Table of Revisions from SDC 1.4 to SDC 1.5

Section	**Revision**
Table of Contents	New captions for Sections 2.1.1, 6.1.1, 6.1.2 and Appendix B; New Sections 6.1.3 and 6.1.4 added; Minor Repagination
1.1	Updated definition of an ordinary standard bridge
1.2	Updated definition of types of components addressed in the SDC
2.1	Major revision on "Ground Motion Representation"
2.1.1	Major revision – new material on "Design Spectrum"
3.1.3	Major revision in the definition of Curvature capacity ϕ_u ; Units added to symbol definitions
3.2.4	Correction to value of E_s (prestressing steel) in Figure 3.5
3.2.6	Symbol correction: f_c' changed to f_{ce}'
3.5	Change in the definition of "Minimum Lateral Strength"
6.1.1	New material on Ground Shaking and Design Spectrum
6.1.2	New material on Liquefaction
6.1.3	New SDC Section: "Fault Rupture Hazard"
6.1.4	New SDC Section: "Additional Seismic Hazards"
7.4.4.3	Symbol correction: A_{jv} changed to A_s^{jv}
7.6.1	Equation (7.24) augmented to include guideline on Footing Depth-to-Column Depth ratio
7.6.2 (c)	Correction of typographical error in Equation 7.27
7.7.1.3	Correction to Equation 7.32
7.7.3.5	Correction in the definition of Type II pile shafts
8.2.4	Major revision on "Development length for column reinforcement extended into enlarged Type II shafts"
Appendix A	Minor revision
Appendix B	Major revision – New material: "Design Spectrum Development"

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1.1 Definition Of An Ordinary Standard Bridge

A structure must meet all of the following requirements to be classified as an Ordinary Standard bridge:

- Span lengths less than 300 feet (90 m)
- Constructed with normal weight concrete girder, and column or pier elements
- Horizontal members either rigidly connected, pin connected, or supported on conventional bearings; Isolation bearings and dampers are considered nonstandard components
- Dropped bent caps or integral bent caps terminating inside the exterior girder; C-bents, outrigger bents, and offset columns are nonstandard components
- Foundations supported on spread footing, pile cap w/piles, or pile shafts
- Soil that is not susceptible to liquefaction, lateral spreading, or scour
- Bridge systems with a fundamental period greater than or equal to 0.7 seconds in the transverse and longitudinal directions of the bridge

1.2 Types Of Components Addressed In The SDC

The SDC is focused on concrete bridges. Seismic criteria for structural steel bridges are being developed independently and will be incorporated into the future releases of the SDC. In the interim, inquiries regarding the seismic performance of structural steel components shall be directed to the Structural Steel Technical Specialist and the Structural Steel Committee.

The SDC includes seismic design criteria for Ordinary Standard bridges constructed with the types of components listed in Table 1.

Table 1

ABUTMENTS Diaphragm Short Seat High Cantilever	**SUBSTRUCTURE SUPPORT SYSTEMS** Single Column Multi-Column Pier Walls Pile Extensions
SUPERSTRUCTURES Cast-In-Place • Reinforced Concrete • Post-tensioned concrete Precast • Reinforced Concrete • Pre-tensioned Concrete • Post-tensioned Concrete	**FOUNDATIONS** Spread Footing Driven Piles • Steel H/HP and Pipe • Precast P/S • CISS Drilled Shafts • CIDH • Large Diameter Types I and II Proprietary

2.1 Ground Motion Representation

For structural applications, seismic demand is represented using an elastic 5% damped response spectrum. In general, the Design Spectrum (DS) is defined as the greater of: (1) a probabilistic spectrum based on a 5% in 50 years probability of exceedance (or 975-year return period), (2) a deterministic spectrum based on the largest median response resulting from the maximum rupture (corresponding to **M**max) of any fault in the vicinity of the bridge site, or (3) a statewide minimum spectrum defined as the median spectrum generated by a magnitude 6.5 earthquake on a strike-slip fault located 12 kilometers from the bridge site. A detailed discussion of the development of both the probabilistic and deterministic design spectra as well as possible adjustment factors is given in Appendix B.

2.1.1 Design Spectrum

Several aspects of design spectrum development require special knowledge related to the determination of fault location (utilization of original source mapping where appropriate) and interpretation of the site profile and geologic setting for incorporation of site effects. Consequently, Geotechnical Services or a qualified geo-professional is responsible for providing final design spectrum recommendations.

Several design tools are available to the engineer for use in preliminary and final specification of the design spectrum. These tools include the following:

- Deterministic PGA map (http://dap3.dot.ca.gov/shake_stable/references/Deterministic_PGA_Map_8-12-09.pdf)
- Preliminary spectral curves for several magnitudes and soil classes (Appendix B, Figures B.13 – B.27)
- Spreadsheet with preliminary spectral curve data (http://dap3.dot.ca.gov/shake_stable/references/Preliminary_Spectral_Curves_Data_073009.xls)
- Recommended fault parameters for California faults meeting criteria specified in Appendix B (http://dap3.dot.ca.gov/shake_stable/references/2007_Fault_Database_120309.xls)
- Deterministic Response Spectrum spreadsheet (http://dap3.dot.ca.gov/shake_stable/references/Deterministic_Response_Spectrum_072809.xls)
- Probabilistic Response Spectrum spreadsheet (http://dap3.dot.ca.gov/shake_stable/references/Probabilistic_Response_Spectrum_080409.xls)
- Caltrans ARS Online (Caltrans intranet: http://10.160.173.178/shake2/shake_index2.php, internet: http://dap3.dot.ca.gov/shake_stable/)
- USGS Earthquake Hazards Program website (http://earthquake.usgs.gov/research/hazmaps/index.php)

3.1.3 Local Member Displacement Capacity

The local displacement capacity of a member is based on its rotation capacity, which in turn is based on its curvature capacity. The curvature capacity shall be determined by M-ϕ analysis, see Section 3.3.1. The local displacement capacity $\varDelta_c$ of any column may be idealized as one or two cantilever segments presented in Equations 3.1-3.5 and 3.1a-3.5a, respectively. See Figures 3.1 and 3.2 for details.

$$\varDelta_c = \varDelta_Y^{col} + \varDelta_p \qquad (3.1)$$

$$\varDelta_Y^{col} = \frac{L^2}{3} \times \phi_Y \qquad (3.2)$$

$$\Delta_p = \theta_p \times \left(L - \frac{L_p}{2} \right) \qquad (3.3)$$

$$\theta_p = L_p \times \phi_p \qquad (3.4)$$

$$\phi_p = \phi_u - \phi_Y \qquad (3.5)$$

$$\varDelta_{c1} = \varDelta_{Y1}^{col} + \varDelta_{p1} \quad , \quad \varDelta_{c2} = \varDelta_{Y2}^{col} + \varDelta_{p2} \qquad (3.1a)$$

$$\varDelta_{Y1}^{col} = \frac{L_1^2}{3} \times \phi_{Y1} \quad , \quad \varDelta_{Y2}^{col} = \frac{L_2^2}{3} \times \phi_{Y2} \qquad (3.2a)$$

$$\varDelta_{p1} = \theta_{p1} \times \left(L_1 - \frac{L_{p1}}{2} \right) \quad , \quad \varDelta_{p2} = \theta_{p2} \times \left(L_2 - \frac{L_{p2}}{2} \right) \qquad (3.3a)$$

$$\theta_{p1} = L_{p1} \times \phi_{p1} \quad , \quad \theta_{p2} = L_{p2} \times \phi_{p2} \qquad (3.4a)$$

$$\phi_{p1} = \phi_{u1} - \phi_{Y1} \quad , \quad \phi_{p2} = \phi_{u2} - \phi_{Y2} \qquad (3.5a)$$

Where:

L = Distance from the point of maximum moment to the point of contra-flexure (in)

L_P = Equivalent analytical plastic hinge length as defined in Section 7.6.2 (in)

$\varDelta_p$ = Idealized plastic displacement capacity due to rotation of the plastic hinge (in)

$\varDelta_Y^{col}$ = The idealized yield displacement of the column at the formation of the plastic hinge (in)

ϕ_Y = Idealized yield curvature defined by an elastic-perfectly-plastic representation of the cross section's M-ϕ curve, see Figure 3.7 (rad/in)

ϕ_p = Idealized plastic curvature capacity (assumed constant over L_p) (rad/in)

ϕ_u = Curvature capacity at the Failure Limit State, defined as the concrete strain reaching ε_{cu} or the longitudinal reinforcing steel reaching the reduced ultimate strain ε_{su}^R (rad/in)

θ_p = Plastic rotation capacity (radian)

3.1.4.1 *Minimum Local Displacement Ductility Capacity*

Each ductile member shall have a minimum local displacement ductility capacity of $\mu_c = 3$ to ensure dependable rotational capacity in the plastic hinge regions regardless of the displacement demand imparted to that member. The local displacement ductility capacity shall be calculated for an equivalent member that approximates a fixed base cantilever element as defined in Figure 3.3.

The minimum displacement ductility capacity $\mu_c = 3$ may be difficult to achieve for columns and Type I pile shafts with large diameters D_c > 10 ft (3m) or components with large L/D ratios. Local displacement ductility capacity less than 3 requires approval as specified in MTD 20-11.

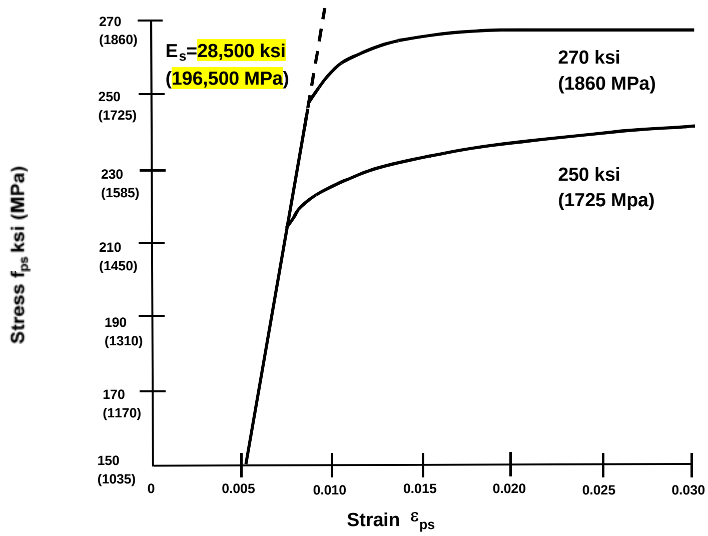

Figure 3.5 Prestressing Strand Stress Strain Model

3.2.6 Normal Weight Portland Cement Concrete Properties

Modulus of Elasticity, $E_c = 33 \times w^{1.5} \times \sqrt{f'_{ce}} \quad \text{(psi)}, \quad E_c = 0.043 \times w^{1.5} \times \sqrt{f'_{ce}} \quad \text{(MPa)}$ (3.11)

Where w = unit weight of concrete in lb/ft^3 and kg/m^3, respectively. For w = 143.96 lb/ft^3 (2286.05 kg/m^3), Equation 3.11 results in the form presented in other Caltrans documents.

Shear Modulus $G_c = \dfrac{E_c}{2 \times (1 + \nu_c)}$ (3.12)

Poisson's Ratio $\nu_c = 0.2$

Expected concrete compressive strength f'_{ce} = the greater of: $\begin{cases} 1.3 \times f'_c \\ \quad \text{or} \\ 5000 \ \text{(psi)} \quad 34.5 \ \text{(MPa)} \end{cases}$ (3.13)

Unconfined concrete compressive strain at the maximum compressive stress $\varepsilon_{c0} = 0.002$

Ultimate unconfined compression (spalling) strain $\varepsilon_{sp} = 0.005$

Confined compressive strain $\varepsilon_{cc} = *$

Ultimate compression strain for confined concrete $\varepsilon_{cu} = *$

* Defined by the constitutive stress strain model for confined concrete, see Figure 3.6.

3.5 Minimum Lateral Strength

Each bent shall have a minimum lateral flexural capacity (based on expected material properties) to resist a lateral force of $0.1 \times P_{dl}$, where P_{dl} is the tributary dead load applied at the center of gravity of the superstructure.

6.1.1 Ground Shaking

Generally, ground shaking hazard is characterized for design by the Design Response Spectrum. Methodology for development of the Design Response Spectrum is described in detail in Appendix B, *Design Spectrum Development*. This spectrum reflects the shaking hazard at or near the ground surface.

When bridges are founded on either stiff pile foundations or pile shafts and extend through soft soil, the response spectrum at the ground surface may not reflect the motion of the pile cap or shaft. In these instances, special analysis that considers soil-pile/shaft kinematic interaction is required and will be addressed by the geo-professional on a project specific basis.

Soil profiles can vary significantly along the length of bridges resulting in the need to develop multiple Design Spectra. In the case of bridges with lengths greater than 1000 feet, seismic demand can also vary from seismic waves arriving at different bents at different times (i.e., phase lag). Furthermore, complex wave scattering contributes to incoherence between different bridge bents, particularly at higher frequencies. While incoherence in seismic loading is generally thought to reduce seismic demands overall, it does result in increased relative displacement demand between adjacent bridge frames. In cases with either varying soil profile or extended bridge length, the geo-professional must work in close collaboration with the structural engineer to ensure the bridge can withstand the demands resulting from incoherent loading.

6.1.2 Liquefaction

Preliminary investigation performed by Geotechnical Services will include an assessment of liquefaction potential within the project site per MTD 20-14 and 20-15. When locations are identified as being susceptible to liquefaction, the geo-professional will provide recommendations that include a discussion of the following:

- need for additional site investigation and soil testing
- possible consequences of liquefaction including potential horizontal and vertical ground displacements and resulting structural impacts
- possible remediation strategies including ground improvement, avoidance, and/or structural modification

6.1.3 Fault Rupture Hazard

Preliminary investigation of fault rupture hazard includes the identification of nearby active surface faults that may cross beneath a bridge or proposed bridge, per MTD 20-10. In some instances, the exact location of a fault will not be known because it is concealed by a relatively recent man-made or geologic material or the site is located in a region of complex fault structure. In such cases, a geologist will recommend a fault zone with dimensions based on professional judgment. If a fault trace underlies a structure or the structure falls within the specified fault zone, then Geotechnical Services will provide the following recommendations:

- location and orientation of fault traces or zones with respect to structures
- expected horizontal and vertical displacements
- description of additional evaluations or investigations that could refine the above information
- strategies to address ground rupture including avoidance (preferred) and structural design

6.1.4 Additional Seismic Hazards

The following seismic hazards may also exist at a site, and will be addressed by Geotechnical Services if applicable to the location:

- potential for slope instability and rock-fall resulting from earthquakes
- loss of bearing capacity/differential settlement
- tsunami/seiche

7.2.2 Vertical Acceleration

If vertical acceleration is considered, per Section 2.1, a separate analysis of the superstructure's nominal capacity shall be performed based on a uniformly applied vertical force equal to 25% of the dead load applied upward and downward, see Figure 7.3. The superstructure at seat type abutments is assumed to be pinned in the vertical direction, up or down. The superstructure flexural capacity shall be based only on continuous mild reinforcement distributed evenly between the top and bottom slabs. The effects of dead load, primary and secondary prestressing shall be ignored. The continuous steel shall be spliced with "service level" couplers as defined in Section 8.1.3, and is considered effective in offsetting the mild reinforcement required for other load cases. Lap splices equal to two times the standard lap may be substituted for the "service splices," provided the laps are placed away from the critical zones (mid-spans and near supports).

7.4.4.3 Joint Shear Reinforcement

A) Vertical Stirrups:

$$A_s^{jv} = 0.2 \times A_{st} \tag{7.19}$$

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 7.7, 7.8, and 7.10. The stirrups provided in the overlapping areas shown in Figure 7.7 shall count towards meeting the requirements of both areas creating the overlap. These stirrups can be used to meet other requirements documented elsewhere including the shear in the bent cap.

7.6.1 Column Dimensions

Every effort shall be made to limit the column cross sectional dimensions to the depth of the superstructure. This requirement may be difficult to meet on columns with high L/D ratios. If the column dimensions exceed the depth of the bent cap it may be difficult to meet the joint shear requirements in Section 7.4.2, the superstructure capacity requirements in Section 4.3.2.1, and the ductility requirements in Section 3.1.4.1.

The relationship between column cross section, bent cap depth and footing depth specified in equations 7.24a and 7.24b are guidelines based on observation. Maintaining these ratios should produce reasonably well proportioned structures.

$$0.7 \le \frac{D_c}{D_s} \le 1.0 \qquad (7.24a)$$

$$0.7 \le \frac{D_{ftg}}{D_c} \qquad (7.24b)$$

7.6.2 (c) Non-cased Type I Pile Shafts:

$$L_p = D^* + 0.08H_{o-\max} \qquad (7.27)$$

D^* = Diameter for circular shafts or the least cross section dimension for oblong shafts.

7.7.1.3 *Rigid Footing Response*

The length to thickness ratio along the principal axes of the footing must satisfy equation 7.32 if rigid footing behavior and the associated linear distribution of pile forces and deflections is assumed.

$$L_{ftg} / D_{ftg} \leq 2.2 \tag{7.32}$$

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

7.7.3.5 *Enlarged Pile Shafts*

Type II shafts typically are enlarged relative to the column diameter to contain the inelastic action to the column. Enlarged shafts shall be at least 24 inches larger than the column diameter and the reinforcement shall satisfy the clearance requirements for CIP piling specified in Bridge Design Details 13-22.

8.2.4 Development Length For Column Reinforcement Extended Into Enlarged Type II Shafts

Column longitudinal reinforcement shall be extended into Type II (enlarged) shafts in a staggered manner with the minimum recommended embedment lengths of $(D_{c,\max} + l_d)$ and $(D_{c,\max} + 2 \times l_d)$, where $D_{c,\max}$ is the largest cross section dimension of the column, and l_d is the development length in tension of the column longitudinal bars. The development length l_d shall be determined by multiplying the basic tension development length l_{db} as specified in AASHTO LRFD Section 5.11.2.1 by the compounded modification factors of 0.9 and 0.6 for epoxy-coated and non epoxy-coated reinforcement, respectively. Nominal values of f_y = 68 ksi and $f_c^{'}$ = 5 ksi shall be used in calculating l_{db}.

In addition to ensuring adequate anchorage beyond the plastic hinge penetration into the shaft, this provision will ensure that the embedment lengths for a majority of bridge columns supported on Type II shafts are less than 20 ft. Construction cost increases significantly when embedment lengths exceed 20 ft as the shaft excavations are governed by the more stringent Cal-OSHA requirements for tunneling and mining.

Appendix A. Notations & Acronyms

B_{cap} = Bent cap width (Section 7.4.2.1)

L = Member length from the point of maximum moment to the point of contra-flexure (in, mm) (Section 3.1.3)

L_p = Equivalent analytical plastic hinge length (in, mm) (Section 3.1.3)

~~Mm = Earthquake moment magnitude (Section 6.1.2.2)~~

Mmax = Earthquake maximum moment magnitude (Section 2.1)

N = Blow count per foot (0.3 m) for the California Standard Penetration Test (Section 6.2.2, Appendix B-17)

T = Natural period of vibration, in seconds T = $2\pi\sqrt{m/k}$ (Section 7.1.2)

s_u = Undrained shear strength (psf, KPa) (Section 6.2.2, Appendix B-17)

Δ_{eq} = The average displacement at an expansion joint due to earthquake (Section 7.2.5.4)

ρ_s = Ratio of volume of spiral or hoop reinforcement to the core volume confined by the spiral or hoop reinforcement (measured out-to-out), $\rho_s = 4 \times A_b / (D' \times s)$ for circular cross sections (Sections 3.6.2 and 3.8.1)

v_s = Shear wave velocity (ft/sec, m/sec) (Section 6.2.2, Appendix B-17)

Note to Technical Publications

For this publication of SDC, i.e. version 1.5, the entire SDC v.1.4 Appendix B is to be replaced with the following Appendix B.

APPENDIX B - DESIGN SPECTRUM DEVELOPMENT

California Seismic Hazard

Seismic hazard in California is governed by shallow crustal tectonics, with the sole exception of the Cascadia subduction zone along California's northern coastline. In both regimes, the Design Response Spectrum is based on the envelope of a deterministic and probabilistic spectrum. Instructions for the determination of these spectra, including the application of appropriate adjustment factors, are provided in the sections below.

Deterministic Criteria

Shallow crustal tectonics (all faults other than Cascadia subduction zone)

The deterministic spectrum is calculated as the arithmetic average of median response spectra calculated using the Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) ground motion prediction equations (GMPE's). These equations are applied to all faults in or near California considered to be active in the last 700,000 years (late Quaternary age) and capable of producing a moment magnitude earthquake of 6.0 or greater. In application of these ground motion prediction equations, the earthquake magnitude should be set to the maximum moment magnitude **M**Max, as recommended by California Geological Survey (1997, 2005). Recommended fault parameters, including **M**Max, are provided in the *"2007 Fault Database"* (http://dap3.dot.ca.gov/ shake_stable/references/2007_Fault_Database_120309.xls). Updates to these parameters along with additions or deletions to the database of considered faults can be found at *"Errata Report"* (http://dap3.dot.ca.gov/shake_stable/Errata_Report_120309.pdf)

Multi-fault Hazard

In cases where more than one fault contributes maximum spectral values across the period spectrum, an envelope of the spectral values shall be used for the design spectrum.

Eastern California Shear Zone

The Eastern California Shear Zone is a region of distributed shear and complex faulting that makes identification of potential seismic sources challenging. To account for this uncertainty, a minimum response spectrum based on a strike-slip mechanism with moment magnitude **M** 7.6 and a distance to the vertical rupture plane of 10 km (6.2 miles) is imposed. This minimum spectrum is shown for several V_{S30} values in Figure B.1. The Eastern California Shear Zone is shown in Figure B.2.

Cascadia Subduction Zone

Following the general approach of the USGS (Frankel, 2002), the deterministic spectrum for the Cascadia subduction zone is defined by the median spectrum from the Youngs et al. (1997) ground motion prediction equation, with the added criterion that where the Youngs et al. spectrum is less than the average of the Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) models (both without the hanging wall term applied), an arithmetic average of the Youngs et al. and CB-CY average is used.

Minimum Deterministic Spectrum

In recognition of the potential for earthquakes to occur on previously unknown faults, a minimum deterministic spectrum is imposed statewide. This minimum spectrum is defined as the average of the median predictions of Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) for a scenario **M** 6.5 vertical strike-slip event occurring at a distance of 12 km (7.5 miles). While this scenario establishes the minimum spectrum, the spectrum is intended to represent the possibility of a wide range of magnitude-distance scenarios. Although a rupture distance of 12 km strictly meets the criteria for application of a directivity adjustment factor, application of this factor to the minimum spectrum is NOT recommended.

Probabilistic Criteria

The probabilistic spectrum is obtained from the (2008) USGS Seismic Hazard Map (Petersen, 2008) for the 5% in 50 years probability of exceedance (or 975 year return period). Since the USGS Seismic Hazard Map spectral values are published only for V_{S30} = 760m/s, soil amplification factors must be applied for other site conditions. The site amplification factors shall be based on an average of those derived from the Boore-Atkinson (2008), Campbell-Bozorgnia (2008), and Chiou-Youngs (2008) ground motion prediction models (the same models used for the development of the USGS map).

Spectrum Adjustment Factors

The design spectrum may need to be modified to account for seismological effects related to being in close proximity to a rupturing fault and/or placement on top of a deep sedimentary basin. These adjustments are discussed in the following sections.

Near-Fault Factor

Sites located near a rupturing fault may experience elevated levels of shaking at periods longer than 0.5-second due to phenomena such as constructive wave interference, radiation pattern effects, and static fault offset (fling). As a practical matter, these phenomena are commonly combined into a single “near-fault” adjustment factor. This adjustment factor, shown in Figure B.3, is fully applied at locations with a site to rupture plane distance (R_{Rup}) of 15 km (9.4 miles) or less and linearly tapered to zero adjustment at 25 km (15.6 miles). The adjustment consists of a 20% increase in spectral values with corresponding period longer than 1-second. This increase is linearly tapered to zero at a period of 0.5-second.

For application to a probabilistic spectrum, a deaggregation of the site hazard should be performed to determine whether the "probabilistic" distance is less than 25 km. The "probabilistic" distance shall be calculated as the smaller of the mean distance and the mode distance (from the peak R, M bin), but not less than the site to rupture plane distance corresponding to the nearest fault in the Caltrans Fault Database. This latter requirement reflects the intention not to apply a near-fault adjustment factor to a background seismic source used in the probabilistic seismic hazard analysis.

Basin Factor

Both the Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) ground motion prediction models include a depth to rock (Z) parameter that allows each model to better predict ground motion in regions with deep sedimentary structure. The two models use different reference velocities for rock, with Campbell-Bozorgnia using a depth to 2.5 km/s shear wave velocity ($Z_{2.5}$) and Chiou-Youngs using a depth to 1.0 km/s shear wave velocity ($Z_{1.0}$). Numerical models suggest that ground shaking in sedimentary basins is impacted by phenomena such as trapped surface waves, constructive and destructive interference, amplifications at the basin edge, and heightened 1-D soil amplification due to a greater depth of soil. Since neither the Campbell-Bozorgnia nor Chiou-Youngs models consider these phenomena explicitly, it is more accurate to refer to predicted amplification due to the Z parameter as a "depth to rock" effect instead of a basin effect. However, since sites with large depth to rock are located in basin structures the term "basin effect" is commonly used.

Amplification factors for the two models are shown for various depths to rock in Figure B.4. These plots assume a V_{S30} of 270 m/s (typical for many basin locations) but are suitable for other V_{S30} values as well since the basin effect is only slightly sensitive to V_{S30} (primarily at periods less than 0.5 second). It should be noted that both models predict a decrease in long period energy for cases of shallow rock ($Z_{2.5} <$ 1 km or $Z_{1.0} <$ 40 m). Since $Z_{2.5}$ and $Z_{1.0}$ data are generally unavailable at non-basin locations, implementation of the basin amplification factors is restricted to locations with $Z_{2.5}$ larger than 3 km or $Z_{1.0}$ larger than 400 m.

Maps of $Z_{1.0}$ and $Z_{2.5}$

Figures B.5 through B.11 show contour maps of $Z_{1.0}$ and $Z_{2.5}$ for regions with sufficient depth to rock to trigger basin amplification. In Southern California, these maps were generated using data from the Community Velocity Model (CVM) Version 4 (http://www.data.scec.org/3Dvelocity/). In Northern California, the $Z_{2.5}$ contour map was generated using tomography data by Thurber (2009) and a generalized velocity profile by Brocher (2005). Details of the contour map development are provided in the *"Deterministic PGA Map and ARS Online Report"* *(http://dap3.dot.ca.gov/shake_stable/references/Deterministic_PGA_Map_and_ARS_Online_*

Report_071409.pdf). A $Z_{1.0}$ contour map could not be created in Northern California due to insufficient data.

Application of the models

For Southern California locations an average of the Campbell-Bozorgnia and Chiou-Youngs basin amplification factors is applied to both the deterministic and probabilistic spectra. For Northern California locations only the Campbell-Bozorgnia basin amplification factor is applied.

Directional Orientation of Design Spectrum

When recorded horizontal components of earthquake ground motion are mathematically rotated to different orientations, the corresponding response spectrum changes as well. Both the deterministic and probabilistic spectra defined above reflect a spectrum that is equally probable in all orientations. The maximum response spectrum, occurring at a specific but unpredictable orientation, is approximately 15% to 25% larger than the equally probable spectrum calculated using the procedures described above. Since a narrow range of directional orientations typically define the critical loading direction for bridge structures, the equally probable component spectrum is used for design.

Selection of V_{S30} for Site Amplification

The Campbell-Bozorgnia (2008), Chiou-Youngs (2008), and Boore-Atkinson (2008) ground motion prediction models (the latter is included for application to the probabilistic spectrum) use the parameter V_{S30} to characterize near surface soil stiffness as well as infer broader site characteristics. V_{S30} represents the average small strain shear wave velocity in the upper 100 feet (30 meters) of the soil column. This parameter, along with the level of ground shaking, determines the estimated site amplification in each of the above models. If the shear wave velocity (V_S) is known (or estimated) for discrete soil layers, then V_{S30} can be calculated as

$$V_{s30} = \frac{100\ ft}{\frac{D_1}{V_1} + \frac{D_2}{V_2} + \ldots + \frac{D_n}{V_n}}$$

where D_n represents the thickness of layer n (ft), V_n represents the shear wave velocity of layer n (fps), and the sum of the layer depths equals 100 feet. It is recommended that direct shear wave velocity measurements be used, or, in the absence of available field measurements, correlations to available parameters such as undrained shear strength, cone penetration tip resistance, or standard penetration test blow counts be utilized. Additional recommendations pertaining to determination of V_{S30} for development of the preliminary and final design spectrum are given in *"Geotechnical Services Design Manual"* (http://dap3.dot.ca.gov/shake_stable/references/GS_Design_Manual _081209.pdf)

Figure B.12 provides a profile classification system that is published in Applied Technology Council–32 (1996) and was adopted in previous versions of SDC. This table includes general guidance on average shear wave velocity that may be useful for development of a preliminary design spectrum. Acceleration and displacement response spectra at V_{S30} values corresponding to the center of the velocity ranges designated for soil profile types B, C, and D are provided at several magnitudes in Figures B.13-B.24. The data for these curves can be found in the *"Preliminary Spectral Curves Data"* spreadsheet (http://dap3.dot.ca.gov/shake_stable/references/ Preliminary_Spectral_Curves_Data_073009.xls).

The Campbell-Bozorgnia and Chiou-Youngs ground motion prediction equations are applicable for V_{S30} ranging from 150 m/s (500 fps) to 1500 m/s (5000 fps). For cases where V_{S30} exceeds 1500 m/s (very rare in California), a value of 1500 m/s should be used. For cases where either (1) V_{S30} is less than 150 m/s, (2) one or more layers of at least 5 feet thickness has a shear wave velocity less than 120 m/s, or (3) the profile conforms to Soil Profile Type E criteria per Figure B.12, a site-specific response analysis is required for determination of the final design spectrum.

For cases where the site meets the criteria prescribed for Soil Profile Type E, the response spectra presented in Figures B.25-B.27, originally presented in ATC-32, can be used for development of a preliminary design spectrum. In most cases, however, Type E spectra will significantly exceed spectra developed using site response analysis methods. For this reason it is preferred that a site response analysis be performed for the determination of the preliminary design spectrum in Type E soils.

When a soil profile meets the criteria prescribed for Soil Profile Type F (in Figure B.12), a site response analysis is required for both preliminary and final design.

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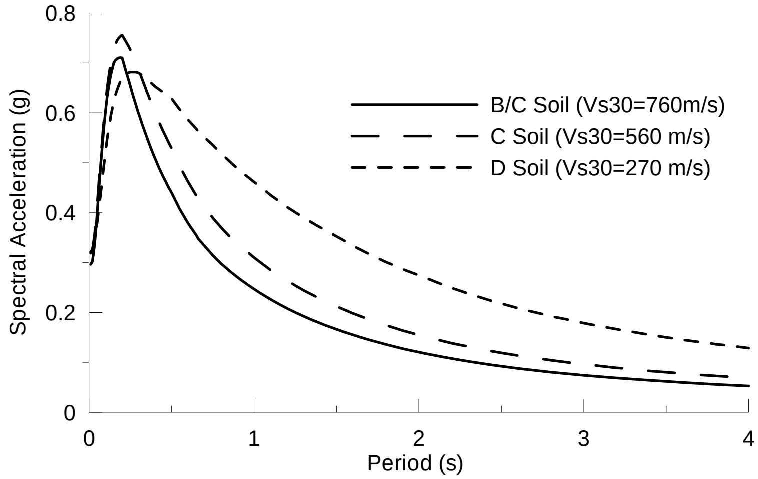


Figure B.1 Minimum response spectrum for Eastern Shear Zone (V_{S30} = 760, 560, and 270 m/s)

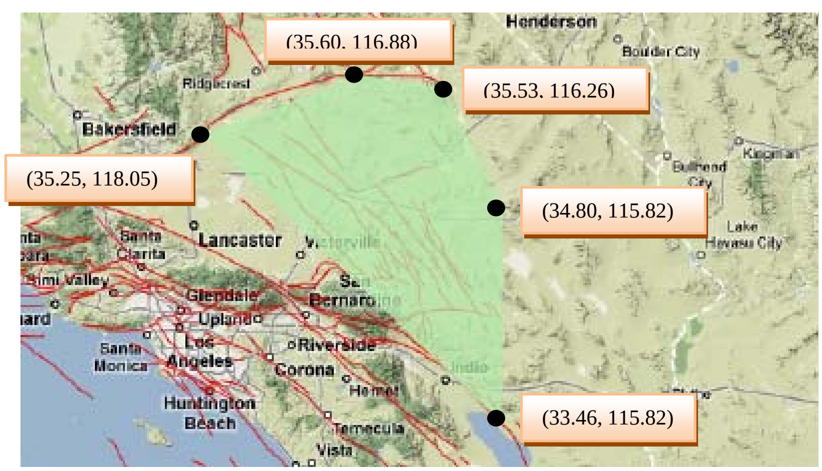


Figure B.2 Boundaries of Eastern Shear Zone. Coordinates in decimal degrees (Lat, Long)

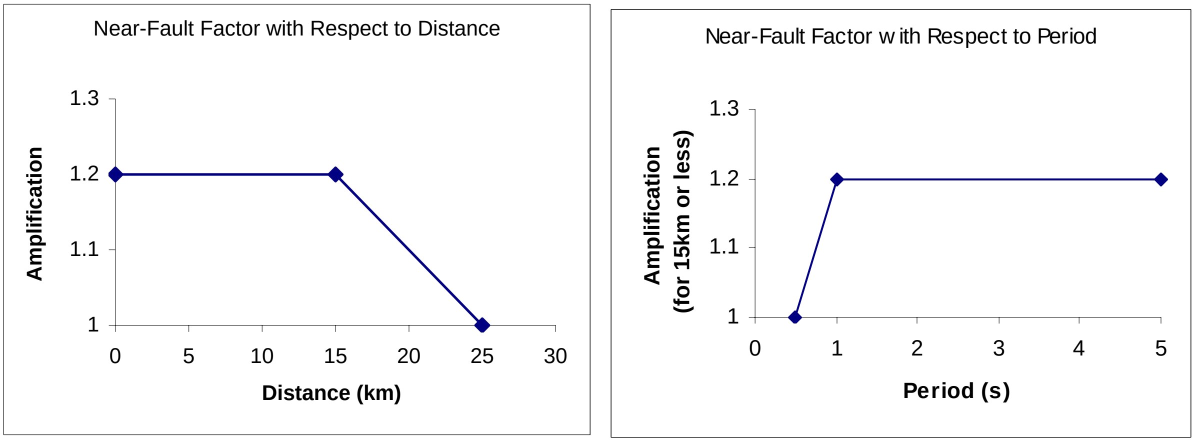


Figure B.3 Near-Fault adjustment factor as a function of distance and spectral period. The distance measure is based on the closest distance to any point on the fault plane.

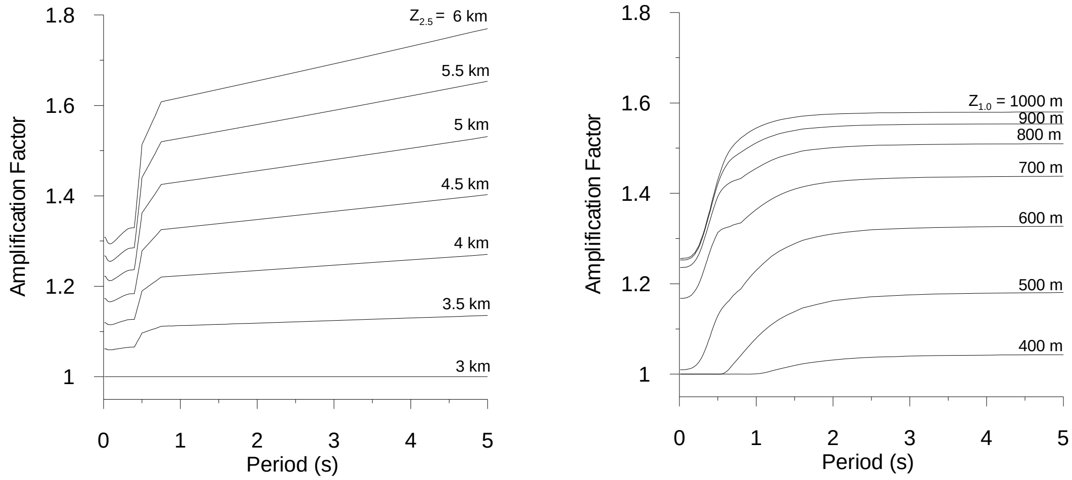


Figure B.4 Basin amplification factors for the Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) ground motion prediction equations. Curves may be slightly conservative at periods less than 0.5 seconds.

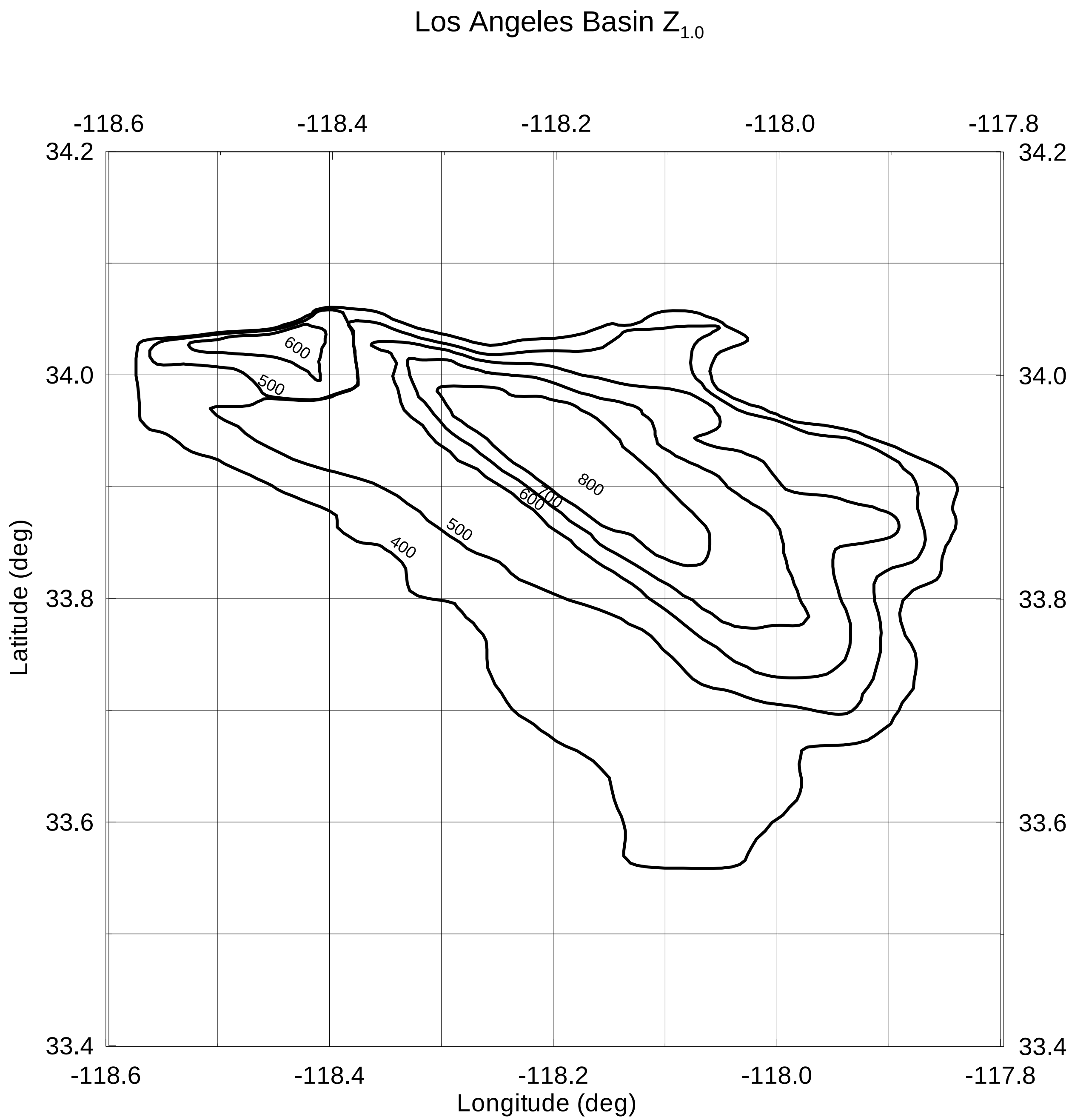


Figure B.5 Contours of depth (meters) to shear wave velocity 1 km/s ($Z_{1.0}$) in the Los Angeles basin.

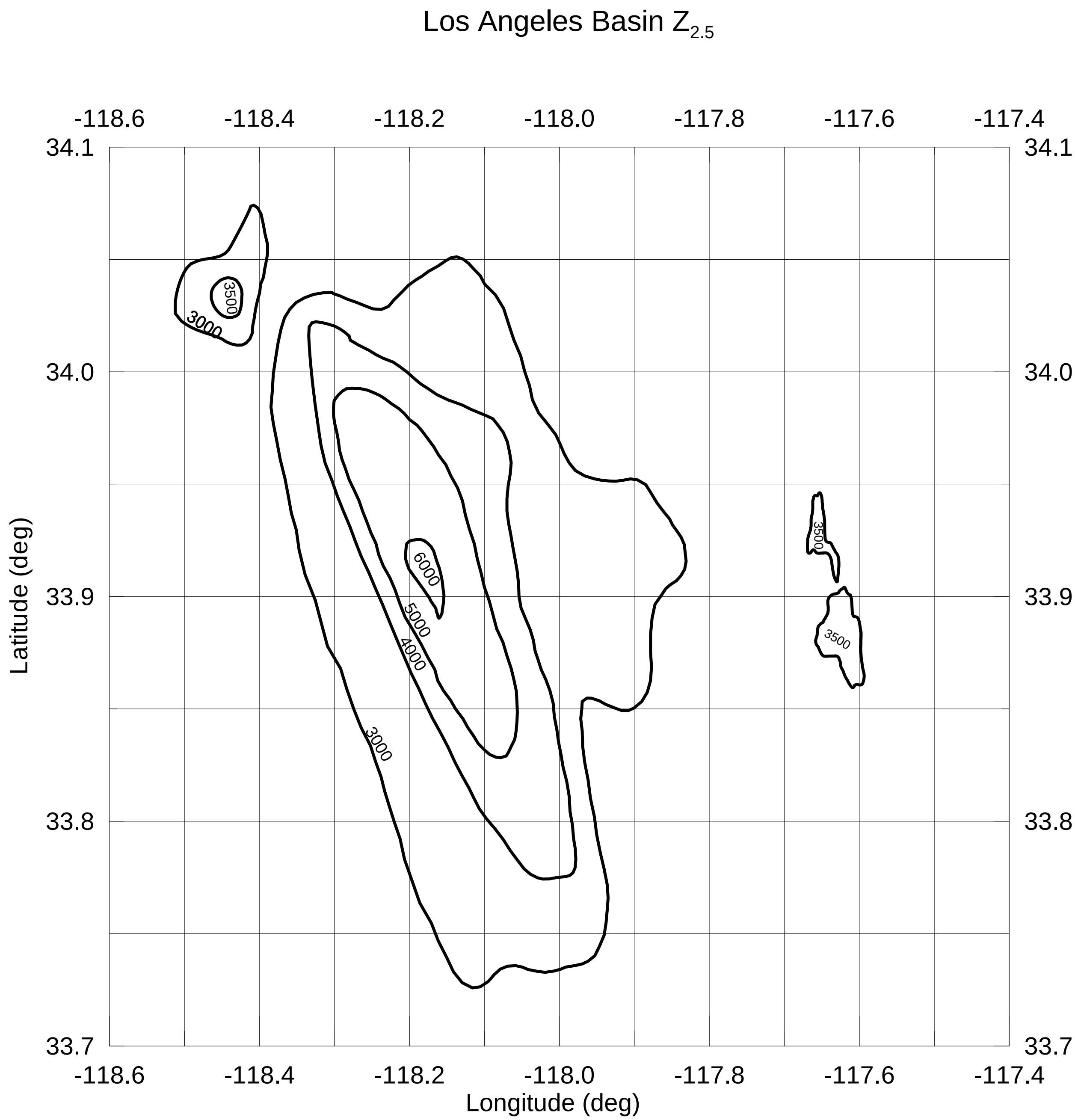


Figure B.6 Contours of depth (meters) to shear wave velocity 2.5 km/s ($Z_{2.5}$) in the Los Angeles basin.

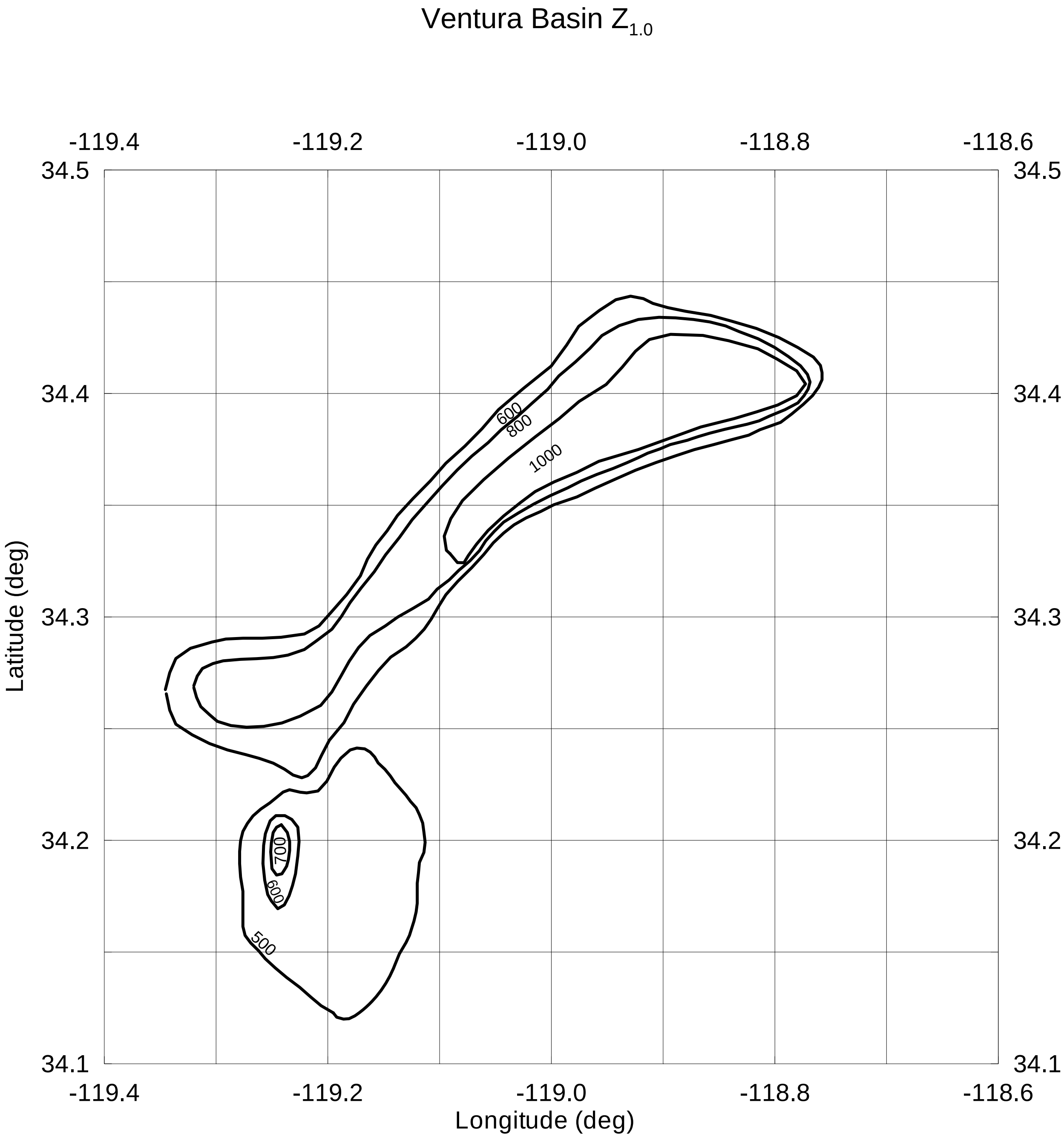


Figure B.7 Contours of depth (meters) to shear wave velocity 1 km/s ($Z_{1.0}$) in the Ventura basin.

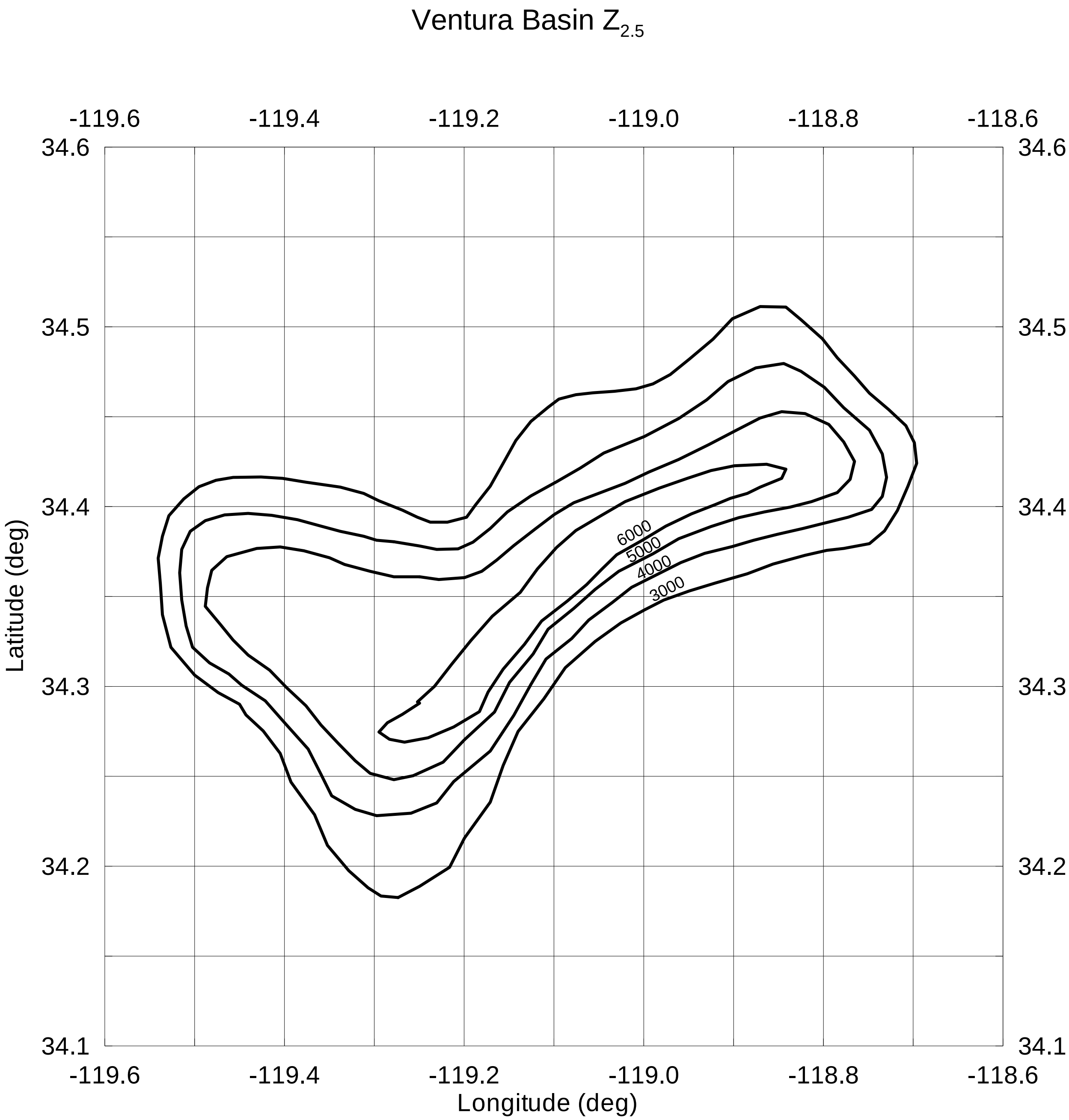


Figure B.8 Contours of depth (meters) to shear wave velocity 2.5 km/s ($Z_{2.5}$) in the Ventura basin.

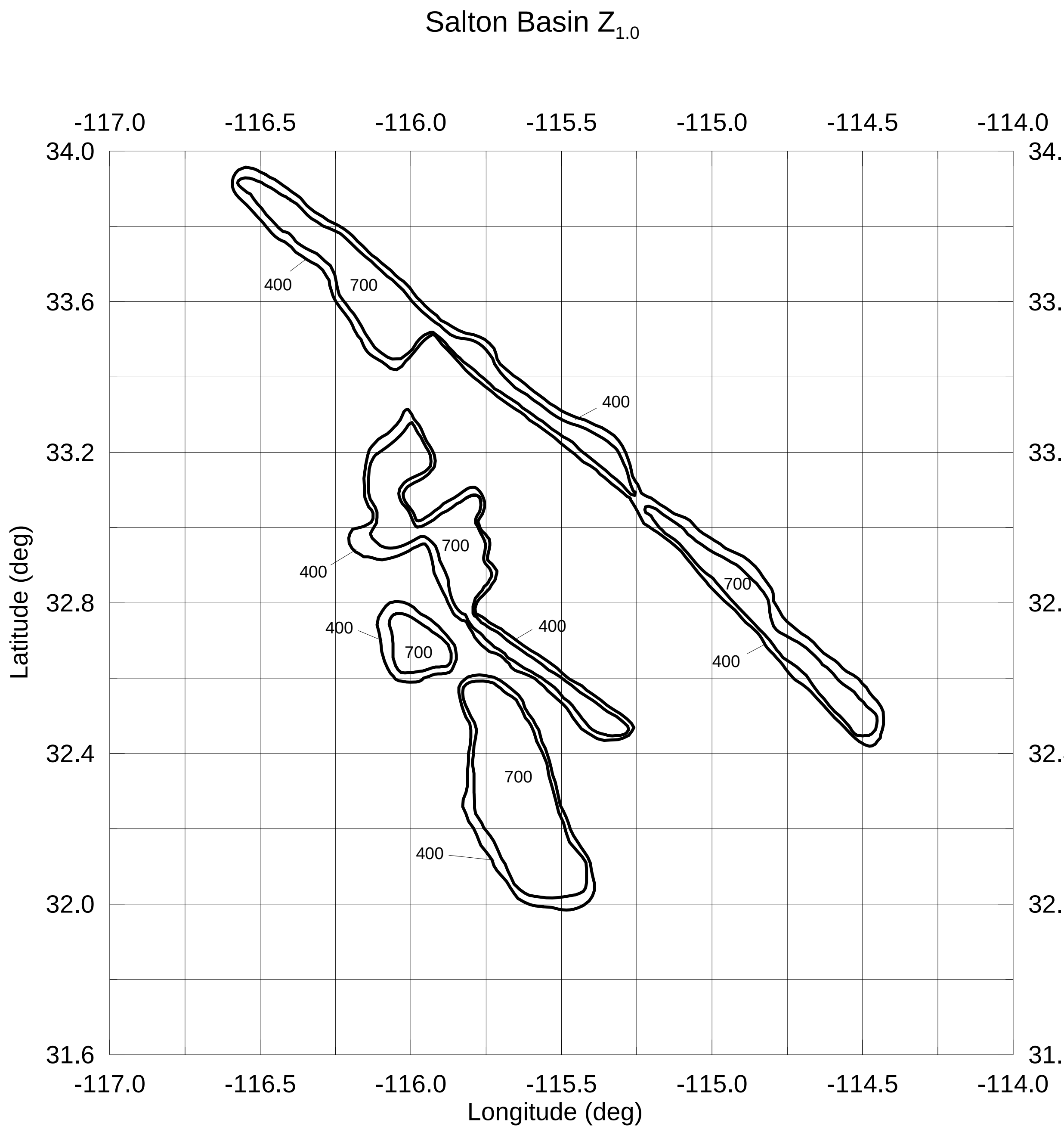


Figure B.9 Contours of depth (meters) to shear wave velocity 1 km/s ($Z_{1.0}$) in the Salton basin (Imperial Valley).

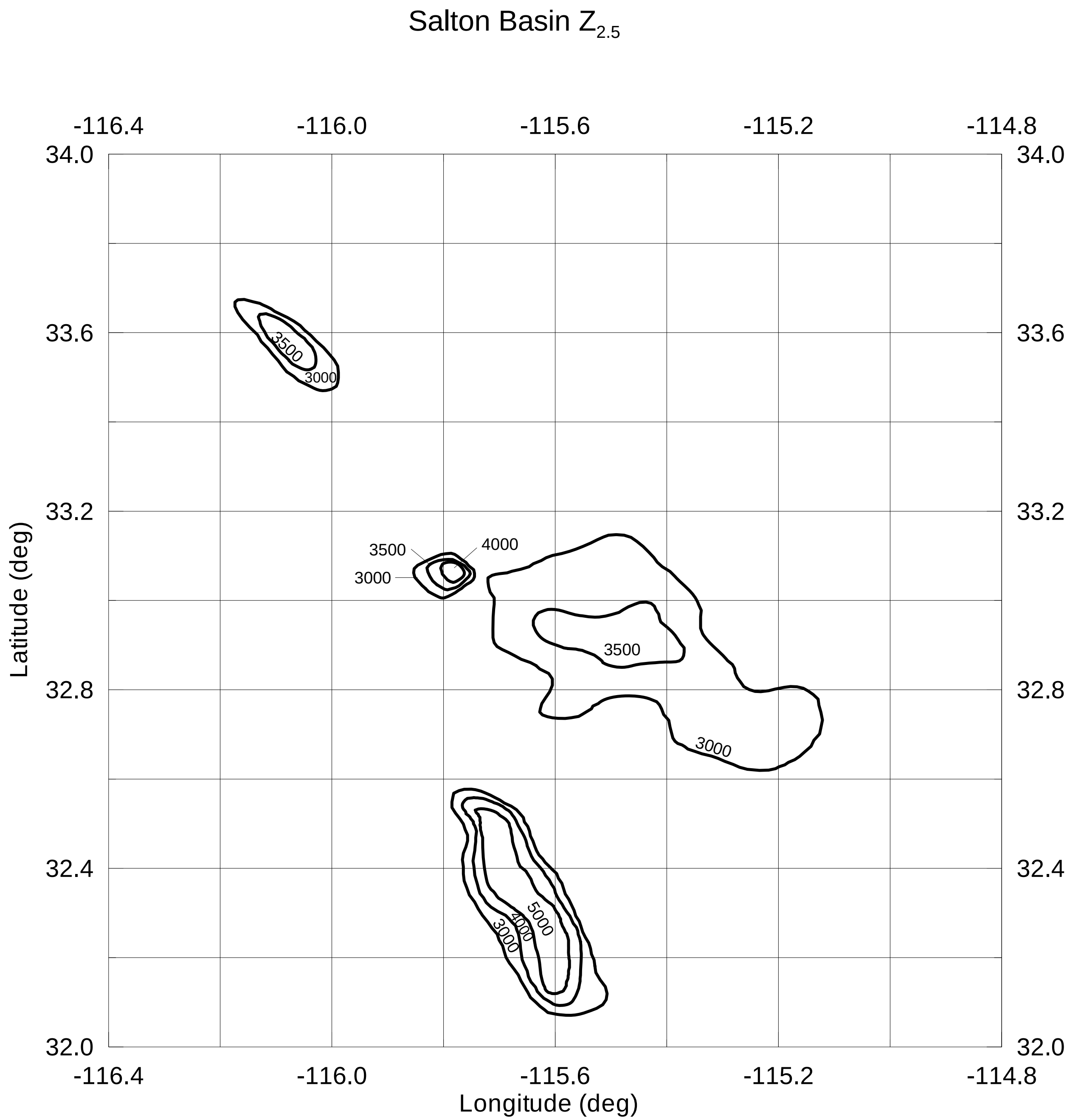


Figure B.10 Contours of depth (meters) to shear wave velocity 2.5 km/s ($Z_{2.5}$) in the Salton basin (Imperial Valley).

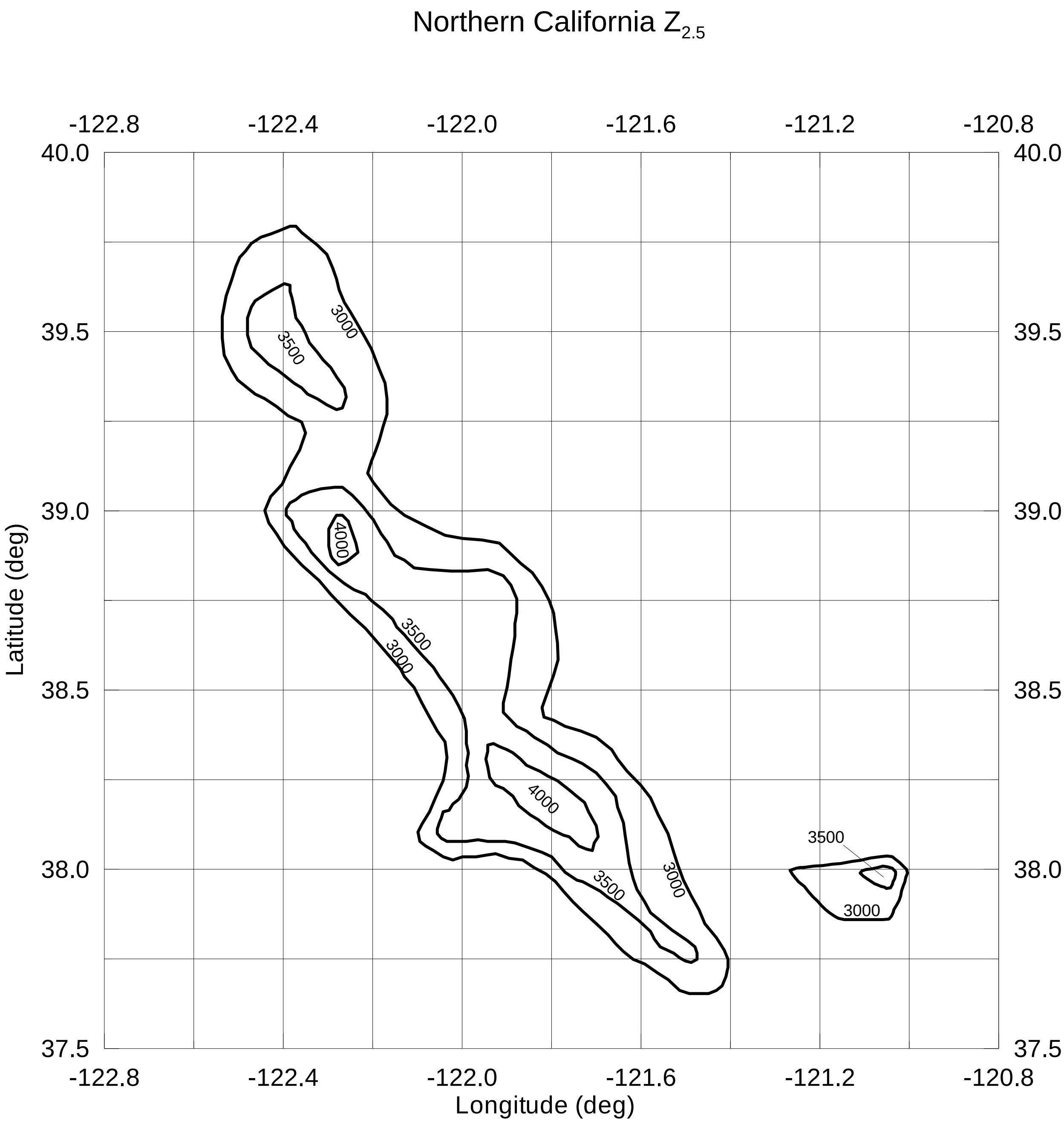


Figure B.11 Contours of depth (meters) to shear wave velocity 2.5 km/s ($Z_{2.5}$) in Northern California.

Soil Profile Type	Soil Profile Description[a]
A	**Hard rock with measured shear wave velocity v_{S30} > 5000 ft/s (1,500 m/s)**
B	**Rock with shear wave velocity 2,500 < v_{S30} < 5000 ft/s (760m/s < v_{S30} < 1,500 m/s)**
C	**Very dense soil and soft rock with shear wave velocity 1,200 < v_{S30} < 2,500 ft/s (360m/s < v_{S30} < 760 m/s) or with either standard penetration resistance N > 50 or undrained shear strength $s_u \geq$ 2,000 psf (100 kPa)**
D	**Stiff soil with shear wave velocity 600 < v_{S30} < 1,200 ft/s (180 m/s < v_{S30} < 360 m/s) or with either standard penetration resistance 15 ≤ N ≤ 50 or undrained shear strength 1,000 < s_u < 2,000 psf (50 < s_u < 100 kPa)**
E	**A soil profile with shear wave velocity v_{S30} < 600 ft/s (180 m/s) or any profile with more than 10 ft (3 m) of soft clay, defined as soil with plasticity index *PI* > 20, water content *w* ≥ 40 percent, and undrained shear strength s_u < 500 psf (25 kPa)**
F	**Soil requiring site-specific evaluation:** **1. Soils vulnerable to potential failure or collapse under seismic loading; i.e. liquefiable soils, quick and highly sensitive clays, collapsible weakly-cemented soils** **2. Peat and/or highly organic clay layers more than 10 ft (3 m) thick** **3. Very high-plasticity clay (*PI* > 75) layers more than 25 ft (8 m) thick** **4. Soft-to-medium clay layers more than 120 ft (36 m) thick**

Figure B.12 Soil profile types (after Applied Technology Council-32-1, 1996)

[a] The soil profile types shall be established through properly substantiated geotechnical data.

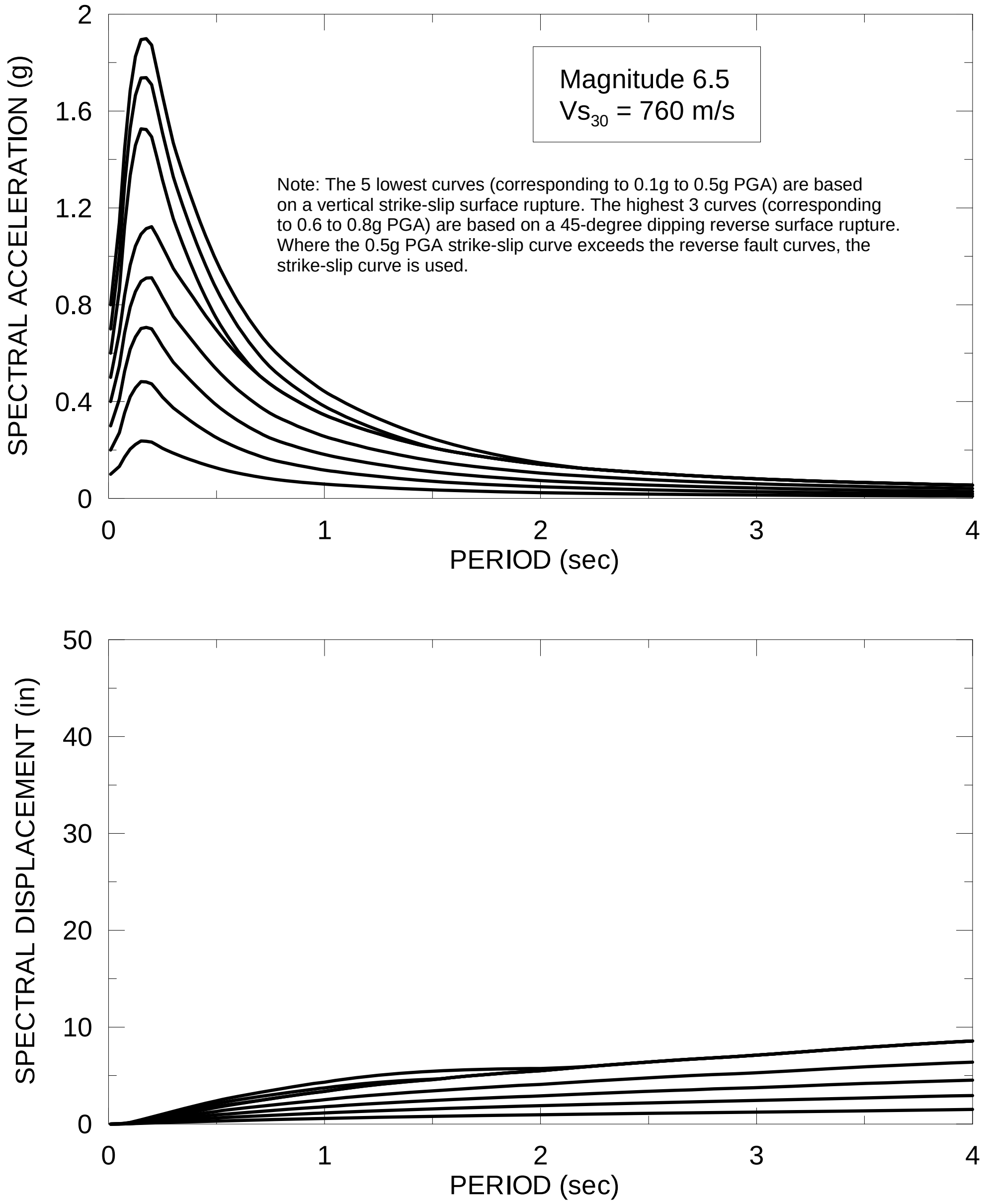


Figure B.13 Spectral Acceleration and Displacement for Vs30 = 760 m/s (*M* = 6.5)

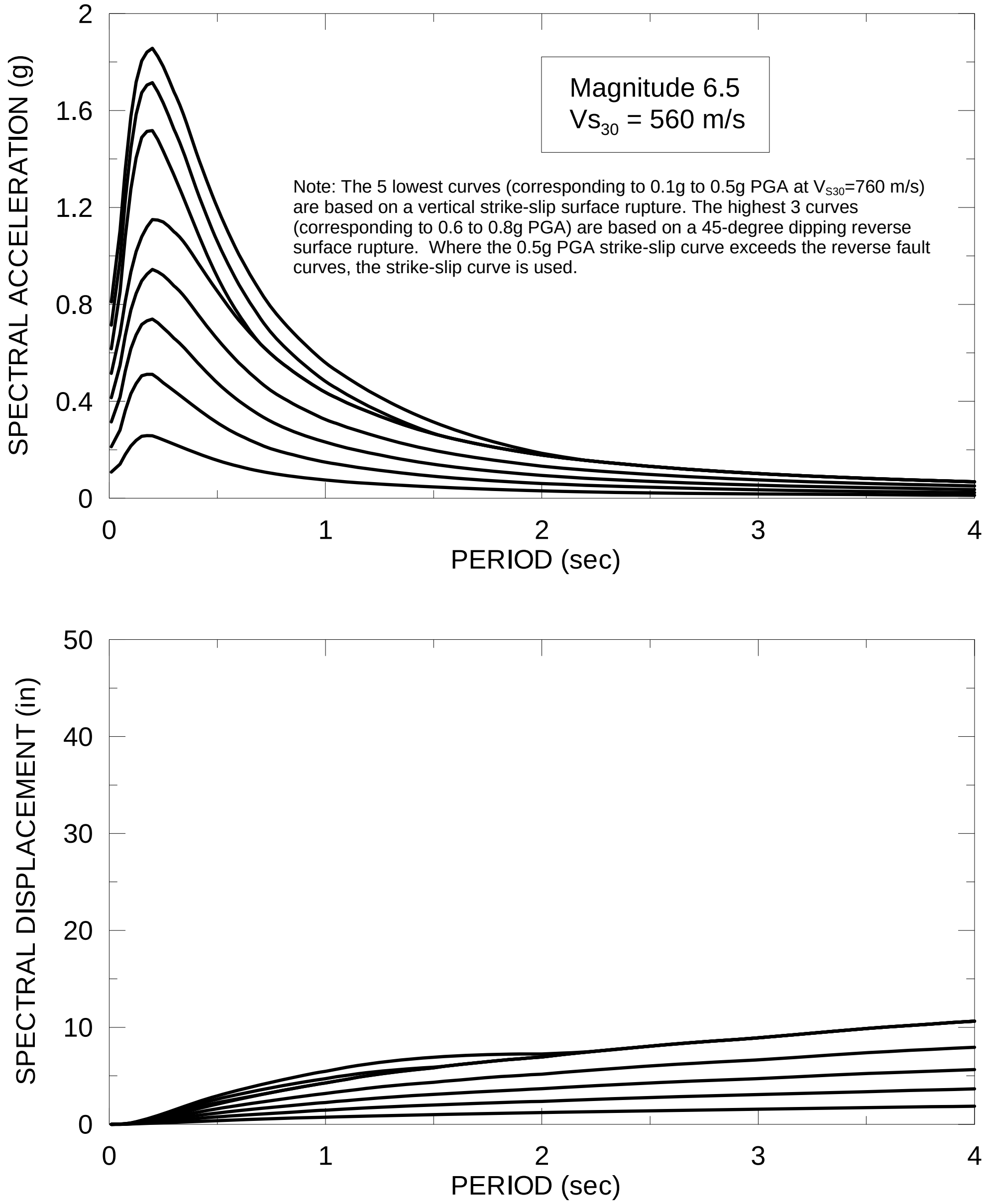


Figure B.14 Spectral Acceleration and Displacement for Vs30 = 560 m/s (M = 6.5)

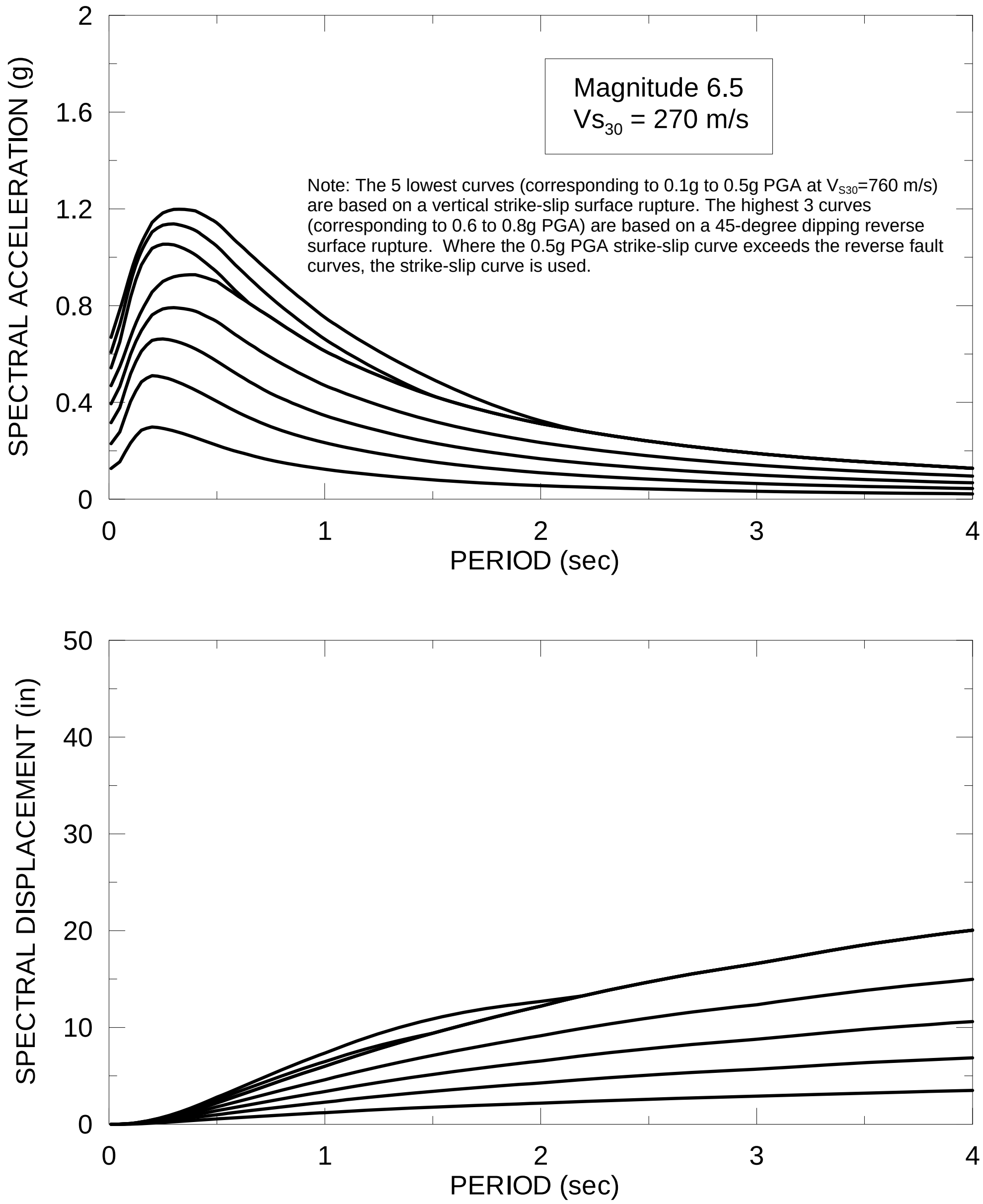


Figure B.15 Spectral Acceleration and Displacement for Vs30 = 270 m/s (*M* = 6.5)

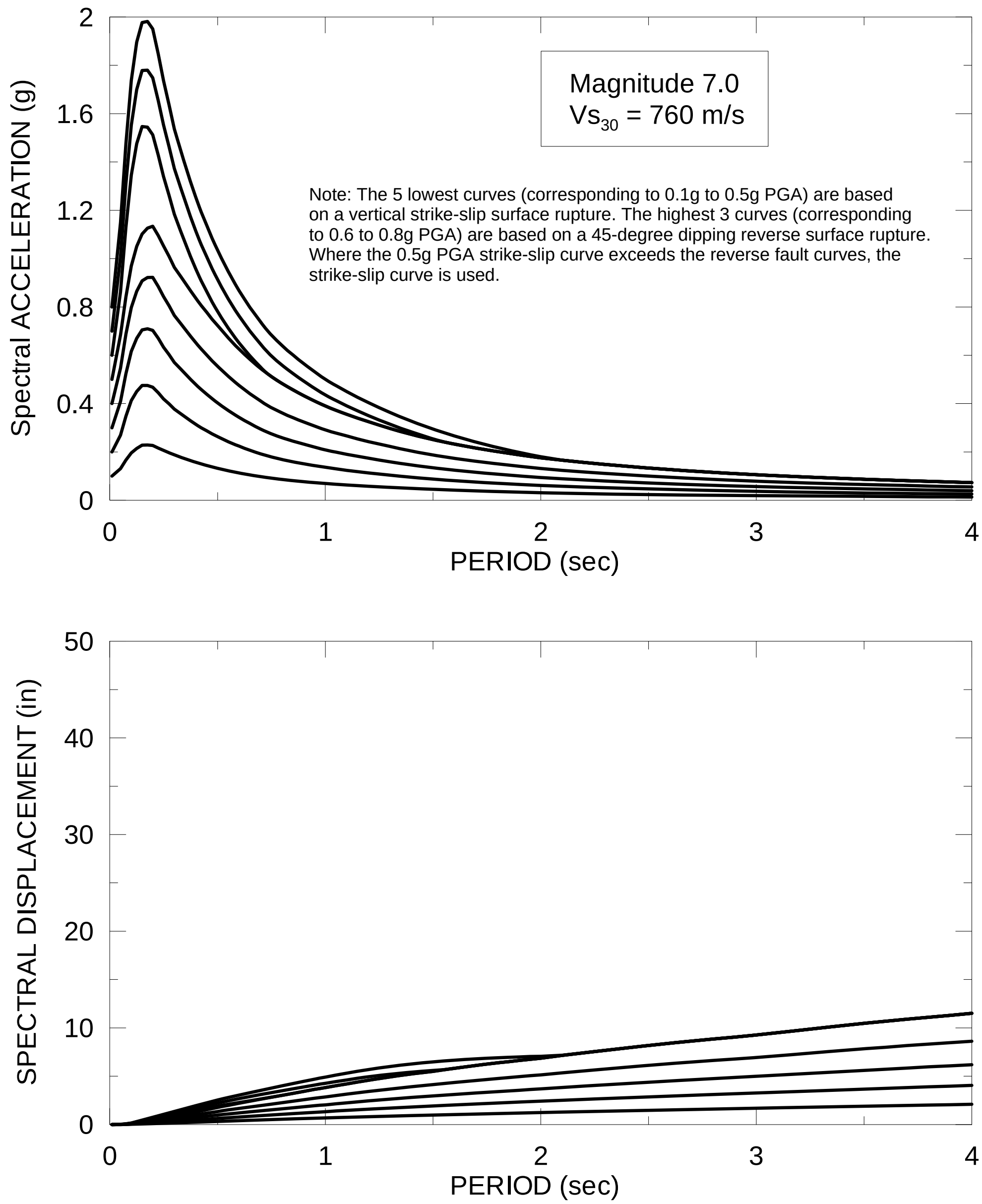
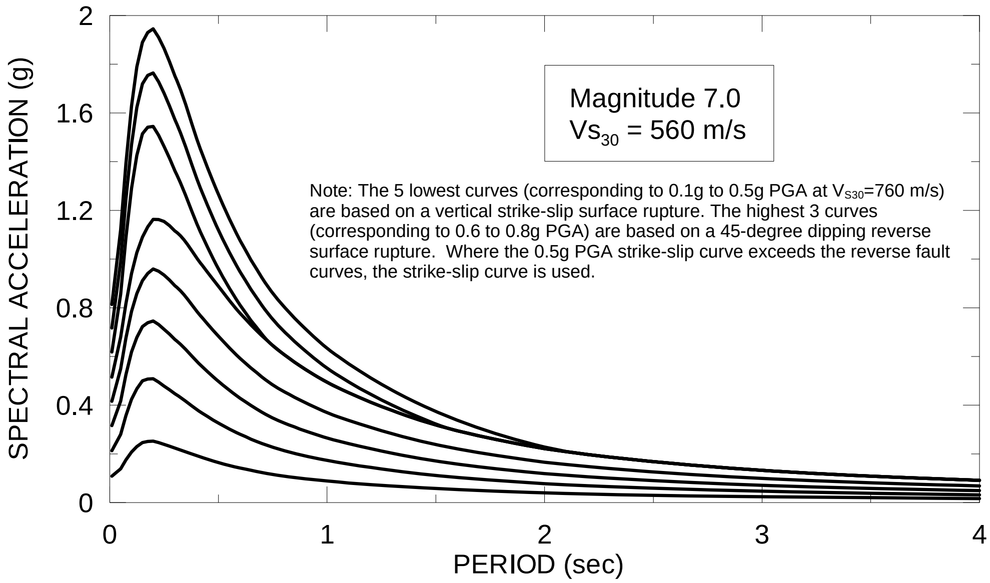


Figure B.16 Spectral Acceleration and Displacement for Vs30 = 760 m/s (*M* = 7.0)



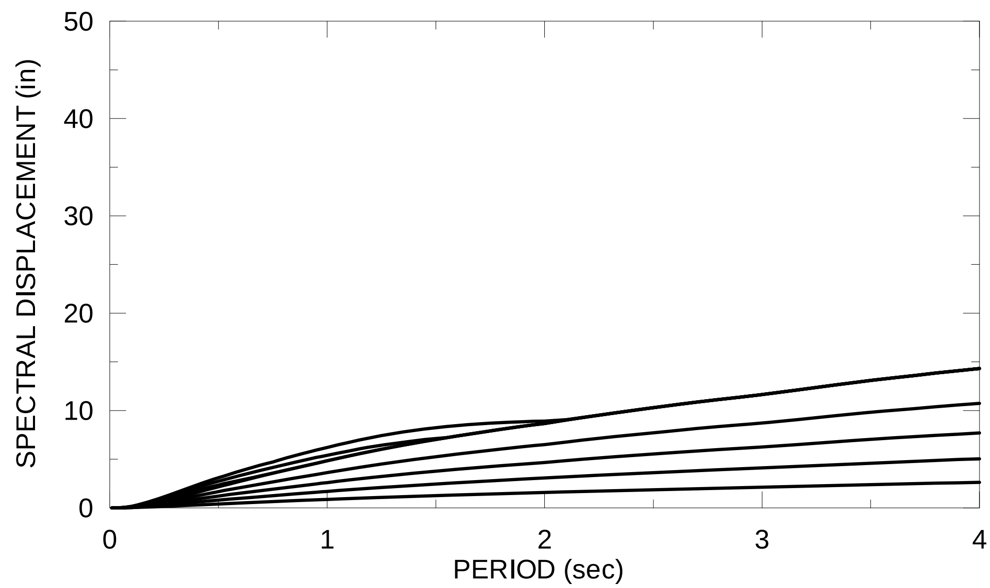


Figure B.17 Spectral Acceleration and Displacement for Vs30 = 560 m/s (M = 7.0)

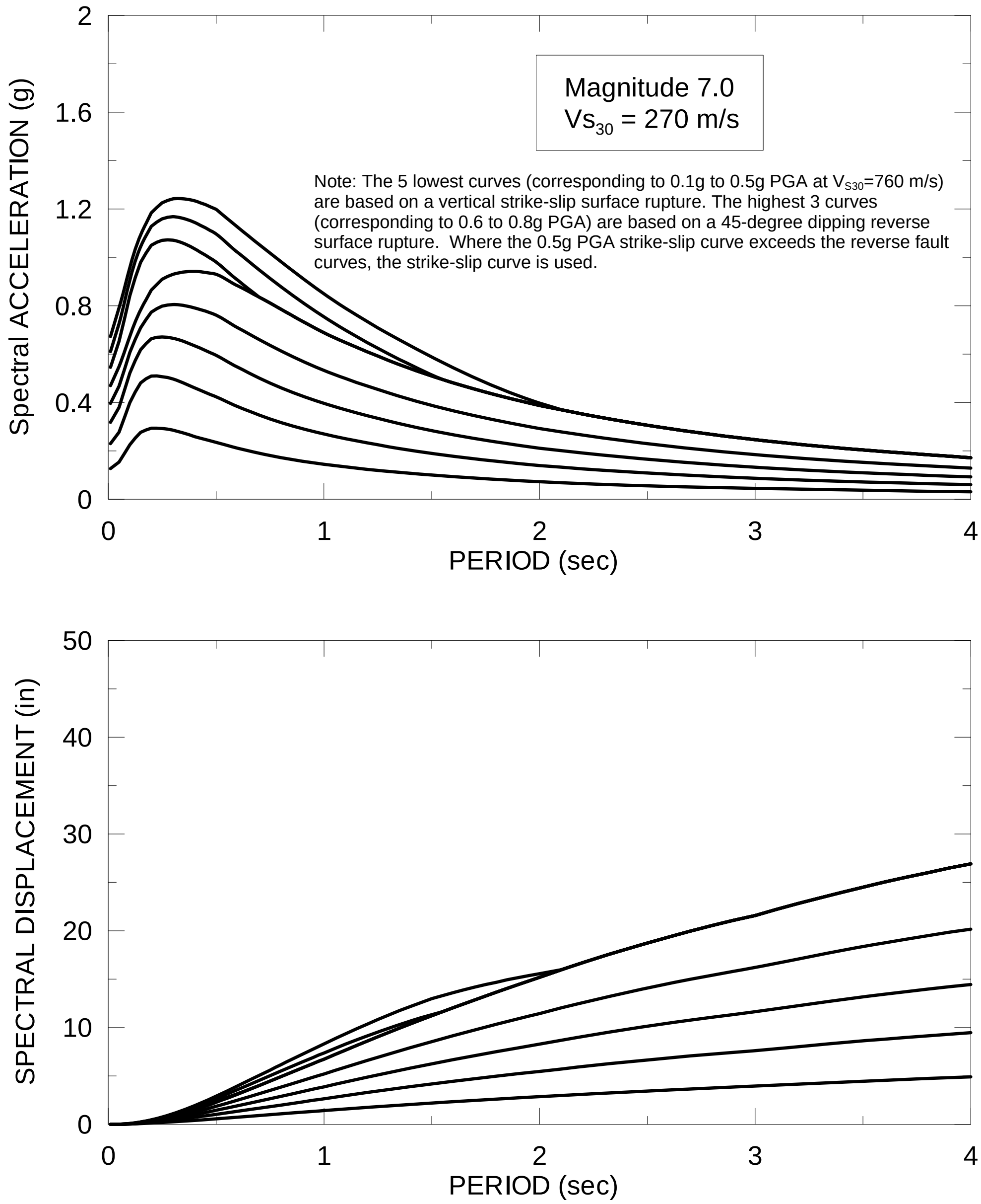


Figure B.18 Spectral Acceleration and Displacement for Vs30 = 270 m/s (M = 7.0)

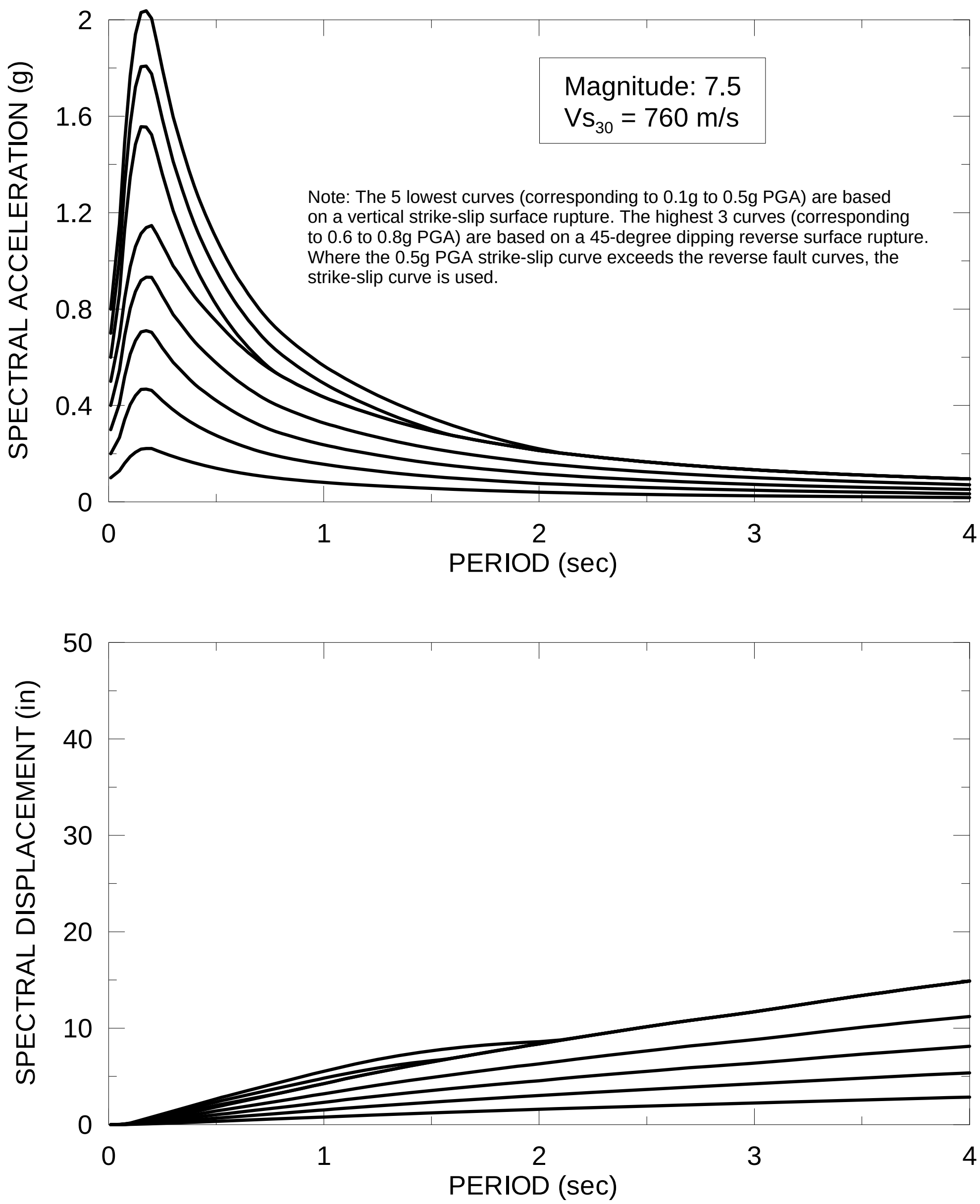


Figure B.19 Spectral Acceleration and Displacement for Vs30 = 760 m/s (*M* = 7.5)

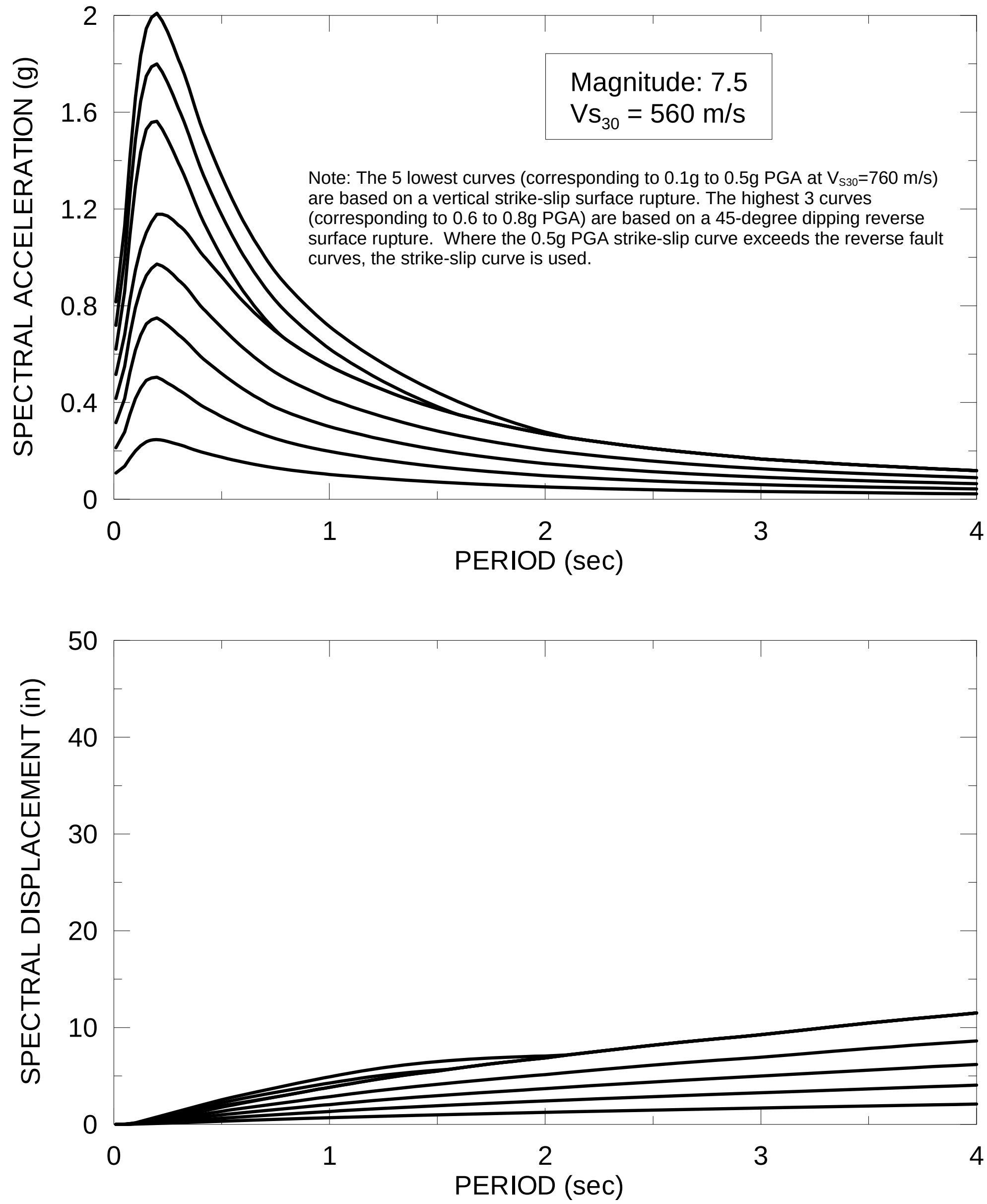


Figure B.20 Spectral Acceleration and Displacement for Vs30 = 560 m/s (*M* = 7.5)

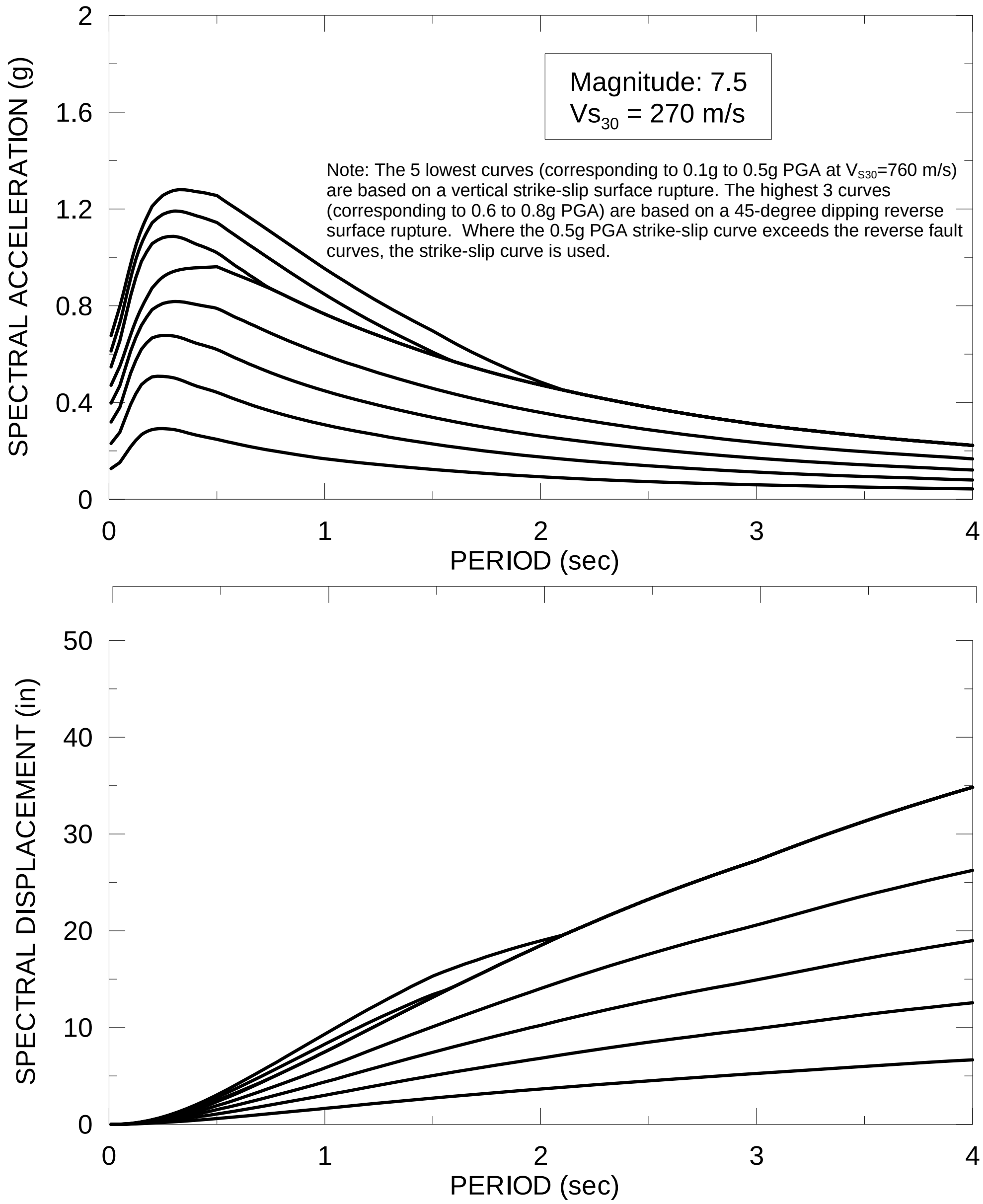


Figure B.21 Spectral Acceleration and Displacement for Vs30 = 270 m/s (*M* = 7.5)

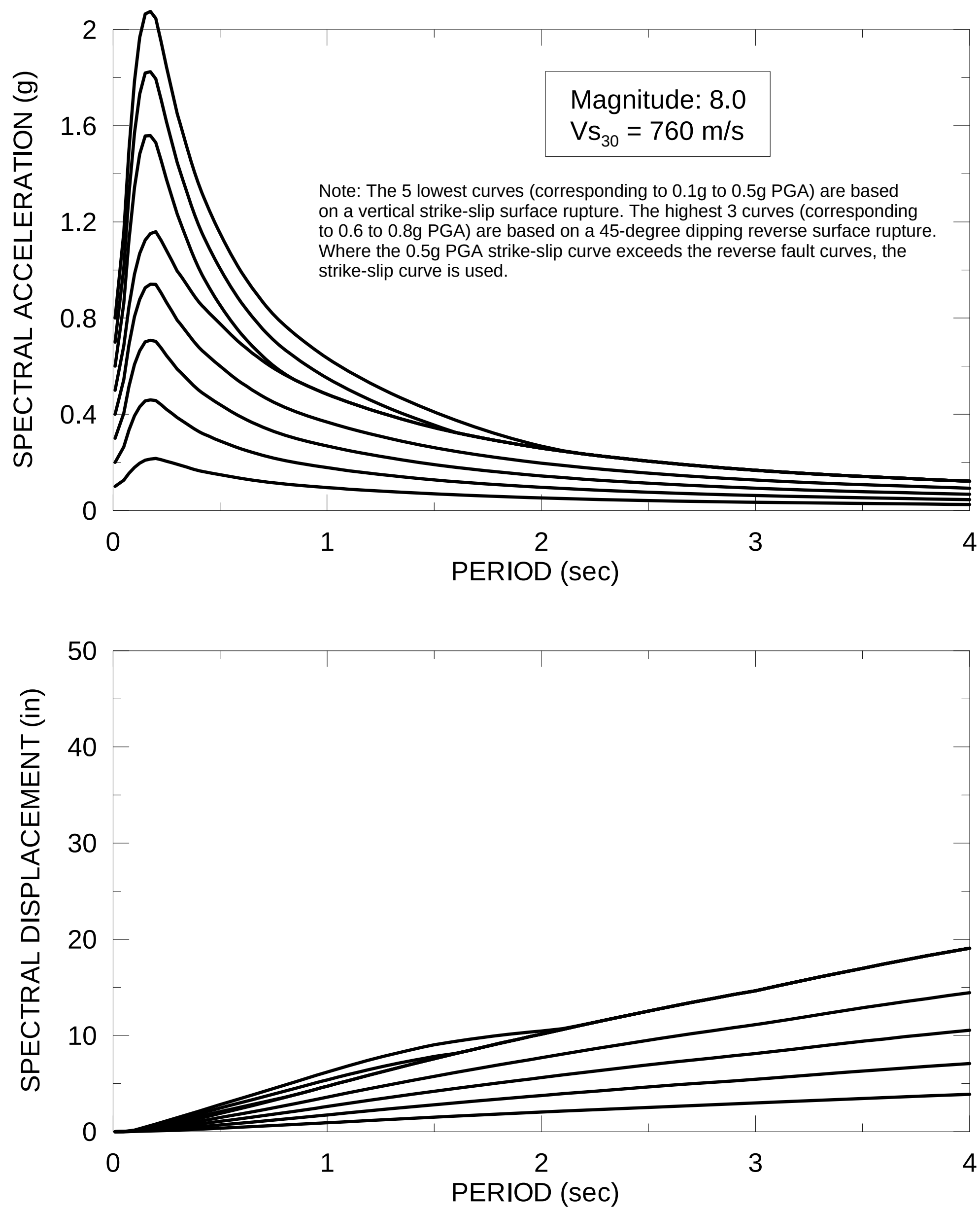


Figure B.22 Spectral Acceleration and Displacement for Vs30 = 760 m/s (M = 8.0)

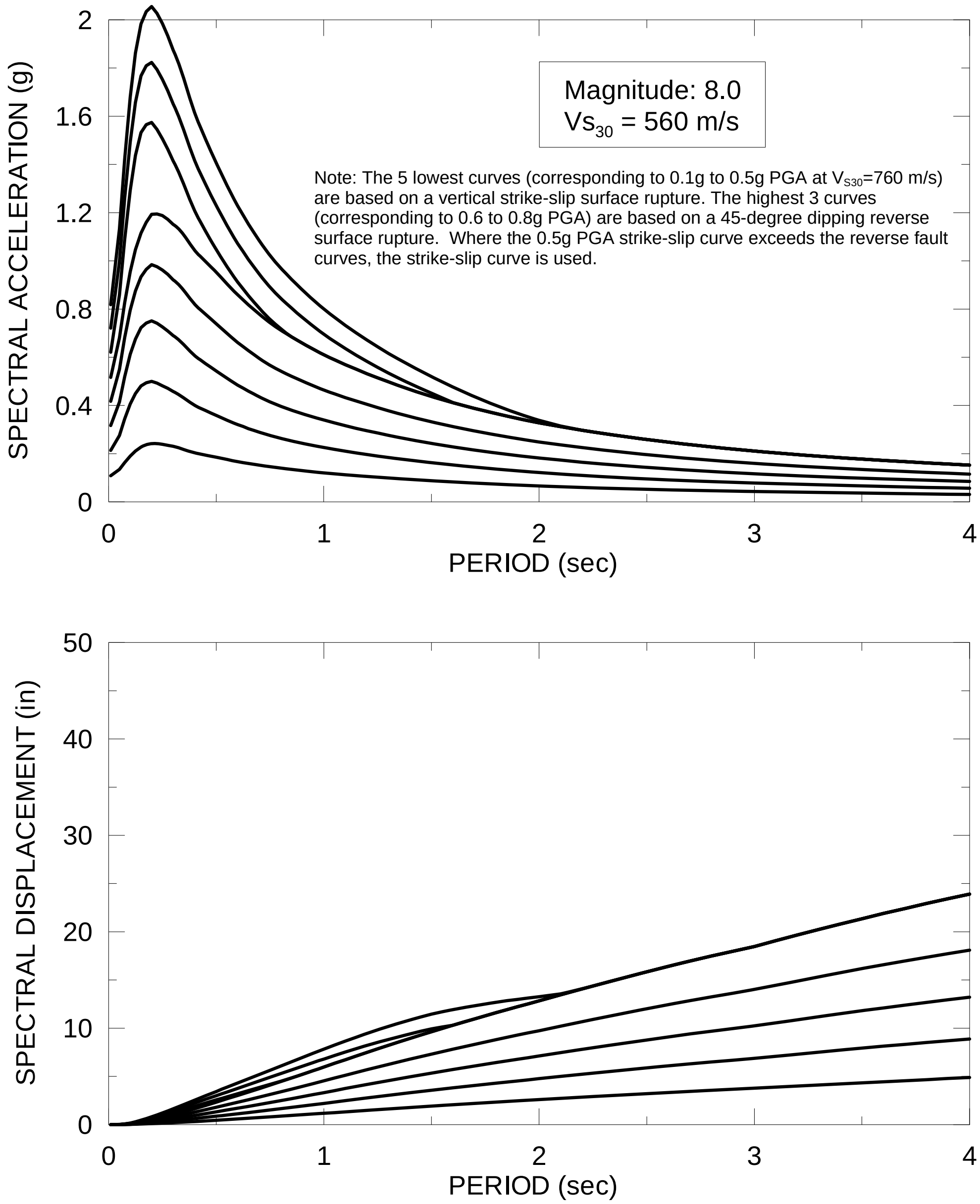
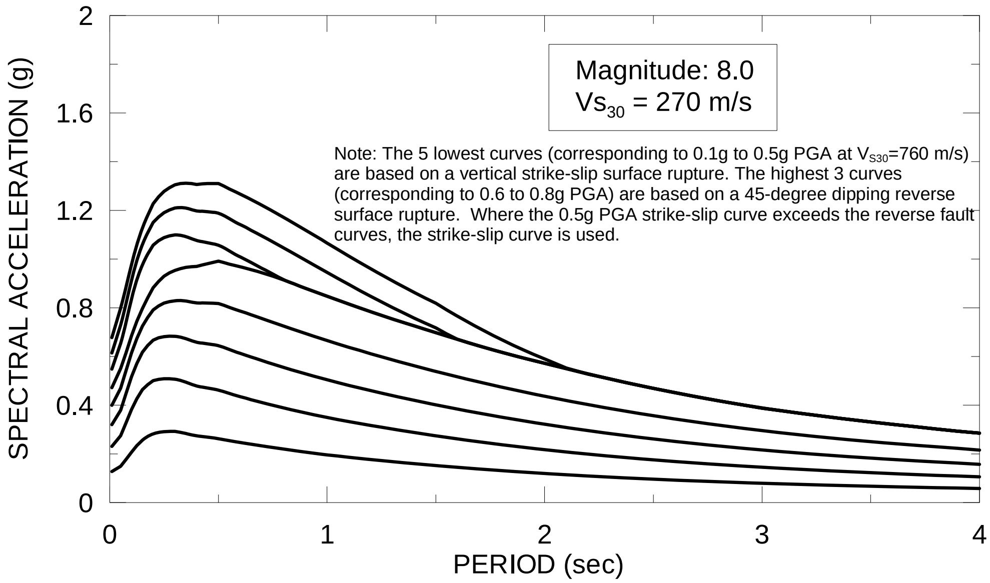


Figure B.23 Spectral Acceleration and Displacement for Vs30 = 560 m/s (*M* = 8.0)



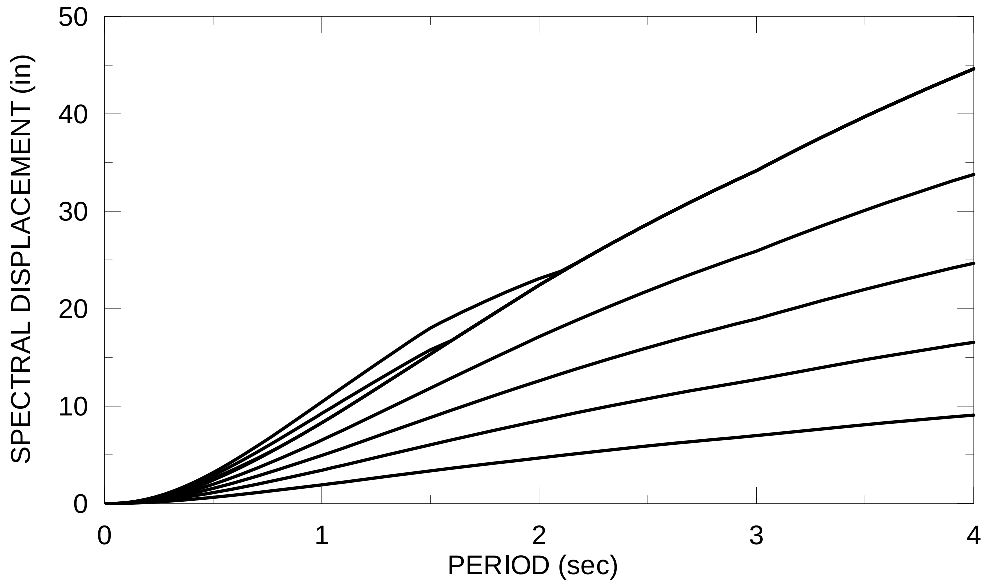


Figure B.24 Spectral Acceleration and Displacement for Vs30 = 270 m/s (*M* = 8.0)

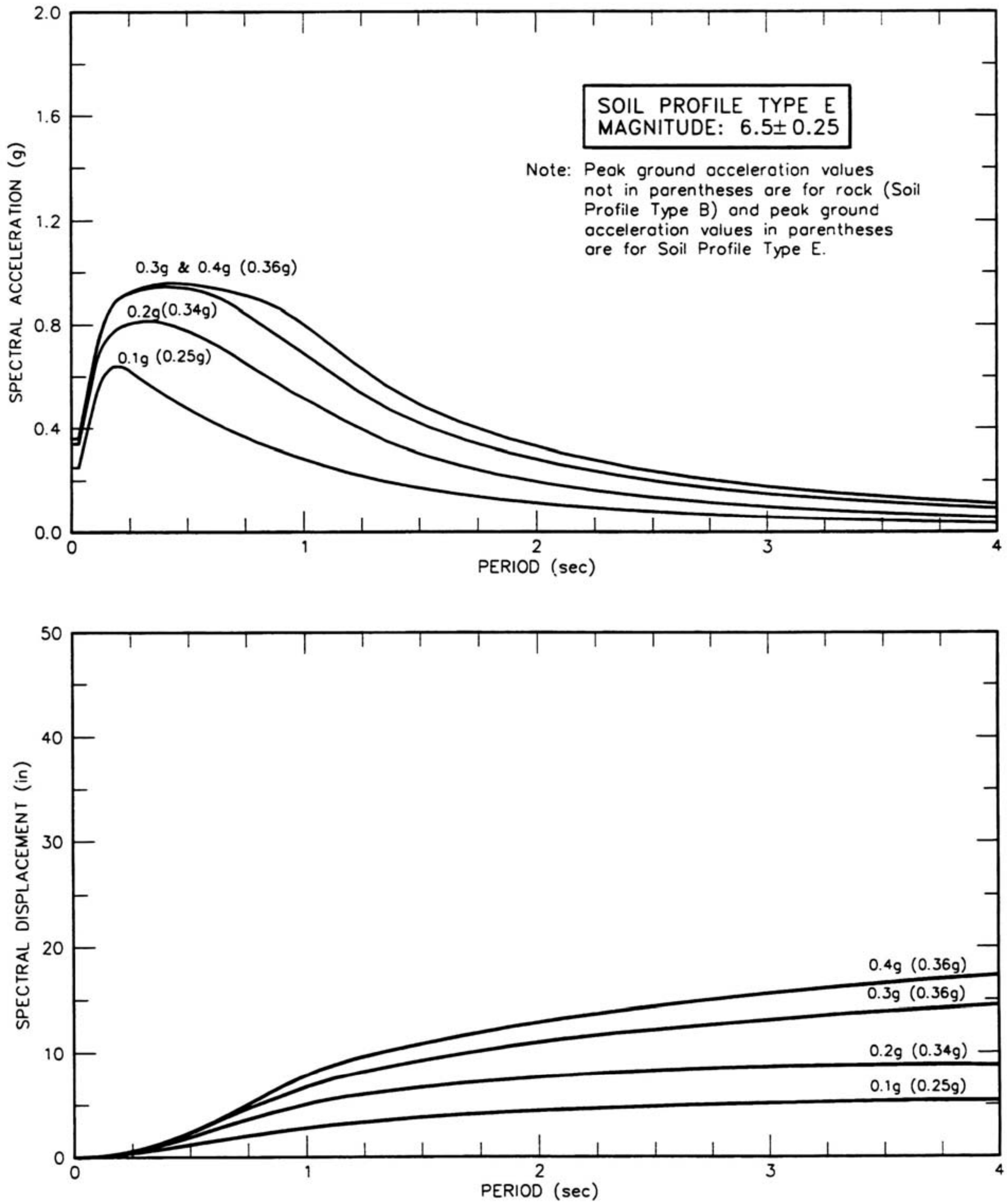


Figure B.25 Spectral Acceleration and Displacement for Soil Profile E (M = 6.5±0.25)

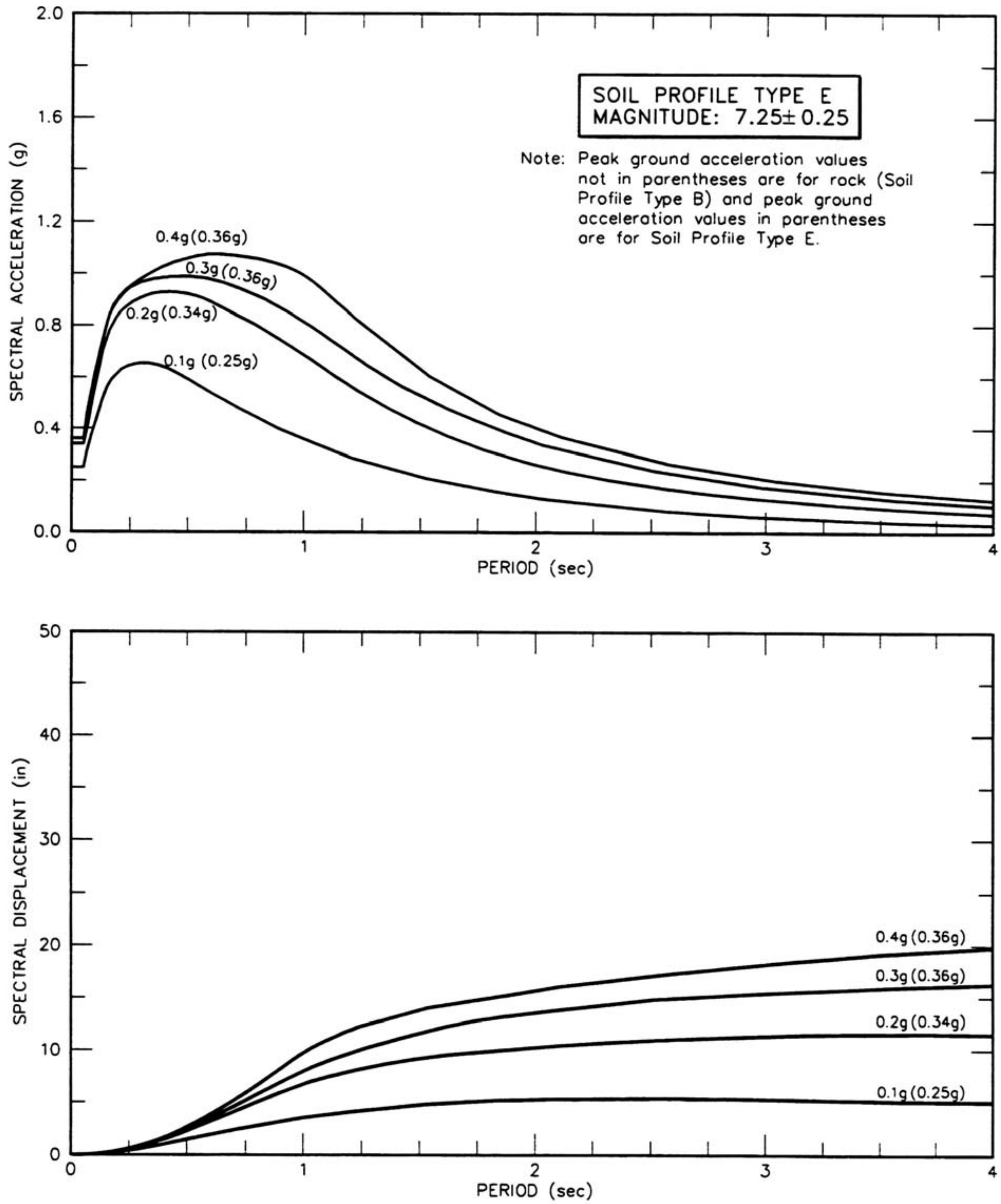


Figure B.26 Spectral Acceleration and Displacement for Soil Profile E (M = 7.25±0.25)

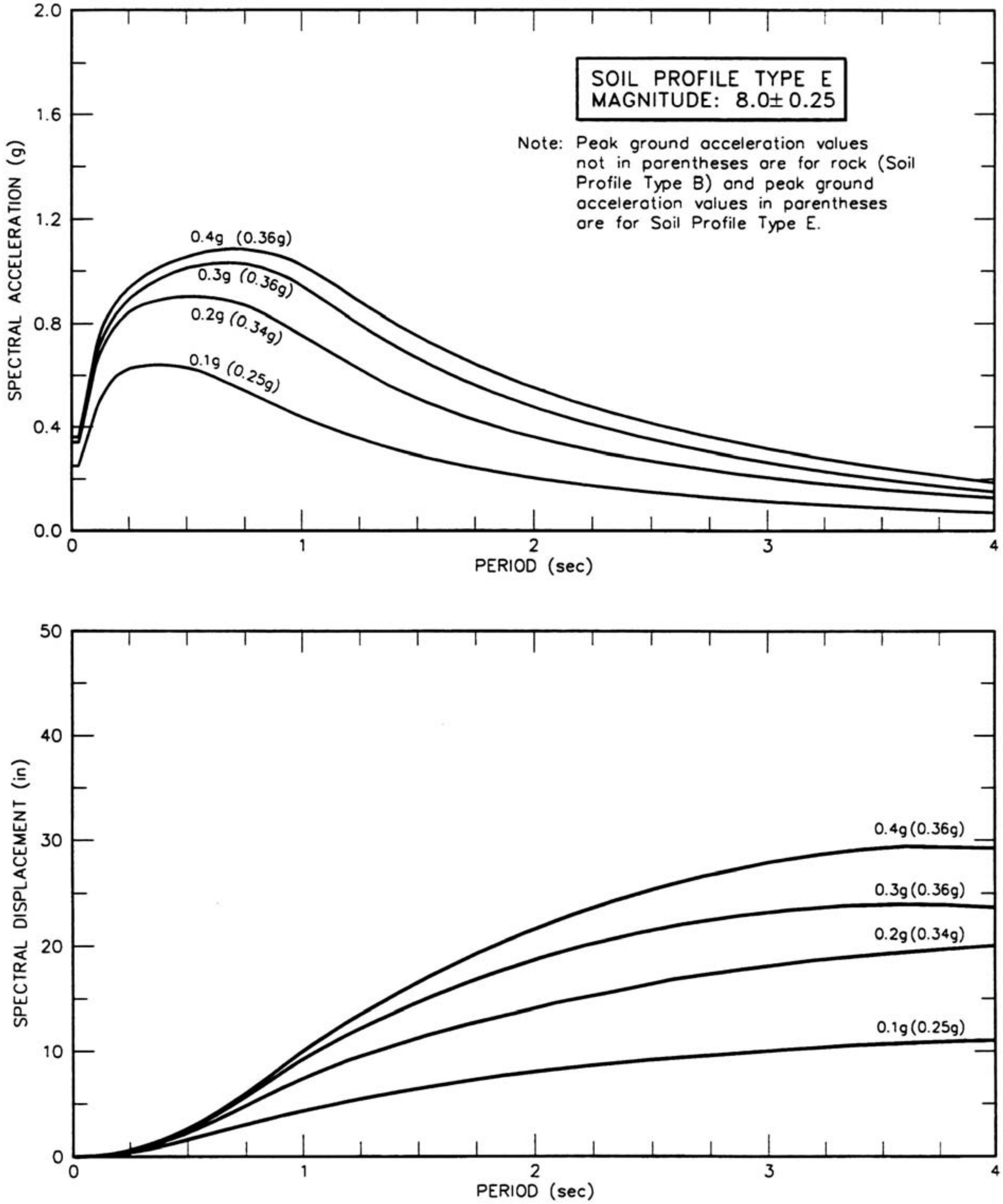


Figure B.27 Spectral Acceleration and Displacement for Soil Profile E (M = 8.0±0.25)