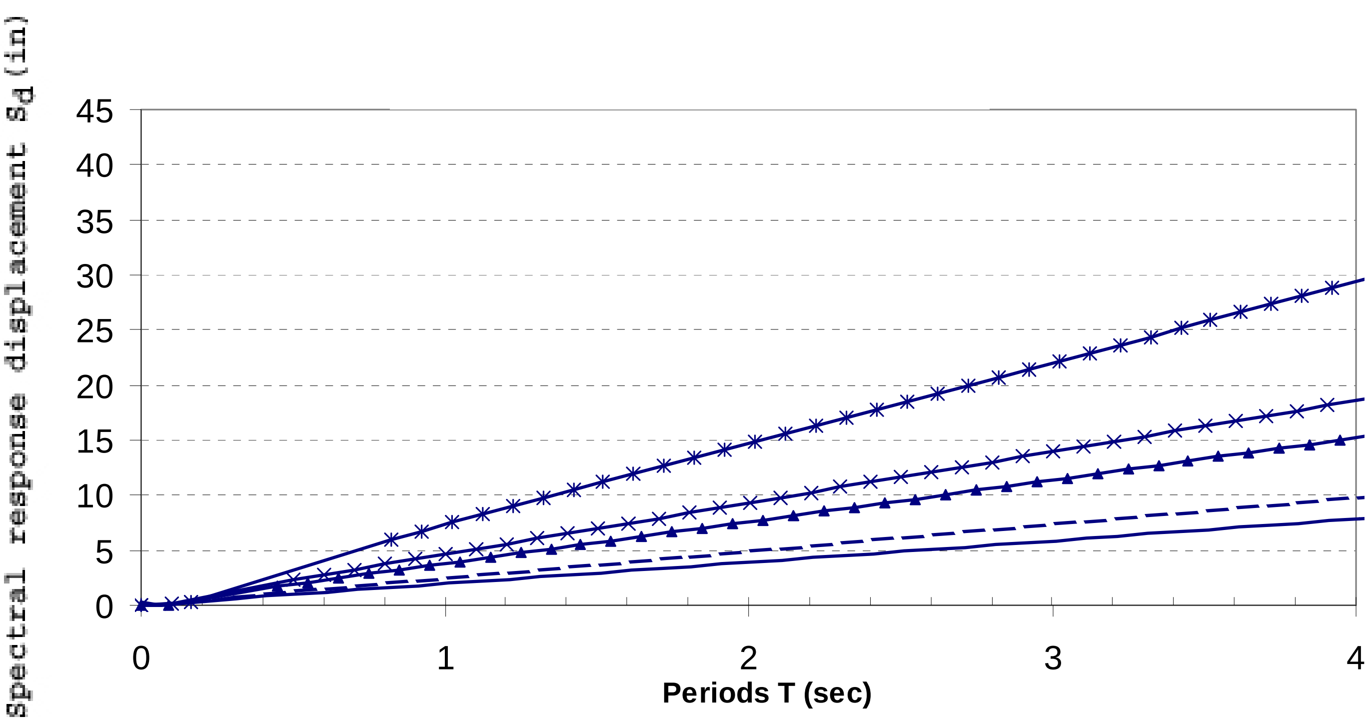


FIGURE 3.4.5D Design Spectra, S_a and S_d, for Site Class A, B, C, D and E, 5% Damping

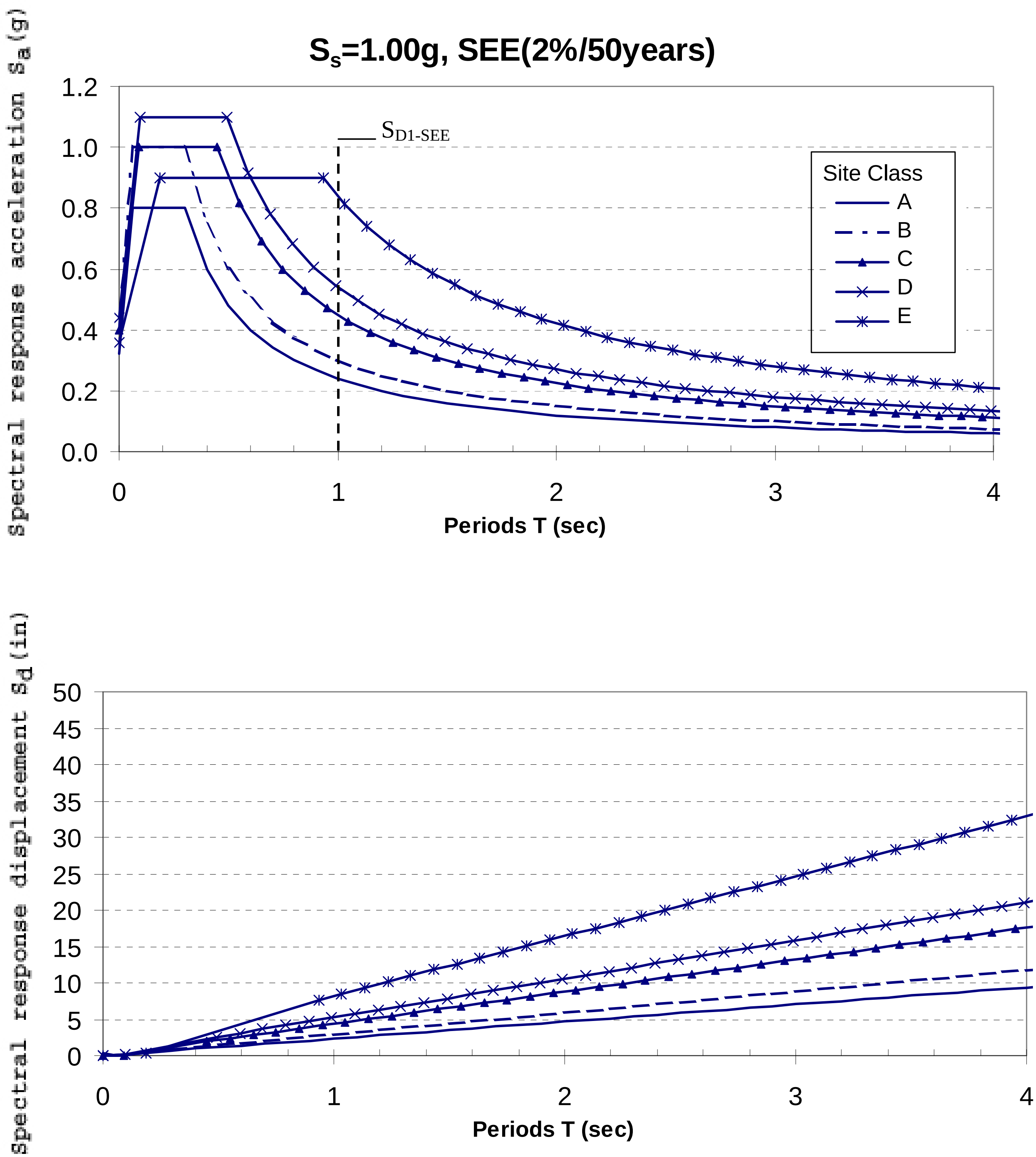
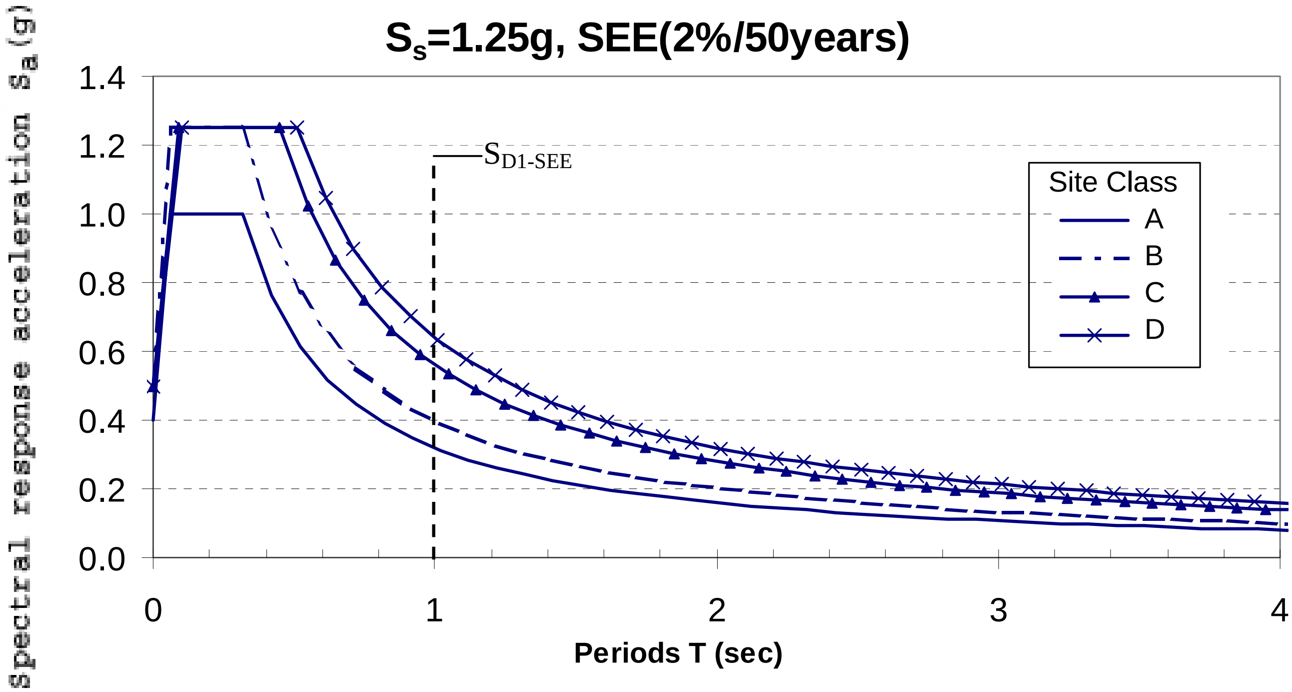


FIGURE 3.4.5E Design Spectra, S_a and S_d, for Site Class A, B, C, D and E, 5% Damping



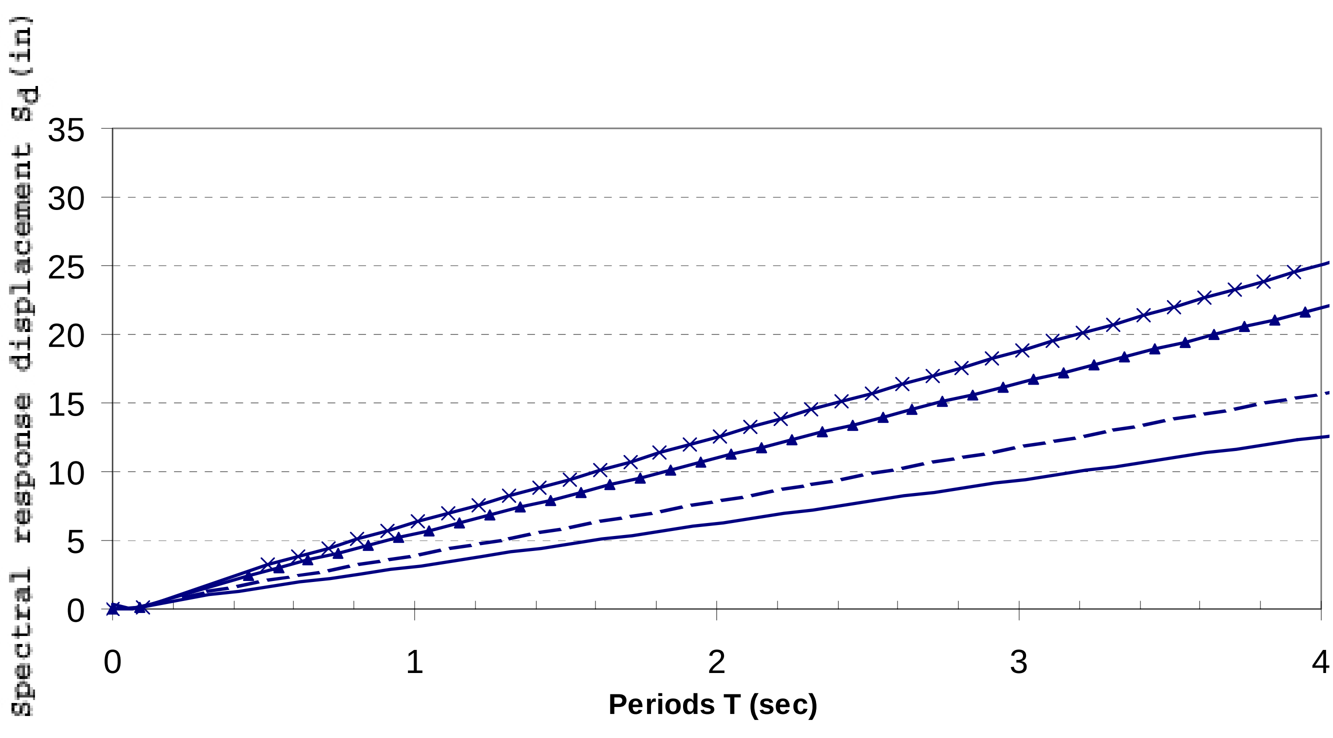


FIGURE 3.4.5F Design Spectra, S_a and S_d, for Site Class A, B, C and D, 5% Damping

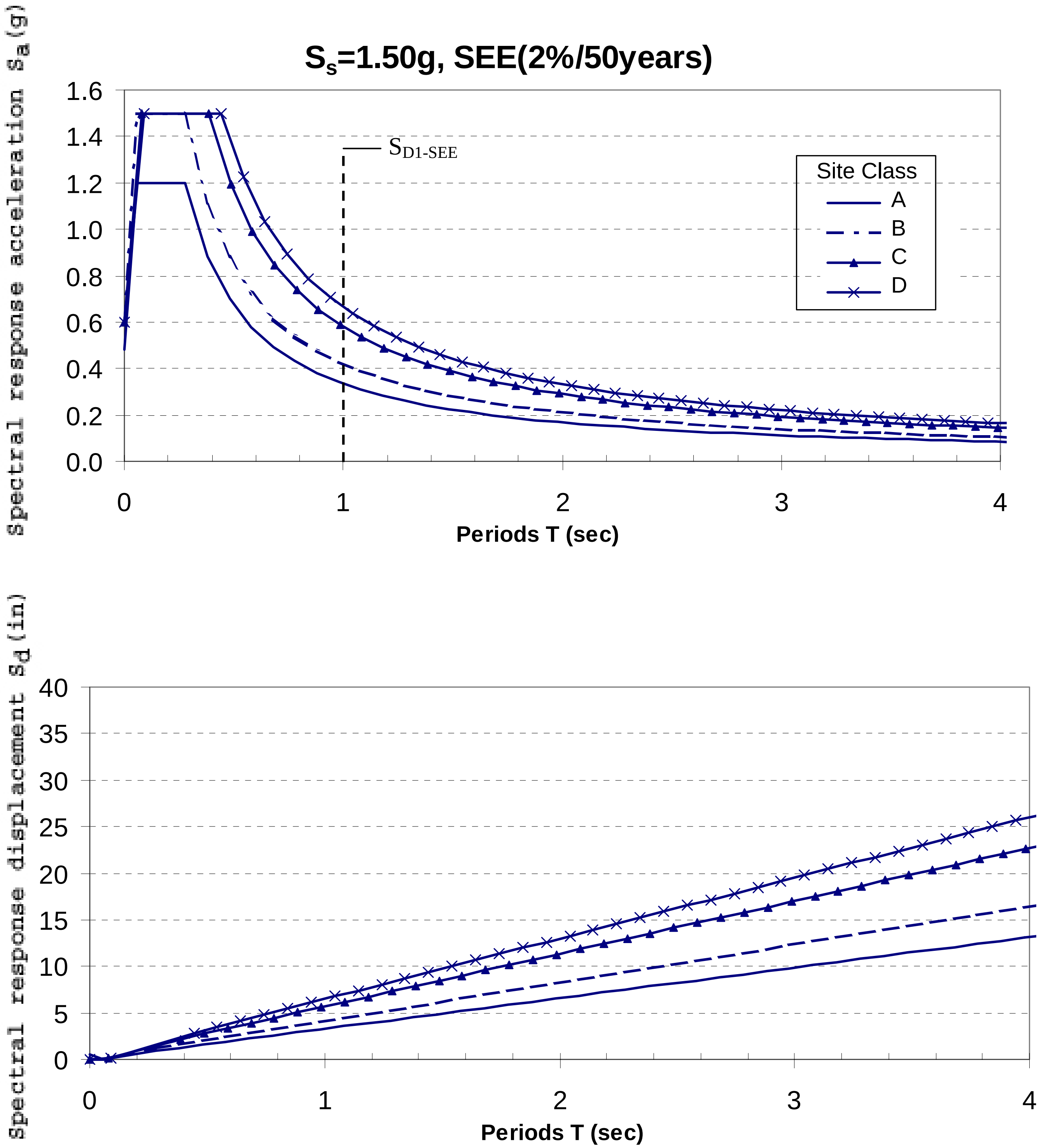
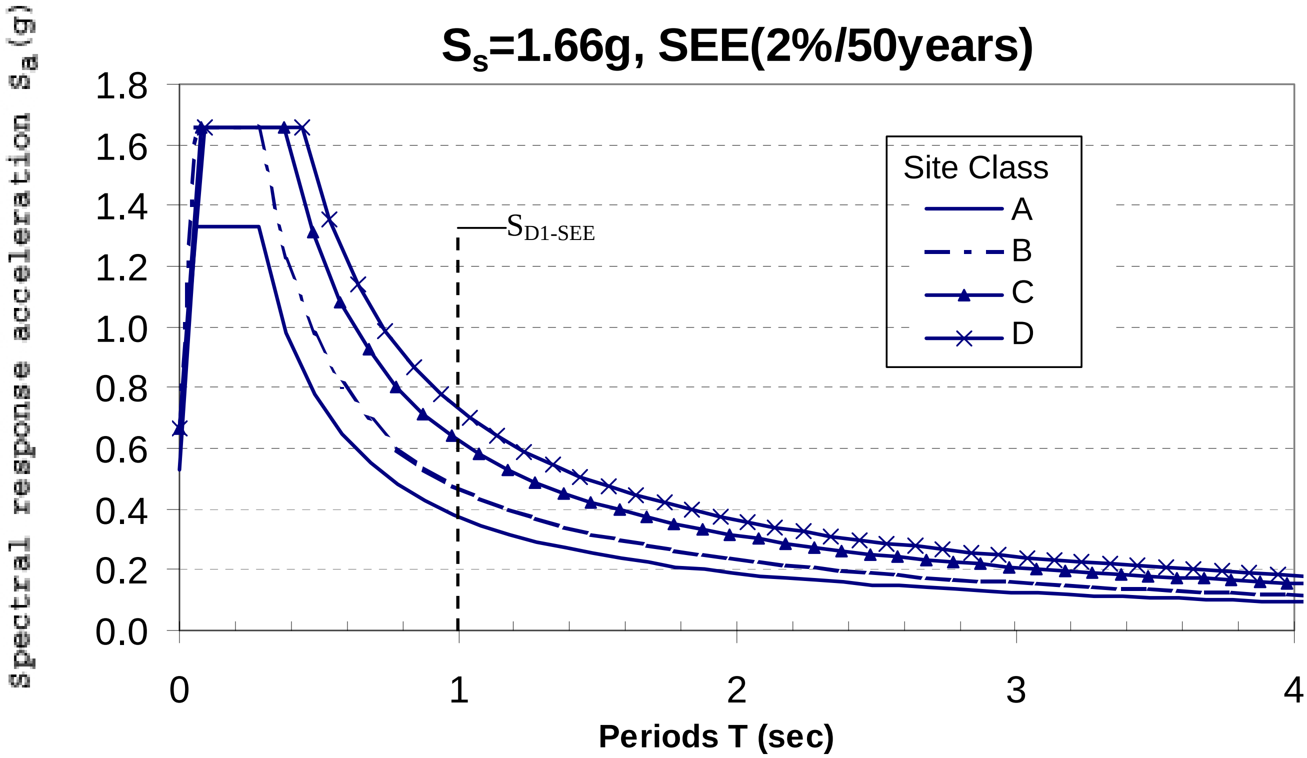


FIGURE 3.4.5G Design Spectra, S_a and S_d, for Site Class A, B, C and D, 5% Damping



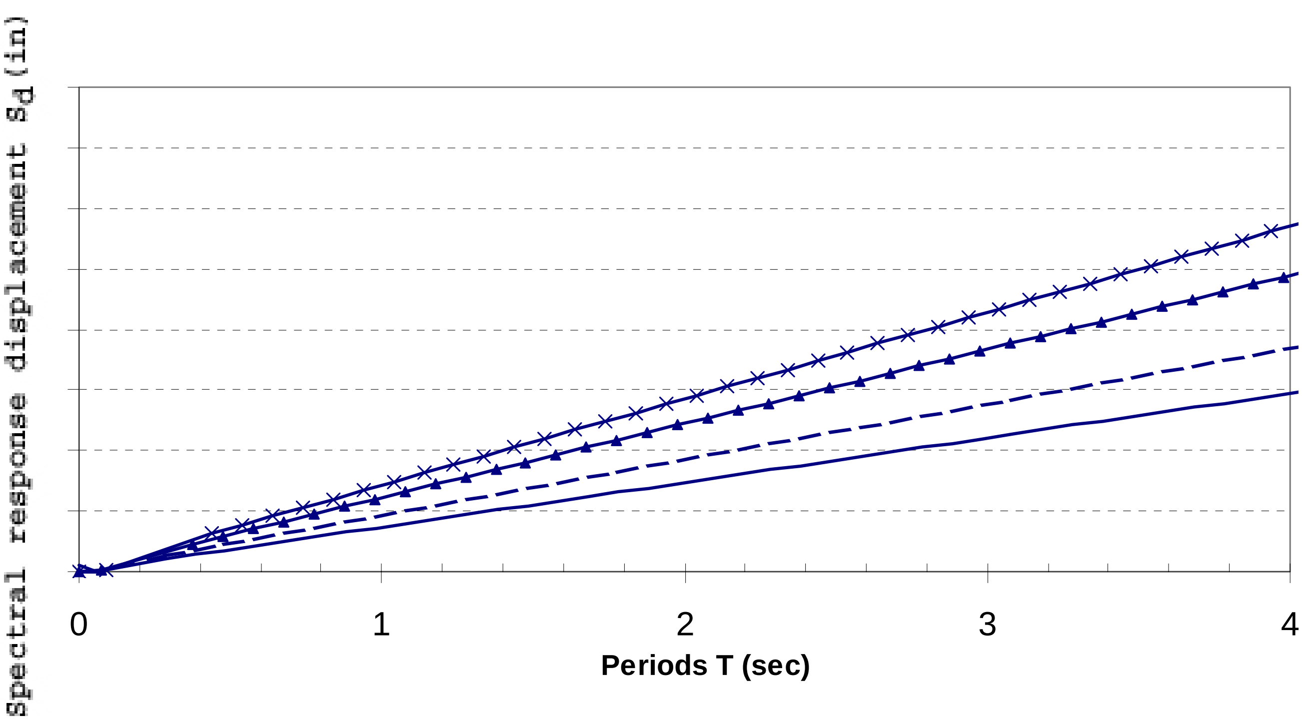


FIGURE 3.4.5H Design Spectra, S_a and S_d, for Site Class A, B, C and D, 5% Damping

SECTION 4

ANALYSIS AND DESIGN REQUIREMENTS

4.1 GENERAL

The requirements of this chapter shall control the selection and method of seismic analysis of bridges and seismic design and displacements. The seismic design 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.

EXCEPTION:
Seismic design requirements for single span bridges are given in Sections 4.5 and 4.8 and design requirements for bridges classified as SPC A are given in Sections 4.6 and 4.8.

The following are recommendations that the designer should consider at the preliminary phase of setting a bridge layout and framing configuration. Recommendations typically need to be satisfied in order to obtain enhanced seismic performance but need not to be strictly satisfied to achieve the criteria of collapse prevention.

4.1.1 Balanced Stiffness

It is strongly 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>

$$\frac{k_i^e}{k_j^e} \geq 0.5 \quad \text{(4-1a)} \qquad \frac{k_i^e \times m_j}{k_j^e \times m_i} \geq 0.5 \quad \text{(4-1b)}$$

$$\frac{k_i^e}{k_j^e} \geq 0.75 \quad \text{(4-2a)} \qquad \frac{k_i^e \times m_j}{k_j^e \times m_i} \geq 0.75 \quad \text{(4-2b)}$$

k_i^e = The smaller effective bent or column stiffness

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: 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

It is strongly recommended that the ratio of fundamental periods of vibration for adjacent frames in the longitudinal and transverse direction satisfy Equation 4-3.

$$\frac{T_i}{T_j} \geq 0.7 \quad \text{(4-3)}$$

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 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 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

A careful evaluation of the local ductility demands and capacities is required 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 on the transverse stiffness of the columns near the abutment, particularly when calculating shear demand, shall be considered.

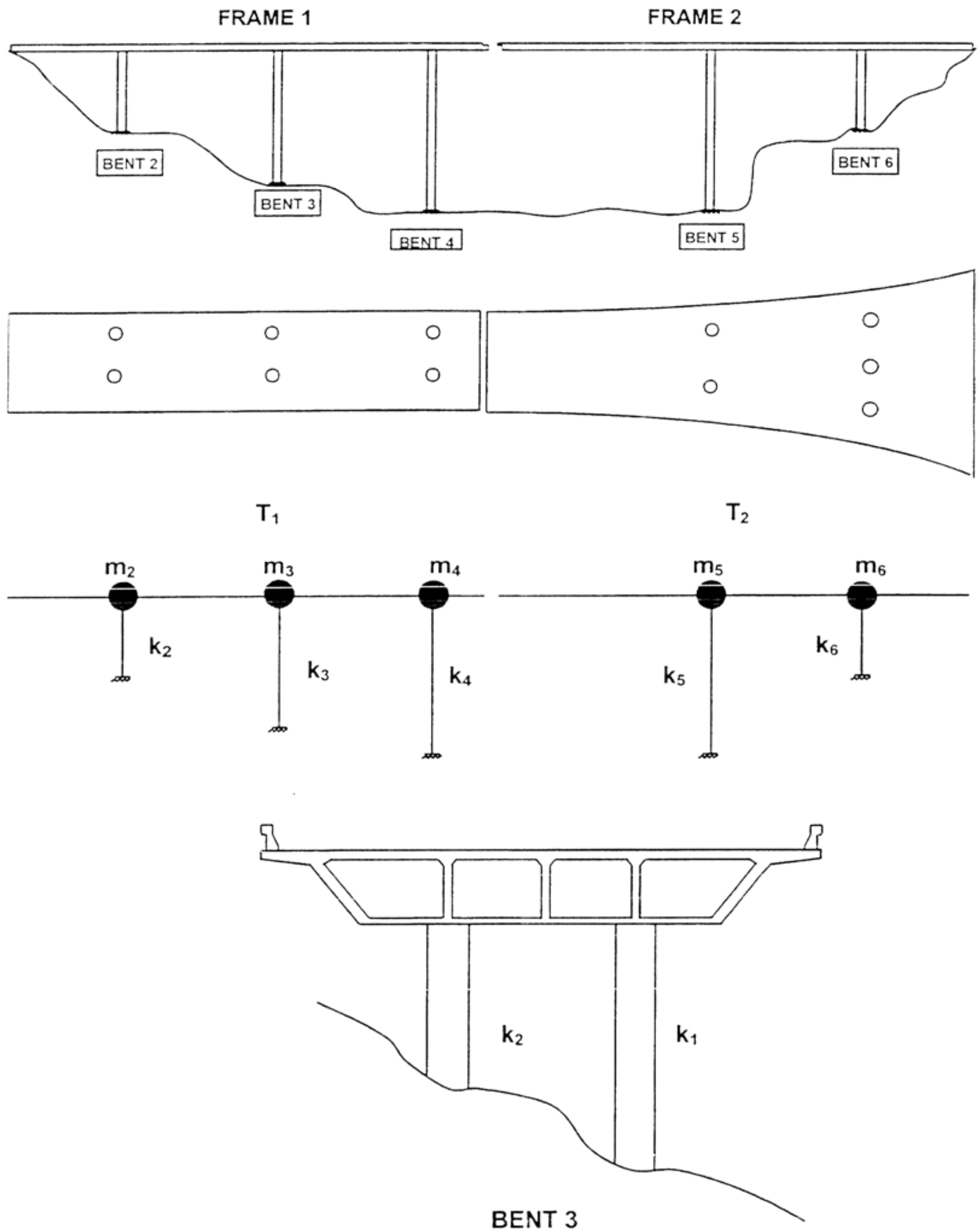


FIGURE 4.1 Balanced Stiffness

4.2 SELECTION OF ANALYSIS PROCEDURE

Minimum requirements for the selection of an analysis method 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 have less than seven spans, no abrupt or unusual changes in weight, stiffness, or geometry and no larger changes in these parameters from span-to-span or support-to-support (abutments excluded). They are defined in Table 4.2. Any bridge not satisfying the requirements of Table 4.2 is considered to be "not regular". A more rigorous, generally accepted procedure may be used in lieu of the recommended minimum.

TABLE 4.1 Analysis Procedures

Seismic Performance category	Regular Bridges with 2 Through 6 Spans	Not Regular Bridges with 2 or More Spans
A	Not required	Not required
B, C, D	Use Procedure 1 or 2	Use Procedure 2

Details of these procedures are given in Section 5.

The analysis procedures to be used are as follows:

PROCEDURE 1: Equivalent Static Analysis Method
PROCEDURE 2: Multimode Spectral Method
PROCEDURE 3: Inelastic Static Analysis
PROCEDURE 4: Nonlinear Time History Method

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

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 30°: and
(c) the span lengths of the equivalent straight bridge are equal to the arc lengths of the curved bridge.

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)	90°	90°	90°	90°	90°
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 Special Requirements for Critical and Essential Bridges

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. Time history methods of analysis are recommended for this purpose, provided care is taken with both the modeling of the structure and the selection of the input time histories of ground motion. Time history methods of analysis are described in Section 5 of the specifications.

4.3 DETERMINATION OF SEISMIC DISPLACEMENTS DEMANDS

4.3.1 Horizontal Ground Motions

For bridges classified as SPC B, C or D the seismic displacements demands shall be determined independently along two perpendicular axes by the use of the analysis procedure specified in Section 4.2. 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 but the choice is open to the designer. The longitudinal axis of a curved bridge may be selected along a chord connecting the two abutments.

4.3.2 Vertical Ground Motions

Bridges, under seismic performance Category D, shall have at least 25% 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. The continuous steel reinforcement shall be spliced with “service load” couplers capable of achieving a minimum of 80 ksi strength capacity. A case-by-case determination on the effect of vertical ground motions is required for essential and critical bridges.

4.3.3 Damping Considerations

Damping ratios on the order of 10% can be used for bridges that are heavily influenced by energy dissipation 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 spectrum coefficient used to calculate the displacement demand.

The following characteristics are typically good indicators that higher damping can be used.

- Total length less than 300 feet (90 m)
- Three spans or less
- Abutments designed for sustained soil mobilization
- Normal or slight skew (less than 20 degrees)
- Continuous superstructure without hinges or expansion joints

$$R_D = \frac{1.5}{(40c+1)} + .5 \qquad (4\text{-}4)$$

c = damping ratio (maximum of 10%)

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.

4.3.4 Displacement Magnification For Short Period Structures

Displacements calculated from elastic analysis shall be multiplied by the factor R_d obtained from Equation 4-5 to obtain the design displacements. This magnification applies where the fundamental period of the structure T is less than the characteristic ground motion period corresponding to the peak energy input spectrum.

Values T^* are given in Table 4.3.

$$R_d = \left(1 - \frac{1}{R}\right)\frac{T^*}{T} + \frac{1}{R} \geq 1 \qquad (4\text{-}5)$$

The value of R_d used shall be taken based on the maximum value of R expected in the design of the subject bridge. This value can be obtained by dividing the spectral force by the plastic capacity of the bridge component where plastic hinging is expected.

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 design earthquake moment magnitude.

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 30 m (100 ft) of the ground, as listed in Table 4.4 below.

TABLE 4.4 Site Classes for Response Spectral (UBC, 1997, and NEHRP, 1997)

Site Class	**Description**	**Mean Shear Wave Velocity V_s (ft/sec)**
B (Rock)	Firm to Hard Rock	5,000>V_s≥2,500
C (Soil)	Very Dense Soil and Soft Rock	2,500>V_s≥1,200
D (Soil)	Stiff Soils	1,200>V_s≥600
E (Soil)	Soft Soils	600>V_s (see Note 1)
F (Soil)	Special-Investigation Soils	(see Note 2)

[1]Soil Site Class E could also be any profile with more than 10 ft of soft clay defined as having Plasticity Index PI>20, water content Wn>40%, and undrained shear strength less than 0.5 ksf.

[2]Special-Investigation Soils include the following: collapsible, liquefiable, highly sensitive soils; more than 10 ft of peat or highly organic soils; more than 25 ft of very high plasticity soils with PI. >75%; very thick (more than 120 ft) soft/medium clays.

4.4 COMBINATION OF ORTHOGONAL SEISMIC DISPLACEMENTS

A combination of orthogonal seismic displacements 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 of Section 4.3 shall be combined to form two load cases as follows:

LOAD CASE 1: Seismic displacements on 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 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 minimum support lengths shall be as specified in Section 4.8.

4.6 DESIGN FORCES FOR SEISMIC PERFORMANCE CATEGORY A

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

4.7 DESIGN REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORIES B,C,D

4.7.1 Design Approaches

For design purpose, each structure shall be categorized according to its intended structural action in terms of damage level. Table 4-5 illustrates the selection available to the designer. The following approaches are further defined as follows:

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

Under horizontal loading, a plastic mechanism is intended to develop. The plastic mechanism shall be defined clearly as part of the design. Yielding may occur in areas that are not readily accessible for inspection. Inelastic action is intended to be restricted to flexural plastic hinges in columns and pier walls and inelastic soil deformation behind abutment walls and wing walls. Details and proportions shall ensure large ductility capacity under load reversals without significant strength loss.

(b) Limited-Ductility Design

Under horizontal loading, a plastic mechanism as described for Full-Ductility Structures is intended to develop, but with reduced ductility demands. 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 the same as those required for Full-Ductility Structures.

(c) Limited-Ductility Design in Configuration with Protective Systems

This is a structure incorporating seismic isolation, passive energy dissipating devices, or other mechanical devices to control seismic response. Under horizontal loading, a plastic mechanism may or may not be intended to form. The occurrence of a plastic mechanism shall be determined by analysis.

4.7.2 Global Structure Displacement Requirement

The global structure displacement, Δ_D, is the total displacement at a particular location within the structure or subsystem. The global displacement 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_p respectively. The analytical model for determining the displacement demands shall include as many of the structural characteristics and boundary conditions affecting the structure's global displacements as possible.

Each bridge or frame shall satisfy Equation 4-6.

$$\Delta_D < \Delta_C \qquad (4\text{-}6)$$

where

Δ_D is the displacement along the local principal axes of a ductile member generated by seismic design applied to the structural system.

Δ_C is the corresponding displacement capacity obtained along the same axis as the displacement demand Δ_D.

For SPC B the displacement capacity, Δ_c, of each bent shall be calculated based on:

$$\Delta_c(ft) = \frac{H}{100} * 5.3 * (.0013)^{X} \qquad (4\text{-}6a)$$

where,

$$X = \Lambda \, D/H$$

Λ is a fixity factor for the column equal to:

a. Λ = 1 for fixed-free (pinned on one end).
b. Λ = 2 for fixed top and bottom.

D = Column Diameter (ft.).

H = Height from top of footing to C.G. of superstructure (ft.).

4.7.3 Member Ductility Requirement

Local member displacements such as column displacements, Δ_{col} are defined as the portion of global displacement attributed to the elastic displacement Δ_y and plastic displacement Δ_p of an equivalent member form 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_y and Δ_p.

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

$$\mu_D = 1 + \frac{\Delta_p}{\Delta_y} \qquad (4\text{-}7)$$

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

Single Column Bents	$\mu_D \le 6$
Multi Column Bents	$\mu_D \le 8$
Pier Walls Weak Ductile	$\mu_D \le 6$
Pier Walls Strong Ductile	$\mu_D \le 1$

Pile shafts are treated similar to columns.

4.7.4 Capacity Ductility Requirement

All ductile members in a bridge shall satisfy the displacement ductility capacity requirements specified in Chapter 8.

4.7.5 Capacity Requirement Against P-Δ

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 4-8 is satisfied:

$$P_{dl} \times \Delta_r \le 0.25 \times M_P \qquad (4\text{-}8)$$

Where:

Δ_r = The relative lateral offset between the point of contra-flexure and the end of the plastic hinge. For single column pile shaft where $\Delta_r = \Delta_D - \Delta_S$

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

4.7.6 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 Δ_p 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, or an oversized shaft:

$$L_p = 0.08L + 0.15 f_{ye} d_{bl} \geq 0.3 f_{ye} d_{bl} \quad \text{(in,ksi)} \qquad (4\text{-}9)$$

(b) Prismatic Pile Shafts:

$$L_p = D^* + 0.06H' \qquad (4\text{-}10)$$

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

4.7.7 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 cross sectional dimension in the direction of bending
- The region of column where the moment exceeds 75% of the maximum plastic moment
- The analytical plastic hinge length L_p.

TABLE 4.5 Design Approach

Design Approach	Ductility Demand	Protection Systems	Repairability
Minimal Plastic Action	**Limited** $\mu_D < 2$	**May be Used**	**Not required to Maintain**
Moderate Plastic Action	**Limited** $\mu_D < 4$	**May be Used**	**May require closure or limited usage**
Significant Plastic Action	μ_D **May be higher**	**Not warranted**	**May require closure or removal**

4.8 MINIMUM SEAT WIDTH

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

4.8.1 Seismic Performance Category A

Bridges classified as SPC A shall meet the following requirement: Bearing seats supporting the expansion ends of girders, as shown in Figure 4.2, shall be designed to provide a minimum support length N (in or mm) measured normal to the face of an abutment or pier, not less than specified below.

$$N = 12 + 0.03L_l + 0.12H_h \quad (1 + 0.000125S^2)\ (\text{in}) \tag{4-11}$$

where

- L_l= length, in feet for Equation 4-11 of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck. For hinges within a span, L_l shall be the sum of L_1 and L_2, the distances to either side of the hinge. For single span bridges L_l equals the length of the bridge deck. These lengths are shown in Figure 4.2.
- S= angle of skew of support in degrees, measured from a line normal to the span.

and H_h is given by one of the following:

For abutments, H_h is the average height, in feet for Equation 4-11, of columns supporting the bridge deck to the next expansion joint. $H_h = 0$ for the single span bridges.
For columns/or piers, H_h is the column or pier height in feet for Equation 4-11.
For hinges within a span, H_h is the average height of the adjacent two columns or piers in feet for Equation 4-11.

4.8.2 SEISMIC PERFORMANCE CATEGORY B, C, D

For seismic categories B, C and D, hinge seat or support width 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

$$N \geq \text{the larger of} \begin{cases} \Delta_{p/s} + \Delta_{cr+sh} + \Delta_{temp} + \Delta_{eq} \\ \text{or} \\ 24\ (\text{in})\ \ 600(\text{mm}) \end{cases} \tag{4-12}$$

where

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

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

Δ_{temp} = Displacement due to temperature variation

Δ_{eq} = Design displacement at the expansion joint due to earthquake

$\Delta_{eq} = \sqrt{(\Delta_{D1})^2 + (\Delta_{D2})^2}$

N = Minimum seat width normal to the centerline of bearing

Δ_{eq} = Averaged relative earthquake displacement demand

Δ_{Di} = The larger earthquake displacement demand for each frame calculated by the global analysis for the 2% in 50 years safety evaluation

Δ_{D2} = The smaller earthquake displacement demand for each frame calculated by the global analysis for the 2% in 50 years safety evaluation

4.9 SUPPORT RESTRAINTS FOR SPC B, C AND D

Support restraints shall be provided for longitudinal linkage at expansion joints within the space and at adjacent sections of simply supported superstructures. 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 multiframe system. Restrainer units shall be designed and detailed as described in the following sections.

4.9.1 Expansion Joints within a Span

A restrainer unit with a minimum of five cables shall 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, shall be used at each intermediate expansion joint hinge.

Force demands predicted by elastic response analysis for continuous superstructures are generally too conservative and should not be used.

4.9.2 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 0.25g shall be used as a minimum.

4.9.3 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 expansion.
- Yield indicators shall be used on cable restrainers to facilitate post earthquake investigation

4.9.4 Existing Bridges (Optional Section)

For existing bridges, support restrainers can be considered primary in preventing unseating of simply supported composite steel, precast concrete, and cast-in-place concrete spans.

Also, for existing hinges of reinforced concrete box superstructure, pipe seat extenders can be installed across the hinge diaphragms to achieve the minimum seat width requirement. Cable Restrainers can be used in conjunction with pipe seat extenders but shall not be used as the sole primary means for preventing unseating. Pipes seat extenders shall be designed for the induced moments under single or double curvature depending on how the pipe is anchored. Pipe seat extenders will substantially increase the shear transfer capacity across expansion joints if significant out-of-phase displacements are anticipated. If this is the case, care must be taken to insure stand-alone frame capacity is not adversely affected by the additional demand transmitted between frames through the pipe seat extenders.

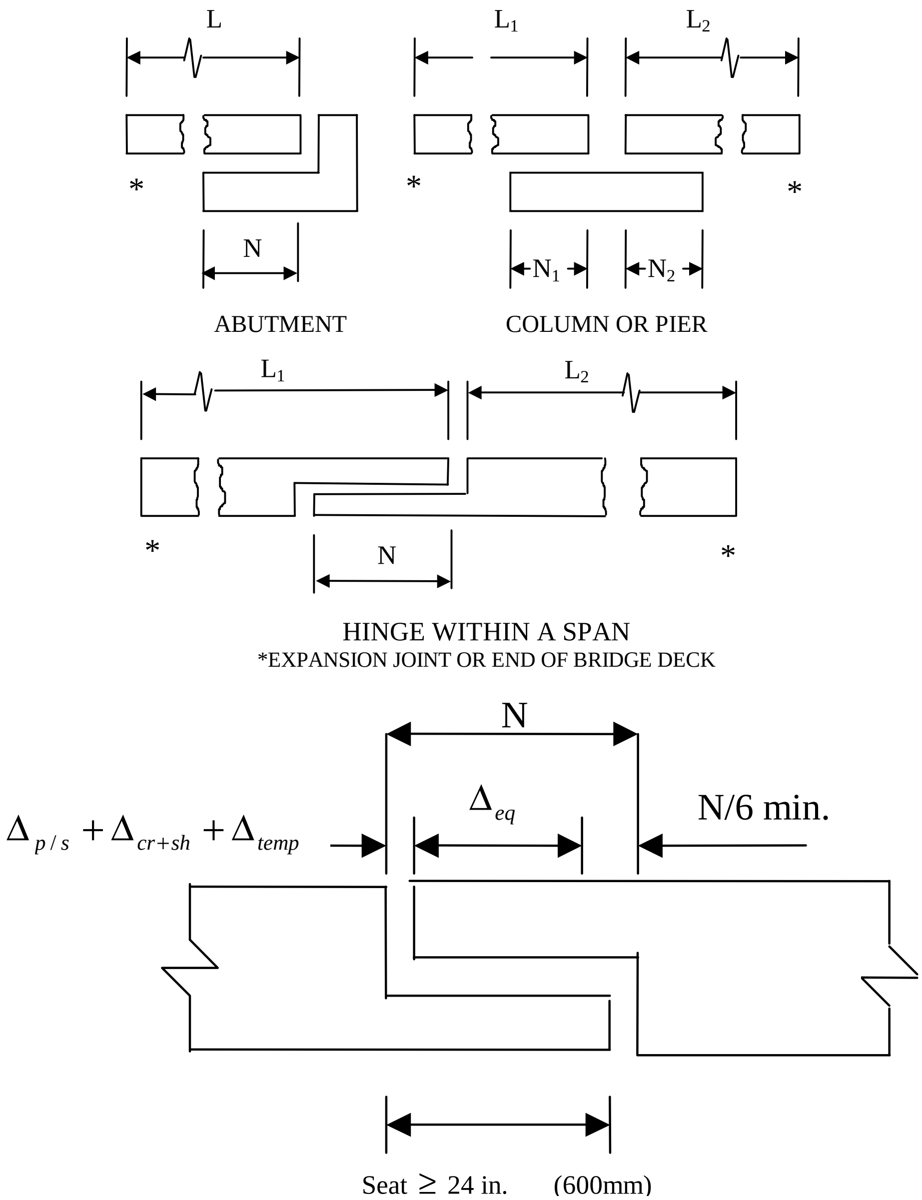


FIGURE 4.2 Dimensions for Minimum Support Length Requirements

4.10 SUPERSTRUCTURE SHEAR KEYS

Shear keys are typically designed to fuse at the SEE hazard level and to stay elastic at the FEE hazard level. 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 can still function elastically at the SEE 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 μ considering concrete placed monolithically for the shear key. The overstrength shear key capacity V_{ok} shall be calculated using:

$$V_{ok} = 1.5V_{nk} \tag{4-13}$$

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

For cases where shear keys are needed to achieve a higher performance criteria at the SEE hazard level, non-linear analysis shall be conducted to derive the design forces of the shear keys.

4.11 REQUIREMENTS FOR TEMPORARY BRIDGES AND STAGED CONSTRUCTION

The requirement that an earthquake shall not cause collapse of all or part of a bridge applies to temporary bridges that are expected to carry traffic and/or pass over routes that carry traffic. It also applies to those bridges that are constructed in stages and expected to carry traffic and/or pass over routes that carry traffic. However, in view of the limited exposure period, the Acceleration Coefficient given in FEE may be used in order to calculate the component elastic forces and displacements. Note that Acceleration Coefficients for construction sites that are close to active faults shall be subject of special study.

The minimum seat-width provisions of Section 4.8 are recommended to temporary bridges and staged construction.

It is recommended that any bridge or partially constructed bridge that is expected to be temporary for more than 5 years be designed using the requirements for permanent structures and shall not use the provisions of Section 4.11.

The final requirements for temporary bridges and staged construction will be directed as required by the particular project on a case by case basis and in consultation with SCDOT.

SECTION 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.

The separate 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 analysis 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. Both the development of the analytical model and the selected analysis procedure are dependent on the seismic hazard, desired performance and the complexity of the bridge. There are various levels or degrees of refinement in the analytical model and analytical procedures that are available for the designer.

5.1.1 General

The entire bridge system 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" describes the overall behavior of the bridge system including the effects of adjacent components, subsystems, or boundary conditions. The term "local" referring to the behavior of an individual component or subsystem constitutes its response independent of the effects of adjacent components, subsystems or boundary conditions.

The analysis of seismic bridge response under these Specifications shall consider global models and local models including those of individual frames and bents, in quantifying earthquake bridge response.

Bridge components displacement capacities shall be greater than displacements demands derived from the "global" analysis or the stand-alone "local" analysis.

The displacement demands of a bridge system consisting of multiple simple spans can be derived using the equivalent static analysis outlined in Section 5.2.2. The requirements of a global analysis in Section 5.1.2 and a stand-alone analysis in Section 5.1.3 need not to be applied in this case. Bridge components displacement capacities shall be greater than displacement demands derived from the Equivalent Static Analysis.

5.1.2 Global Model

A global model is required when it is necessary to capture the response of the entire bridge system. Bridge systems with irregular geometry, in particular curved bridges and skew bridges, multiple transverse expansion joints, massive substructures components, and foundations supported by soft soil can exhibit dynamic response characteristics that are not necessarily intuitively obvious and may not be captured in a separate subsystem analysis.

Linear elastic dynamic analysis procedures are generally 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.

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 versus compression.

In the tension model, the superstructure joints including the abutments are released longitudinally with truss elements connecting the joints to capture the effects of the 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 dictate if both a tension model and a compression model are required. Structures with appreciable superstructure curvature may require additional models, which combine the characteristics identified for the tension and compression models.

Long multi-frame bridges shall be analyzed with multiple elastic models. A single multi-frame model may not be realistic since it cannot account for out-of-phase movement among the frames.

Each multi-frame model should 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.

The boundary frames provide some continuity between adjacent models but are considered redundant and their analytical results are ignored. A massless spring should be attached to the dead end of the boundary frames to represent the stiffness of the remaining structure. Engineering judgement should be exercised when interpreting the deformation results among various sets of frames since the boundary frame method does not fully account for the continuity of the structure.

5.1.3 Stand-Alone " Local" Analysis

Stand-alone analysis shall be performed in both the transverse and longitudinal directions on each individual frame separately.

(a) Transverse Stand-Alone Analysis

Transverse stand-alone frame models shall assume lumped mass at the columns. Hinge spans shall be modeled as rigid elements with half of their mass lumped at the adjacent column, see Figure 5.2. The transverse analysis of end frames shall include a realistic estimate of the abutment stiffness consistent with the abutment's expected performance. The transverse displacement demand at each bent in a frame shall include the effects of rigid body rotation around the frame's center of rigidity.

(b) Longitudinal Stand-Alone Analysis

Longitudinal stand-alone frame models shall include the short side of hinges with a concentrated dead load, and the entire long side of hinges supported by rollers at their ends, see Figure 5.2. Typically the abutment stiffness is ignored in the stand-alone longitudinal model for structures with more than two frames, an overall length greater than 300 feet or significant in plane curvature since the controlling displacement occurs when the frame is moving away from the abutment. A realistic estimate of the abutment stiffness may be incorporated into the stand-alone analysis for single frame tangent bridges and two frame tangent bridges less than 300 feet in length.

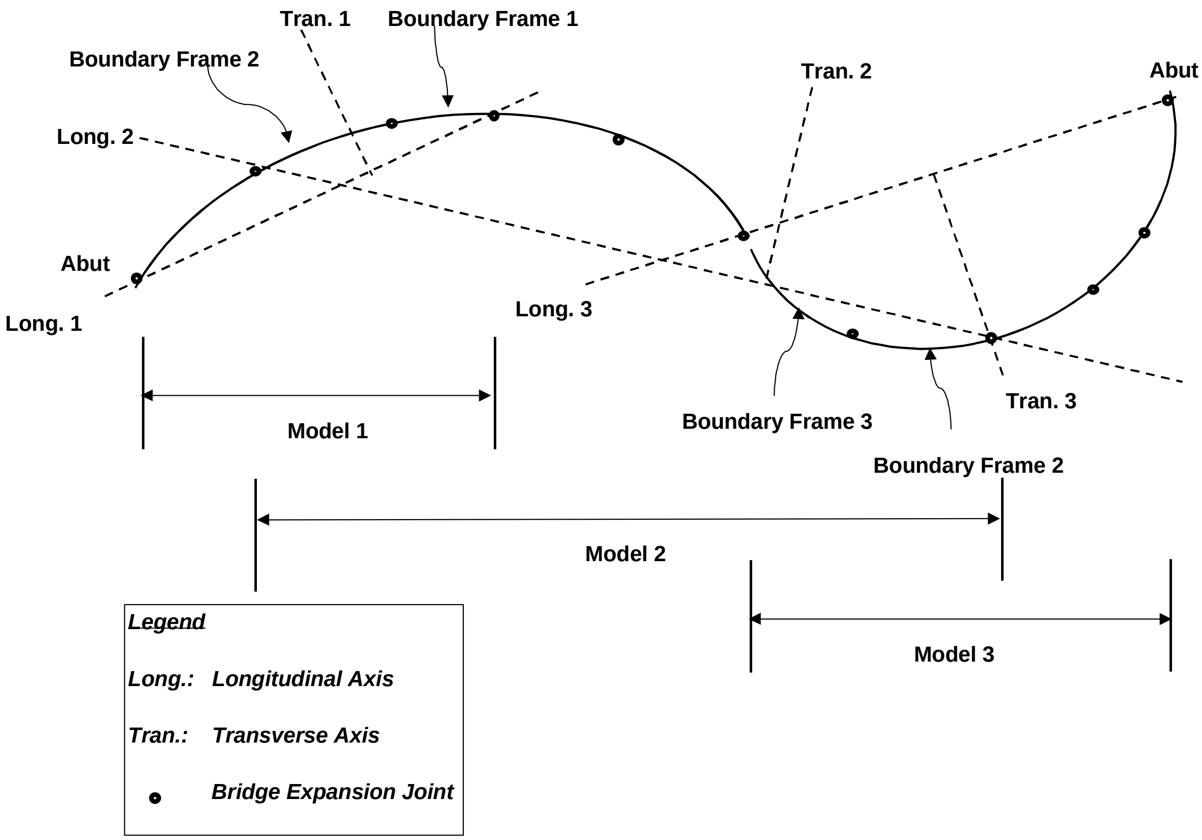
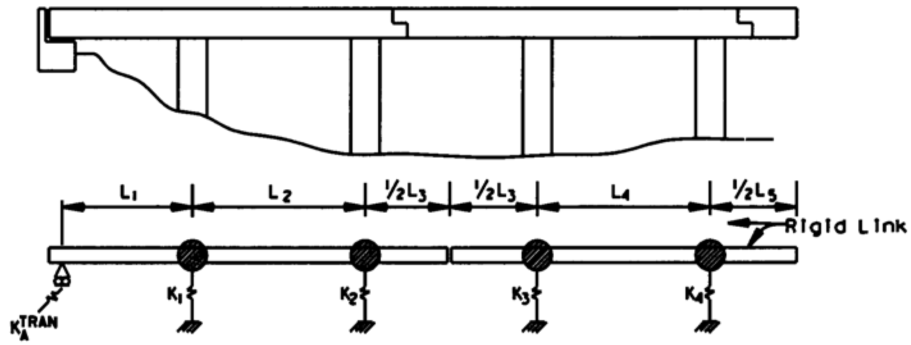
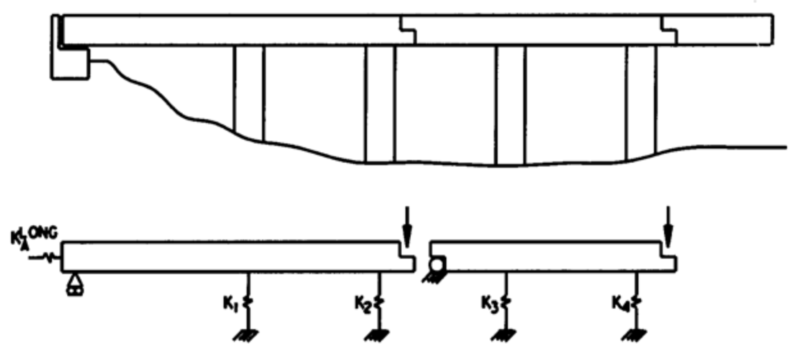


FIGURE 5.1 Elastic Dynamic Analysis Modeling Techniques



Transverse Stand – Alone Model



Longitudinal Stand – Alone Model

FIGURE 5.2 Stand-Alone Analysis

5.2 ANALYTICAL PROCEDURES

5.2.1 General

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

Nonlinear Time History analysis is used for critical or essential bridges. In this type of analysis, components capacities are characterized in the mathematical model used for the seismic response analysis. The procedures mentioned above are presented in more detail.

5.2.2 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 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 equivalent static analysis procedures (see AASHTO, Division I-A, Sections 4.3 and 4.4 respectively).

5.2.3 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 sophistication to estimate the dynamic behavior. A linear elastic multi-modal spectral analysis utilizing the appropriate response spectrum 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 the longitudinal and transverse directions. A minimum of three elements per 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 considerable from the actual force demands on the structure. 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.

Multi-frame analysis shall include a minimum of two boundary frames or one frame and an abutment beyond the frame uner consideration. See Figure 5.1.

5.2.4 Procedure 3 Inelastic Static Analysis (ISA)

ISA, commonly referred to as “push over” analysis, shall be used to determine the reliable displacement capacities of a structure or frame as it reaches its limit of structural stability. ISA 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 pushes the frame laterally, through all possible stages, until the potential collapse mechanism is achieved. Because the analytical model accounts for the redistribution of internal actions as components respond inelastically, ISA 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, the two-dimensional plane frame “push over” 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 overturning on the column axial load and associated member capacities must be considered in the simplified model.

5.2.5 Procedure 4 Time History Analysis 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:

(a) 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 shall be selected and called from on fewer than three recorded events. Appropriate time histories shall have magnitude, fault distances and source mechanisms that are consistent with those that control the design earthquake ground motion. Where three appropriate recorded ground motion time history data sets are not available, appropriate simulated time history data sets may be used to make up the total number required. For each data set. the square root of the sum of the squares (SRSS) of the 5%-damped site-specific spectrum of the scaled horizontal components shall be constructed. The data sets shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5%-damped spectrum for the design earthquake for periods between 0.2T seconds and 1.5T seconds (where T is the fundamental period of the structure).

(b) 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.

(c) 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.

5.3 MATHEMATICAL MODELING USING EDA

5.3.1 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.

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.)

5.3.2 Superstructure

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

5.3.3 Substructure

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

5.4 EFFECTIVE SECTION PROPERTIES

5.4.1 Effective Section Properties For Seismic Analysis

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

Section properties, flexural rigidity, $E_c I$ and torsional rigidity, $G_c J$, 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.

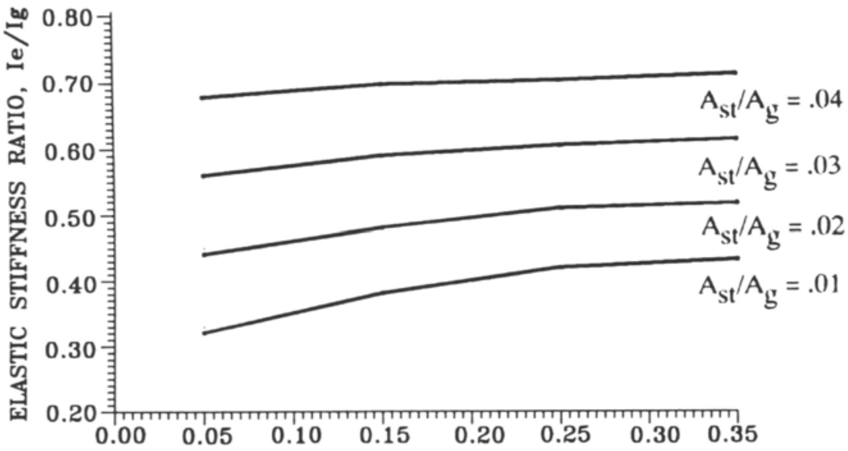
5.4.1.1 I_{eff} For Ductile Members

The cracked flexural stiffness I_{eff} should be used when modeling ductile elements. I_{eff} can be estimated by Figure 5.3 or the initial slope of the $M-\phi$ curve between the origin and the point designating the first reinforcing bar yield as defined by Equation 5.1.

$$E_c \times I_{eff} = \frac{M_y}{\phi_y} \qquad (5\text{-}1)$$

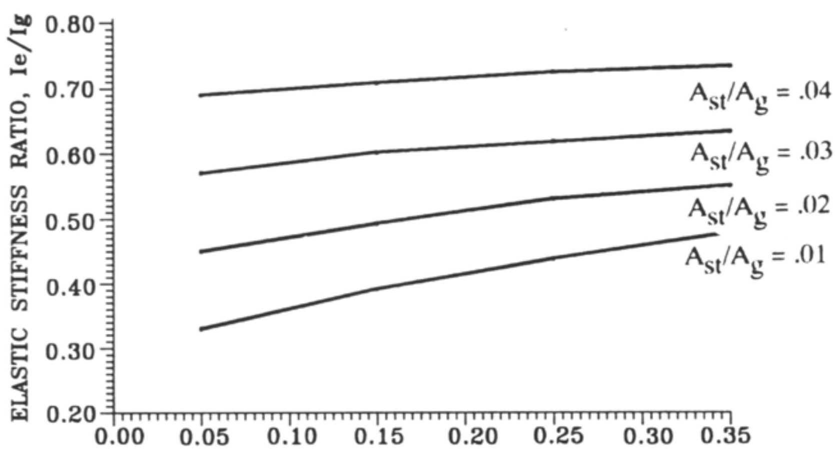
M_y = Moment capacity of the section at first yield of the reinforcing steel.

ϕ_y = Yield Curvature



AXIAL LOAD RATIO $P/f'_{ce}A_g$

a) Circular Sections



AXIAL LOAD RATIO $P/f'_{ce}A_g$

b) Rectoangular Sections

FIGURE 5.3 Effective Stiffness of Cracked Reinforced Concrete Sections

5.4.1.2 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.

5.4.1.3 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.

5.4.2 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 5-2.

$$J_{eff} = 0.2 \times J_g \tag{5-2}$$

SECTION 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 SEISMIC PERFORMANCE CATEGORY A

There are no special seismic design requirements for this category.

6.3 FOUNDATIONS FOR SPC B, C, D

6.3.1 (A) Investigation for SPC B

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, (2) liquefaction, (3) fill settlement, and (4) increases in lateral earth pressure, all as a result of earthquake motions. Seismically induced slope instability in approach fills or cuts may displace abutments and lead to significant differential settlement and structural damage. Fill settlement and abutment displacements due to lateral pressure increases may lead to bridge access problems and structural damage. Liquefaction of saturated cohesionless fills or foundation soils may contribute to slope and abutment instability, and could lead to a loss of foundation-bearing capacity and lateral pile support. Liquefaction failures of the above types have led to many bridge failures during past earthquakes.

6.3.1(B) Investigation for SPC C

In addition to the normal site investigation report, the Engineer may require the submission of a report which shall include, in addition to the requirement of Section 6.3.l (A), a determination of the potential for surface rupture due to faulting or differential ground displacement (lurching), all as a result of earthquake motions.

6.3.1 (C) Investigation for SPC D

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

6.3.2 Foundation Design

For columns with monolithic fixed connections to the footings designed to have a plastic hinge formation at the base, the foundations shall be designed to resist the overstrength column capacity M_o and the associated plastic shear V_o.

The design of pile foundations in competent soil can be greatly simplified using elastic analysis.

A linear distribution of forces (see Figure 6.1) at different rows of piles is considered adequate provided a rigid footing response can be ensured. The rigid response of a footing can be assumed provided:

$$\frac{L_{ftg}}{D_{ftg}} \leq 2.5 \qquad (6\text{-}1)$$

where

L_{ftg} = The cantilever length of the pile cap measured from the face of the column to the edge 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.

The axial demand on an individual pile is found using Equations 6-2 and 6-3.

$$\left.\begin{matrix} C_{(i)}^{pile} \\ T_{(i)}^{pile}\end{matrix}\right\} = \frac{P_c}{N_p} \pm \frac{M_{p(y)}^{col}\times c_{x(i)}}{I_{p.g.(y)}} \pm \frac{M_{p(x)}^{col}\times c_{y(i)}}{I_{p.g.(x)}} \qquad (6\text{-}2)$$

$$I_{p.g.(y)} = \sum n \times c_{y(i)}^2 \qquad I_{p.g.(x)} = \sum n \times c_{x(i)}^2 \qquad (6\text{-}3)$$

Where:

$I_{p.g.}$ = Moment of inertia of the pile group defined by Equation 6.3

$M_{P(y).(x)}^{col}$ = 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

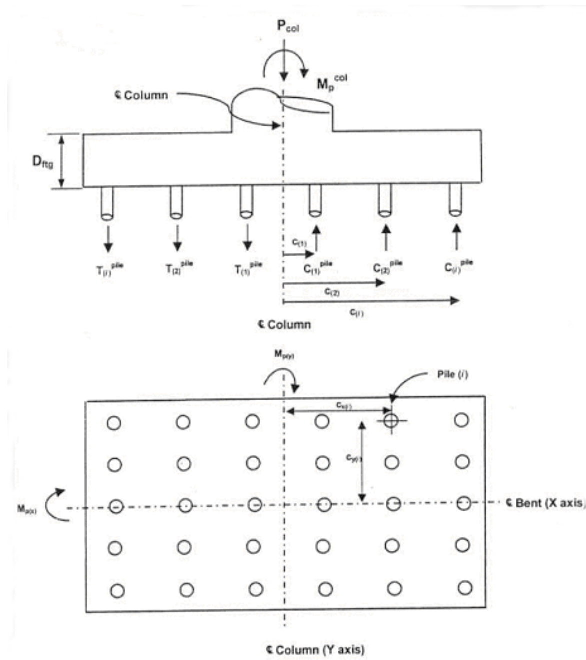


FIGURE 6.1 Simplified Pile Model for Foundations in Competent Soil

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 lateral capacity of the foundation exceeds the lateral demand transmitted by the columns, including the piles capability of sustaining the imposed displacements.

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. For single-column bents, the drift on the column/footing subsystem shall not be greater than 4%. This drift shall be calculated based on the flexibility of the column in addition to the effect of the footing rocking mechanism. For multi-column bents monolithic to the substructure, the effect of rocking shall be examined on the framing configuration of the subject bent.

Multi-column bents that are not monolithic to the superstructure shall be treated in the longitudinal direction similarly to a single column bent.

6.3.3 Special Pile Requirements

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

When reliable uplift pile capacity from skin-friction is present, and when the pile/footing connection detail is present, and when 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. Friction piles may be considered to resist an intermittent but not sustained uplift. For seismic loads, 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 SCDOT standard details.

Footings shall be proportioned to provide the required minimum spacing, clearance and embedment of piles. The minimum center-to-center spacing of piles shall be two times either the diameter or the maximum dimension of the pile, but not less than 3 feet. The spacing shall be increased when required by subsurface conditions. The minimum distance from the center of the exterior pile to the nearest edge of the footing shall be equal to either the diameter or the maximum dimension of the pile, but not less than 1 foot 6 inches. Embedment of concrete and steel piles in the footing cap shall be in accordance with SCDOT Standard Details. In soft soils, piles shall be designed and detailed to accommodate imposed displacements and axial forces based on analysis findings.

6.3.4 Footing Joint Shear

All footing/column moment resisting joints shall be proportioned so the principal stresses meet the following criteria:

Principal compression:

$$p_c \le 0.25 f'_{ce} \qquad (6\text{-}4)$$

Principal tension:

$$p_t \le 12x\sqrt{f'_{ce}} \ \text{(psi)} \qquad (6\text{-}5)$$

Where:

$$p_t = \frac{f_v}{2} - \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \qquad (6\text{-}6)$$

$$p_c = \frac{f_v}{2} + \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \qquad (6\text{-}7)$$

and

$$v_{jv} = \frac{T_{jv}}{B_{eff}^{ftg} \times D_{ftg}} \qquad (6\text{-}8)$$

$$T_{jv} = T_c - \sum T_{(i)}^{pile} \qquad (6\text{-}9)$$

T_c = Column tensile force associated with M_o

$\sum T_{(i)}^{pile}$ = Summation of the hold down force in the tension piles.

$$B_{eff}^{ftg} = \begin{cases} \sqrt{2}xD_c & \text{Circular Column} \\ B_c + D_c & \text{Rectangular Column} \end{cases} \qquad (6\text{-}10)$$

$$f_v = \frac{P_{col}}{A_{jh}^{ftg}} \qquad (6\text{-}11)$$

P_{col} = Column axial force including the effects of overturning

$$A_{jh}^{ftg} = \begin{cases} \left(D_c + D_{ftg}\right)^2 & \text{for Circular Column} \\ \left(B_c + \frac{D_{ftg}}{2}\right)x\left(D_c + \frac{D_{ftg}}{2}\right) & \text{for Rectangular Column} \end{cases} \qquad (6\text{-}12)$$

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 6.2.

6.3.5 Effective Footing Width For Flexure

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.

6.4 ABUTMENTS

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.

Abutment participation in the overall dynamic response of bridge systems shall reflect the structural configuration, the load-transfer mechanism from bridge to abutment system, the effective stiffness and force capacity of wall-soil systems, and the level of expected abutment damage.

The capacity of 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. Soil capacity shall be evaluated based on an applicable passive earth pressure theory.

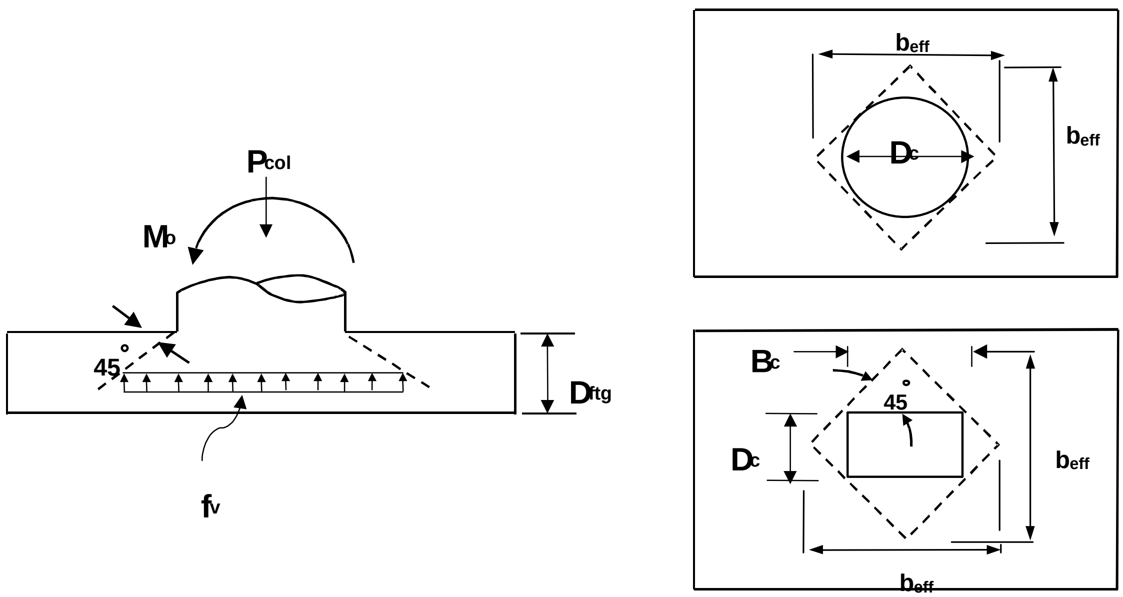


FIGURE 6.2 Effective Joint Width for Footing Joint Stress Calculation

6.4.1 Free-Standing Abutments

For free-standing abutments or retaining walls which may displace horizontally without significant restraint (e.g., superstructure supported by sliding bearings), the pseudostatic Mononobe-Okabe method of analysis is recommended for computing lateral active soil pressures during seismic loading. A seismic coefficient equal to one-half the acceleration coefficient ($k_h = 0.5S_{DS}$) is recommended. The effects of vertical acceleration may be omitted. Abutments should be proportioned to slide rather than tilt, and provisions should be made to accommodate small horizontal seismically induced abutment displacements when minimum damage is desired at abutment supports. Abutment displacements causing a maximum drift of 4% can be tolerated under the No Collapse Performance Criteria.

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

For free-standing abutments which are restrained from horizontal displacement by anchors or batter piles, the magnitudes of seismically-induced lateral earth pressures are higher than those given by the Mononobe-Okabe method of analysis. As a first approximation, it is recommended that the maximum lateral earth pressure be computed by using a seismic coefficient $k_h = 1.0S_{DS}$ in conjunction with the Mononobe-Okabe analysis method.

6.4.2 Monolithic Abutments

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

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

6.4.3 Design Requirements of Abutments

In addition to the provisions outlined in Sections 6.4.1 and 6.4.2, consideration should be given to the mechanism of transfer of superstructure transverse inertial forces to the bridge abutments. Adequate resistance to lateral pressure should be provided by wing walls or abutment keys to minimize lateral abutment displacements where desired.

To minimize potential loss of bridge access arising from abutment damage, monolithic or end diaphragm construction is strongly recommended for short span bridges.

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

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 skewed abutment is needed, sufficient seat width in conjunction with an adequate shear key, shall be designed to ensure against any possible unseating of the bridge superstructure.

SECTION 7

SUPERSTRUCTURE STEEL COMPONENT

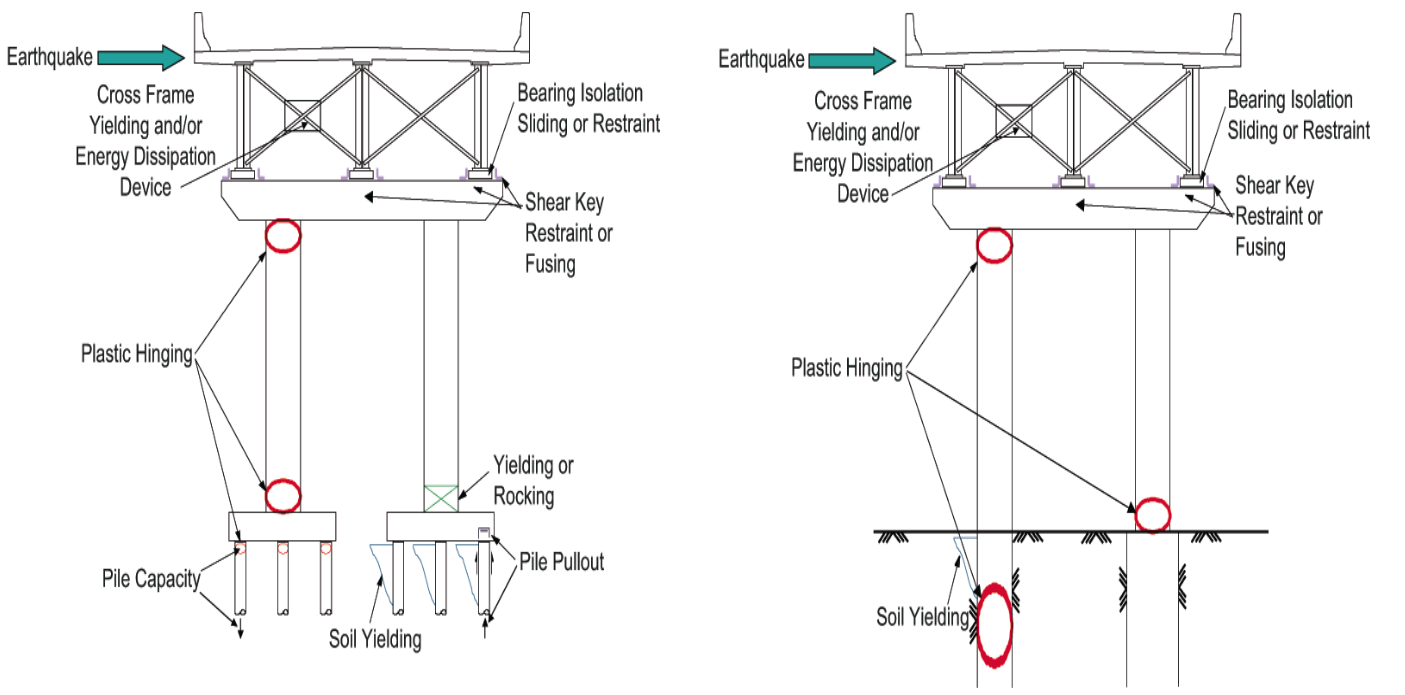
7.1 GENERAL

The Engineer shall demonstrate that a clear, straight-forward load path to the substructure exists and that all components and connections are capable of resisting the imposed seismic load effects consistent with the chosen load path.

The flow of forces (see Figure 7.1) in the assumed load path must be accommodated through all affected components and details 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 confinement of anchor bolts or similar devices in the substructure. The substructure shall also be designed to transmit the imposed force effects into the ground.

The design of end diaphragms and cross-frames shall include analysis cases with horizontal supports at an appropriate number of bearings, consistent with Section 7.7.2.

A viable load path 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:



a) Pile Footing **b) Drilled Shaft**

FIGURE 7.1 Seismic Load Path and Affected Components

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.

The seismic 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.

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 choices:

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.

For Type 1 choice, the designer shall refer back to Section 8 of this document on designing for a ductile substructure. For Type 2 choice, the design of the superstructure is accomplished using a force reduction approach. Those factors are used for the design of transverse bracing members, top laterals and bottom laterals. The reduction factors shown in Table 7.1 shall be used.

Table 7.1 Reduction Factors for Steel Superstructure Bracings

	Essential or Critical Bridges	Normal Bridges
Functional Evaluation	1	2
Safety Evaluation	2	4

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 overstrength 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 7.6).

In this section, reference to an essentially elastic component is used where the force demand to capacity ratio of any member in the superstructure is less than 1.3.

7.3 MATERIALS

Structural steel in ductile components shall meet one of the following AASHTO M270 (ASTM A709) Grade 36, Grade 50 and Grade 50W. The overstrength factor over the specified minimum yield strength of steel shall be taken as 1.5 for A36 and 1.3 for A572 steel.

7.4 MEMBER REQUIREMENTS

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 shall be used unless a lower value can be justified by an appropriate analysis. The slenderness parameter λ_c for axial 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 7.2.

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 7.3. For ductile components, width-thickness ratios shall not exceed the value λ_p as specified in Table 7.3.

7.4.3 Flexural Ductility for Members with Combined Flexural and Axial Load.

Ductility in bending may be utilized only if axial loads are small; 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 check shall be checked using interaction equations following AASHTO-LRFD Method 1998.

TABLE 7.2 Limiting Slenderness Parameters

Member Classification		Limiting Slenderness Parameters (λ_r or λ_b)	
Ductile	Axial Load Dominant $P / P_n \geq M / M_{ns}$	λ_{cp}	0.75
	Flexural Moment Dominant $M / M_n \geq P / P_n$	λ_{bp}	2500/ F_y (AISC 1997)
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}	LTB - AASHTO - LRFD (1998) $750 / \sqrt{F_y}$

The following symbols are used in Table 7.2

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 = $\left(\frac{KL}{r\pi}\right)\sqrt{\frac{F_y}{E}}$ (slenderness parameter of axial load dominant members)
λ_b = $\frac{KL}{r_y}$ (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 7.3 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 AASHTO-LRFD (AASHTO, 1999)

for n=1, $k = (8I_s / bt^3)^{1/3} \le 4.0$ for n=2,3,4 and 5, $k = (14.3I_s / bt^3 n^4)^{1/3} \le 4.0$

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.

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 Diaphragm

A ductile diaphragm can be a concentrically braced frame (CBF) or an eccentrically braced frame (EBF). Special design provisions for CBF or EBF, following the LRFD AISC Seismic Provisions for Structural Steel Buildings 1997, shall be used in addition to requirements stated in this document.

7.5 CONNECTIONS

7.5.1 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.

7.5.2 Yielding of Gross Section for Connectors to Ductile Members

Yielding of the gross section shall be checked (see Section 7.5.6). Fracture in the net section and the block shear rupture failure shall be prevented.

7.5.3 Welded Connections

Partial penetration 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).

7.5.4 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.

7.5.5 Limiting Unsupported Edge Length to Thickness Ratio for a Gusset Plate

The unsupported edge length to thickness ratio of a gusset plate shall satisfy:

$$\frac{L_g}{t} \leq 2.06\sqrt{\frac{E}{F_y}} \qquad (7\text{-}1)$$

where,

L_g = unsupported edge length of a gusset plate (in.)
t = thickness of a gusset plate (in.)

7.5.6 Gusset Plate Tension Strength

The tension strength of the gusset plates shall be:

$$\phi P_n = \phi A_g F_y \leq \phi_{tf} A_n F_u \, or \, \phi_{bs} P_{bs} \qquad (7\text{-}2)$$

where

$$P_{bs} = 0.58 F_y A_{vg} + F_u A_n$$

for $A_{tn} \geq 0.58 A_{vn}$

or

$$P_{bs} = 0.58 F_u A_{vn} + F_y A_{tg}$$

for $A_{tn} \geq 0.58 A_{vn}$

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.

7.5.7 Compression Strength of a Gusset Plate

The nominal compression strength of the gusset plates, P_{ng}, shall be calculated according to Article 6.9.4.1 of AASHTO-LRFD (1998).

7.5.8 In-Plate Moment (Strong Axis)

The nominal moment strength of a gusset plate, M_{ng}, shall be:

$$M_{ng} = S_{sm} F_y \tag{7-3}$$

where

S_{sm} = elastic section modulus about strong axis (in.^{3})

7.5.9 In-Plate Shear Strength

The nominal moment strength of a gusset plate, V_n, shall be:

$$V_n = 0.58 F_y A_{gg} \tag{7-4}$$

where

A_{gg} = gross area of a gusset plate (in.^{2})

7.5.10 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:

$$\frac{M_g}{M_{ng}} + \frac{P_g}{P_{yg}} \leq 1 \tag{7-5}$$

and

$$\left(\frac{V_g}{V_{ng}}\right)^2 + \left(\frac{P_g}{P_{yg}}\right)^2 \leq 1 \tag{7-6}$$

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:

$$\frac{M_g}{M_{Pg}} + \left(\frac{P_g}{P_{yg}}\right)^2 + \frac{\left(\frac{V_g}{V_{pg}}\right)^4}{\left[1-\left(\frac{P_g}{P_{yg}}\right)^2\right]} = 1 \tag{7-7}$$

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)

7.5.11 Fasteners Capacity

Fasteners capacity shall be determined using AASHTO-LRFD (1998) under combined shear and tension.

7.6 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.

7.7 FIXED AND EXPANSION BEARINGS

7.7.1 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.

7.7.2 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 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 designed explicitly as base isolators or fuses have demonstrated a reduction in force transmission to the substructure, and may be used under any circumstances. if used, they shall be designed to accommodate imposed seismic loads.

7.7.3 Load Distribution

The Engineer shall determine the number of bearings needed to resist the loads specified with consideration of the potential for unequal participation due to tolerances, unintended misalignments, the capacity of the individual bearings, and the skew.

Consideration should be given to the use of field-adjustable elements to provide near-simultaneous engagement of the intended number of bearings.

7.7.4 Design and Detail Requirements

Roller bearings or Rocker bearings shall not be used in new bridge construction.

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 sliding surfaces shall be neglected where it contributes to resisting seismic loads, and shall be conservatively estimated (i.e., overestimated) where friction results in the application of force effects to structural components as a result of seismic movements.

Elastomeric expansion bearings shall be provided with adequate seismically resistant anchorage to resist 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.

7.7.5 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.

SECTION 8

REINFORCED CONCRETE COMPONENTS

8.1 GENERAL

Design and construction of cast-in-place monolithic reinforced concrete columns, piers, footings and connections shall conform to the requirements of this section.

8.2 SEISMIC PERFORMANCE CATEGORY A

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

8.3 SEISMIC PERFORMANCE CATEGORIES B, C, D

8.3.1 Force Demands

The design forces shall be the lesser of forces resulting from plastic hinging or unreduced elastic seismic forces in columns or pier walls. Those forces shall be less than capacities established in this section. Initial sizing of columns can be performed using service load combinations.

8.3.1.1 Flexural Demands

The column design moments shall be determined by the idealized plastic capacity of the column's cross section, M_p. The overstrength moment M_{po} defined in Section 8.5, the associated shear V_{po}, and the moment distribution characteristics of the structural system shall determine the design moments for the Essentially Elastic Components connecting to the column.

8.3.1.2 Column Shear Demand

The column shear demand and the shear demand transferred to adjacent components shall be the shear force, V_{po} associated with the overstrength column moment, M_{po}. The designer shall consider all potential plastic hinge locations to insure the maximum possible shear demand has been determined.

8.3.1.3 Pier Wall Shear Demand

The shear demand for pier walls in the weak direction shall be determined in the same manner as the shear demand in columns. The shear demand for pier walls in the strong direction is dependent upon the boundary conditions of the pier wall. If foundation yielding is the selected strategy the pier walls with fixed-fixed end conditions shall be designed to resist the lesser of the shear generated by the unreduced elastic seismic demand or 130% of the ultimate shear capacity of the foundation (based on most probable geotechnical properties). Also, for the same selected strategy pier walls with fixed-pinned end conditions shall be designed for 130% of the lesser of either the shear capacity of the pinned connection or the ultimate capacity of the foundation.

8.3.1.4 Shear Demand For Essentially Elastic Members

The shear demand for essentially elastic members adjacent to plastic hinging locations shall be determined by the distribution of overstrength moments and associated shear when the frame or structure reaches its Collapse Limit State.

8.3.2 Local Ductility Demands

The local displacement ductility demands of Equivalent Members shall be determined based on the analysis adopted in Section 5. The ductility demands shall be lesser than the ductility capacities determined based on parameters established in Section 5 in addition to the maximum allowable ductilities established in Section 4.7.3.

8.4 EXPECTED MATERIAL PROPERTIES

Expected Material Properties shall be used to determine section properties for the purpose of establishing displacement capacity of the bridge system and the ductility capacities of the various components.

8.4.1 Reinforcing Steel

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. A 706 reinforcing steel shall be used with the following expected properties:

$$f_{ye} = 1.1 f_y \qquad (8\text{-}1)$$

where

f_{ye} = the expected yield strength

f_y = the specified minimum yield strength

$$f_{ue} = 1.4\ f_{ye} \qquad (8\text{-}2)$$

where

f_{ue} is the expected tensile strength

The ultimate tensile strain ε_{su} shall be:

ε_{su} = 0.12(0.09 when reduced) for #10 bars or smaller

ε_{su} = 0.09 (0.06 when reduced) for #11 bars or larger

The onset of strain hardening ε_{sh} shall be:

$$\varepsilon_{sh} \begin{cases} 0.0150 \text{ #8 bars or smaller} \\ 0.0125 \text{ #9 bars} \\ 0.0115 \text{ #10 \& #11 bars} \\ 0.0075 \text{ #14 bars} \\ 0.0050 \text{ #18 bars} \end{cases}$$

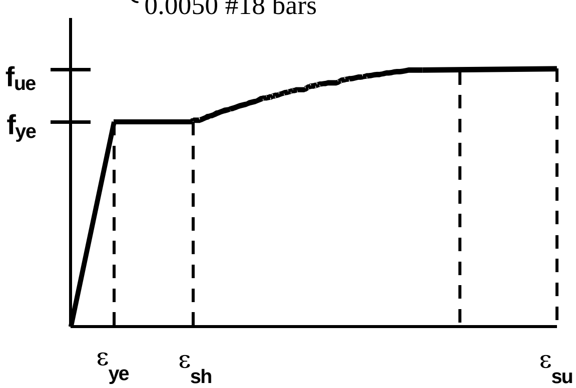


FIGURE 8.1 Steel Stress-Strain Model

8.4.2 Prestressing Steel

Prestressing steel shall be modeled with an idealized nonlinear stress-strain model. The ultimate prestress steel strain ε_{psu} shall not exceed 0.04. Figure 8.2 is an idealized stress-strain model for 7-wire low-relaxation prestressing strand.

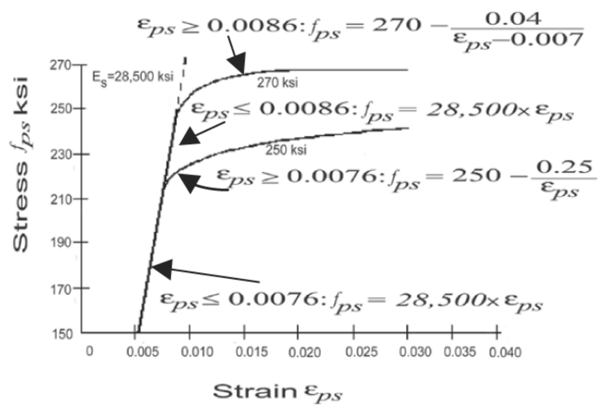


FIGURE 8.2 Prestressing Strand Stress-Strain Model

8.4.3 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 f'_{ce} shall be the greater of:

$$f'_{ce} = \begin{cases} 1.3 \times f'_c \\ \text{or} \\ 5000 \text{ (psi)} \end{cases} \qquad (8\text{-}3)$$

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.

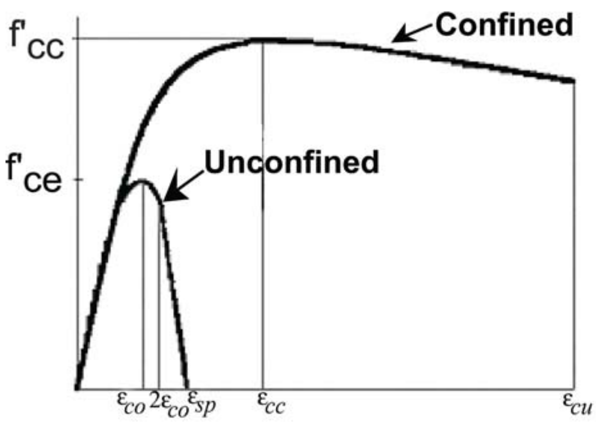


FIGURE 8.3 Concrete Stress-Strain Model

8.5 PLASTIC MOMENT CAPACITY FOR DUCTILE CONCRETE MEMBERS

The plastic moment capacity of all ductile concrete members shall be calculated by $(M-\phi)$ 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 M–ϕ 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 balancing the areas between the actual and the idealized M–ϕ curves beyond the first reinforcing bar yield point. See Figure 8.4.

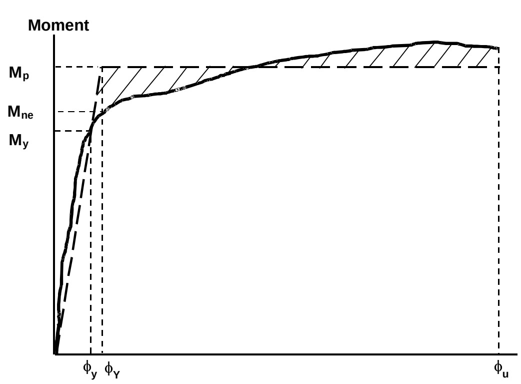


FIGURE 8.4 Moment-Curvature Model

In order to determine force demands on Essentially Elastic Members connected to a yielding, 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

$$M_{po} = 1.2 \times M_p$$

8.6 SHEAR CAPACITY FOR DUCTILE CONCRETE MEMBERS

8.6.1 Nominal Shear Capacity

The seismic shear demand shall be based on the overstrength shear V_{po} associated with the overstrength moment M_{po} defined in Section 8.5. The shear capacity for ductile concrete members shall be based on the nominal material strengths.

$$\phi V_n \geq V_{po} \quad (8\text{-}4)$$

$$V_n = V_c + V_s \quad (8\text{-}5)$$

where

$\phi = 0.85$

8.6.2 Concrete Shear Capacity

The concrete shear capacity of members designed for ductility shall consider the effects of flexure and axial load (see Figure 8.5) as specified in Equation 8-6 through 8-11.

$$V_c = v_c \times A_e \quad (8\text{-}6)$$

$$A_e = 0.8 \times A_g \quad (8\text{-}7)$$

- Inside the plastic hinge zone

$$v_c = \frac{Factor1 \times Factor2}{1000} \times \sqrt{f_c'} \leq 4\sqrt{f_c'}\,(ksi) \quad (8\text{-}8)$$

- Outside the plastic hinge zone

$$v_c = \frac{3 \times Factor2}{1000} \times \sqrt{f_c^{'}} \leq 4\sqrt{f_c^{'}}(ksi) \qquad (8\text{-}9)$$

$$Factor1 = 0.3 \leq \frac{\rho_s f_{yh}}{150} + 3.67 - \mu_D < 3 \qquad (8\text{-}10)$$

$$Factor2 = 1 + \frac{P_c}{2000 \times A_g} < 1.5 \qquad (8\text{-}11)$$

For members whose net axial load is in tension, $v_c = 0$

The displacement ductility μ_D used to derive Factor 1 shall be based on the maximum local ductility demand in either of the principal local member axes.

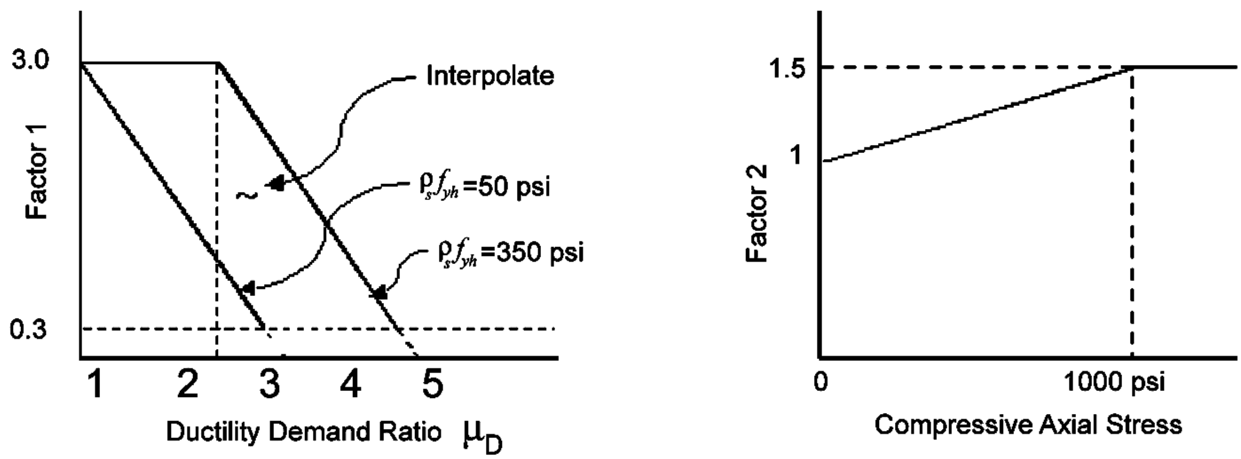


FIGURE 8.5 Concrete Shear Factors (Factor 1 and Factor 2)

8.6.3 Shear Reinforcement Capacity

For confined circular or interlocking core sections, as described in Section 8.6.4.

$$V_s = \frac{\pi}{2} \times \frac{A_v f_{yh} D'}{s} \qquad (8\text{-}12)$$

For pier walls (in the weak direction)

$$V_s = \frac{A_v f_y d}{s} \qquad (8\text{-}13)$$

A_v = Area of hoop or spiral for circular columns or the area of one cross tie for pier walls.

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 8-12.

8.6.5 Maximum Shear Reinforcement

The shear strength provided by the reinforcing steel, V_s, shall not be taken greater than:

$$8x\sqrt{f'_c}\ A_e \text{ (psi)} \qquad (8\text{-}14)$$

8.6.6 Minimum Shear Reinforcement

The area of shear reinforcement, A_v, provided in columns shall be greater than the area required by Equation 8-15. The area of shear reinforcement for each individual core of columns confined by interlocking spirals or hoops shall be greater than the area required by Equation 8-15.

$$A_v \geq 25 \times \frac{D's}{f_{yh}} \quad (\text{in}^2) \qquad (8\text{-}15)$$

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 Sections 8.6.2 & 8.6.3.

8.6.8 Pier Wall Shear Capacity in the Strong Direction

The shear capacity of pier walls in the strong direction shall resist the maximum shear demand specified in Section 8.3.1.3.

$$\phi V_n > V_u \qquad (8\text{-}16)$$

where

$\phi = 0.85$

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 8-17.

$$\frac{V_n}{0.8xA_g} < 8x\sqrt{f'_c} \text{ (psi)} \qquad (8\text{-}17)$$

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 at the bottom of the column.

The requirement for pier wall flexural capacity in the weak direction is similar to a column. Piles where ductility demand is greater than one shall have the same requirement.

8.7.2 Maximum Axial Load In A Ductile Member

The maximum axial load in a column, a pier wall, or a pile where ductility demand is greater than one shall not be greater than when $0.2 f'_{ce} A_g$ where f'_{ce} is expected concrete strength and A_g is the gross cross-sectional area.

8.8 MAXIMUM AND MINIMUM LONGITUDINAL REINFORCEMENT

8.8.1 Maximum Longitudinal Reinforcement

The area of longitudinal reinforcement for compression members shall not exceed the value specified in Equation 8-18.

$$0.04 \times A_g \qquad (8\text{-}18)$$

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 8-19 and 8-20.

$$0.01 \times A_g \quad \text{for Columns} \qquad (8\text{-}19)$$

$$0.005 \times A_g \quad \text{for Pier Walls} \qquad (8\text{-}20)$$

8.8.3 Splicing of Longitudinal Reinforcement in Columns Subject to Ductility Demands

Splicing of longitudinal column reinforcement subject to ductility demands greater than one shall be outside the plastic hinging region as defined in Section 4.7.7. For SPC C and D ultimate strength splicing of reinforcement shall be used by means of mechanical couplers approved by SCDOT.

8.8.4 Minimum Development Length of Reinforcing Steel for Seismic Loads

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.

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).

The anchorage length shall not be reduced by the addition of hooks or mechanical anchorage devices.

8.8.5 Anchorage of Bundled Bars in Ductile Components

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 Flexural Bond Requirement

8.8.6.1 Maximum Bar Diameter

The nominal diameter of longitudinal reinforcement in columns shall not exceed $\dfrac{25 \times \sqrt{f_c'} \times (L - 0.5D_C)}{f_{ye}}$ (in, psi), where L is the length of column from the point of maximum moments to the point of contra-flexure. Where longitudinal bars in columns are bundled, this requirement of 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.

8.8.6.2 Development Length for Column Bars Extended into Shafts

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.

8.8.7 Lateral Reinforcement Inside The Plastic Hinge Region

The volume of lateral reinforcement typically defined by the volumetric ratio, ρ_s provided inside the plastic hinge length shall be sufficient to ensure the column or pier wall has adequate shear capacity and confinement level. ρ_s for circular columns is defined by Equation 8-21.

$$\rho_s = \frac{4A_v}{D' x\ s} \qquad (8\text{-}21)$$

The lateral reinforcement shall be either butt-welded hoops or continuous spiral. Only hoops are permitted in columns having a diameter of three feet or larger. 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.

8.8.8 Lateral Column Reinforcement Outside The Plastic Hinge Region

The volume of lateral reinforcement required outside of the plastic hinge region, shall not be less than 50% of the amount specified in Section 8.8.7.

The lateral reinforcement shall be butt-welded hoops. 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

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 or 8 inches for bundled hoops.

8.8.10 Lateral Reinforcement Requirements For Columns Supported On Oversized Pile Shafts

The volumetric ratio of lateral reinforcement for columns supported on oversized pile shafts shall meet the requirements specified in Sections 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.

8.8.11 Lateral Confinement For Oversized Pile Shafts

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.

8.8.12 Lateral Confinement for Non Oversized Strengthened Pile Shafts

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 oversized shaft confinement can be doubled beyond the column cage termination length.

8.9 REQUIRMENTS FOR ESSENTIALLY ELASTIC 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 an $(M-\phi)$ 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 either the concrete strain reaches a magnitude of 0.005 or the reinforcing steel strain reaches ε_{su} as defined in Section 8.4.1.

8.10 SUPERSTRUCTURE DESIGN FOR LONGITUDINAL DIRECTION

The superstructure shall be designed as an essentially elastic 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 superstructure 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 8-22. 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 8.6).

$$B_{eff} = \begin{cases} D_c + 2xD_s & \text{Box girders \& Solid Superstructures} \\ D_c + D_s & \text{Open Soffit Superstructure} \end{cases} \qquad (8\text{-}22)$$

Additional superstructure width can be considered effective if the designer verifies that the torsional capacity of the cap can distribute the rotational demands beyond the effective width stated in Equation 8-22.

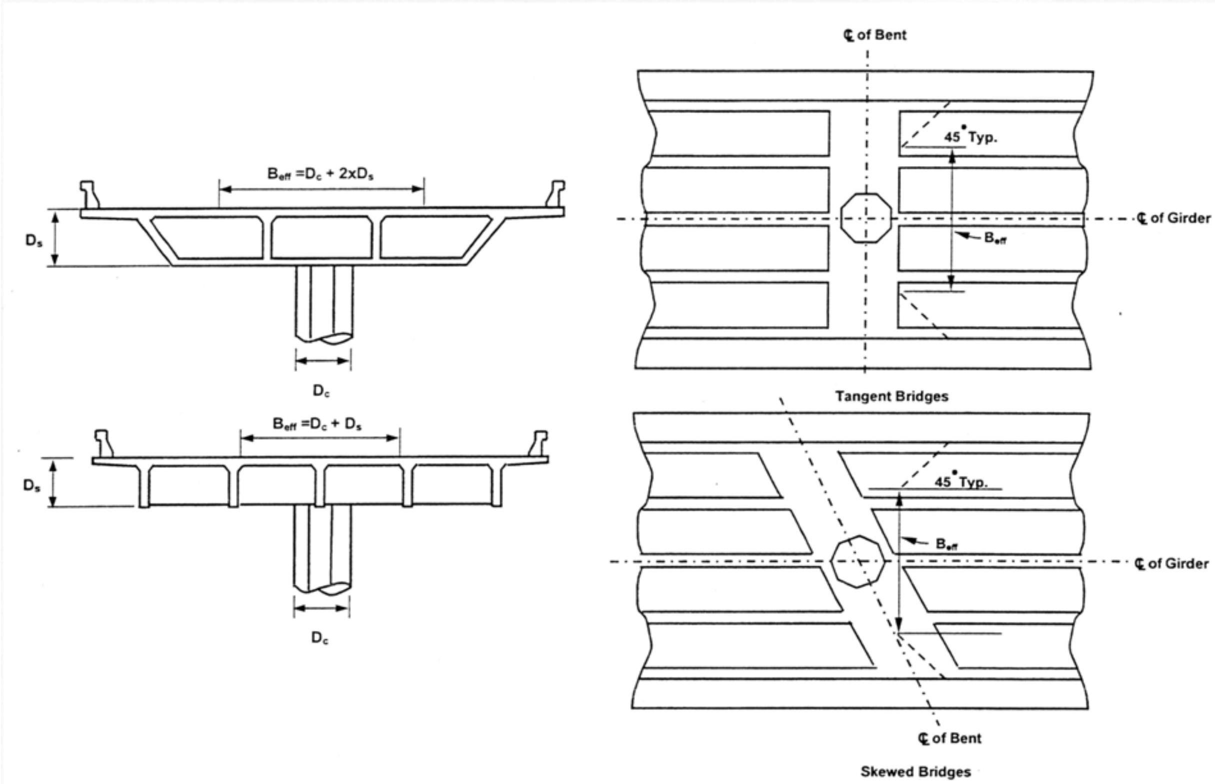


FIGURE 8.6 Effective Superstructure Width

8.11 SUPERSTRUCTURE DESIGN FOR TRANSVERSE DIRECTION (INTEGRAL BENT CAP)

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 8.7. 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 by Equation 8-23, (see Figure 8.7).

$$B_{eff} = B_{cap} + (12xt) \quad (8\text{-}23)$$

t = thickness of the top or bottom slab

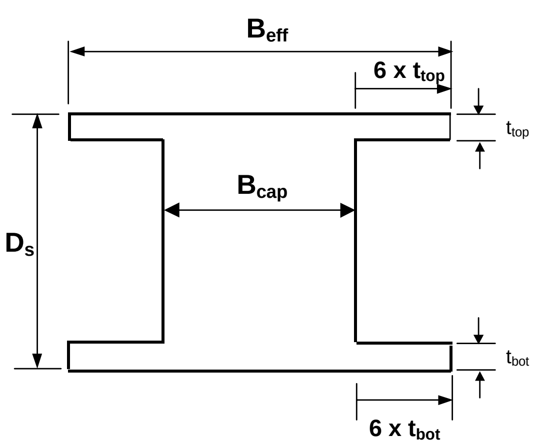


FIGURE 8.7 Effective Bent Cap Width

Cutting off bent cap reinforcement is discouraged. Splicing of reinforcement shall be done using service couplers at a minimum.

8.12 SUPERSTRUCTURE DESIGN FOR NONINTEGRAL BENT CAP

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 an acceleration of 0.4g in addition to the overstrength capacity of shear keys or the elastic seismic force whichever is smaller.

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 Section 7.2.

Drop caps supporting superstructures with expansion joints at the cap shall have sufficient width to prevent unseating. The minimum seat width for non-integral bent caps shall be determined based on Section 4.8. 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

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 8-24 and 8-25

Principal compression:

$$p_c \le 0.25xf'_{ce} \quad (8\text{-}24)$$

Principal tension:

$$p_t \le 12\,x\sqrt{f'_{ce}}\,(psi) \quad (8\text{-}25)$$

8.13.2.1 Minimum Bent Cap Width

The minimum bent cap width required for adequate joint shear transfer is specified in Equation 8-26. Larger cap widths may be required to develop the compression strut outside the joint for large diameter columns.

$$B_{cap} = D_c + 2 \quad \text{(ft)} \quad (8\text{-}26)$$

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.

8.13.4 T Joint Shear Design

8.13.4.1 Principal Stress Definition

The principal tension and compression stresses (see Figure 8.8) in a joint are defined as follows:

$$p_t = \frac{(f_h + f_v)}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} \quad (8\text{-}27)$$

$$p_c = \frac{(f_h + f_v)}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} \quad (8\text{-}28)$$

$$v_{jv} = \frac{T_c}{A_{jv}} \quad (8\text{-}29)$$

$$A_{jv} = l_{ac} \times B_{cap} \quad (8\text{-}30)$$

$$f_v = \frac{P_{ac}}{A_{jh}} \quad (8\text{-}31)$$

$$A_{jh} = (D_c + D_s)B_{cap} \quad (8\text{-}32)$$

$$f_h = \frac{P_b}{B_{cap} \times D_s} \quad (8\text{-}33)$$

Where:

A_{jh} = The effective horizontal area

A_{jv} = The effective vertical joint area

B_{cap} = Bent cap width

D_c = Cross–sectional dimension of column in the direction of bending

l_{ac} = Length of column reinforcement embedded into the bent cap

P_{ac} = The column axial force including the effects of overturning

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

T_c = The column tensile force associated with M_o^{col}

Note: Unless the prestressing is specifically designed to provide horizontal joint compression, f_h can typically be ignored without significantly effecting the principal stress calculation.

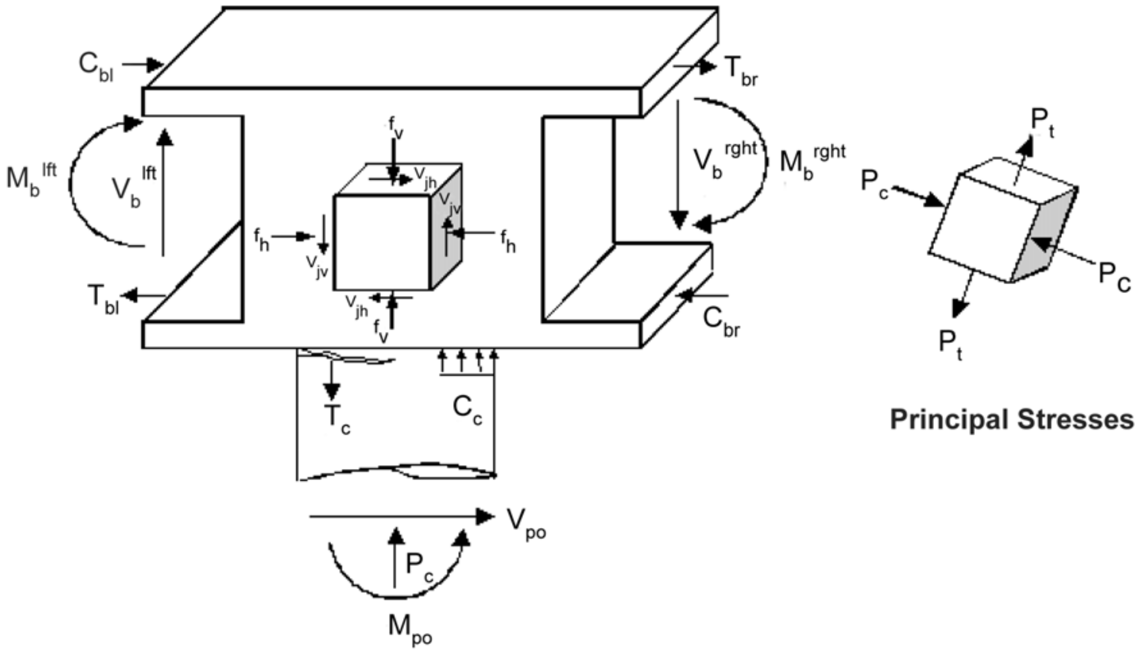


FIGURE 8.8 Joint Shear Stresses in T Joints

8.13.4.2 Minimum Joint Shear Reinforcement

If the principal tension stress, $p_t \leq 3.5 \times \sqrt{f_c'}$ psi, no additional joint reinforcement is required. The volumetric ratio of transverse column reinforcement ρ_s continued into the cap shall not be less than the value specified by Equation 8-34.

$$\rho_{s,\min} = \frac{3.5 \times \sqrt{f_c'}}{f_{yh}} \quad \text{(psi)} \tag{8-34}$$

The reinforcement shall be in the form of spirals, hoops, or intersecting spirals or hoops.

If the principal tension stress, $p_t > 3.5 \times \sqrt{f_c'}$ psi the joint reinforcement specified in Section 8.13.4.3 is required.

8.13.4.3 Joint Shear Reinforcement

A) Vertical Stirrups:

$$A_s^{jv} = 0.2 \times A_{st} \tag{8-35}$$

where,

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 8.9 and 8.10.

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 shall be placed within a distance D_c extending from either side of the column centerline, see Figure 8.11.

$$A_s^{jh} = 0.1 \times A_{st} \tag{8-36}$$

C) Horizontal Side Reinforcement:

Longitudinal side face reinforcement in the bent cap shall be at least equal to the greater of the areas specified in Equation 8-37, see Figure 8.10.

$$A_s^{sf} \geq \begin{cases} 0.1 \times A_{cap}^{top} \\ \text{or} \\ 0.1 \times A_{cap}^{bot} \end{cases} \tag{8-37}$$

where,

A_{cap} = Area of bent cap top or bottom flexural steel

D) J-Dowels

For bents skewed greater than 20°, J-dowels with 135 degree seismic hooks, tied to 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 8-38.

$$A_s^{j-bar} = 0.08xA_{st} \quad (8\text{-}38)$$

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 8.12.

E) Transverse Reinforcement

Transverse reinforcement in the joint region shall consist of hoops with a minimum reinforcement ratio specified by Equation 8-39.

$$\rho_s = 0.005 \quad (8\text{-}39)$$

All vertical column bars shall be extended as close as possible to the top bent cap reinforcement.

F) Main Column Reinforcement

The main Column reinforcement shall extend into the cap as deep as possible to fully develop the compression strut mechanism in the joint.

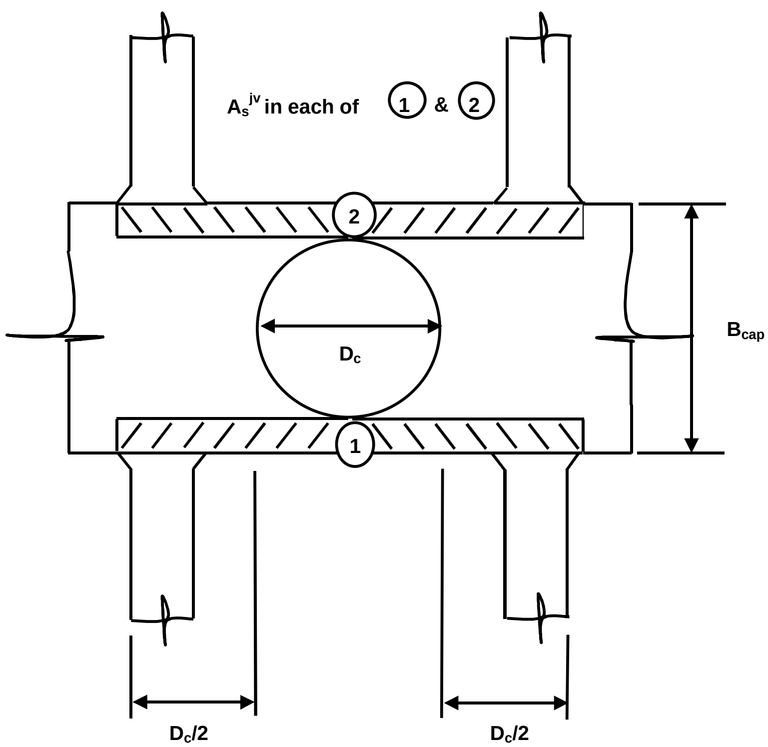
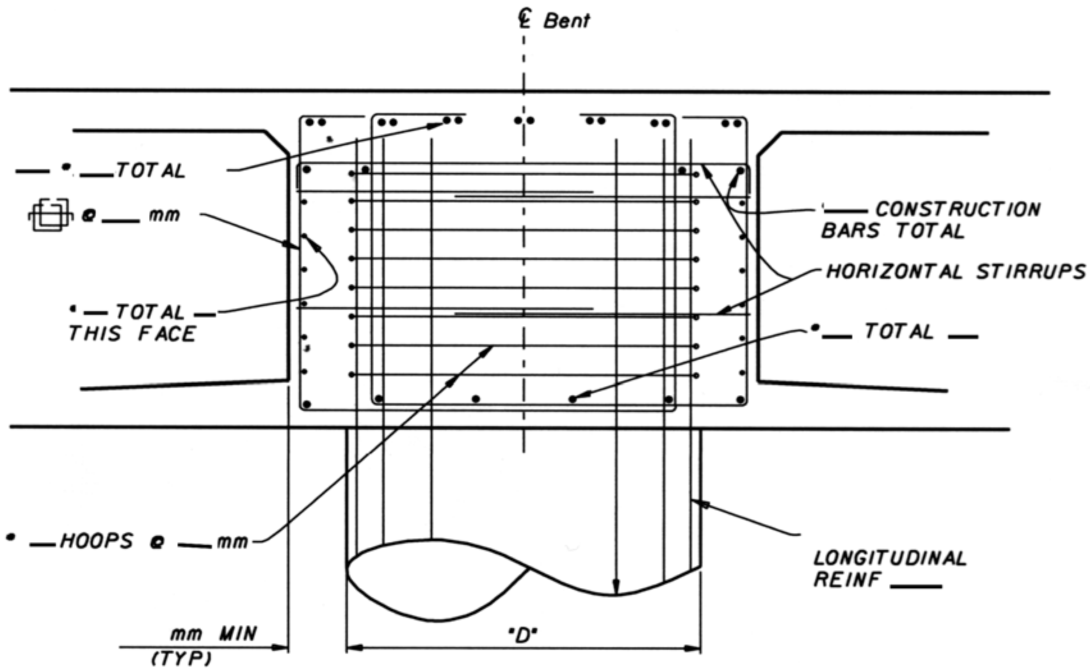
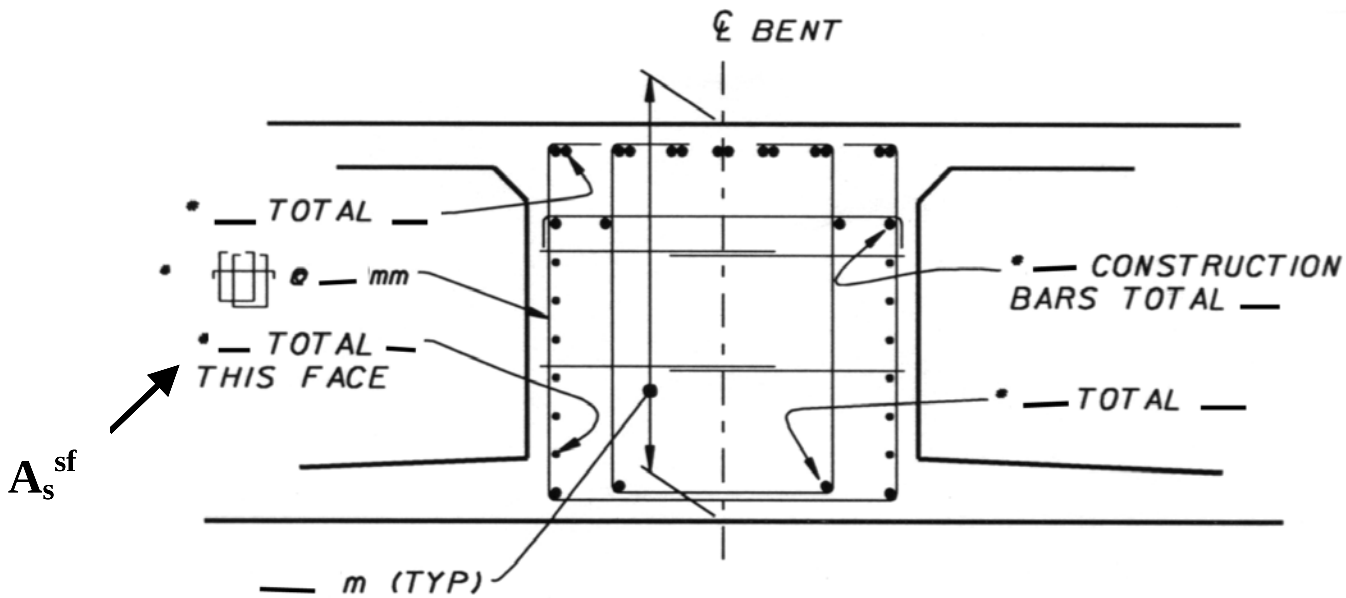


FIGURE 8.9 Location of Vertical Joint Reinforcement



SECTION WITHIN "D" FROM COLUMN ℄



SECTION >D/2 FROM COLUMN FACE

FIGURE 8.10 Joint Shear Reinforcement Details

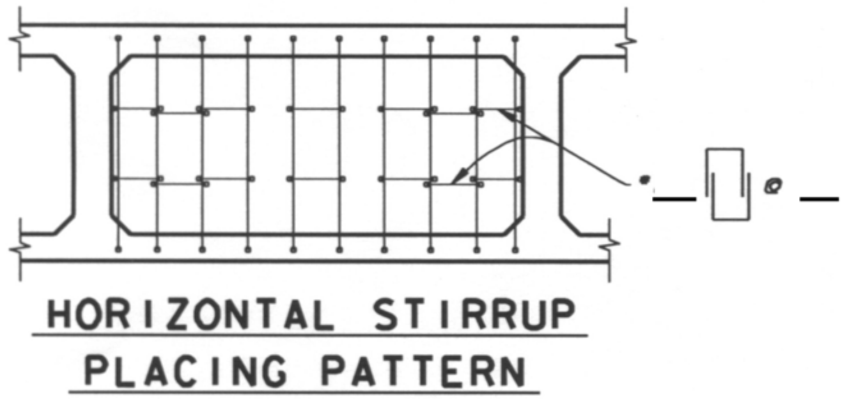


FIGURE 8.11 Location of Horizontal Joint Shear Steel

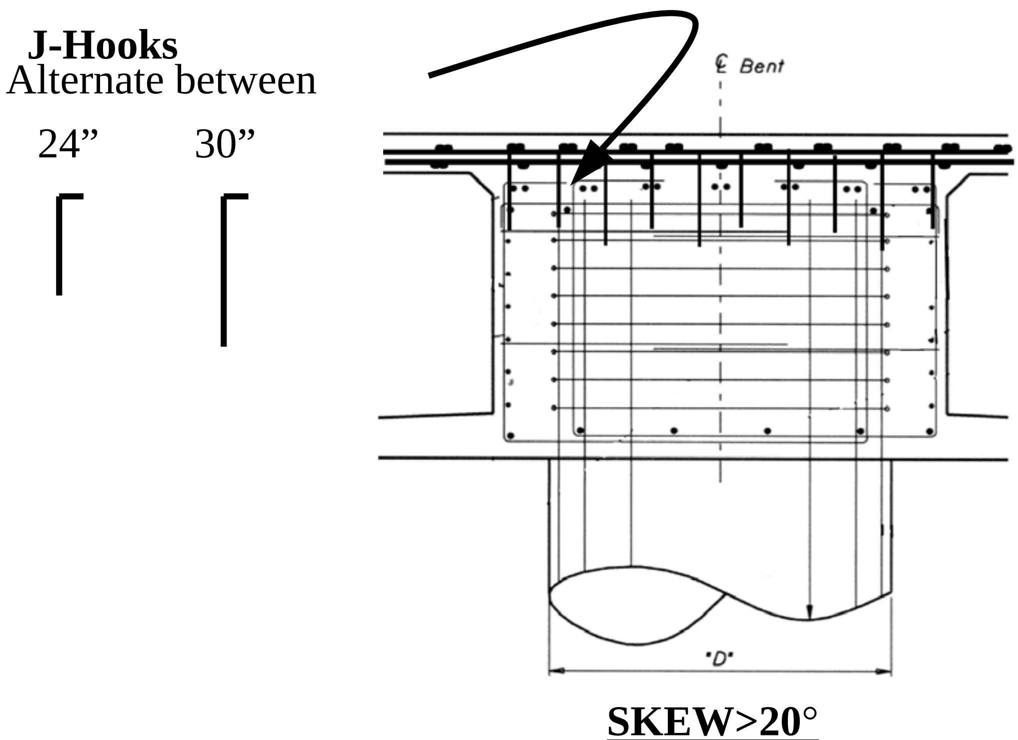


FIGURE 8.12 Additional Joint Shear Steel For Skewed Bridges

8.14 COLUMN FLARES

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. The gap shall be large enough so that it will not close during a seismic event. The gap thickness, G shall be based on the estimated ductility demand and the corresponding plastic hinge rotation. The minimum gap thickness shall be 2 inches (50 mm).

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 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 #6 bars @ 18 inch spacing.

8.15 COLUMN KEY DESIGN

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 steel should be considered in the footing design.

APPENDIX A

ACCELERATION AND DISPLACEMENT TABLES OF DATA

Acceleration Ss=0.15g, FEE(10%/50Years)

A		B		C		D		E	
T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)
0.00	0.05	0.00	0.06	0.00	0.07	0.00	0.10	0.00	0.15
0.07	0.12	0.07	0.15	0.09	0.18	0.10	0.24	0.09	0.38
0.33	0.12	0.33	0.15	0.47	0.18	0.50	0.24	0.47	0.38
0.43	0.09	0.43	0.12	0.57	0.15	0.60	0.20	0.57	0.31
0.53	0.08	0.53	0.09	0.67	0.13	0.70	0.17	0.67	0.26
0.63	0.06	0.63	0.08	0.77	0.11	0.80	0.15	0.77	0.23
0.73	0.05	0.73	0.07	0.87	0.10	0.90	0.13	0.87	0.20
0.83	0.05	0.83	0.06	0.97	0.09	1.00	0.12	0.97	0.18
0.93	0.04	0.93	0.05	1.07	0.08	1.10	0.11	1.07	0.16
1.03	0.04	1.03	0.05	1.17	0.07	1.20	0.10	1.17	0.15
1.13	0.04	1.13	0.04	1.27	0.07	1.30	0.09	1.27	0.14
1.33	0.03	1.33	0.04	1.47	0.06	1.50	0.08	1.47	0.12
1.53	0.03	1.53	0.03	1.67	0.05	1.70	0.07	1.67	0.11
1.73	0.02	1.73	0.03	1.87	0.05	1.90	0.06	1.87	0.09
1.93	0.02	1.93	0.03	2.07	0.04	2.10	0.06	2.07	0.08
2.43	0.02	2.43	0.02	2.57	0.03	2.60	0.05	2.57	0.07
2.93	0.01	2.93	0.02	3.07	0.03	3.10	0.04	3.07	0.06
3.43	0.01	3.43	0.01	3.57	0.02	3.60	0.03	3.57	0.05
3.93	0.01	3.93	0.01	4.07	0.02	4.10	0.03	4.07	0.04

Displacement Ss=0.15g, FEE(10%/50Years)

A		B		C		D		E	
T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.07	0.01	0.04	0.01	0.09	0.02	0.10	0.02	0.09	0.03
0.33	0.13	0.22	0.14	0.47	0.39	0.50	0.59	0.47	0.80
0.43	0.17	0.32	0.20	0.57	0.48	0.60	0.70	0.57	0.97
0.53	0.21	0.42	0.27	0.67	0.56	0.70	0.82	0.67	1.14
0.63	0.25	0.52	0.33	0.77	0.64	0.80	0.94	0.77	1.31
0.73	0.29	0.62	0.39	0.87	0.73	0.90	1.06	0.87	1.48
0.83	0.33	0.72	0.46	0.97	0.81	1.00	1.17	0.97	1.66
0.93	0.37	0.82	0.52	1.07	0.89	1.10	1.29	1.07	1.83
1.03	0.40	0.92	0.58	1.17	0.98	1.20	1.41	1.17	2.00
1.13	0.44	1.02	0.65	1.27	1.06	1.30	1.53	1.27	2.17
1.33	0.52	1.22	0.77	1.47	1.22	1.50	1.76	1.47	2.51
1.53	0.60	1.42	0.90	1.67	1.39	1.70	2.00	1.67	2.85
1.73	0.68	1.62	1.03	1.87	1.56	1.90	2.23	1.87	3.20
1.93	0.76	1.82	1.16	2.07	1.72	2.10	2.47	2.07	3.54
2.43	0.95	2.32	1.47	2.57	2.14	2.60	3.05	2.57	4.40
2.93	1.15	2.82	1.79	3.07	2.56	3.10	3.64	3.07	5.25
3.43	1.34	3.32	2.11	3.57	2.97	3.60	4.23	3.57	6.11
3.93	1.54	3.82	2.43	4.07	3.39	4.10	4.82	4.07	6.97

Acceleration Ss=0.20g, FEE(10%/50Years)

A		B		C		D		E	
T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)
0.00	0.06	0.00	0.08	0.00	0.10	0.00	0.13	0.00	0.20
0.06	0.16	0.06	0.20	0.08	0.24	0.08	0.32	0.08	0.50
0.28	0.16	0.28	0.20	0.39	0.24	0.41	0.32	0.39	0.50
0.38	0.12	0.38	0.15	0.49	0.19	0.51	0.26	0.49	0.40
0.48	0.09	0.48	0.12	0.59	0.16	0.61	0.22	0.59	0.33
0.58	0.08	0.58	0.10	0.69	0.14	0.71	0.19	0.69	0.28
0.68	0.07	0.68	0.08	0.79	0.12	0.81	0.16	0.79	0.25
0.78	0.06	0.78	0.07	0.89	0.11	0.91	0.14	0.89	0.22
0.88	0.05	0.88	0.06	0.99	0.09	1.01	0.13	0.99	0.20
0.98	0.05	0.98	0.06	1.09	0.09	1.11	0.12	1.09	0.18
1.08	0.04	1.08	0.05	1.19	0.08	1.21	0.11	1.19	0.16
1.28	0.03	1.28	0.04	1.39	0.07	1.41	0.09	1.39	0.14
1.48	0.03	1.48	0.04	1.59	0.06	1.61	0.08	1.59	0.12
1.68	0.03	1.68	0.03	1.79	0.05	1.81	0.07	1.79	0.11
1.88	0.02	1.88	0.03	1.99	0.05	2.01	0.07	1.99	0.10
2.38	0.02	2.38	0.02	2.49	0.04	2.51	0.05	2.49	0.08
2.88	0.02	2.88	0.02	2.99	0.03	3.01	0.04	2.99	0.06
3.38	0.01	3.38	0.02	3.49	0.03	3.51	0.04	3.49	0.06
3.88	0.01	3.88	0.01	3.99	0.02	4.01	0.03	3.99	0.05

Displacement Ss=0.20g, FEE(10%/50Years)

A		B		C		D		E	
T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.06	0.00	0.04	0.01	0.08	0.01	0.08	0.02	0.08	0.03
0.28	0.12	0.21	0.16	0.39	0.36	0.41	0.53	0.39	0.73
0.38	0.16	0.31	0.23	0.49	0.45	0.51	0.66	0.49	0.92
0.48	0.20	0.41	0.30	0.59	0.54	0.61	0.79	0.59	1.11
0.58	0.25	0.51	0.38	0.69	0.63	0.71	0.92	0.69	1.29
0.68	0.29	0.61	0.45	0.79	0.73	0.81	1.05	0.79	1.48
0.78	0.33	0.71	0.52	0.89	0.82	0.91	1.18	0.89	1.67
0.88	0.38	0.81	0.60	0.99	0.91	1.01	1.31	0.99	1.86
0.98	0.42	0.91	0.67	1.09	1.00	1.11	1.44	1.09	2.05
1.08	0.46	1.01	0.74	1.19	1.09	1.21	1.57	1.19	2.24
1.28	0.55	1.21	0.89	1.39	1.28	1.41	1.82	1.39	2.62
1.48	0.64	1.41	1.04	1.59	1.46	1.61	2.08	1.59	2.99
1.68	0.72	1.61	1.19	1.79	1.65	1.81	2.34	1.79	3.37
1.88	0.81	1.81	1.33	1.99	1.83	2.01	2.60	1.99	3.75
2.38	1.02	2.31	1.70	2.49	2.29	2.51	3.25	2.49	4.69
2.88	1.24	2.81	2.07	2.99	2.75	3.01	3.89	2.99	5.64
3.38	1.45	3.31	2.43	3.49	3.21	3.51	4.54	3.49	6.58
3.88	1.67	3.81	2.80	3.99	3.67	4.01	5.18	3.99	7.53

Acceleration Ss=0.25g, FEE(10%/50Years)

A		B		C		D		E	
T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)
0.00	0.08	0.00	0.10	0.00	0.12	0.00	0.16	0.00	0.25
0.05	0.20	0.05	0.25	0.07	0.30	0.07	0.40	0.07	0.63
0.24	0.20	0.24	0.25	0.34	0.30	0.36	0.40	0.34	0.63
0.34	0.14	0.34	0.18	0.44	0.23	0.46	0.31	0.44	0.48
0.44	0.11	0.44	0.14	0.54	0.19	0.56	0.26	0.54	0.39
0.54	0.09	0.54	0.11	0.64	0.16	0.66	0.22	0.64	0.33
0.64	0.08	0.64	0.09	0.74	0.14	0.76	0.19	0.74	0.29
0.74	0.06	0.74	0.08	0.84	0.12	0.86	0.17	0.84	0.25
0.84	0.06	0.84	0.07	0.94	0.11	0.96	0.15	0.94	0.23
0.94	0.05	0.94	0.06	1.04	0.10	1.06	0.14	1.04	0.20
1.04	0.05	1.04	0.06	1.14	0.09	1.16	0.12	1.14	0.19
1.24	0.04	1.24	0.05	1.34	0.08	1.36	0.11	1.34	0.16
1.44	0.03	1.44	0.04	1.54	0.07	1.56	0.09	1.54	0.14
1.64	0.03	1.64	0.04	1.74	0.06	1.76	0.08	1.74	0.12
1.84	0.03	1.84	0.03	1.94	0.05	1.96	0.07	1.94	0.11
2.34	0.02	2.34	0.03	2.44	0.04	2.46	0.06	2.44	0.09
2.84	0.02	2.84	0.02	2.94	0.04	2.96	0.05	2.94	0.07
3.34	0.01	3.34	0.02	3.44	0.03	3.46	0.04	3.44	0.06
3.84	0.01	3.84	0.02	3.94	0.03	3.96	0.04	3.94	0.05

Displacement Ss=0.25g, FEE(10%/50Years)

A		B		C		D		E	
T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.05	0.00	0.06	0.05	0.07	0.01	0.07	0.02	0.07	0.03
0.24	0.11	0.28	1.30	0.34	0.34	0.36	0.51	0.34	0.69
0.34	0.16	0.38	1.76	0.44	0.44	0.46	0.65	0.44	0.90
0.44	0.21	0.48	2.22	0.54	0.54	0.56	0.79	0.54	1.11
0.54	0.25	0.58	2.68	0.64	0.65	0.66	0.93	0.64	1.31
0.64	0.30	0.68	3.14	0.74	0.75	0.76	1.07	0.74	1.52
0.74	0.35	0.78	3.60	0.84	0.85	0.86	1.21	0.84	1.73
0.84	0.39	0.88	4.06	0.94	0.95	0.96	1.35	0.94	1.93
0.94	0.44	0.98	4.52	1.04	1.05	1.06	1.49	1.04	2.14
1.04	0.49	1.08	4.98	1.14	1.15	1.16	1.63	1.14	2.35
1.24	0.58	1.28	5.90	1.34	1.35	1.36	1.92	1.34	2.76
1.44	0.68	1.48	6.82	1.54	1.55	1.56	2.20	1.54	3.17
1.64	0.77	1.68	7.74	1.74	1.75	1.76	2.48	1.74	3.59
1.84	0.86	1.88	8.66	1.94	1.96	1.96	2.76	1.94	4.00
2.34	1.10	2.38	10.96	2.44	2.46	2.46	3.47	2.44	5.03
2.84	1.33	2.88	13.26	2.94	2.96	2.96	4.17	2.94	6.06
3.34	1.57	3.38	15.56	3.44	3.47	3.46	4.88	3.44	7.10
3.84	1.80	3.88	17.86	3.94	3.97	3.96	5.58	3.94	8.13

Acceleration Ss=0.30g, FEE(10%/50Years)

A		B		C		D		E	
T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)
0.00	0.10	0.00	0.12	0.00	0.14	0.00	0.19	0.00	0.28
0.04	0.24	0.04	0.30	0.06	0.36	0.07	0.47	0.06	0.70
0.22	0.24	0.22	0.30	0.31	0.36	0.33	0.47	0.32	0.70
0.32	0.16	0.32	0.21	0.41	0.27	0.43	0.36	0.42	0.54
0.42	0.12	0.42	0.16	0.51	0.22	0.53	0.29	0.52	0.44
0.52	0.10	0.52	0.13	0.61	0.18	0.63	0.25	0.62	0.37
0.62	0.08	0.62	0.11	0.71	0.16	0.73	0.21	0.72	0.31
0.72	0.07	0.72	0.09	0.81	0.14	0.83	0.19	0.82	0.28
0.82	0.06	0.82	0.08	0.91	0.12	0.93	0.17	0.92	0.25
0.92	0.06	0.92	0.07	1.01	0.11	1.03	0.15	1.02	0.22
1.02	0.05	1.02	0.06	1.11	0.10	1.13	0.14	1.12	0.20
1.22	0.04	1.22	0.05	1.31	0.08	1.33	0.12	1.32	0.17
1.42	0.04	1.42	0.05	1.51	0.07	1.53	0.10	1.52	0.15
1.62	0.03	1.62	0.04	1.71	0.07	1.73	0.09	1.72	0.13
1.82	0.03	1.82	0.04	1.91	0.06	1.93	0.08	1.92	0.12
2.32	0.02	2.32	0.03	2.41	0.05	2.43	0.06	2.42	0.09
2.82	0.02	2.82	0.02	2.91	0.04	2.93	0.05	2.92	0.08
3.32	0.02	3.32	0.02	3.41	0.03	3.43	0.05	3.42	0.07
3.82	0.01	3.82	0.02	3.91	0.03	3.93	0.04	3.92	0.06

Displacement Ss=0.30g, FEE(10%/50Years)

A		B		C		D		E	
T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.04	0.00	0.06	0.05	0.06	0.01	0.07	0.02	0.06	0.03
0.22	0.11	0.28	1.15	0.31	0.33	0.33	0.51	0.32	0.72
0.32	0.16	0.38	1.56	0.41	0.44	0.43	0.66	0.42	0.95
0.42	0.21	0.48	1.97	0.51	0.55	0.53	0.81	0.52	1.17
0.52	0.26	0.58	2.38	0.61	0.66	0.63	0.97	0.62	1.39
0.62	0.31	0.68	2.80	0.71	0.77	0.73	1.12	0.72	1.62
0.72	0.36	0.78	3.21	0.81	0.88	0.83	1.27	0.82	1.84
0.82	0.42	0.88	3.62	0.91	0.99	0.93	1.43	0.92	2.06
0.92	0.47	0.98	4.03	1.01	1.09	1.03	1.58	1.02	2.29
1.02	0.52	1.08	4.44	1.11	1.20	1.13	1.73	1.12	2.51
1.22	0.62	1.28	5.26	1.31	1.42	1.33	2.04	1.32	2.95
1.42	0.72	1.48	6.08	1.51	1.64	1.53	2.34	1.52	3.40
1.62	0.82	1.68	6.91	1.71	1.85	1.73	2.65	1.72	3.85
1.82	0.92	1.88	7.73	1.91	2.07	1.93	2.95	1.92	4.29
2.32	1.18	2.38	9.78	2.41	2.61	2.43	3.72	2.42	5.41
2.82	1.43	2.88	11.84	2.91	3.16	2.93	4.48	2.92	6.53
3.32	1.69	3.38	13.89	3.41	3.70	3.43	5.24	3.42	7.64
3.82	1.94	3.88	15.95	3.91	4.24	3.93	6.01	3.92	8.76

Acceleration Ss=0.35g, FEE(10%/50Years)

A		B		C		D		E	
T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)
0.00	0.11	0.00	0.14	0.00	0.17	0.00	0.21	0.00	0.31
0.04	0.28	0.04	0.35	0.06	0.42	0.07	0.53	0.07	0.76
0.21	0.28	0.21	0.35	0.30	0.42	0.34	0.53	0.34	0.76
0.31	0.19	0.31	0.24	0.40	0.32	0.44	0.41	0.44	0.59
0.41	0.14	0.41	0.18	0.50	0.25	0.54	0.33	0.54	0.48
0.51	0.12	0.51	0.15	0.60	0.21	0.64	0.28	0.64	0.41
0.61	0.10	0.61	0.12	0.70	0.18	0.74	0.24	0.74	0.35
0.71	0.08	0.71	0.11	0.80	0.16	0.84	0.21	0.84	0.31
0.81	0.07	0.81	0.09	0.90	0.14	0.94	0.19	0.94	0.28
0.91	0.07	0.91	0.08	1.00	0.13	1.04	0.17	1.04	0.25
1.01	0.06	1.01	0.07	1.10	0.12	1.14	0.16	1.14	0.23
1.21	0.05	1.21	0.06	1.30	0.10	1.34	0.13	1.34	0.20
1.41	0.04	1.41	0.05	1.50	0.09	1.54	0.12	1.54	0.17
1.61	0.04	1.61	0.05	1.70	0.08	1.74	0.10	1.74	0.15
1.81	0.03	1.81	0.04	1.90	0.07	1.94	0.09	1.94	0.14
2.31	0.03	2.31	0.03	2.40	0.05	2.44	0.07	2.44	0.11
2.81	0.02	2.81	0.03	2.90	0.04	2.94	0.06	2.94	0.09
3.31	0.02	3.31	0.02	3.40	0.04	3.44	0.05	3.44	0.08
3.81	0.02	3.81	0.02	3.90	0.03	3.94	0.05	3.94	0.07

Displacement Ss=0.35g, FEE(10%/50Years)

A		B		C		D		E	
T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)
0.00	0.11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.04	0.01	0.04	0.01	0.06	0.02	0.07	0.02	0.07	0.04
0.21	0.13	0.21	0.16	0.30	0.38	0.34	0.60	0.34	0.88
0.31	0.18	0.31	0.23	0.40	0.51	0.44	0.77	0.44	1.14
0.41	0.24	0.41	0.30	0.50	0.63	0.54	0.95	0.54	1.40
0.51	0.30	0.51	0.38	0.60	0.76	0.64	1.12	0.64	1.66
0.61	0.36	0.61	0.45	0.70	0.88	0.74	1.30	0.74	1.92
0.71	0.42	0.71	0.52	0.80	1.01	0.84	1.48	0.84	2.17
0.81	0.48	0.81	0.60	0.90	1.13	0.94	1.65	0.94	2.43
0.91	0.54	0.91	0.67	1.00	1.26	1.04	1.83	1.04	2.69
1.01	0.60	1.01	0.74	1.10	1.38	1.14	2.01	1.14	2.94
1.21	0.71	1.21	0.89	1.30	1.63	1.34	2.36	1.34	3.46
1.41	0.83	1.41	1.04	1.50	1.88	1.54	2.71	1.54	3.97
1.61	0.95	1.61	1.19	1.70	2.13	1.74	3.06	1.74	4.49
1.81	1.07	1.81	1.33	1.90	2.38	1.94	3.41	1.94	5.00
2.31	1.36	2.31	1.70	2.40	3.01	2.44	4.30	2.44	6.29
2.81	1.65	2.81	2.07	2.90	3.64	2.94	5.18	2.94	7.58
3.31	1.95	3.31	2.43	3.40	4.26	3.44	6.06	3.44	8.87
3.81	2.24	3.81	2.80	3.90	4.89	3.94	6.94	3.94	10.15

Acceleration Ss=0.40g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)
0.00	0.13	0.00	0.16	0.00	0.19	0.00	0.24	0.00	0.32
0.08	0.32	0.08	0.40	0.10	0.48	0.11	0.59	0.12	0.81
0.38	0.32	0.38	0.40	0.51	0.48	0.56	0.59	0.62	0.81
0.48	0.25	0.48	0.32	0.61	0.41	0.66	0.50	0.72	0.70
0.58	0.21	0.58	0.26	0.71	0.35	0.76	0.44	0.82	0.61
0.68	0.18	0.68	0.22	0.81	0.31	0.86	0.38	0.92	0.55
0.78	0.15	0.78	0.19	0.91	0.27	0.96	0.34	1.02	0.49
0.88	0.14	0.88	0.17	1.01	0.25	1.06	0.31	1.12	0.45
0.98	0.12	0.98	0.15	1.11	0.22	1.16	0.29	1.22	0.41
1.08	0.11	1.08	0.14	1.21	0.21	1.26	0.26	1.32	0.38
1.18	0.10	1.18	0.13	1.31	0.19	1.36	0.24	1.42	0.35
1.38	0.09	1.38	0.11	1.51	0.16	1.56	0.21	1.62	0.31
1.58	0.08	1.58	0.10	1.71	0.15	1.76	0.19	1.82	0.28
1.78	0.07	1.78	0.08	1.91	0.13	1.96	0.17	2.02	0.25
1.98	0.06	1.98	0.08	2.11	0.12	2.16	0.15	2.22	0.23
2.48	0.05	2.48	0.06	2.61	0.10	2.66	0.12	2.72	0.18
2.98	0.04	2.98	0.05	3.11	0.08	3.16	0.10	3.22	0.16
3.48	0.03	3.48	0.04	3.61	0.07	3.66	0.09	3.72	0.14
3.98	0.03	3.98	0.04	4.11	0.06	4.16	0.08	4.22	0.12

Displacement Ss=0.40g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.08	0.02	0.08	0.02	0.10	0.05	0.11	0.07	0.12	0.12
0.38	0.44	0.38	0.55	0.51	1.21	0.56	1.80	0.62	3.06
0.48	0.56	0.48	0.70	0.61	1.48	0.66	2.12	0.72	3.55
0.58	0.68	0.58	0.84	0.71	1.72	0.76	2.45	0.82	4.05
0.68	0.79	0.68	0.99	0.81	1.96	0.86	2.77	0.92	4.54
0.78	0.91	0.78	1.14	0.91	2.20	0.96	3.09	1.02	5.03
0.88	1.03	0.88	1.28	1.01	2.45	1.06	3.42	1.12	5.52
0.98	1.15	0.98	1.43	1.11	2.69	1.16	3.74	1.22	6.02
1.08	1.26	1.08	1.58	1.21	2.93	1.26	4.06	1.32	6.51
1.18	1.38	1.18	1.73	1.31	3.17	1.36	4.38	1.42	7.00
1.38	1.61	1.38	2.02	1.51	3.66	1.56	5.03	1.62	7.98
1.58	1.85	1.58	2.31	1.71	4.15	1.76	5.68	1.82	8.97
1.78	2.08	1.78	2.61	1.91	4.63	1.96	6.32	2.02	9.95
1.98	2.32	1.98	2.90	2.11	5.12	2.16	6.97	2.22	10.94
2.48	2.91	2.48	3.63	2.61	6.33	2.66	8.58	2.72	13.40
2.98	3.49	2.98	4.37	3.11	7.54	3.16	10.20	3.22	15.86
3.48	4.08	3.48	5.10	3.61	8.76	3.66	11.81	3.72	18.32
3.98	4.67	3.98	5.84	4.11	9.97	4.16	13.43	4.22	20.79

Acceleration Ss=0.50g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)
0.00	0.16	0.00	0.20	0.00	0.24	0.00	0.28	0.00	0.34
0.07	0.40	0.07	0.50	0.09	0.60	0.11	0.70	0.13	0.85
0.35	0.40	0.35	0.50	0.47	0.60	0.53	0.70	0.67	0.85
0.45	0.31	0.45	0.39	0.57	0.49	0.63	0.59	0.77	0.74
0.55	0.25	0.55	0.32	0.67	0.42	0.73	0.51	0.87	0.66
0.65	0.22	0.65	0.27	0.77	0.37	0.83	0.45	0.97	0.59
0.75	0.19	0.75	0.23	0.87	0.32	0.93	0.40	1.07	0.53
0.85	0.16	0.85	0.21	0.97	0.29	1.03	0.36	1.17	0.49
0.95	0.15	0.95	0.18	1.07	0.26	1.13	0.33	1.27	0.45
1.05	0.13	1.05	0.17	1.17	0.24	1.23	0.30	1.37	0.42
1.15	0.12	1.15	0.15	1.27	0.22	1.33	0.28	1.47	0.39
1.35	0.10	1.35	0.13	1.47	0.19	1.53	0.24	1.67	0.34
1.55	0.09	1.55	0.11	1.67	0.17	1.73	0.21	1.87	0.31
1.75	0.08	1.75	0.10	1.87	0.15	1.93	0.19	2.07	0.28
1.95	0.07	1.95	0.09	2.07	0.14	2.13	0.17	2.27	0.25
2.45	0.06	2.45	0.07	2.57	0.11	2.63	0.14	2.77	0.21
2.95	0.05	2.95	0.06	3.07	0.09	3.13	0.12	3.27	0.18
3.45	0.04	3.45	0.05	3.57	0.08	3.63	0.10	3.77	0.15
3.95	0.04	3.95	0.04	4.07	0.07	4.13	0.09	4.27	0.13

Displacement Ss=0.50g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.07	0.02	0.07	0.02	0.09	0.05	0.11	0.08	0.13	0.15
0.35	0.48	0.35	0.60	0.47	1.28	0.53	1.89	0.67	3.78
0.45	0.62	0.45	0.77	0.57	1.55	0.63	2.25	0.77	4.34
0.55	0.75	0.55	0.94	0.67	1.83	0.73	2.61	0.87	4.90
0.65	0.89	0.65	1.11	0.77	2.10	0.83	2.97	0.97	5.46
0.75	1.03	0.75	1.28	0.87	2.38	0.93	3.33	1.07	6.02
0.85	1.16	0.85	1.46	0.97	2.65	1.03	3.69	1.17	6.59
0.95	1.30	0.95	1.63	1.07	2.92	1.13	4.05	1.27	7.15
1.05	1.44	1.05	1.80	1.17	3.20	1.23	4.41	1.37	7.71
1.15	1.58	1.15	1.97	1.27	3.47	1.33	4.77	1.47	8.27
1.35	1.85	1.35	2.31	1.47	4.02	1.53	5.49	1.67	9.39
1.55	2.12	1.55	2.65	1.67	4.57	1.73	6.21	1.87	10.51
1.75	2.40	1.75	3.00	1.87	5.12	1.93	6.93	2.07	11.63
1.95	2.67	1.95	3.34	2.07	5.66	2.13	7.65	2.27	12.75
2.45	3.36	2.45	4.20	2.57	7.03	2.63	9.45	2.77	15.56
2.95	4.04	2.95	5.05	3.07	8.40	3.13	11.26	3.27	18.36
3.45	4.73	3.45	5.91	3.57	9.77	3.63	13.06	3.77	21.17
3.95	5.41	3.95	6.77	4.07	11.14	4.13	14.86	4.27	23.97

Acceleration Ss=0.60g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)
0.00	0.19	0.00	0.24	0.00	0.28	0.00	0.32	0.00	0.36
0.07	0.48	0.07	0.60	0.09	0.70	0.10	0.79	0.14	0.90
0.33	0.48	0.33	0.60	0.46	0.70	0.51	0.79	0.71	0.90
0.43	0.37	0.43	0.46	0.56	0.57	0.61	0.66	0.81	0.79
0.53	0.30	0.53	0.38	0.66	0.49	0.71	0.57	0.91	0.70
0.63	0.25	0.63	0.32	0.76	0.42	0.81	0.50	1.01	0.63
0.73	0.22	0.73	0.27	0.86	0.37	0.91	0.44	1.11	0.58
0.83	0.19	0.83	0.24	0.96	0.33	1.01	0.40	1.21	0.53
0.93	0.17	0.93	0.21	1.06	0.30	1.11	0.36	1.31	0.49
1.03	0.15	1.03	0.19	1.16	0.28	1.21	0.33	1.41	0.45
1.13	0.14	1.13	0.18	1.26	0.25	1.31	0.31	1.51	0.42
1.33	0.12	1.33	0.15	1.46	0.22	1.51	0.27	1.71	0.37
1.53	0.10	1.53	0.13	1.66	0.19	1.71	0.23	1.91	0.33
1.73	0.09	1.73	0.12	1.86	0.17	1.91	0.21	2.11	0.30
1.93	0.08	1.93	0.10	2.06	0.16	2.11	0.19	2.31	0.28
2.43	0.07	2.43	0.08	2.56	0.13	2.61	0.15	2.81	0.23
2.93	0.05	2.93	0.07	3.06	0.10	3.11	0.13	3.31	0.19
3.43	0.05	3.43	0.06	3.56	0.09	3.61	0.11	3.81	0.17
3.93	0.04	3.93	0.05	4.06	0.08	4.11	0.10	4.31	0.15

Displacement Ss=0.60g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.07	0.02	0.07	0.03	0.09	0.06	0.10	0.08	0.14	0.18
0.33	0.52	0.33	0.65	0.46	1.44	0.51	1.98	0.71	4.45
0.43	0.68	0.43	0.85	0.56	1.75	0.61	2.37	0.81	5.08
0.53	0.84	0.53	1.04	0.66	2.07	0.71	2.76	0.91	5.71
0.63	0.99	0.63	1.24	0.76	2.38	0.81	3.15	1.01	6.33
0.73	1.15	0.73	1.44	0.86	2.69	0.91	3.54	1.11	6.96
0.83	1.31	0.83	1.63	0.96	3.01	1.01	3.93	1.21	7.59
0.93	1.46	0.93	1.83	1.06	3.32	1.11	4.33	1.31	8.21
1.03	1.62	1.03	2.02	1.16	3.63	1.21	4.72	1.41	8.84
1.13	1.77	1.13	2.22	1.26	3.95	1.31	5.11	1.51	9.47
1.33	2.09	1.33	2.61	1.46	4.57	1.51	5.89	1.71	10.72
1.53	2.40	1.53	3.00	1.66	5.20	1.71	6.68	1.91	11.97
1.73	2.71	1.73	3.39	1.86	5.82	1.91	7.46	2.11	13.22
1.93	3.03	1.93	3.78	2.06	6.45	2.11	8.24	2.31	14.48
2.43	3.81	2.43	4.76	2.56	8.02	2.61	10.20	2.81	17.61
2.93	4.59	2.93	5.74	3.06	9.58	3.11	12.16	3.31	20.74
3.43	5.38	3.43	6.72	3.56	11.15	3.61	14.11	3.81	23.87
3.93	6.16	3.93	7.70	4.06	12.72	4.11	16.07	4.31	27.01

Acceleration Ss=0.80g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)
0.00	0.26	0.00	0.32	0.00	0.35	0.00	0.38	0.00	0.36
0.06	0.64	0.06	0.80	0.09	0.86	0.10	0.94	0.16	0.91
0.31	0.64	0.31	0.80	0.45	0.86	0.50	0.94	0.82	0.91
0.41	0.48	0.41	0.61	0.55	0.71	0.60	0.79	0.92	0.81
0.51	0.39	0.51	0.49	0.65	0.60	0.70	0.68	1.02	0.73
0.61	0.33	0.61	0.41	0.75	0.52	0.80	0.59	1.12	0.67
0.71	0.28	0.71	0.35	0.85	0.46	0.90	0.53	1.22	0.61
0.81	0.25	0.81	0.31	0.95	0.41	1.00	0.47	1.32	0.57
0.91	0.22	0.91	0.27	1.05	0.37	1.10	0.43	1.42	0.53
1.01	0.20	1.01	0.25	1.15	0.34	1.20	0.39	1.52	0.49
1.11	0.18	1.11	0.22	1.25	0.31	1.30	0.36	1.62	0.46
1.31	0.15	1.31	0.19	1.45	0.27	1.50	0.32	1.82	0.41
1.51	0.13	1.51	0.17	1.65	0.24	1.70	0.28	2.02	0.37
1.71	0.12	1.71	0.15	1.85	0.21	1.90	0.25	2.22	0.34
1.91	0.10	1.91	0.13	2.05	0.19	2.10	0.23	2.42	0.31
2.41	0.08	2.41	0.10	2.55	0.15	2.60	0.18	2.92	0.26
2.91	0.07	2.91	0.09	3.05	0.13	3.10	0.15	3.42	0.22
3.41	0.06	3.41	0.07	3.55	0.11	3.60	0.13	3.92	0.19
3.91	0.05	3.91	0.06	4.05	0.10	4.10	0.12	4.42	0.17

Displacement Ss=0.80g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)
0.00	0.26	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.06	0.02	0.06	0.03	0.09	0.07	0.10	0.09	0.16	0.24
0.31	0.61	0.31	0.76	0.45	1.70	0.50	2.34	0.82	6.04
0.41	0.81	0.41	1.01	0.55	2.08	0.60	2.80	0.92	6.77
0.51	1.00	0.51	1.25	0.65	2.46	0.70	3.27	1.02	7.50
0.61	1.20	0.61	1.50	0.75	2.84	0.80	3.73	1.12	8.24
0.71	1.39	0.71	1.74	0.85	3.22	0.90	4.20	1.22	8.97
0.81	1.59	0.81	1.99	0.95	3.60	1.00	4.66	1.32	9.71
0.91	1.79	0.91	2.23	1.05	3.98	1.10	5.13	1.42	10.44
1.01	1.98	1.01	2.48	1.15	4.36	1.20	5.59	1.52	11.18
1.11	2.18	1.11	2.72	1.25	4.74	1.30	6.06	1.62	11.91
1.31	2.57	1.31	3.21	1.45	5.50	1.50	6.99	1.82	13.38
1.51	2.96	1.51	3.70	1.65	6.26	1.70	7.92	2.02	14.85
1.71	3.35	1.71	4.19	1.85	7.02	1.90	8.85	2.22	16.31
1.91	3.74	1.91	4.68	2.05	7.78	2.10	9.78	2.42	17.78
2.41	4.72	2.41	5.90	2.55	9.68	2.60	12.10	2.92	21.45
2.91	5.70	2.91	7.13	3.05	11.58	3.10	14.43	3.42	25.12
3.41	6.68	3.41	8.35	3.55	13.48	3.60	16.75	3.92	28.79
3.91	7.66	3.91	9.57	4.05	15.37	4.10	19.08	4.42	32.46

Acceleration Ss=1.00g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)
0.00	0.32	0.00	0.40	0.00	0.40	0.00	0.44	0.00	0.36
0.06	0.80	0.06	1.00	0.09	1.00	0.10	1.10	0.19	0.90
0.30	0.80	0.30	1.00	0.45	1.00	0.49	1.10	0.93	0.90
0.40	0.60	0.40	0.75	0.55	0.82	0.59	0.91	1.03	0.81
0.50	0.48	0.50	0.60	0.65	0.69	0.69	0.78	1.13	0.74
0.60	0.40	0.60	0.50	0.75	0.60	0.79	0.68	1.23	0.68
0.70	0.34	0.70	0.43	0.85	0.53	0.89	0.61	1.33	0.63
0.80	0.30	0.80	0.38	0.95	0.47	0.99	0.54	1.43	0.59
0.90	0.27	0.90	0.33	1.05	0.43	1.09	0.50	1.53	0.55
1.00	0.24	1.00	0.30	1.15	0.39	1.19	0.45	1.63	0.51
1.10	0.22	1.10	0.27	1.25	0.36	1.29	0.42	1.73	0.48
1.30	0.18	1.30	0.23	1.45	0.31	1.49	0.36	1.93	0.43
1.50	0.16	1.50	0.20	1.65	0.27	1.69	0.32	2.13	0.39
1.70	0.14	1.70	0.18	1.85	0.24	1.89	0.29	2.33	0.36
1.90	0.13	1.90	0.16	2.05	0.22	2.09	0.26	2.53	0.33
2.40	0.10	2.40	0.13	2.55	0.18	2.59	0.21	3.03	0.28
2.90	0.08	2.90	0.10	3.05	0.15	3.09	0.17	3.53	0.24
3.40	0.07	3.40	0.09	3.55	0.13	3.59	0.15	4.03	0.21
3.90	0.06	3.90	0.08	4.05	0.11	4.09	0.13	4.53	0.19

Displacement Ss=1.00g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)
0.00	0.32	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.06	0.03	0.06	0.04	0.09	0.08	0.10	0.10	0.19	0.31
0.30	0.70	0.30	0.88	0.45	1.98	0.49	2.59	0.93	7.67
0.40	0.94	0.40	1.17	0.55	2.42	0.59	3.12	1.03	8.50
0.50	1.17	0.50	1.47	0.65	2.86	0.69	3.65	1.13	9.32
0.60	1.41	0.60	1.76	0.75	3.30	0.79	4.18	1.23	10.14
0.70	1.64	0.70	2.06	0.85	3.74	0.89	4.71	1.33	10.96
0.80	1.88	0.80	2.35	0.95	4.18	0.99	5.24	1.43	11.78
0.90	2.11	0.90	2.64	1.05	4.62	1.09	5.77	1.53	12.61
1.00	2.35	1.00	2.94	1.15	5.07	1.19	6.29	1.63	13.43
1.10	2.58	1.10	3.23	1.25	5.51	1.29	6.82	1.73	14.25
1.30	3.05	1.30	3.82	1.45	6.39	1.49	7.88	1.93	15.90
1.50	3.52	1.50	4.40	1.65	7.27	1.69	8.94	2.13	17.54
1.70	3.99	1.70	4.99	1.85	8.15	1.89	9.99	2.33	19.18
1.90	4.46	1.90	5.58	2.05	9.03	2.09	11.05	2.53	20.83
2.40	5.64	2.40	7.05	2.55	11.23	2.59	13.69	3.03	24.94
2.90	6.81	2.90	8.52	3.05	13.43	3.09	16.34	3.53	29.05
3.40	7.99	3.40	9.98	3.55	15.64	3.59	18.98	4.03	33.16
3.90	9.16	3.90	11.45	4.05	17.84	4.09	21.62	4.53	37.27

Acceleration Ss=1.25g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)
0.00	0.40	0.00	0.50	0.00	0.50	0.00	0.50		
0.06	1.00	0.06	1.25	0.09	1.25	0.10	1.25		
0.32	1.00	0.32	1.25	0.45	1.25	0.51	1.25		
0.42	0.76	0.42	0.95	0.55	1.02	0.61	1.05		
0.52	0.62	0.52	0.77	0.65	0.86	0.71	0.90		
0.62	0.52	0.62	0.65	0.75	0.75	0.81	0.79		
0.72	0.44	0.72	0.56	0.85	0.66	0.91	0.70		
0.82	0.39	0.82	0.49	0.95	0.59	1.01	0.63		
0.92	0.35	0.92	0.43	1.05	0.53	1.11	0.58		
1.02	0.31	1.02	0.39	1.15	0.49	1.21	0.53		
1.12	0.29	1.12	0.36	1.25	0.45	1.31	0.49		
1.32	0.24	1.32	0.30	1.45	0.39	1.51	0.42		
1.52	0.21	1.52	0.26	1.65	0.34	1.71	0.37		
1.72	0.19	1.72	0.23	1.85	0.30	1.91	0.33		
1.92	0.17	1.92	0.21	2.05	0.27	2.11	0.30		
2.42	0.13	2.42	0.17	2.55	0.22	2.61	0.25		
2.92	0.11	2.92	0.14	3.05	0.18	3.11	0.21		
3.42	0.09	3.42	0.12	3.55	0.16	3.61	0.18		
3.92	0.08	3.92	0.10	4.05	0.14	4.11	0.16		

Displacement Ss=1.25g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)
0.00	0.40	0.00	0.00	0.00	0.00	0.00	0.00		
0.06	0.04	0.06	0.05	0.09	0.10	0.10	0.13		
0.32	1.00	0.32	1.25	0.45	2.46	0.51	3.21		
0.42	1.32	0.42	1.64	0.55	3.00	0.61	3.83		
0.52	1.63	0.52	2.04	0.65	3.55	0.71	4.46		
0.62	1.94	0.62	2.43	0.75	4.10	0.81	5.09		
0.72	2.26	0.72	2.82	0.85	4.65	0.91	5.71		
0.82	2.57	0.82	3.21	0.95	5.20	1.01	6.34		
0.92	2.88	0.92	3.60	1.05	5.74	1.11	6.97		
1.02	3.19	1.02	3.99	1.15	6.29	1.21	7.59		
1.12	3.51	1.12	4.38	1.25	6.84	1.31	8.22		
1.32	4.13	1.32	5.17	1.45	7.94	1.51	9.47		
1.52	4.76	1.52	5.95	1.65	9.03	1.71	10.72		
1.72	5.39	1.72	6.73	1.85	10.13	1.91	11.98		
1.92	6.01	1.92	7.52	2.05	11.23	2.11	13.23		
2.42	7.58	2.42	9.47	2.55	13.97	2.61	16.36		
2.92	9.15	2.92	11.43	3.05	16.71	3.11	19.49		
3.42	10.71	3.42	13.39	3.55	19.45	3.61	22.63		
3.92	12.28	3.92	15.35	4.05	22.19	4.11	25.76		

Acceleration Ss=1.50g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)
0.00	0.48	0.00	0.60	0.00	0.60	0.00	0.60		
0.06	1.20	0.06	1.50	0.08	1.50	0.09	1.50		
0.28	1.20	0.28	1.50	0.39	1.50	0.44	1.50		
0.38	0.88	0.38	1.11	0.49	1.19	0.54	1.22		
0.48	0.70	0.48	0.88	0.59	0.99	0.64	1.03		
0.58	0.58	0.58	0.72	0.69	0.84	0.74	0.89		
0.68	0.49	0.68	0.62	0.79	0.74	0.84	0.79		
0.78	0.43	0.78	0.54	0.89	0.65	0.94	0.70		
0.88	0.38	0.88	0.48	0.99	0.59	1.04	0.64		
0.98	0.34	0.98	0.43	1.09	0.53	1.14	0.58		
1.08	0.31	1.08	0.39	1.19	0.49	1.24	0.53		
1.28	0.26	1.28	0.33	1.39	0.42	1.44	0.46		
1.48	0.23	1.48	0.28	1.59	0.37	1.64	0.40		
1.68	0.20	1.68	0.25	1.79	0.32	1.84	0.36		
1.88	0.18	1.88	0.22	1.99	0.29	2.04	0.33		
2.38	0.14	2.38	0.18	2.49	0.23	2.54	0.26		
2.88	0.12	2.88	0.15	2.99	0.19	3.04	0.22		
3.38	0.10	3.38	0.12	3.49	0.17	3.54	0.19		
3.88	0.09	3.88	0.11	3.99	0.15	4.04	0.16		

Displacement Ss=1.50g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)
0.00	0.48	0.00	0.00	0.00	0.00	0.00	0.00		
0.06	0.04	0.06	0.05	0.08	0.09	0.09	0.11		
0.28	0.92	0.28	1.15	0.39	2.19	0.44	2.87		
0.38	1.25	0.38	1.56	0.49	2.76	0.54	3.53		
0.48	1.58	0.48	1.97	0.59	3.33	0.64	4.17		
0.58	1.91	0.58	2.38	0.69	3.90	0.74	4.82		
0.68	2.24	0.68	2.80	0.79	4.46	0.84	5.47		
0.78	2.57	0.78	3.21	0.89	5.03	0.94	6.12		
0.88	2.89	0.88	3.62	0.99	5.60	1.04	6.77		
0.98	3.22	0.98	4.03	1.09	6.17	1.14	7.42		
1.08	3.55	1.08	4.44	1.19	6.73	1.24	8.07		
1.28	4.21	1.28	5.26	1.39	7.87	1.44	9.37		
1.48	4.87	1.48	6.08	1.59	9.01	1.64	10.67		
1.68	5.52	1.68	6.91	1.79	10.14	1.84	11.97		
1.88	6.18	1.88	7.73	1.99	11.28	2.04	13.27		
2.38	7.83	2.38	9.78	2.49	14.11	2.54	16.52		
2.88	9.47	2.88	11.84	2.99	16.95	3.04	19.77		
3.38	11.12	3.38	13.89	3.49	19.79	3.54	23.02		
3.88	12.76	3.88	15.95	3.99	22.63	4.04	26.27		

Acceleration Ss=1.66g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)	T(s)	Sa(g)
0.00	0.53	0.00	0.66	0.00	0.66	0.00	0.66		
0.06	1.33	0.06	1.66	0.08	1.66	0.09	1.66		
0.28	1.33	0.28	1.66	0.38	1.66	0.44	1.66		
0.38	0.98	0.38	1.23	0.48	1.31	0.54	1.35		
0.48	0.78	0.48	0.97	0.58	1.08	0.64	1.14		
0.58	0.64	0.58	0.81	0.68	0.92	0.74	0.99		
0.68	0.55	0.68	0.69	0.78	0.80	0.84	0.87		
0.78	0.48	0.78	0.60	0.88	0.71	0.94	0.78		
0.88	0.43	0.88	0.53	0.98	0.64	1.04	0.70		
0.98	0.38	0.98	0.48	1.08	0.58	1.14	0.64		
1.08	0.35	1.08	0.43	1.18	0.53	1.24	0.59		
1.28	0.29	1.28	0.37	1.38	0.45	1.44	0.51		
1.48	0.25	1.48	0.32	1.58	0.40	1.64	0.44		
1.68	0.22	1.68	0.28	1.78	0.35	1.84	0.40		
1.88	0.20	1.88	0.25	1.98	0.32	2.04	0.36		
2.38	0.16	2.38	0.20	2.48	0.25	2.54	0.29		
2.88	0.13	2.88	0.16	2.98	0.21	3.04	0.24		
3.38	0.11	3.38	0.14	3.48	0.18	3.54	0.21		
3.88	0.10	3.88	0.12	3.98	0.16	4.04	0.18		

Displacement Ss=1.66g, SEE(2%/50Years)

A		B		C		D		E	
T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)	T(s)	Sd(in)
0.00	0.53	0.00	0.00	0.00	0.00	0.00	0.00		
0.06	0.04	0.06	0.05	0.08	0.09	0.09	0.13		
0.28	1.04	0.28	1.30	0.38	2.30	0.44	3.13		
0.38	1.41	0.38	1.76	0.48	2.92	0.54	3.84		
0.48	1.78	0.48	2.22	0.58	3.53	0.64	4.56		
0.58	2.15	0.58	2.68	0.68	4.14	0.74	5.27		
0.68	2.51	0.68	3.14	0.78	4.75	0.84	5.99		
0.78	2.88	0.78	3.60	0.88	5.36	0.94	6.70		
0.88	3.25	0.88	4.06	0.98	5.97	1.04	7.41		
0.98	3.62	0.98	4.52	1.08	6.59	1.14	8.13		
1.08	3.99	1.08	4.98	1.18	7.20	1.24	8.84		
1.28	4.72	1.28	5.90	1.38	8.42	1.44	10.27		
1.48	5.46	1.48	6.82	1.58	9.64	1.64	11.69		
1.68	6.19	1.68	7.74	1.78	10.87	1.84	13.12		
1.88	6.93	1.88	8.66	1.98	12.09	2.04	14.55		
2.38	8.77	2.38	10.96	2.48	15.15	2.54	18.12		
2.88	10.61	2.88	13.26	2.98	18.21	3.04	21.68		
3.38	12.45	3.38	15.56	3.48	21.27	3.54	25.25		
3.88	14.29	3.88	17.86	3.98	24.33	4.04	28.82		