PROPOSAL 3-121 (2009)

SCOPE: Part 2, Commentary Chapter 21

PROPOSAL FOR CHANGE:

Add Chapter 21 to Part 2, of the 2009 Commentary:

Proposed Chapter is attached. Text is not underlined to allow easier review.

REASON FOR PROPOSAL:

One of the basic tasks of the 2009 NEHRP *Provisions* update is to develop a viable commentary to Part 1. Since Part 1 adopts ASCE 7-05 and lists any exceptions to it, the Commentary is developed in accordance with the format and sections of ASCE 7-05.

TS 3 VOTE:

TS 3 developed this commentary chapter and approved for submission. The chapter was edited and accepted by TS 3 .

Chapter 21

SITE-SPECIFIC GROUND MOTION PROCEDURES FOR SEISMIC DESIGN

GENERAL

Site-specific procedures for computing earthquake ground motions include (1) dynamic site response analyses, and (2) probabilistic and deterministic seismic hazard analyses (PSHA and DSHA), which may include dynamic site response analysis as part of the calculation. Use of site-specific procedures may be required in lieu of the general procedure in Sections 11.4.1 through 11.4.6; Section C11.4.7 explains the conditions under which the use of these procedures is required. Such studies must be comprehensive and incorporate current scientific interpretations. Because there is typically more than one scientifically credible alternative for models and parameter values used to characterize seismic sources and ground motions, it is important to formally incorporate these uncertainties in a site-specific analysis. For example, uncertainties may exist in seismic source location, extent and geometry; maximum earthquake magnitude; earthquake recurrence rate; ground-motion attenuation; local site conditions, including soil layering and dynamic soil properties; and possible two- or three-dimensional wave-propagation effects. The use of peer review for a site-specific ground-motion analysis is encouraged.

Site-specific ground-motion analysis can consist of one of the following approaches: (1) PSHA and possibly DSHA if the site is near an active fault, (2) PSHA/DSHA followed by dynamic site-response analysis, and (3) dynamic site response analysis only. The first approach is used to compute ground motions for bedrock or stiff soil conditions (not softer than Site Class D). In this approach, if the site consists of stiff soil overlying bedrock, for example, the analyst has the option of either (1) computing the bedrock motion from the PSHA/DSHA and then using the site-coefficient (F_a and F_v) tables in Section 11.4.3 to adjust for the stiff soil overburden, or (2) computing the response spectrum at the ground surface directly from the PSHA/DSHA. The latter requires the use of attenuation equations for computing stiff soil-site response spectra (instead of bedrock response spectra).

The second approach is used where softer soils overlie the bedrock or stiff soils. The third approach assumes that a site-specific PSHA/DSHA is not necessary, but that a dynamic site response analysis should or must be performed. This analysis requires the definition of an outcrop ground motion, which can be based on the 5 percent damped response spectrum computed from the PSHA/DSHA or obtained from the general procedure in Chapter 11.4. A representative set of acceleration time histories are selected and scaled to be compatible with this outcrop spectrum. Dynamic site response analyses using these acceleration histories as input~~,~~ are used to compute motions at the ground surface. The response spectra of these surface motions are used to define a Maximum Considered Earthquake (MCE) ground motion response spectrum.

The approaches described above have advantages and disadvantages. In many cases, user preference governs the selection, but geotechnical conditions at the site may dictate the use of one approach over the other. On the one hand, if bedrock is at a depth much greater than the extent of the site geotechnical investigations, the direct approach of computing the ground-surface motion in the PSHA/DSHA may be more reasonable. On the other hand, if bedrock is shallow and a large impedance contrast exists between it and the overlying soil (that is, density times shear-wave velocity of bedrock is much greater than that of the soil), the two-step approach might be more appropriate.

Use of peak ground acceleration as the anchor for a generalized site-dependent response spectrum is discouraged because sufficiently robust ground-motion attenuation relations are available for computing response spectra in western ~~U.S.~~United States and eastern ~~U.S.~~United States tectonic environments.

C21.1 SITE RESPONSE ANALYSIS

C21.1.1 Base Ground Motions. Ground motion acceleration histories that are representative of horizontal rock motions at the site are required as input to the soil model. Where a site-specific ground motion hazard analysis is not performed, the ~~maximum considered earthquake (~~MCE~~)~~ response spectrum for Site Class B (rock) is defined using the general procedure described in Section 11.4.1. If the model is terminated in material of Site Class A, C, or D, the input MCE response spectrum is adjusted in accordance with Section 11.4.3. The ~~U.S.~~United States Geological Survey national seismic hazard mapping project website (http://earthquake.cr.usgs.gov/research/hazmaps/) includes hazard deaggregation options that can be used to evaluate the predominant types of earthquake sources, magnitudes, and distances contributing to the probabilistic ground-motion hazard. Sources of recorded acceleration time histories include the databases of the Consortium of Organizations for Strong Motion Observation Systems (COSMOS) Virtual Data Center web~~-~~site (db.cosmos-eq.org) and the Pacific Earthquake Engineering Research Center (PEER) Strong Motion Data Base website (http://peer.berkeley.edu/products/strong_ground_motion_db.html/), and the ~~U.S.~~United States National Center for Engineering Strong Motion Data (NCESMD) website (http://www.strongmotioncenter.org). Ground motion acceleration histories at these sites generally were recorded at the ground surface and hence apply for an outcropping condition and should be specified as such in the input to the site response analysis code (see Kwok et al., 2007, for additional details).

C21.1.2 Site Condition Modeling. Modeling criteria are established by site-specific geotechnical investigations that should include (~~i~~1) borings with sampling, (~~ii~~2) standard penetration tests (SPTs), cone penetrometer tests (CPTs), and/or other subsurface investigative techniques, and (~~iii~~3) laboratory testing to establish the soil types, properties, and layering. The depth to rock or stiff soil material should be established from these investigations. Investigation should extend to bedrock or, for very deep soil profiles, to material in which the model will be terminated. While it is preferable to measure shear wave velocities in all soil layers, it is also possible to estimate shear wave velocities based on measurements available for similar soils in the local area or through correlations with soil types and properties. A number of such correlations are summarized by Kramer (1996).

Typically, a one-dimensional soil column extending from the ground surface to bedrock is adequate to capture first-order site response characteristics. For very deep soils, the model of the soil columns may extend to very stiff or very dense soils at depth in the column. Two- or three-dimensional models should be considered for critical projects when two- or three-dimensional wave propagation effects may be significant (for example, sloping ground sites). The soil layers in a one-dimensional model are characterized by their total unit weights and shear wave velocities from which low-strain (maximum) shear moduli may be obtained, and by relationships defining the nonlinear shear stress-strain behavior of the soils. The required relationships for analysis are often in the form of curves that describe the variation of soil shear modulus with shear strain (modulus reduction curves) and by curves that describe the variation of soil damping with shear strain (damping curves). In a two- or three-dimensional model, compression wave velocities or moduli or Poisson ratios also are required. In an analysis to estimate the effects of liquefaction on soil site response, the nonlinear soil model also must incorporate the buildup of soil pore water pressures and the consequent reductions of soil stiffness and strength. Typically, modulus reduction curves and damping curves are selected on the basis of published relationships for similar soils (for example, Vucetic and Dobry, 1991; Electric Power Research Institute, 1993; Darendeli, 2001; Menq, 2003; Zhang et al., 2005). Site-specific laboratory dynamic tests on soil samples to establish nonlinear soil characteristics can be considered where published relationships are judged to be inadequate for the types of soils present at the site. Shear and compression wave velocities and associated maximum moduli should be selected based on field tests to determine these parameters or, if such tests are not possible, on published relationships and experience for similar soils in the local area. The uncertainty in the selected maximum shear moduli, modulus reduction and damping curves, and other soil properties should be estimated (see Darendeli, 2001;~~,~~ and Zhang et al., 2008).

C21.1.3 Site Response Analysis and Computed Results. Analytical methods may be equivalent linear or nonlinear. Frequently used computer programs for one-dimensional analysis include the equivalent linear program SHAKE (Schnabel et al., 1972; Idriss and Sun, 1992) and the nonlinear programs FLAC (Itasca, 2005), DESRA-2 (Lee and Finn, 1978), MARDES (Chang et al., 1991), SUMDES (Li et al., 1992), DMOD_2 (Matasovic, 2006), DEEPSOIL (Hashash and Park, 2001), TESS (Pyke, 2000), and OpenSees (Ragheb, 1994; Parra, 1996; Yang, 2000). If the soil response induces large strains in the soil (such as for high acceleration levels and soft soils), nonlinear programs may be preferable to equivalent linear programs. For analysis of liquefaction effects on site response, computer programs incorporating pore water pressure development (effective stress analyses) should be used (for example, FLAC, DESRA-2, SUMDES, D-MOD, TESS, DEEPSOIL, and OpenSees). Response spectra of output motions at the ground surface are calculated as the ratios of response spectra of ground-surface motions to input outcropping rock motions. Typically, an average of the response spectral ratio curves is obtained and multiplied by the input MCE response spectrum to obtain the MCE ground-surface response spectrum. Alternatively, the results of site-response analyses can be used as part of the PSHA using procedures described by Goulet et al. (2007) and programmed for use in OpenSHA (www.opensha.org; Field et al., 2005). Sensitivity analyses to evaluate effects of soil-property uncertainties should be conducted and considered in developing the final MCE response spectrum.

C21.2 GROUND MOTION HAZARD ANALYSIS

Uncertainties in the characterizations of the key seismic sources (tectonic provinces, zones of seismicity, and active faults), with respect to location, earthquake recurrence, and maximum earthquake magnitude, must be considered in the ground motion hazard analysis. Uncertainties in the ground-motion models are typically included by incorporating more than one ground-motion attenuation equation. However, these equations may underestimate the intermediate- and long-period motion from large earthquakes on nearby active faults due to directivity and directionality effects mentioned in C11.4.7. The probabilistic seismic hazard analysis code can be modified to account for these effects in a consistent probabilistic manner, or a deterministic adjustment can be made to the probabilistic MCE response spectrum using methods in Somerville et al. (1997) and Abrahamson (2000) or more recent procedures. If the deterministic adjustment is used, then judgment must be exercised in selecting the parameters comprising these methods. The worst-case scenario yielding the maximum possible increase in motion from directivity/directionality effects is acknowledged to be conservative, but it offers an ~~upper~~ upper-bound solution to help gauge the appropriate level for the MCE response spectrum.

Site-response effects in PSHA generally should be evaluated by using the site term in the ground-motion prediction equations. This term is generally a scale factor or a function of V_{s30} = average shear-wave velocity in the upper 30 meters. Site-specific dynamic response analyses can also be performed as described in Section C21.1.

C21.2.1 Probabilistic MCE.

~~Probabilistic seismic hazard analysis (~~PSHA~~)~~ methods are sufficient to define the MCE ground motion at all locations except those near highly active faults. Descriptions of current PSHA methods can be found in McGuire (2004).

C21.2.2 Deterministic MCE.

Ground motions for the deterministic MCE shall be based on characteristic earthquakes on all known active faults in a region. The magnitude of a characteristic earthquake on a given fault should be a best estimate of the maximum magnitude capable for that fault but not less than the largest magnitude that has occurred historically on the fault. The maximum magnitude should be estimated considering all seismic-geologic evidence for the fault, including fault length and paleoseismic observations. For faults characterized as having more than a single segment, the potential for rupture of multiple segments in a single earthquake should be considered in assessing the characteristic maximum magnitude for the fault.

For consistency, the same attenuation equations used in the PSHA should be used in the DSHA. Adjustments for directivity/directional effects should also be made, when appropriate. In some cases, ground-motion simulation methods may be appropriate for the estimation of long-period motions at sites in deep sedimentary basins or from great ($M \geq 8$) or giant ($M \geq 9$) earthquakes, for which recorded ground-motion data are lacking.

As a point of clarification, the deterministic lower limit spectrum on the MCE (Figure 21.2-1) extends to zero period in the same manner as the design response spectrum of Figure 11.4-1. The spectrum in Figure 21.2-1 is simply a schematic illustrating the lower bounds for the constant spectral acceleration ($S_{aM} = 1.5F_a$) and constant spectral velocity ($S_{aM} = 0.6F_v/T$) portions of the spectrum. The transition in the deterministic lower limit spectrum from the $1.5F_a$ plateau to zero period occurs at a period (in seconds) of $0.08F_v/F_a$ which is derived in the same manner as T_0 in Section 11.4.5. From this period to zero period, where the ordinate is $0.6F_a$, the deterministic lower limit spectrum is a straight line, similar to the design response spectrum in the period band, 0 to T_0.

C21.3 DESIGN RESPONSE SPECTRUM

Eighty percent of the design response spectrum determined in accordance with Section 11.4.5 was established as the lower limit to prevent the possibility of site-specific studies generating unreasonably low ground motions from potential misapplication of site-specific procedures or misinterpretation or mistakes in the quantification of the basic inputs to these procedures. Even if site-specific studies were correctly performed and resulted in ground-motion response spectra less than the 80 percent lower limit, the uncertainty in the seismic potential and ground-motion attenuation across the ~~U.S.~~United States was recognized in setting this limit. Under these circumstances, the allowance of up to a 20 percent reduction in the design response spectrum based on site-specific studies was considered reasonable.

C21.4 DESIGN ACCELERATION PARAMETERS

The 90 percent lower limit rule, which can affect the determination of S_{DS}, was inserted because it was recognized that site-specific studies could produce response spectra with ordinates at periods greater than 0.2 s that were significantly greater than those at 0.2 s. Similarly, the rule that requires that S_{D1} be taken as the larger of the spectral acceleration at a period of 1 s and two times the spectral acceleration at a period of 2 s accounts for the possibility that the assumed $1/T$ proportionality for the constant velocity portion of the design response spectrum begins at periods greater than 1 s or is actually $1/T^n$ (where $n < 1$). Thus, this rule leads to more accurate spectral ordinates at periods around 2 s and conservative estimates at shorter periods. However, the conservatism is unlikely to be excessive.

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