OCTOBER 2019 INTERIM REVISIONS

TO

CALTRANS
SEISMIC DESIGN CRITERIA
VERSION 2.0

State of California
Department of Transportation

APRIL 2019

CHANGED SECTIONS

Revisions have been made to Caltrans Seismic Design Criteria, Version 2.0 (April 2019). The affected sections are included in this package. The revised sections replace the contents of the corresponding sections of SDC 2.0 (April 2019). The pages of these Interim Revisions are not paginated to replace the corresponding pages of SDC 2.0 (April 2019).

SECTION 3: GENERAL REQUIREMENTS

3.2.1
3.2.1.1
C3.2.1.1
3.2.1.2
C3.2.1.2

SECTION 4: SEISMIC DEFORMATION DEMANDS AND ANALYSIS METHODS

4.2 (Table 4.2-1)

SECTION 6: FOUNDATIONS, ABUTMENTS, AND SOIL-FOUNDATION-STRUCTURE INTERACTION

6.2.6
C6.2.6

APPENDIX B: DESIGN SPECTRUM DEVELOPMENT

SECTION 3: GENERAL REQUIREMENTS

3.2.1 Ground Shaking

Ground shaking shall be characterized for design by the design spectrum. The design spectrum shall be based on the 2014 U.S. Geological Survey Seismic Hazard Maps. A qualified geo-professional shall provide final design spectrum recommendations.

C3.2.1

The design spectrum reflects the shaking hazard at or near the ground surface.

For design spectrum development, refer to Appendix B.

3.2.1.1 Safety Evaluation Earthquake

The design spectrum for Safety Evaluation Earthquake (SEE) shall be taken as a spectrum based on a 975-year return period (i.e., 5% probability of exceedance in 50 years).

C3.2.1.1

For a 75-year bridge design life, the design spectrum based on a 975-year return period represents a ground motion event that has approximately 7% probability of exceedance in 75 years.

A web-based design tool is available for use in the specification of the design spectrum: (https://arsonline.dot.ca.gov/).

A detailed discussion of the development of the design spectrum is given in Appendix B.

The deterministic criterion considered in previous editions of the SDC to determine the design spectrum is eliminated from SDC 2.0.

3.2.1.2 Functional Evaluation Earthquake

The Design Spectrum for Functional Evaluation Earthquake (FEE) shall be taken as a spectrum based on a 225-year return period (i.e., 20 % probability of exceedance in 50 years).

C3.2.1.2

The FEE design spectrum can be obtained using the USGS Uniform Hazard Tool at: (https://earthquake.usgs.gov/hazards/interactive/). The FEE spectrum is constructed by running the hazard tool for a 225-year return period hazard. The FEE design spectrum must include near-fault and basin amplification factors as specified in Appendix B for the SEE Design Spectrum.

SECTION 4:
SEISMIC DEFORMATION DEMANDS AND ANALYSIS METHODS

Table 4.2-1 Applicability of Methods for Displacement Demand Analysis

PARAMETER	ANALYSIS METHOD		
	ESA	**EDA**	**NTHA**
Maximum bridge length	1000 ft	3000 ft*1	No restriction
Maximum skew angle	30°	No restriction	No restriction
Maximum bearing difference between any two supports*2	5°	20°	No restriction

*1 The maximum bridge length requirement shall not apply when EDA is used for viaducts with repeating frame systems and geometry.

*2 The maximum bearing difference between any two supports shall apply to the entire bridge irrespective of the number of frames in the bridge. The maximum bearing difference between any two supports shall not be applicable for bridges supported by round or square single or multi column bents.

SECTION 6:

FOUNDATIONS, ABUTMENTS, AND SOIL-FOUNDATION-STRUCTURE INTERACTION

6.2.6 Lateral Stability of Shafts

In order to determine the tip elevation for horizontal loading, lateral stability analysis shall be performed for shafts supporting single-column bents in Class S2 soils based on the following assumptions:

- The static lateral load and the dead load are applied at the top of the column
- The effects of scour and liquefaction are considered, if applicable

The critical length for lateral stability shall be taken as the embedded length of the shaft for which greater lengths do not result in a reduction of 5% or more in the deflection at the shaft cut-off elevation.

The length of shaft for lateral stability shall be greater than or equal to the critical length multiplied by the Embedment Factor shown in Table 6.2.6-1.

C6.2.6

Lateral stability analysis is accomplished by applying static lateral loads with dead load at the top of the column for a range of shaft lengths and recording the resulting top of shaft deflections as shown in Figure C6.2.6-1.

Types I and II shafts and shaft/pile groups founded in Class S1 soil or supporting multicolumn bents are laterally stable, and do not need to be analyzed for lateral stability.

Table 6.2.6-1 Embedment Factor for Shaft Tip Elevation

Shaft		Embedment Factor
Shafts (Types I and II) without rock sockets supporting single-column bents in Class S2 soil		1.2
Shafts (Types I and II) with rock sockets supporting single-column bents in Class S2 soil	portion not in rock socket	1.0
	portion in rock socket	1.2

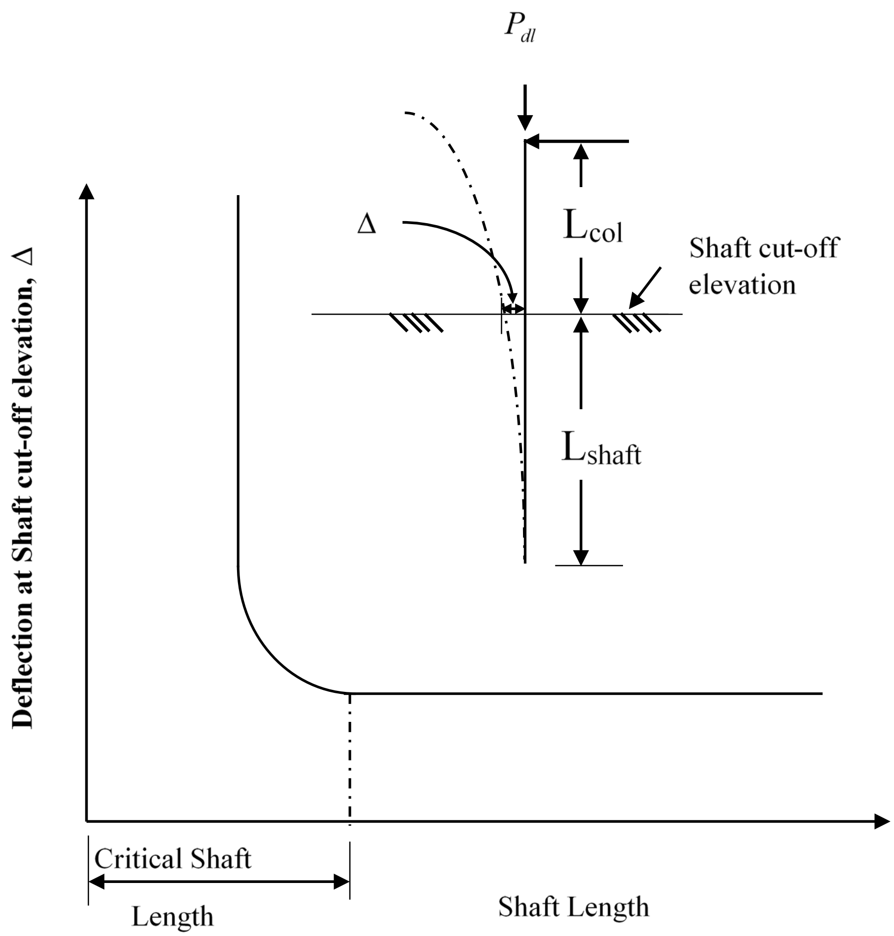

Figure C6.2.6-1 Lateral Stability Analysis of Shafts

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. The design spectrum for a Safety Evaluation Earthquake is based on the 2014 USGS Seismic Hazard Map for the 5% in 50 years probability of exceedance (or 975-year return period) with adjustment factors for near-fault and basin amplification effects as described in the sections below.

Spectrum Adjustment Factors

The design spectrum may need to account for seismological effects related to the 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.1, 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 no adjustment at 25 km (15.6 miles). The adjustment consists of a 20% increase in spectral values with corresponding period longer than one second. This increase is linearly tapered to zero at a period of 0.5 second. Since the design spectrum is probabilistically based and includes the influence of multiple faults, the site to rupture plane distance is based on the deaggregated mean distance for spectral acceleration at a period of 1.0 second.

Basin Factor

Both the Campbell-Bozorgnia (2014) and Chiou-Youngs (2014) 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 tend to be located in basin structures, the term "basin effect" is commonly used.

Amplification factors for the two models (2014 versions) are shown for various depths to rock in Figure B.2. These plots assume a shear wave velocity for the upper 30 m of the soil profile, v_{s30} of 259 m/s (typical for many basin locations) but are suitable for other v_{s30} values as well since the basin effect is only moderately sensitive to v_{s30}. 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 450 m. Basin amplification factors less than 1.0 are not allowed.

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

Figures B.3 through B.9 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://scec.usc.edu/scecpedia/Community_Velocity_Model). 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). 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 (2014) and Chiou-Youngs (2014) basin amplification factors is applied. For Northern California locations, only the Campbell-Bozorgnia (2014) 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. The probabilistic median (rotated) response spectra (Boore, 2010) defined above reflect a spectrum that is equally probable in all orientations. The maximum response spectrum, occurring at an unpredictable orientation, is approximately 15% to 25% larger than the equally probable spectrum calculated using the procedures described above.

Selection of v_{s30} for Site Amplification

Recent generations of ground motion prediction models 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 follows:

$$V_{s30} = \frac{100\,ft}{\frac{D_1}{v_1} + \frac{D_2}{v_2} + \ldots + \frac{D_n}{v_n}}$$

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 used.

Figure B.10 provides a profile classification system that was published in Applied Technology Council-32 (1996) and was adopted in previous versions of SDC. USGS 2014 hazard maps provide hazard results for v_{s30} ranging from 180 m/s (590 fps) to 1150 m/s (3775 fps). For cases where v_{s30} exceeds 1150 m/s (very rare in California), a value of 1150 m/s should be used. A site-specific ground response analysis is required for determination of the final design spectrum for cases where (1) v_{s30} is less than 180 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.10.

For cases where the site meets the criteria prescribed for Soil Profile Type E, the response spectra presented in Figures B.11 - B.13, 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-specific ground response analysis methods. For this reason, it is preferred that a site-specific ground 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.10), a site-specific ground response analysis is required for final design.

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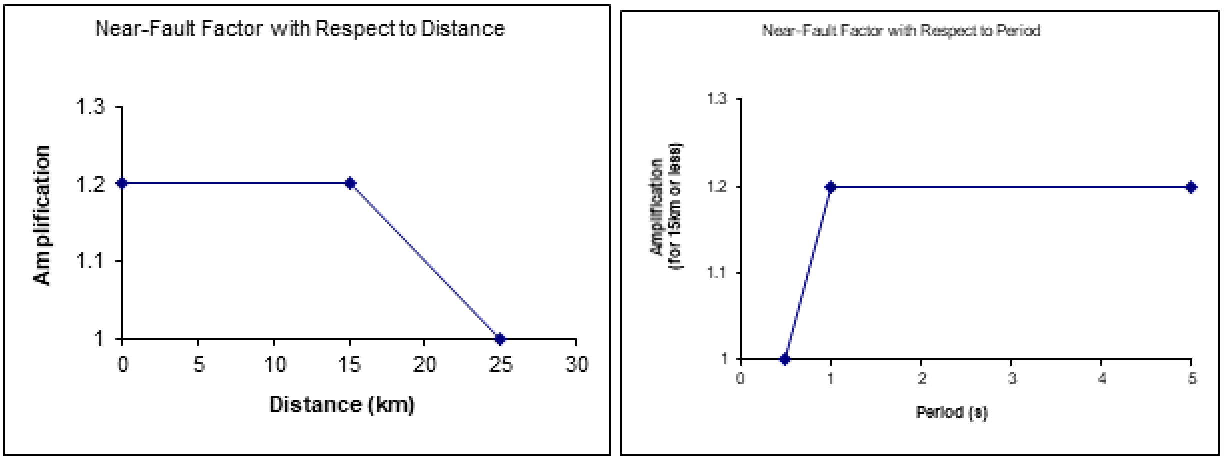

Figure B.1 Near-Fault adjustment factor as a function of distance and spectral period. The distance measure is based on the closest distance to any point on the fault plane

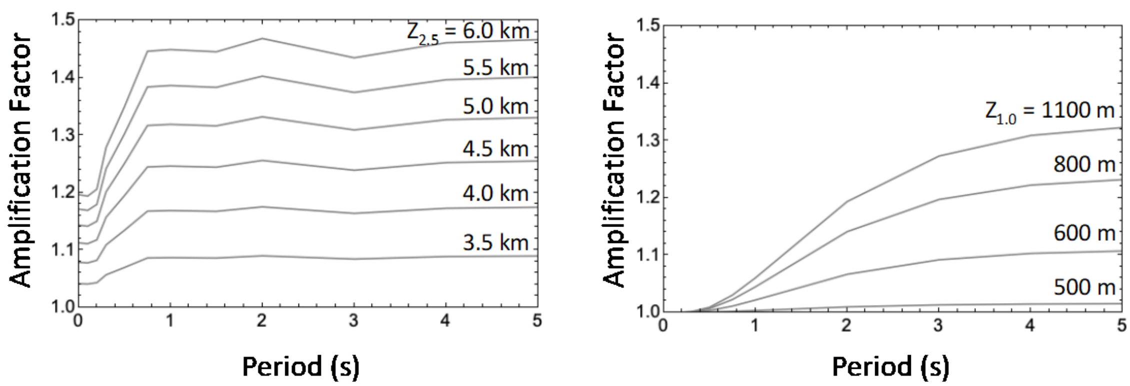

Figure B.2 Basin amplification factors for the Campbell-Bozorgnia (2014) and Chiou-Youngs (2014) ground motion prediction equations. Chiou-Youngs is V_{S30} dependent. Plotted are basin amplification factors for V_{S30} = 259 m/s.

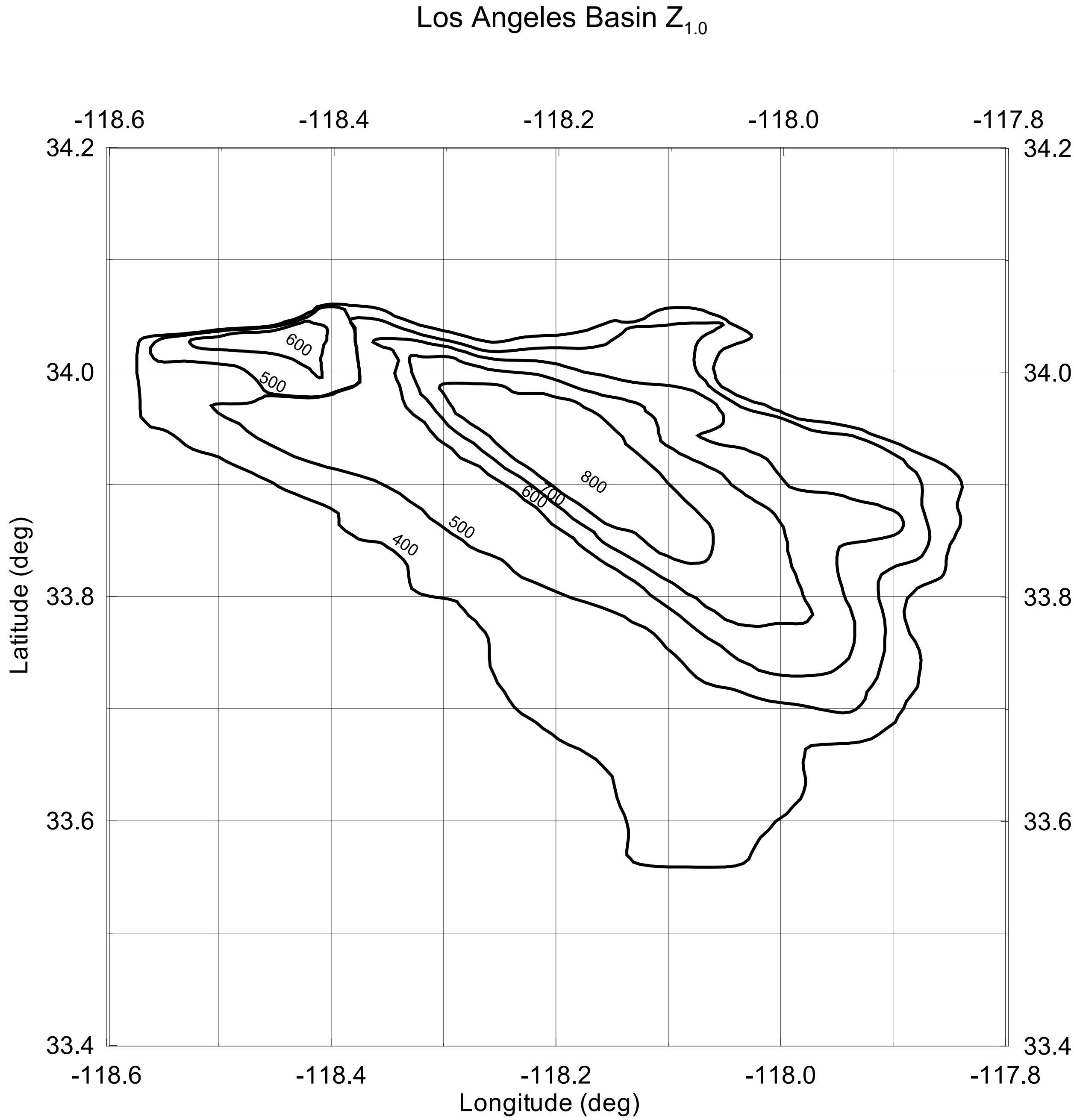

Figure B.3 Contours of depth (meters) to shear wave velocity 1 km/s ($Z_{1.0}$) in the Los Angeles Basin

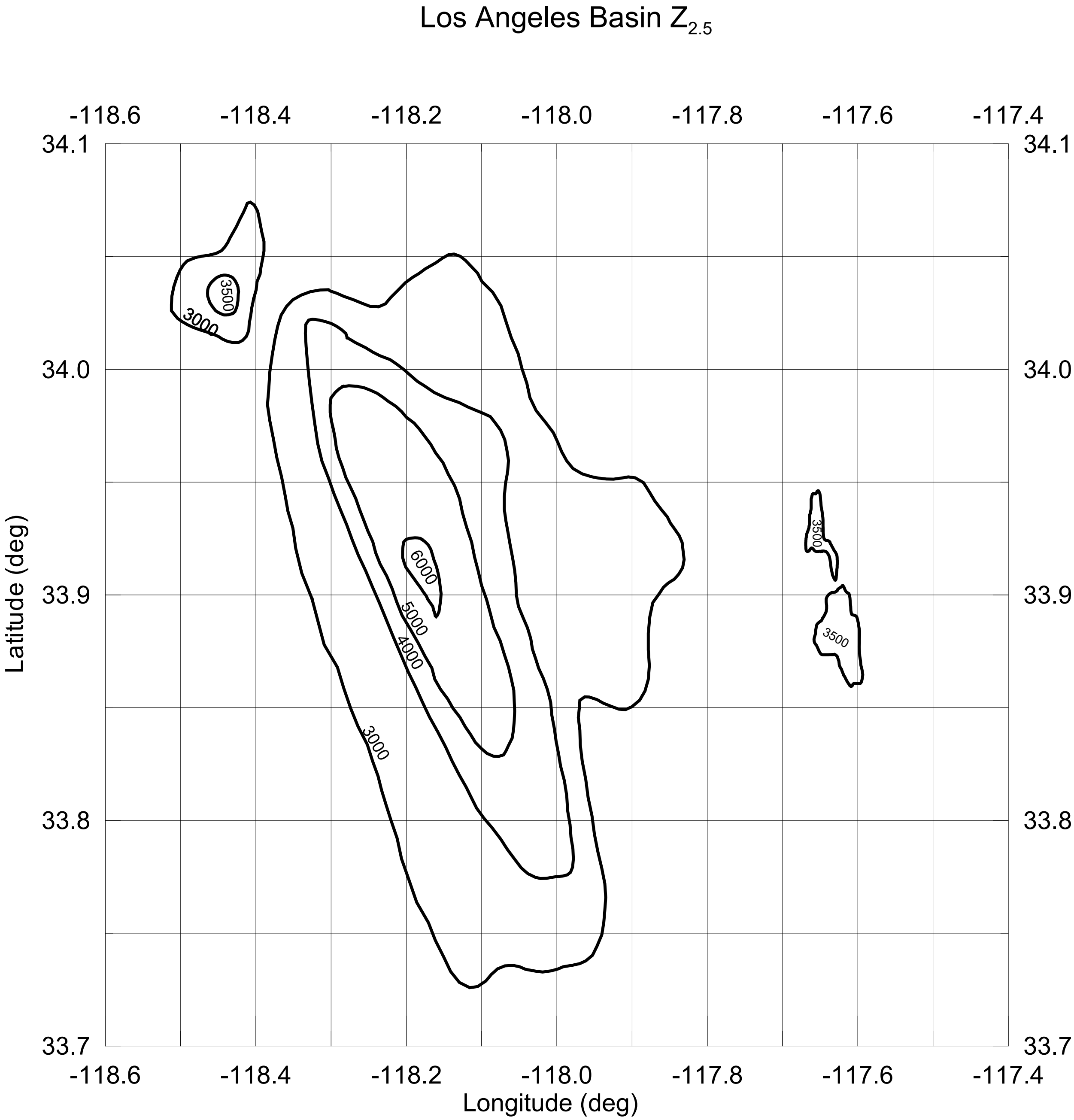

Figure B.4 Contours of depth (meters) to shear wave velocity 2.5 km/s ($Z_{2.5}$) in the Los Angeles Basin

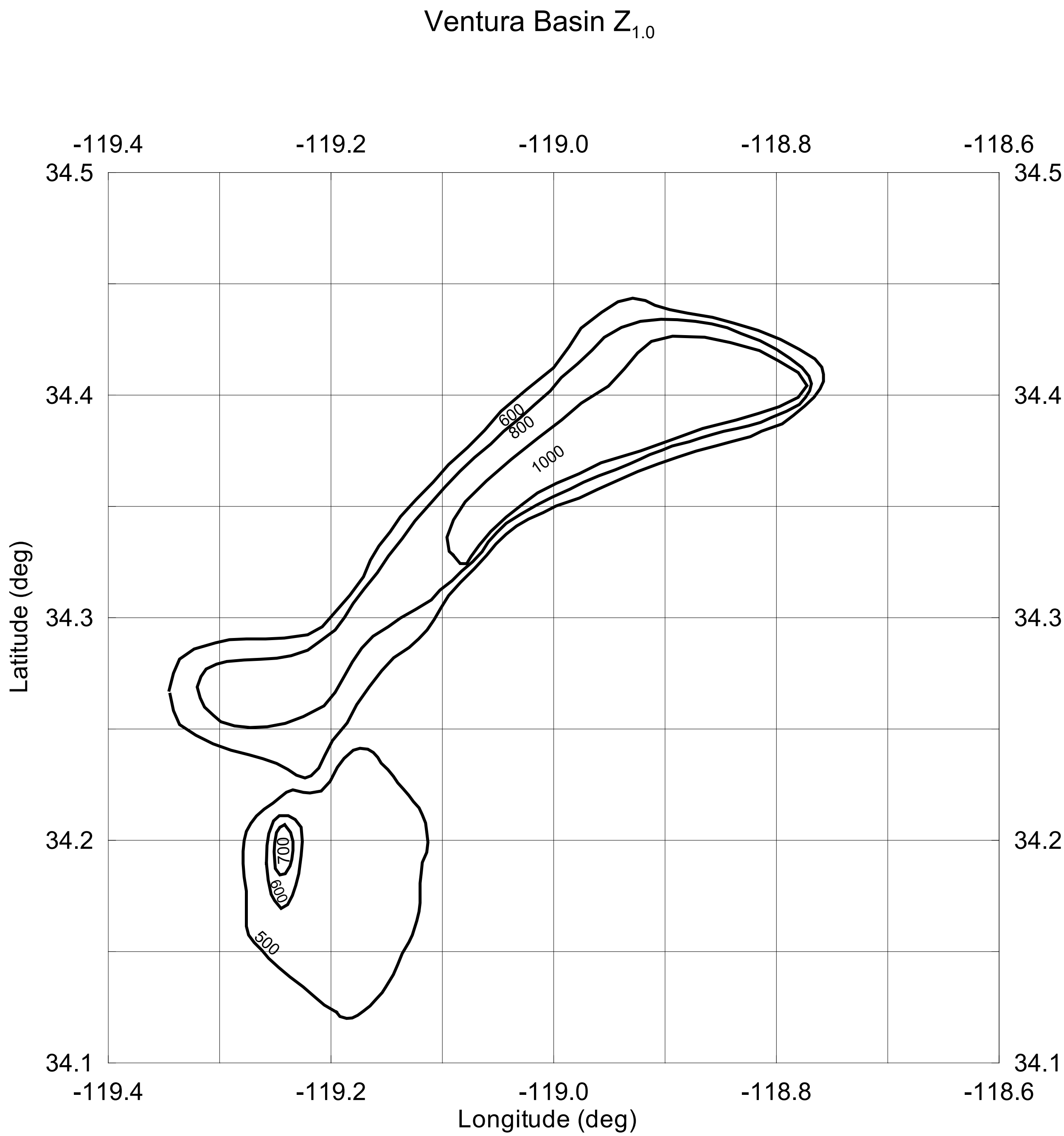

Figure B.5 Contours of depth (meters) to shear wave velocity 1 km/s ($Z_{1.0}$) in the Ventura Basin

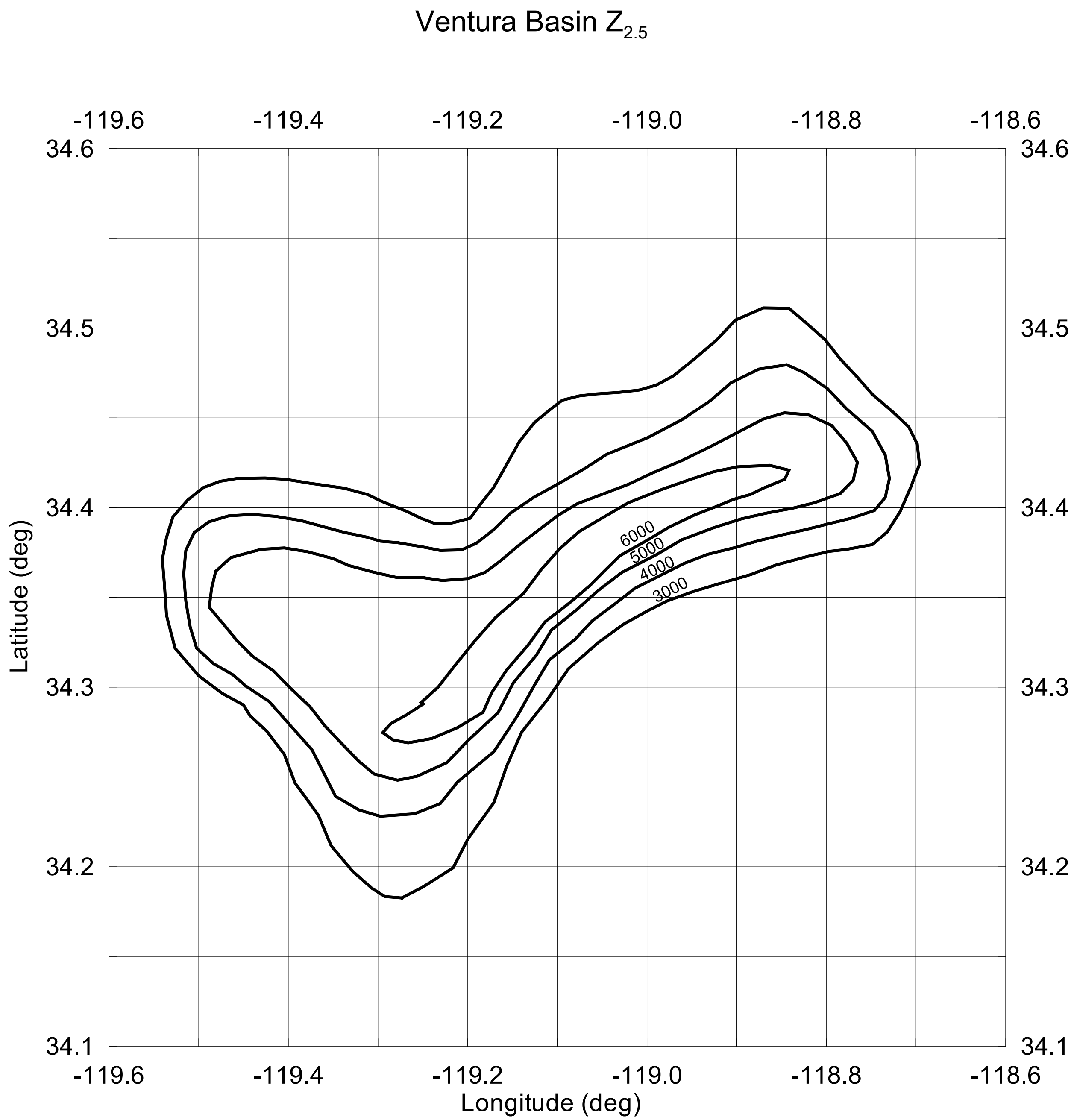

Figure B.6 Contours of depth (meters) to shear wave velocity 2.5 km/s ($Z_{2.5}$) in the Ventura Basin

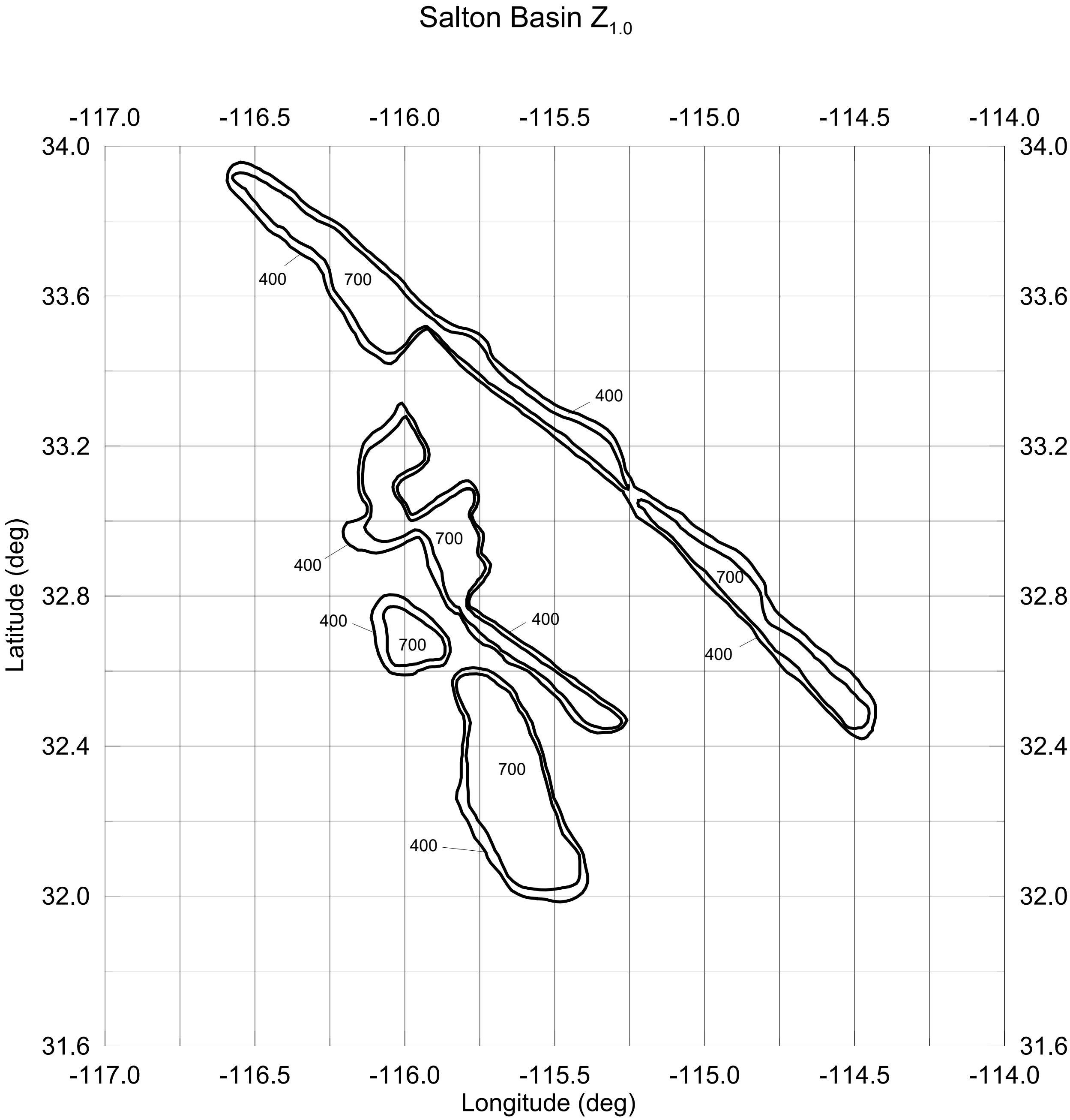

Figure B.7 Contours of depth (meters) to shear wave velocity 1 km/s ($Z_{1.0}$) in the Salton Basin (Imperial Valley)

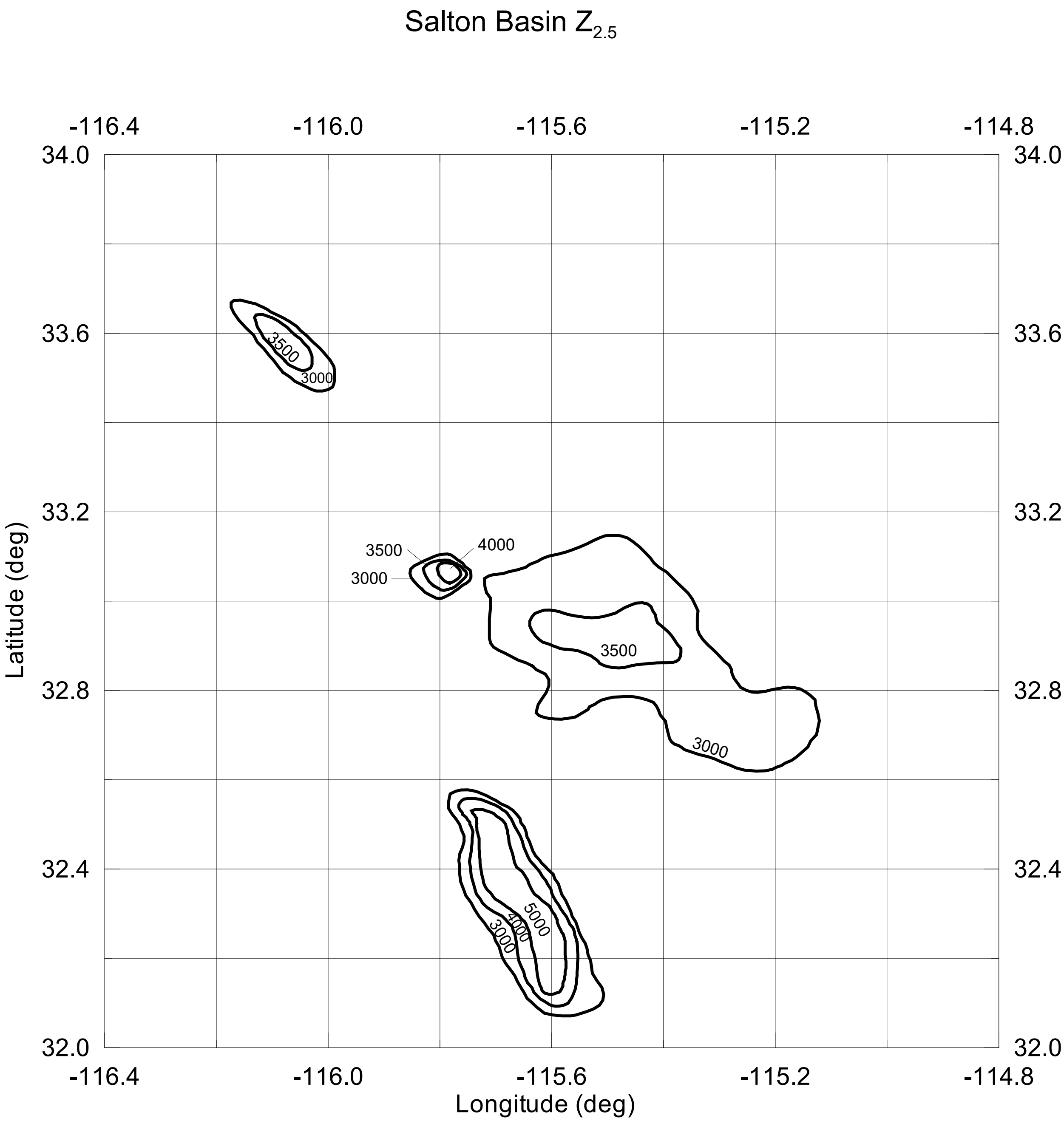

Figure B.8 Contours of depth (meters) to shear wave velocity 2.5 km/s ($Z_{2.5}$) in the Salton Basin (Imperial Valley)

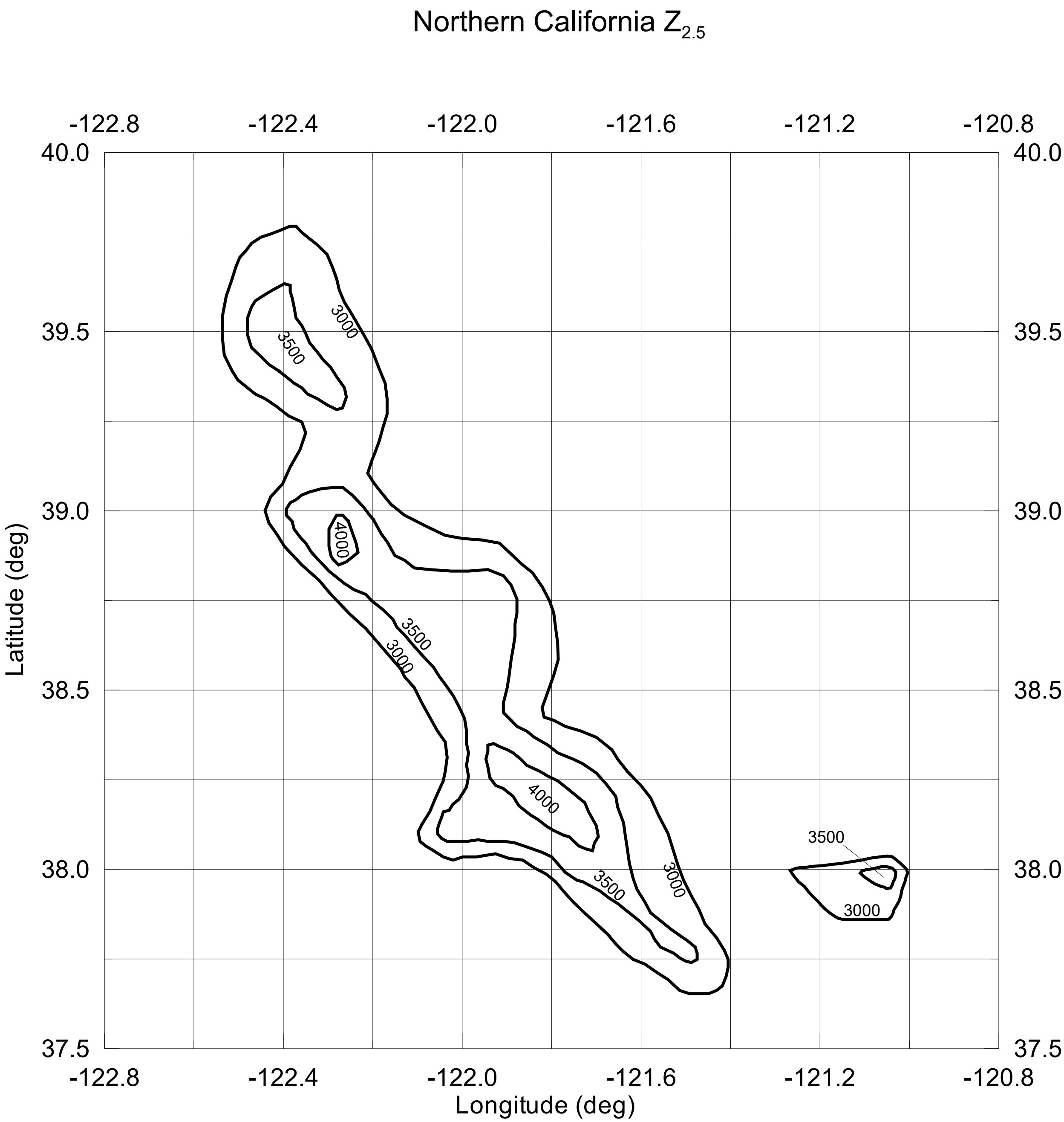

Figure B.9 Contours of depth (meters) to shear wave velocity 2.5 km/s ($Z_{2.5}$) in Northern California

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 $w \geq$ 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

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

Figure B.10 Soil profile types (after Applied Technology Council-32-1, 1996)

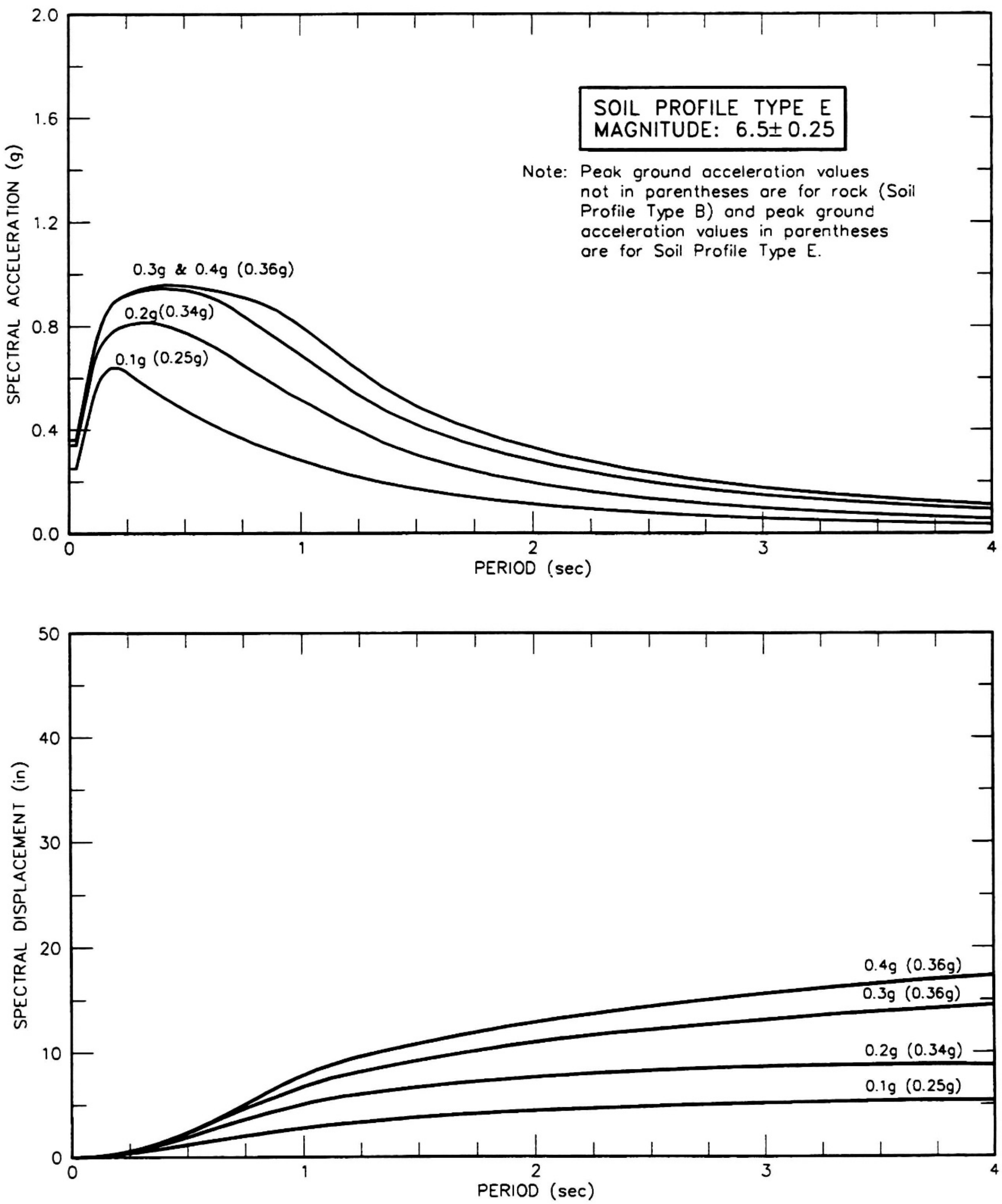

Figure B.11 Spectral Acceleration and Displacement for Soil Profile E (M = 6.5±0.25)

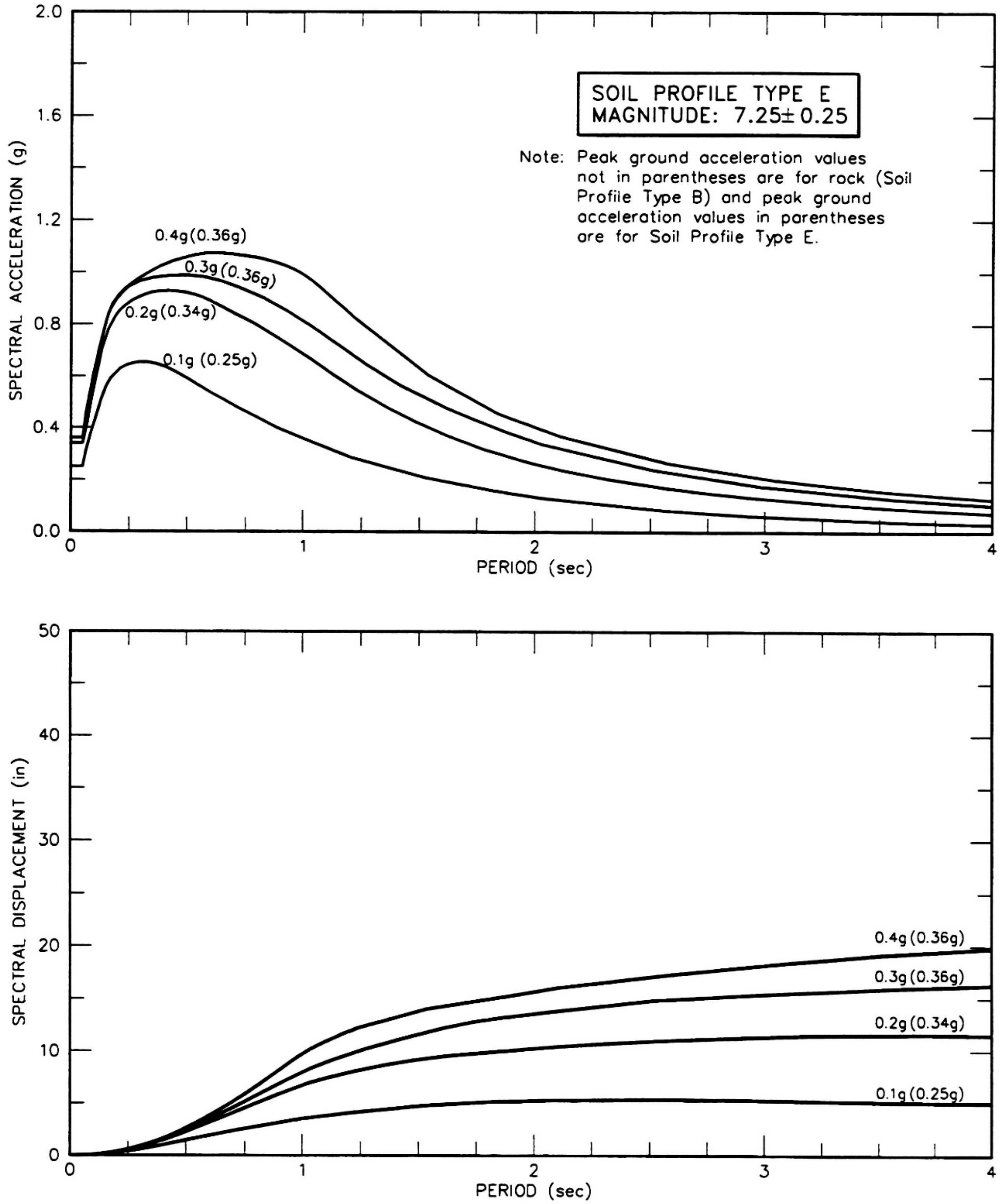

Figure B.12 Spectral Acceleration and Displacement for Soil Profile E (M = 7.25±0.25)

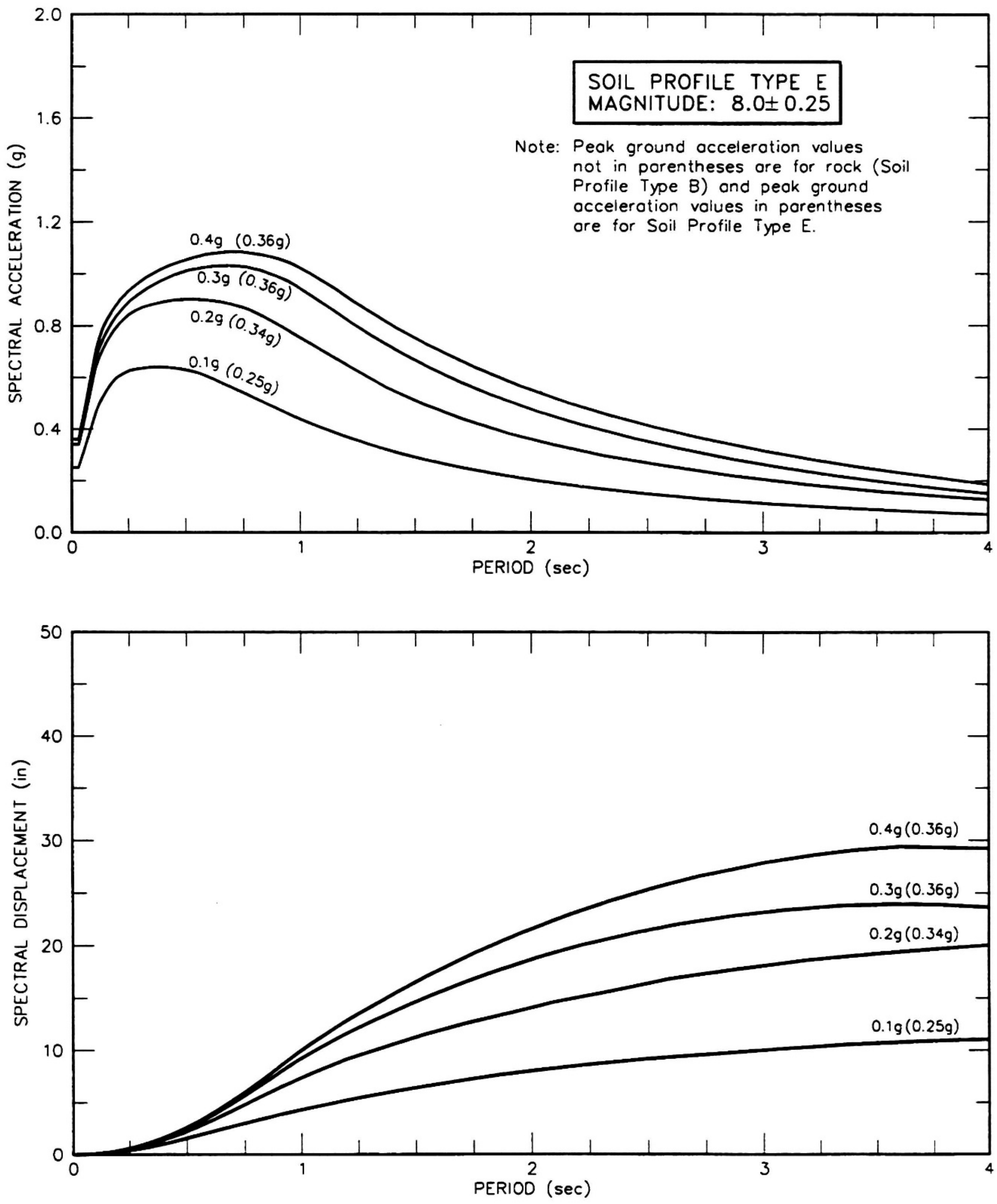

Figure B.13 Spectral Acceleration and Displacement for Soil Profile E (M = 8.0±0.25)