RAI Volume 2, Chapter 2.1.1.7, Eleventh Set, Number 7:

Provide technical bases to support the use of a percentage of critical soil damping values of 20 percent in the structural analysis of surface facilities.

Basis: DOE estimated the percentage of critical soil damping as high as 140 percent in the horizontal direction (BSC, 2008af; Section 7.1.5) and 150 percent in the vertical direction (BSC, 2008ar; Section 7.2), even after reducing the damping by 25 percent to approximately account for the reduction in energy dissipation in non-homogenous media (Hadjian and Ellison, 1985).

For the structural analysis of ITS buildings, DOE further reduced the soil damping values to 20 percent (e.g., BSC, 2007ba; Section C.3 and BSC, 2008af; Section 7.1.5) based on its interpretation of ASCE (1998; Section 3.1.5.4). However, even these soil damping values are higher than those reported in the literature available to the staff. For instance, Stewart et al. (1998) compiled empirical foundation damping factors for shear wall buildings, obtaining soil damping values from 0 to 16 percent (with the exception of the Lotung site, which cannot be compared to Yucca Mountain site conditions).

1. RESPONSE

The seismic response analyses of the important to safety (ITS) surface facilities account for the dissipation of energy in the supporting medium by limiting the percentage of critical soil damping (damping ratio) value to 20%. This damping ratio is conservative, based on a comparison of damping values calculated using different methods. Additionally, evaluation of the data in *Empirical Evaluation of Inertial Soil-Structure Interaction Effects* (Stewart et al. 1998) shows that the 20% value used is appropriate.

1.1 DAMPING CALCULATIONS

In the soil spring calculations of the ITS facilities, variation of shear modulus with depth is considered, and an equivalent shear modulus is determined for the upper bound, median, and lower bound soil cases. The equivalent dynamic shear moduli are used to determine frequency independent spring constants and damping coefficients, using the equations in ASCE 4-98, *Seismic Analysis of Safety-Related Nuclear Structures and Commentary*. The critical damping ratios D_i are calculated from the equation:

$$D_i = C_i / C_{cr} \qquad \text{(Eq. 1)}$$

where C_i is the damping coefficient and C_{cr} is the critical damping of the soil-structure interaction system, calculated from the equation:

$$C_{cr} = 2\sqrt{M_s \cdot K_i} \qquad \text{(Eq. 2)}$$

where M_s is the mass of the structure and K_i is the stiffness of the soil.

The translational damping ratios are reduced by 25% to account for nonhomogenous media, per the recommendations of "Equivalent Properties for Layered Media" (Hadjian and Ellison 1985). The damping ratios calculated for the Canister Receipt and Closure Facility (CRCF) are shown in Table 1 for the controlling upper bound, 100-ft-thick alluvium soil case (UB100). The damping ratios calculated in *CRCF Soil Springs—2007 Strain Compatible Soil Properties* (BSC 2008) are substantially greater than 20%. Thus, the damping ratios used for the soil-structure interaction modes in the seismic analysis of the CRCF are limited to 20%, as shown in Table 1.

1.2 DETERMINATION OF DAMPING RATIO BASED ON FOUNDATION IMPEDANCE STUDY

For comparison with the values determined in the soil spring calculation, damping ratios for the CRCF are determined using a different approach. Using SASSI2000 software program and considering the layered media, a massless, rigid foundation representation of the CRCF is evaluated.

The foundation response is determined by applying unit harmonic loads with varying frequency on a finite element model of the CRCF foundation resting on a soil site represented by horizontally infinite soil layers. The soil-structure interaction impedance curves are developed from the foundation responses. Using an iterative procedure, the dominant frequency of the soil-structure interaction system is calculated for all six degrees of freedom. The values of the soil-structure interaction impedance functions at dominant frequencies (for stiffness K_i and damping coefficient C_i) are used to determine the percentage of critical damping using the equations given in Section 1.1 of this response. These damping ratios are shown in Table 1 for the UB100 soil case. This table shows that the damping ratios obtained from the impedance study are lower than the values obtained in the soil spring calculation, indicating the effect of layering. The differences are smaller for the translational values and further apart for the rotational ones. However, the damping ratios are all greater than 20%, indicating that the value used in the seismic analysis is appropriate and conservative.

1.3 DETERMINATION OF DAMPING RATIO USING SIMPLIFIED METHOD OF THE EMPIRICAL EVALUATION STUDY

Empirical Evaluation of Inertial Soil-Structure Interaction Effects (Stewart et al. 1998) evaluates soil-structure interaction effects based on strong motion data obtained from sites with instrumented structures and free-field accelerographs. System identification analysis techniques are used to develop empirical values of damping ratios, natural periods, period elongation, and other soil-structure interaction related information.

Chapter 2 of the empirical evaluation study (Stewart et al. 1998) presents a simplified method for determining spring stiffness values and damping coefficients. To provide confirmation of the damping ratios in the soil spring calculation, the CRCF is evaluated using this simplified method. Damping ratios for the UB100 soil case are determined for translation along the x-axis and rotation about the x-axis using this simplified method and are shown in Table 2.

The results shown in Table 2 indicate that damping ratios determined by the ASCE 4-98 and the empirical evaluation study (Stewart et al. 1998) methods are reasonably close to each other. This is expected because the frequency independent impedance equations in the two methods are similar.

1.4 COMPARISON OF PERTINENT RESULTS FROM THE EMPIRICAL EVALUATION STUDY

The empirical evaluation of soil-structure interaction effects in the empirical evaluation study (Stewart et al. 1998) used strong motion data from 58 sites. Forty-four sites are located in the coastal region of California, five are located in the central valley of California, eight are located in the inland San Bernardino area, and one is located in Taiwan. None is in the vicinity of the proposed Yucca Mountain site. The sites represented a wide range of structural systems, soil conditions, and ground motions. Fifteen California earthquakes contributed data to the study, with magnitudes ranging from 4.8 to 7.4.

Using system identification analysis techniques, the investigators determined first-mode periods and damping ratios for a flexible-base case that includes soil-structure interaction effects and a fixed-base case for each site. The soil damping results varied significantly.

Structures at nine sites in the study are similar to the CRCF, a shear wall structure founded on soil without piles and with little or no embedment. These sites are shown in Table 3. The flexible-base cases for these structures are selected for comparison with the CRCF. It is noted that the shear wave velocities at these sites are much lower than for the CRCF.

The relevant data from the study are shown in Table 3, along with similar parameters for the CRCF. For larger structures subjected to moderate to strong ground motions, the study calculated damping ratios near or above 20%, as shown in the results for sites A2, A7, and A44 (shaded rows in Table 3). Because these structures have larger base dimensions (equivalent radius close to or greater than 100 ft) and the maximum horizontal accelerations produced by earthquake shaking at these sites are of moderate to strong values, the results for sites A2, A7, and A44 are more representative for comparison with the CRCF. The average damping ratio for these three sites is 28.5%. Thus, based on the empirical evaluation study, a 20% damping ratio is a realistic value for a large shear wall structure founded on soil without piles and little or no embedment and subject to moderate to strong ground motions.

1.5 CONCLUSION

Results of damping ratio calculations using various methods demonstrate that limiting the damping ratio value to 20% for the fundamental modes that are dominated by soil-structure interaction effects in the seismic analyses of the ITS facilities is appropriate and conservative.

2. COMMITMENTS TO NRC

None.

3. DESCRIPTION OF PROPOSED LA CHANGE

None.

4. REFERENCES

ASCE 4-98. 2000. *Seismic Analysis of Safety-Related Nuclear Structures and Commentary*. Reston, Virginia: American Society of Civil Engineers. TIC: 253158.

BSC (Bechtel SAIC Company) 2008. *CRCF Soil Springs—2007 Strain Compatible Soil Properties*. 060-SYC-CR00-00700-000-00C. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20080229.0002.

Hadjian, A.H. and Ellison, B. 1985. "Equivalent Properties for Layered Media." *Soil Dynamics and Earthquake Engineering, 4*, (4), 203–209. Southampton, England: CML Publications. TIC: 255744.

Stewart, J.P.; Seed, R.B.; and Fenves, G.L. 1998. *Empirical Evaluation of Inertial Soil-Structure Interaction Effects*. PEER-98/07. Berkeley, California: Pacific Earthquake Engineering Research Center. TIC: 260378.

Table 1. Comparison of Damping Ratios for CRCF

Mode of Vibration	Percentage of Critical Soil Damping		
	Damping Used in Seismic Analysis	Damping Calculated with Soil Spring Calculation[a, b]	Damping Calculated with Impedance Study[a]
X Translation	20.0%	62.5%[c]	39.7%
Y Translation	20.0%	64.1%[c]	42.7%
Vertical Translation	20.0%	104.4%[c]	76.1%
Rocking About X	20.0%	69.9%	20.2%
Rocking About Y	20.0%	88.5%	33.0%
Torsion	20.0%	35.2%	32.3%

NOTE: [a]Values are for UB100 soil case.

[b]The z- and y-axes are rotated between the soil spring calculation and the seismic analysis. The damping ratios shown correlate to the axes in the seismic analysis.

[c]Translational values include 25% reduction to account for the reduction in energy dissipation of non-homogeneous material.

Table 2. Comparison of Damping Ratios for CRCF by Different Methods

	Evaluation Method		
Mode of Vibration	ASCE 4-98	Impedance Study	Stewart et al. 1998
X Translation	62.5%[a]	39.7%	74.2%[a]
Rocking About X	69.9%	20.2%	75.5%

NOTE: [a]Translational values include 25% reduction to account for the reduction in energy dissipation of nonhomogeneous material.

Table 3. Comparison of Data from Stewart et al. 1998 and CRCF Seismic Analysis

					Transverse			Longitudinal		
Site	**Earthquake**	**MHA**	**V_S (fps)**	**β**	**r_1 (ft)**	**r_2 (ft)**	**D (%)**	**r_1 (ft)**	**r_2 (ft)**	**D (%)**
A1	PT	0.18	701	5.1	57	42	19.6	57	77	12.8
A2[a]	PT	0.12	772	5.6	115	112	34.0	115	122	39.4
A7[a]	LP	0.36	502	7.7	97	75	26.9	97	130	U
A31	NR	0.11	969	3.2	61	52	3.2	61	74	3.9
A34	NR	0.08	1575	1.7	69	49	18.5	69	100	12.4
A44[a]	NR	0.84	1119	9.1	126	132	18.9	126	132	23.4
B1	LP	0.14	649	4.1	6.4	10.9	15.3	Not reported		
B2	LP	0.11	916	3.6	72	59	12.5	Not reported		
B14	NR	0.17	842	3.2	64	28.5	12.1	Not reported		
CRCF	DBGM-2	0.45	2838	NA	174	159	20.0[b]	174	194	20.0[b]

NOTE: [a]Shaded rows indicate sites more representative for comparison with the CRCF.

[b]Damping ratio used in the initial seismic analysis of the CRCF.

β = soil hysteretic damping ratio (not used in the determination of the damping ratios for the CRCF); D = damping ratio; DBGM-2 = Design Basis Ground Motion-2; LP = 1989 Loma Prieta; MHA = maximum horizontal acceleration (for the CRCF, MHA = peak horizontal ground acceleration); NR = 1994 Northridge; PT = 1992 Petrolia; r_1 = equivalent foundation radius for translation; r_2 = equivalent foundation radius for rocking; U = unacceptably low confidence results that were not reported; V_s = soil shear wave velocity.

RAI Volume 2, Chapter 2.1.1.7, Eleventh Set, Number 11:

> For the IHF steel structure, provide technical basis to justify not including soil-structure interaction (SSI) in the analysis. The response should include the rationale for assuming that modeling the steel structure supports as pinned connections at the mat guarantees no moment transfer to the foundation (BSC, 2008am; Section 3.1.2), particularly if braced steel members are included in the design. Also, provide the rationale for not including SSI effects on the structural analysis of the IHF concrete structures (BSC, 2007aq).

1. RESPONSE

The response spectrum analyses of the Initial Handling Facility (IHF) steel and concrete structures use fixed-base support conditions to determine the design forces and moments. In fixed-base analyses, soil-structure interaction effects are not included. However, neglecting soil-structure interaction effects in the initial analyses of the IHF still provides design forces that are reasonable and appropriate, based on the fixed-base frequencies of the steel and concrete structures.

1.1 IHF STEEL STRUCTURE

The response spectrum analyses of the IHF steel structure do not include the effects of soil-structure interaction in accordance with the provisions of Section 3.3.1.1 of ASCE 4-98, *Seismic Analysis of Safety-Related Nuclear Structures and Commentary*. Per Section 3.3.1.1 of ASCE 4-98, a fixed-base support may be assumed in modeling structures for seismic response analysis when the frequency obtained from a soil-structure interaction analysis of a rigid structure (with soil springs representing the supporting soil medium) is more than twice the dominant frequency obtained from a fixed-base analysis of the flexible structure. The results of the fixed-base modal analyses in Table 6.6.3 of *IHF Steel Structure Seismic Analysis and Steel Member Design* (BSC 2008) show that the fundamental frequencies of the steel structure representing the response in the horizontal north–south and east–west directions are 2.45 and 3.77 Hz, respectively.

The stiffness of the soil springs is established using the methodology of ASCE 4-98. Using the ASCE 4-98 impedances for the best estimate (median) soil properties, the calculated soil-structure interaction frequencies range from 7.68 to 9.17 Hz in the horizontal directions. The ratio of the soil-rigid structure frequency to the dominant steel structure frequency is greater than two for these cases. Therefore, not including soil-structure interaction effects in the response spectrum analysis of the steel structure is reasonable and in accordance with ASCE 4-98.

The IHF steel structure model uses pinned connections at the column bases. The use of pinned connections is an accepted practice for analysis of braced-frame structures like the IHF. The pinned connections are fixed against translation but are free to rotate about the horizontal axes. This releases the bending moment reactions to the foundation. In the actual connection, partial rotational fixity may occur. The moments that result from this partial fixity are secondary in nature and will not impact the overall design of the foundation. The local elements of the base connection (e.g., anchor rods, base plates) will be designed considering the effects of this partial

fixity in the detailed design for construction. The detailed design of column base connections will conform to the requirements of ANSI/AISC 341-02, *Seismic Provisions for Structural Steel Buildings*.

1.2 IHF CONCRETE STRUCTURES

There are two concrete structures for the IHF: the separate west structure, and the east structure that shares a common basemat with the IHF steel structure. The fixed-base frequencies in the north–south direction are 9.1 and 13.4 Hz for the west and east structures, respectively. The ground design response spectrum peaks at 10 Hz gradually decreasing above and below the peak frequency. Thus, the dominant fixed-base mode response is at or near peak spectral acceleration levels. Because the participation factor for this mode is over 60% for both structures, the in-plane shear forces are close to maximum values and will not be significantly affected by inclusion of soil-structure interaction effects.

The fixed-base frequencies in the east–west direction are 16.8 and 26.0 Hz for the west and east structures, respectively. For the east structure, the dominant fixed-base mode response is approximately 36% less than the peak spectral acceleration. However, because of the inherent strength of the long walls in the east–west direction, minimum reinforcement controls the design for in-plane shear. Thus, the design has significant margin beyond the margin provided by limiting the demand-to-capacity ratio to approximately 0.6. Even if the effects of soil-structure interaction result in the system response near the peak frequency, the resulting in-plane shear forces are still less than the design capacities of these walls.

Therefore, analysis of both the steel and the concrete structures using fixed-base models is adequate for the initial design. The finite element models used for the detailed design for construction will include soil-structure interaction effects.

2. COMMITMENTS TO NRC

None.

3. DESCRIPTION OF PROPOSED LA CHANGE

None.

4. REFERENCES

ANSI/AISC 341-02. 2002. *Seismic Provisions for Structural Steel Buildings*. Chicago, Illinois: American Institute of Steel Construction. TIC: 256615.

ASCE 4-98. 2000. *Seismic Analysis of Safety-Related Nuclear Structures and Commentary*. Reston, Virginia: American Society of Civil Engineers. TIC: 253158.

BSC (Bechtel SAIC Company) 2008. *IHF Steel Structure Seismic Analysis and Steel Member Design*. 51A-SSC-IH00-00600-000-00B. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20080219.0006.

RAI Volume 2, Chapter 2.1.1.7, Eleventh Set, Number 13:

> Provide sensitivity analyses demonstrating that the approximated mesh size of 5 × 5 ft for the design of the foundation (e.g., BSC 2007ae) and IHF walls and slabs (e.g., BSC 2007aq) is adequate.

1. RESPONSE

The approximate 5 ft × 5 ft mesh size used in the initial design of the typical important to safety facility concrete foundation (e.g., the Canister Receipt and Closure Facility) and in the initial design of the Initial Handling Facility (IHF) concrete walls and slabs is sufficiently fine and appropriate to represent the overall structural behavior. The use of a more closely spaced mesh is not expected to significantly affect the calculated design forces. Additionally, localized stress concentrations (e.g., around discontinuities) will be further evaluated in the detailed design for construction to develop the refined reinforcement requirements. As such, sensitivity analyses of the mesh refinement have not been performed for the initial designs but will be evaluated in the detailed design for construction.

The adequacy of a mesh size in a finite element analysis is normally gauged by the number of elements in the model between supports (e.g., between floors or walls). Section 7.2.1.2 of the *Seismic Analysis and Design Approach Document* (BSC 2007a) provides the following generic guidance on mesh size:

1. In-plane forces can be accurately determined using a relatively coarse mesh. In general, two elements are sufficient for determination of the in-plane forces between supports, because the in-plane deformations are nearly linear between supports (i.e., shear deformation). For narrow piers, where in-plane flexure may be dominant, a minimum of three elements may be needed in the vertical direction.

2. Out-of-plane analyses require a more refined mesh to approximate the higher order deformations. The accuracy of out-of-plane moments and shears depends on many parameters (e.g., finite element type, load type, element thickness to span ratio). In general, six elements are needed to obtain accurate results between supports of a shear wall structure.

The finite element models in *CRCF Foundation Design* (BSC 2007b) and *Initial Handling Facility (IHF): Concrete Structure Design* (BSC 2007c) show that the approximate 5 ft × 5 ft mesh size meets the guidance in *Seismic Analysis and Design Approach Document* (BSC 2007a) for in-plane responses. In the case of the out-of-plane responses, the guidance is met for the Canister Receipt and Closure Facility foundation and met for the IHF walls and slabs with two exceptions in the IHF interior concrete structure: the slab at elevation 28′1″ and the slab at elevation 37′0″. For slabs, out-of-plane forces typically dictate the design. In both slab cases noted, the recommended minimum number of elements for determination of out-of-plane forces is used between supporting walls but not between supports and discontinuities (e.g., the openings

in the slab at elevation 37′0″ for the port slide gates). However, the analysis results and subsequent forces used in the initial design show that the following conservatisms are provided:

1. In the design, the demand-to-capacity ratios are limited to approximately 0.6, providing significant margin.

2. The walls and slabs are modeled from centerline-to-centerline, whereas the design is normally based on clear spans. Due to the large wall thicknesses, design moments at the centerlines (which are used without modification) are conservative.

3. Peak element forces are used for design without redistribution.

4. Individual peak element force components are combined for design even if they do not occur at the same location.

Considering the actual models used in analysis and the conservative practices incorporated into the design process, the calculated forces are reasonable, and the resulting design is adequate for use in the preclosure safety analysis in the SAR.

2. COMMITMENTS TO NRC

None.

3. DESCRIPTION OF PROPOSED LA CHANGE

None.

4. REFERENCES

BSC (Bechtel SAIC Company) 2007a. *Seismic Analysis and Design Approach Document*. 000-30R-MGR0-02000-000-001. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071220.0029.

BSC 2007b. *CRCF Foundation Design*. 060-DBC-CR00-00200-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070322.0005.

BSC 2007c. *Initial Handling Facility (IHF): Concrete Structure Design*. 51A-DBC-IH00-00100-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071205.0021.

RAI Volume 2, Chapter 2.1.1.7, Eleventh Set, Number 14:

> Provide technical bases for the use of a uniform alluvium depth in the seismic analysis of the IHF building, instead of an alluvium depth that varies from 30 to 90 ft.

1. RESPONSE

The response spectrum analyses of the Initial Handling Facility (IHF) use fixed-base support conditions to determine the initial design forces and moments for the steel superstructure and the reinforced concrete walls and slabs (excluding the foundations). In fixed-base analyses, soil-structure interaction effects are not included. Thus, determination of design forces above the basemat is independent of alluvium depth. However, the initial foundation designs and time history analyses use flexible base support conditions (i.e., soil springs) that bound the actual alluvium depths under the IHF. As part of the process in the detailed design for construction, effects of the sloping alluvium will be evaluated.

Initial Handling Facility (IHF) Soil Springs and Damping (BSC 2008) provides soil impedance (spring stiffness and damping) values for lower bound (16th percentile), median, and upper bound (84th percentile) soil properties, at 30- and 100-ft-thick alluvium depths. These soil impedances are used in the design of the IHF foundations and in the development of in-structure response spectra.

The IHF foundation design forces are determined from finite element models of the basemats that support the steel and concrete structures. In these models, vertical and horizontal soil springs from the lower bound, 100-ft alluvium soil case are attached to the basemat nodes. The springs for this soil profile have the lowest stiffness values, which results in the maximum bending moments and shear forces in the foundations and the largest lateral movements of the foundation mats. In addition, soil bearing pressures are maximized due to greater deformations. Thus, the IHF foundation designs bound the results for soil profiles at other alluvium depths.

In the direct integration time history analyses of the IHF, in-structure response spectra are developed considering four soil cases: median and upper bound soil properties at 30-ft-thick alluvium depth and median and lower bound soil properties at 100-ft-thick alluvium depth. These cases are selected to bound the seismic response of the IHF. The 30- and 100-ft depths of alluvium bound the actual site conditions under the IHF, where the alluvium varies from approximately 30 to 90 ft. The in-structure response spectra obtained for the four soil cases are then combined for use in design.

2. COMMITMENTS TO NRC

None.

3. DESCRIPTION OF PROPOSED LA CHANGE

None.

4. REFERENCES

BSC (Bechtel SAIC Company) 2008. *Initial Handling Facility (IHF) Soil Springs and Damping*. 51A-SYC-IH00-00500-000-00C. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20080617.0007.