This reflects past practice in the nuclear industry that considered no amplification of the PGA for frequencies equal to or higher than 33 Hz. Recent ground motion models for Eastern North America (Atkinson and Boore, 2006), however, indicate that the seismic response at frequencies higher than 33 Hz can be significantly above PGA. The issue is less relevant when nonlinear behavior is anticipated since the reduced spectra are flatter.

Flexible soil layers modify the ground motion. Typically, these site-effects are quantified with one-dimensional wave propagation analyses that account for the stiffness and damping properties of the soil/rock layers. ASCE 7-05 adopts a simpler approach where sites are assigned a Class A to F based on shear wave velocity, standard penetration blowcounts, and/or undrained shear strength. Then, two modification site coefficients are tabulated for each site class in terms of the spectral amplitude at rock. This approach is adopted by Manual 113 and is sufficient for most substation projects. If necessary, wave propagation analysis can be conducted with linearized procedures (Schnabel et al., 1972). The most important task is the determination of shear modulus and viscous damping that properly represent the dynamic nonlinear response of the soil layers.

For combining modal responses, Section 5.6.2.1 of Manual 113 allows the SRSS method when the relevant natural frequencies differ more than 10 percent. Otherwise, a complete quadratic combination (CQC) method must be used. Since CQC rules can be easily implemented in computer codes, there is no reason to use the SRSS method.

Three modal analyses are conducted with spectra for three orthogonal directions of ground motion. In each analysis, the total response is a CQC combination of modal responses. These three responses are combined with the SRSS or the 100-40-40 rules.

Time history (step-by-step) analysis

The time history analysis consists of numerical integration of the differential equations of motion in small time steps. For linear systems, the time history is simplified because the mass, damping and stiffness matrices remain unchanged. The analysis must consider only the modes with larger participation, and the input consists of three accelerograms matching the design spectra in each direction of ground motion. Uncorrelated records can be input simultaneously, considering all combinations of positive and negative signs to capture the most unfavorable effects on each component. For correlated records, the analysis is performed independently for each ground motion direction and the results are combined with the SRSS or the 100-40-40 rules. Linear time history analyses also have to consider nonlinear effects. One possibility is to develop time histories that match the reduced spectra, but it is preferable to perform the actual nonlinear analyses. Linear step-by-step analysis constitutes a convenient option for systems with non-proportional damping that have complex modes and frequencies; otherwise, the modal response spectrum approach is sufficiently accurate.

Spectral matching must satisfy several requirements. In addition to enveloping the design spectrum, a time history must contain sufficient energy in all relevant frequencies. Typical frequency content, duration and matching requirements are given in Appendix A of IEEE 693. Time histories also must reflect site conditions, usually by means of one-dimensional wave propagation analysis (Schnabel et al., 1972); however, more accurate two and three-dimensional formulations have been developed by Bielak et al. (2003).

Time history analysis is unavoidable if one needs to take into consideration nonlinearities in components or equipment. Appropriate constitutive models must be identified to capture the main nonlinear structural characteristics under cyclic loading. Since the nonlinear response can vary significantly under different input time histories, sufficient earthquake records must be selected to represent the potential seismic events at the site. An added complication is the need to incorporate all combinations of signs and directions of seismic input. Owing to these difficulties, nonlinear analyses are rarely conducted, except for seismic qualification of equipment excessively large to be qualified by testing.

EFFECTS OF DAMPING RATIO ON DESIGN SPECTRA

As prescribed in current codes, R values are applicable to structures with 5 percent of critical damping, usually appropriate for buildings. Different damping ratios, say 2%, may be adequate for substation components. The design seismic coefficient or spectrum can be adjusted using Fig. 1. However, based on the IEEE 693 equations, the spectrum for a prescribed percent damping ratio, d, is obtained by multiplying the 5-percent spectrum by F_d, as follows:

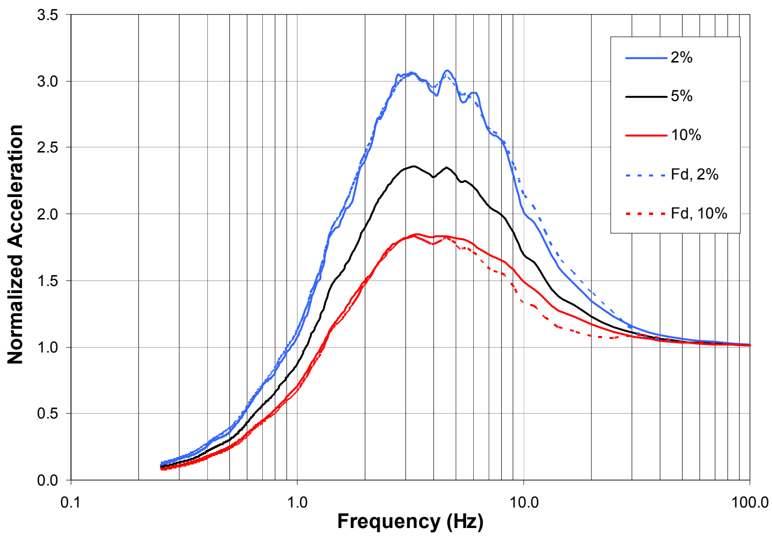

Figure 3. Calculated and Approximate Spectra (IEEE approach).

$F_d = \beta$, for $0 \le f \le 8$ Hz (4)

$F_d = 1 + 0.04\,(\beta - 1)\ (33 - f)$, for $8 \le f \le 33$ Hz (5)

$F_d = 1$, for $f > 33$ Hz (6)

$\beta = 1.5173 - 0.3213\ \ln(d)$ (7)

d = percent of damping ratio

To examine the accuracy of F_d we analyzed elastic single-degree-of-freedom systems excited by a set of 87 Californian earthquake records from stiff to medium stiff sites. The records were normalized to have the same Arias Intensity and to yield average peak ground acceleration equal to that of gravity. We considered damping ratios of 2, 5 and 10 percent. Fig. 3 shows that the average spectra (continuous lines) resemble design spectra and indicates an excellent agreement with spectra calculated with Eqs. 4 through Eq. 7. We have developed a similar approach where the 5-percent spectrum is multiplied by a factor α, as follows:

$$\xi = 0.3 + 0.01d \quad (8)$$
$$\delta = (5/\,d\,)^{\xi} \quad (9)$$
$$\alpha = 1 + (\delta - 1)\exp(-0.05\,f) \quad (10)$$

d = percent of damping ratio
f = frequency in Hz

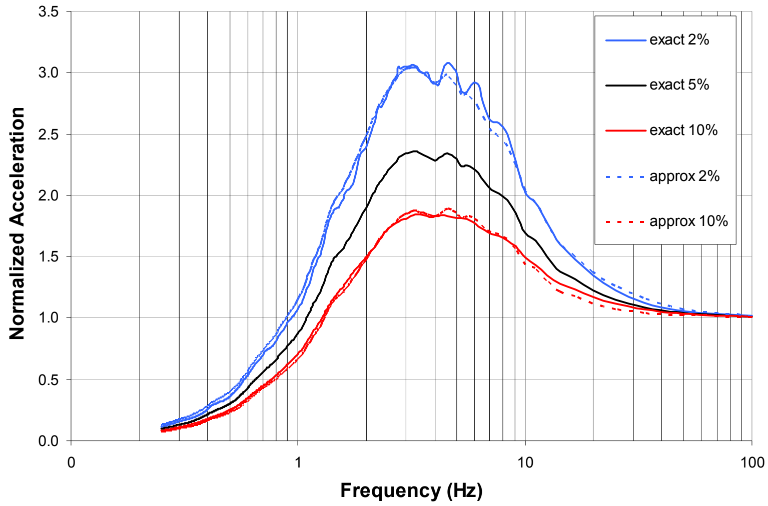

Figure 4. Calculated and Approximate Spectra (This paper approach).

Fig. 4 shows that the spectra (dashed lines) resulting from Eqs. 8 to 10 are in slightly better agreement with the average spectra that the IEEE approach. The advantage of our approach is that it has been extended to adjust spectra reduced due to nonlinear behavior. Indeed, substation structures are designed anticipating inelastic behavior under strong earthquakes with energy dissipation from viscous as well as hysteretic damping. The viscous damping contribution decreases with larger nonlinear energy dissipation. Thus, we have also analyzed single-degree-of-freedom systems with bilinear hysteretic force-displacement curves. The second slope of the curves equals 2 percent of the initial slope. The seismic coefficient is the yield strength, F_y, divided by the weight of the system. The ductility demand, μ, is the maximum displacement divided by the yield displacement.

Figs. 5, 6 and 7 present the average spectra (solid lines) for μ=1, 2 and 4. Note that the spectra for different damping ratios become closer as μ increases, confirming that higher inelastic energy dissipation diminishes the role of viscous damping. Figs. 5, 6 and 7 also show (dashed lines) spectra obtained multiplying the 5-percent spectrum by α, defined by Eq. 10. Reflecting the influence of μ, the parameter ξ is calculated as:

$$\xi = (0.3 + 0.01\,d\,)/\mu^{0.55} \quad (11)$$

μ = average ductility demand

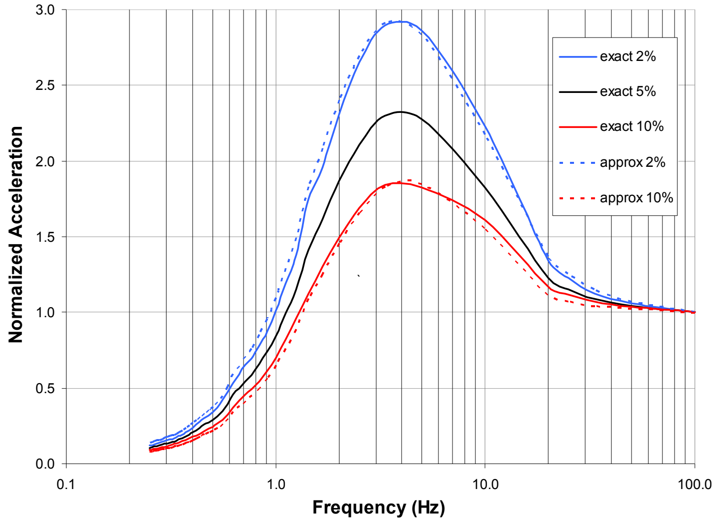

Figure 5. Spectra for Average Ductility Demand μ = 1.

Again, the closeness between continuous and dashed lines in Figs. 5, 6 and 7, demonstrates the accuracy of Eqs. 9, 10 and 11 for estimating spectra for damping ratios between 2 and 10 percent. Note that for $\mu = 1$, Eqs. 8 and 11 yield the same value.

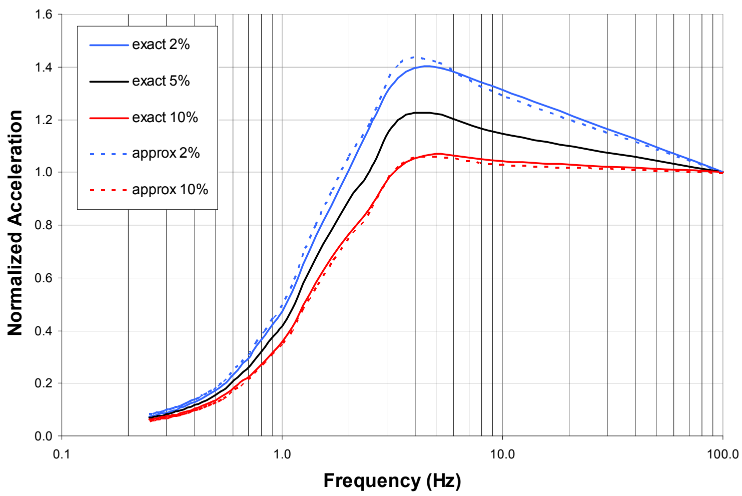

Figure 6. Spectra for Average Ductility Demand μ = 2.

SOIL STRUCTURE INTERACTION

Section 3.1.7 of Manual 113 states: "Designers should be aware of unusual soil conditions, soil structure interaction, and the potential of modified response due to an intermediate structure." Soil-structure interaction (SSI) can appreciably change vibration frequencies, increase damping, and increase displacements. SSI effects can be included using springs with constants, K, and dashpots with damping coefficients, C, that represent the flexibility and energy dissipation characteristics of the soil/foundation system. Up to six values of K and C can be calculated corresponding to the six degrees of freedom of the foundation: two translations in horizontal orthogonal axes, one vertical translation, and three rotations (two rocking and one twisting) about these three axes.

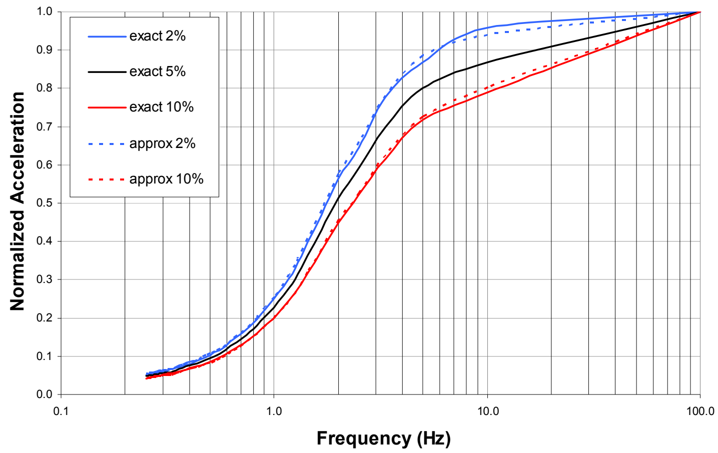

Figure 7. Spectra for Average Ductility Demand $\mu = 4$.

Following pioneering work by Jennings and Bielak (1973), Gazetas (1991) developed formulas for K and C representing shallow foundations in homogeneous soil and considering the plan shape of the foundation and its embedment depth. Procedures for analyzing pile foundations under dynamic loads are presented in Puri and Prakash (2007). The soil-pile behavior under earthquake loading is generally non-linear, which is accounted for by defining soil-pile stiffness in terms of strain dependent soil modulus. In addition to the foundation geometry, these procedures require estimates of the geotechnical properties of subsurface materials, including shear modulus (G), Poisson ratio (ν) and unit weight (γ). While some properties, such as ν and γ, exhibit limited variability, others, such as G, can vary significantly from site to site (even within a site) and with the level of deformations. Thus, it is important to identify the subsurface characteristics at the site by means of geotechnical investigations, laboratory testing and judicious use of correlations and presumptive values. Cantilever support structures, tubular poles and similar systems, can experience increased displacements due to SSI flexibility inducing larger P-delta effects.

SSI springs and dashpots can be readily incorporated in structural models. In static analyses, the most significant impact is increased displacements. In dynamic analysis, the natural vibration frequencies become smaller and the spectral acceleration can change. Indeed, an accurate indicator of the relevance of SSI is the percent of decrease in the fundamental frequency of vibration. A modest decrease indicates negligible SSI effects and vice versa. In any event, SSI stiffness and damping parameters should be calculated and included in the calculation of modes and frequencies of vibration.

Saturated granular loose soils are susceptible to liquefaction during earthquakes. Dramatically settled and tilted structures have been manifestations of this phenomenon in past earthquakes. A recent monograph by Idriss and Boulanger (2008) presents updated methodologies to assess liquefaction potential and to mitigate liquefaction hazards. It should be kept in mind that piles could experience excessive lateral displacements and bending moments in liquefied soils. A possible solution is the used of drilled shafts embedded into deeper non liquefiable soils or in rock.

DISPLACEMENTS

Section 5.2 of Manual 113 highlights the importance of accurately estimating displacements in substation components, warning that "A structure designed for strength may have excessive deflections." By contrast, Section 3.1.11.3 of the Manual states that loads from "earthquakes should not be considered in deflection analysis. It is argued that nonlinear dynamic analyses can be difficult and unreliable. However, seismic regulations, such ASCE 7-05, accept that linear analyses along with structural-response modification factors, is sufficiently reliable for estimating stresses and displacements under seismic loads. Most of the uncertainty in seismic response estimates resides in the assessment of seismic activity, rather than in structural analyses methodologies,

The static and the modal spectral seismic analyses produce estimates of the displacements at any point of a structure. Since the linear analyses use forces reduced by a factor R, on account of nonlinear behavior, the ensuing displacements are also reduced and have to be increased to estimate the inelastic deformations. To this end, seismic regulations stipulate amplifications factors for R-reduced displacements. For instance, ASCE 7-05 requires that displacements be amplified by a factor C_d tabulated along with R. Manual 113 does not provide values for C_d. Upon examination of Table 12.2.1 of ASCE 7-05, we propose that C_d be equal to R.

EXAMPLES

The following examples are based on the Dead-End Structure example of Section 3.6.1 and Fig. 3.5 of Manual 113. Cantilever support poles constitute the structural system in one horizontal direction. We understand that 5 percent damping was considered in this example. Manual 113 considers a Site Class D and provides the following information:

$F_a = 1.33$, $S_S = 0.590$, $F_v = 2.06$, $S_1 = 0.186$
$S_{DS} = 2/3\ F_a\ S_S = 2/3 \times 1.33 \times 0.590 = 0.523$, assumed to control.

Manual 113 assumes R=2 and importance factor I_{FE}=1.25. Thus, the seismic design force, F_E, is equal to $(S_a/2) \times 1.25W = 0.33W$. For the other direction, we assume that the structural system is a moment-resisting steel frame. Now R=4 and $F_E = (S_a/4) \times 1.25\ W = 0.17\ W$, if all other data remain the same. In the vertical direction, lacking any other guidance, we would use F_{EV} equal to 0.8 times the larger horizontal force, i.e., $F_{EV} = 0.8{\times}0.33\ W = 0.26\ W$.

Now let us consider that a damping ratio of 2 (rather than 5) percent is appropriate. Using Figure 5.2 of Manual 113, or Eqs. 6 and Eq. 7, the amplification in the flat region of the design spectra (say at a frequency of 7 Hz), the factor equals 3.25/2.5 = 1.3. However, using equations 9 and 11, with $\mu = R = 2$, and d = 2, we have:

$\xi = (0.3 + 0.01\ d\ ) / 2^{0.55} = 0.22$
$\delta = (5/\ d\ )^{0.22} = (5/2)^{\ 0.22} = 1.22.$
$\alpha = 1 + (\delta - 1) \exp(-0.05\times7)= 1.16$

In the other direction (R=4) the adjustment for a 2 percent damping ratio is:

$\xi = (0.3 + 0.01\ d\ ) / 4^{0.55} = 0.15$
$\delta = (5/\ d\ )^{0.15} = (5/2)^{\ 0.15} = 1.14$
$\alpha = 1 + (\delta - 1) \exp(-0.05\times7)= 1.10$, while the Manual 113 value is still 1.3.

CONCLUDING REMARKS

ASCE Manual 113 is a welcome addition to the technical literature on the design of substation structures and foundations. The specifications of Manual 113 provide a balance between having sufficient criteria available to achieve acceptable uniformity in the seismic design while allowing designers to exercise their experience and judgment. In this paper we have presented our opinions on interpretation of the seismic provisions of Manual 113. We recommend that modal spectral analyses be used for the seismic design of substation structures. The development of required finite element models is facilitated with the variety of available software. The number of modes included in the analyses should ensure that the sum of effective masses equals at least 90 percent of the total mass. A CQC modal combination rule should always be used. These rules are incorporated in most commercial finite element programs.

Both static and dynamic procedures can be simplified. A frequent simplification consists in using two-dimensional models. However, current software facilitates the construction of three-dimensional (3D) models, rendering such simplifications unnecessary. The same model is used for calculating the response to the three components of ground motion, since all three-dimensional features are already represented. The calculation of natural 3D frequencies and modes of vibration is also relevant in the assessment of the structural response to non-seismic dynamic loads. The number of modes included in the analysis should result in a cumulative participating mass of at least 90% of the total mass. The calculation of displacements should reverse the reductions associated with the use of R-reduced design spectra, since these reductions were introduced originally to account for nonlinear hysteretic behavior. Such behavior is undesirable in substation structures. Finally, the response to simultaneous ground motion in three orthogonal directions can be calculated with the square root of the sum of the squares or the 100-40-40 rules.

We recommend that soil-structure interaction (SSI) be always modeled in the seismic analysis. The added complexity in the finite element model is minimal while the benefits can be significant: a more accurate estimation of natural frequencies, which can modify the spectral amplitudes; and more accurate estimates of stresses and displacements.

An original contribution of this paper is the development of an improved approach for adjusting 5-percent damped design spectra to other damping ratios. The procedure of

Standard IEEE 693, adopted in Manual 113, is adequate for modifying elastic spectra. However, substation structures are designed anticipating inelastic behavior with energy dissipation occurring via viscous as well as hysteretic damping. Using inelastic nonlinear analyses of single-degree-of-freedom systems we have verified that viscous damping is less effective when higher hysteretic energy dissipation occurs. This implies that the required modification of the design spectrum due to a change in the damping ratio decreases when the target ductility demand increases. Equations 9, 10 and 11 have been developed herein to calculate spectral adjustment factors for damping ratios between 2 and 10 percent, considering average ductility demands between 1 and 4. These factors correctly approach unity at high frequencies.

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Analytical Techniques to Reduce Magnetic Force from High Fault Current on Rigid Bus

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ABSTRACT

Fault currents are increasing on existing and new substations due to system additions and modifications. Bus designs are typically based on the circuit breaker rating of 40kA to 80kA. Phase spacing is no greater than that required by insulation level, thus for configurations in the 115kV to 138kV range, magnetic forces over 100 pounds per foot are not uncommon when computed per IEEE 605 methods.

The magnetic force calculation in IEEE 605 is proportional to the decrement factor squared and 1.6 is the default decrement factor. The first reduction strategy is to calculate the actual decrement factor. The decrement factor is a function of fault clearing time, system reactance and system resistance. Typical system reactance to resistance ratios (X/R) can reduce magnetic force 12 to 18 per cent for a two cycle breaker. More reduction is available with greater clearing times.

The second force reduction strategy is to compare the natural frequency of the bus to the forcing function frequency. This paper presents a simplified generalized coordinate method to determine the frequency based response of the bus to the magnetic field.

The paper will also review the cost savings as a result of applying these techniques.

DECREMENT FACTOR

Decrement factor defined. The purpose of the decrement factor is to account for the momentary peak effect of the AC and decreasing DC components of the short-circuit current during the first half-cycle of the fault, where the DC component is at a maximum. A full explanation of the theory behind the decrement factor is outside the scope of this document – see section 10.2 of IEEE Std. 605-1998 for further explanation.

Influence of the decrement factor. The following equations show that the short-circuit force is proportional to the square of the decrement factor (See Equation 12 IEEE Std. 605-1998).

$$F_{SC} = K_f \cdot \frac{C \cdot \Gamma \cdot \left(D_f \sqrt{2} \cdot I_{SC}\right)^2}{D} \tag{1}$$

where

F_{SC} = short - circuit force, lbf/ft

K_f = mounting - structure flexibility factor usually taken as unity

$C = 5.4 \times 10^{-7}$ for USCS units

D_f = decrement factor as given in the equation shown below

I_{SC} = symmetrical short - circuit current, A

D = conductor phase spacing center - to - center, in

$$D_f = \sqrt{1 + \frac{T_a}{t_f} \cdot \left(1 - \exp\left(\frac{-2 \cdot t_f}{T_a}\right)\right)} \tag{2}$$

where

$$T_a = \frac{X}{R} \frac{1}{2 \cdot \pi \cdot f} \tag{3}$$

and

t_f = fault current duration, sec

X = system reactance

R = system resistance

f = 60 Hz

Figure 1 plots the decrement factor for a common 5 cycle clearing time, t_f = 0.083 sec, at a fault current of 80 kA over a range of (X/R) ratios with a common 138 kV phase spacing of 8’-0”. Note that even for very high X/R ratios the calculated values approach but do not reach the default maximum decrement factor of 1.6.

Figure 2 plots the corresponding short-circuit force using Equation 1. Note that only for very high X/R ratios do the calculated values approach 160 lbf/ft maximum short-circuit force using the default 1.6-decrement factor.