RAI Volume 2, Chapter 2.1.1.7, Eleventh Set, Number 6:

For Tier # 1 analyses, provide technical bases for not considering concrete cracking of the shear walls.

Basis: According to DOE, concrete cracking is not considered in Tier # 1 analyses because the primary interest is in the in-plane response, which is not significantly affected by concrete cracking (BSC, 2007; Section 7.1.1). However, DOE adopted ACI (2001) for reinforced concrete design. In lieu of refined analyses, this code indicates that a percentage of the moment of inertia of the gross concrete section should be used to indirectly account for concrete cracking (ACI, 2001 and 2006; Section 10.11).

1. RESPONSE

The seismic analyses of the typical reinforced concrete important to safety (ITS) facility (e.g., the Canister Receipt and Closure Facility) do not consider concrete cracking of the shear walls because, as stated in Section 7.1.1 of *Seismic Analysis and Design Approach Document* (BSC 2007), the primary interest is the in-plane response of the structure, and concrete cracking does not significantly affect the in-plane response.

The typical ITS facility is a low-rise shear wall structure that resists lateral load with shear walls and diaphragms in both directions. As such, the primary load transfer mechanism during an earthquake is through in-plane shear. Section C3.1.3.1 of ASCE 4-98, *Seismic Analysis of Safety-Related Nuclear Structures and Commentary*, states that the shear wall stiffness corresponding to the uncracked section is generally satisfactory for the determination of wall design forces. Section C3.1.3.1 goes on to state that the variation in shear wall frequency due to modulus variation (i.e., reduction in stiffness caused by concrete cracking) is less than ±12% and, therefore, the ±15% peak broadening of in-structure response spectra will account for frequency variation due to concrete stiffness changes. Not considering the effects of concrete cracking in the shear walls does not significantly affect the in-plane response for design and generation of in-structure response spectra. Additionally, the ITS structures will remain essentially elastic under the design seismic loads because the design demand is significantly less than the capacity. As such, concrete cracking under design seismic loads is expected to be minimal.

Section 10.11 of ACI 349-01/349R-01, *Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-01) and Commentary (ACI 349R-01)*, is not applicable to the design of low-rise shear wall structures like the typical ITS facility. Section 10.11 presents an alternative to the procedure prescribed in Section 10.10 to evaluate slenderness effects of compression members (e.g., columns) in frames using the magnified moment approach. In this approach, the factored forces and moments are computed using an elastic first-order frame analysis with section properties that take into account cracked regions or alternately use reduced section properties based on a prescribed percentage of the moment of inertia for the gross concrete section. The typical shear wall does not behave as a slender column in a frame (i.e., does not experience a significant reduction in capacity resulting from lateral deflection). As such, the provisions of Section 10.11 are not applicable to the typical ITS facility shear wall design.

Therefore, not considering concrete cracking of the shear walls in the seismic analyses of the ITS facilities is appropriate.

2. COMMITMENTS TO NRC

None.

3. DESCRIPTION OF PROPOSED LA CHANGE

None.

4. REFERENCES

ACI 349-01/349R-01. 2001. *Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-01) and Commentary (ACI 349R-01)*. Farmington Hills, Michigan: American Concrete Institute. TIC: 252732.

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) 2007. *Seismic Analysis and Design Approach Document*. 000-30R-MGR0-02000-000-001 ACN 02. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071220.0029; ENG.20090311.0013.

RAI Volume 2, Chapter 2.1.1.7, Eleventh Set, Number 10:

> For the design of the CRCF and WHF foundation (e.g., BSC, 2007ae), provide technical bases to justify the use of the impedance functions (i.e., soil springs and damping values) derived for a 35 ft thick alluvium depth (e.g., BSC, 2006), instead of using impedance functions obtained for updated alluvium depths (e.g., BSC, 2008af).

1. RESPONSE

Foundation designs for the Canister Receipt and Closure Facility (CRCF) and Wet Handling Facility (WHF) use impedance functions (soil springs and damping values) derived from soil properties for a 35-ft-thick depth of alluvium. These soil properties and corresponding alluvium depth are based on original data qualified in 2004 that were subsequently updated in 2007. A comparison of seismic analyses that use the original data with seismic analyses that use the revised data demonstrates a 35-ft-thick alluvium depth is bounding, resulting in valid foundation designs for updated alluvium depths.

Seismic analyses of the CRCF and WHF use lumped-mass, multiple-stick models to determine design forces and moments. Soil-structure interaction effects are represented by sets of soil impedance functions consisting of spring stiffness and damping values for different soil profiles. Strain-compatible soil property data that were qualified in 2004 consist of two soil profiles: 35- and 110-ft-thick alluvium overlying tuff. Global soil spring stiffness values in each of three translational and three rotational directions are determined for the upper bound (84th percentile), median, and lower bound (16th percentile) sets of dynamic soil properties at each soil profile. Response spectrum analyses for each of the six sets of soil-spring stiffness values are performed to encompass the full range of soil conditions. Results indicate that the 35-ft upper bound soil profile provides the highest story shears and diaphragm accelerations and is the controlling case for the subsequent foundation design calculations. Consequently, foundation designs for the CRCF and WHF only use results of the 35-ft upper bound soil profile.

Strain-compatible soil property data were revised in 2007 for soil profiles at updated alluvium depths. Revised sets of global spring stiffness values were determined from soil properties corresponding to 100- and 200-ft-thick alluvium overlying tuff for the CRCF and 30- and 100-ft-thick alluvium overlying tuff for the WHF. Response spectrum analyses were performed for each revised set of soil spring stiffness values. Comparing results from the seismic analyses that use the revised data to the seismic analyses that use the 35-ft upper bound data indicates that the original case produces higher story shears, diaphragm accelerations, and story drifts than the updated soil profiles, with the exception of the basemat horizontal acceleration in the east-west direction for the CRCF. However, the slight increase (less than 10%) in horizontal acceleration at the basemat does not impact the CRCF foundation design because the resulting horizontal forces primarily affect the sliding stability. Additionally, the vertical acceleration at the basemat that controls the CRCF foundation design forces is substantially reduced (approximately 39%) using the updated soil profiles. Therefore, the foundation designs for the CRCF and WHF, which were based on original data results, bound results for the updated soil profiles.

2. COMMITMENTS TO NRC

None.

3. DESCRIPTION OF PROPOSED LA CHANGE

None.

RAI Volume 2, Chapter 2.1.1.7, Eleventh Set, Number 12:

> The analyses of the foundation using finite element models (e.g., BSC, 2007ae) only include translational springs per unit area derived from global soil springs. Provide technical bases to justify excluding the effects of global rotational springs in the foundation design.

1. RESPONSE

The foundation design of the typical important to safety (ITS) facility (e.g., the Canister Receipt and Closure Facility evaluates both global translational and rotational springs. The smaller (i.e., less stiff) value of the translational and rotational spring per unit area is conservatively used in the design to yield higher bending moments and shear forces in the basemat.

The seismic analysis of the typical ITS facility uses a lumped-mass, multiple-stick model to determine seismic and nonseismic loads. To represent soil-structure interaction effects, a set of six global springs (three translational and three rotational), are used at the foundation's center of rigidity. The foundation design uses a finite element model of the basemat and lower portions of the walls to determine bending moments and shear forces. This model is coupled to the lumped-mass, multiple-stick model used in the seismic analysis to accurately provide seismic and nonseismic loads. To represent the soil underlying the basemat, springs are used at each node of the foundation model. The soil-spring stiffness values are derived from the global springs of the lumped-mass, multiple-stick model using techniques in Appendix C of *Seismic Analysis and Design Approach Document* (BSC 2007). This appendix provides the following formulas for developing the nodal soil-spring stiffness values per unit area in the horizontal and vertical directions (k_h and k_v, respectively) from the horizontal, vertical, and rocking global stiffness values (K_h, K_v, and K_φ, respectively):

$$k_h = K_h/A \qquad \text{(Eq. C-1)}$$

$$k_v = K_v/A \qquad \text{(Eq. C-2)}$$

$$k_v = K_\varphi / I_A \qquad \text{(Eq. C-3)}$$

where A is the plan area of the mat foundation, and I_A is the moment of inertia of the basemat area with respect to its centroidal axis.

In the foundation design of the typical ITS facility, three vertical spring stiffness values per unit area are determined: one based on the vertical global stiffness, a second based on the rotation about the global Y axis, and a third based on rotation about the global X axis. Results indicate that stiffness values based on rotations (Eq. C-3) are significantly higher than those for vertical translation (Eq. C-2). Stiffer springs will yield lower bending moments and shear forces in the mat foundation. As such, the smaller (less stiff) spring values based on the global translational spring stiffness values (Eq. C-2) are conservatively used to maximize design forces. Spring stiffness results are also consistent with Appendix C of *Seismic Analysis and Design Approach*

Document (BSC 2007) that states the use of the values obtained from Eq. C-2 is usually sufficient.

Appendix C of *Seismic Analysis and Design Approach Document* (BSC 2007) also states that if the rocking motion of a structure is dominant, the spring stiffness values from Eq. C-3 should be used. However, using the Canister Receipt and Closure Facility as a representative example, the maximum horizontal displacement due to rigid body rocking at the top of the structure is less than 12% of the total displacement in the North-South direction for seismic excitation in that direction. In comparison, the displacement due to rigid body translation is more than 72% of the total displacement. The relatively small contribution due to rocking indicates that rocking motion is not dominant. Therefore, the use of Eq. C-2 for the spring stiffness values is appropriate.

2. COMMITMENTS TO NRC

None.

3. DESCRIPTION OF PROPOSED LA CHANGE

None.

4. REFERENCES

BSC (Bechtel SAIC Company) 2007. *Seismic Analysis and Design Approach Document*. 000-30R-MGR0-02000-000-001 ACN 02. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071220.0029; ENG.20090311.0013.