RAI Volume 2, Chapter 2.1.1.7, Eleventh Set, Number 1:

The following questions pertain to DOE's design of the surface facilities (SFs) important-to-safety (ITS) systems. These questions are based on the description and operational process provided in SAR Sections 1.1.2 and 1.2.1, respectively, and design information provided in Sections 1.2.2 through 1.2.6 for the SFs. Additional information is required for staff to determine compliance with 10 CFR 63.21(c) and 63.112(f).

SAR Section 1.2.2.1 provides a general description of the structural analysis and design of the GROA facilities, including general methodologies and design criteria. However, the implementation of these methods to specific facilities is not included in the SAR and supporting references. Provide the analysis and design of the following structural components of the ITS facilities:

a) For the Canister Receipt and Closure Facility (CRCF), Wet Handling Facility (WHF), and Initial Handling Facility (IHF):

   i) Analysis and design of steel and concrete components of the floor and roof slabs, and

   ii) Design of shear wall-slab joints and structural discontinuities.

b) For the CRCF:

   i) Design of reinforced concrete shear walls.

c) For the Receipt Facility (RF):

   i) Demonstrate that the analysis and design methods implemented for the RF are identical or similar to the methods implemented for the CRCF since both facilities are similar reinforced concrete structures, or provide the analysis and design information of the RF to the same questions requested in (a)(i) and (ii), and (b)(i) above.

d) For the WHF:

   i) Pool sloshing evaluation,

   ii) Analysis and design of pool-foundation joints, and

   iii) Demonstrate that the analysis and design methods implemented for the shear walls of the WHF are identical to the methods implemented for the CRCF since both facilities are similar reinforced concrete structures, or provide the analysis and design information for the shear walls of the WHF.

e) For the IHF:

i) The IHF design loads document, and

ii) Attachment D of BSC (2008am).

1. RESPONSE

1.1 CANISTER RECEIPT AND CLOSURE FACILITY, WET HANDLING FACILITY, AND INITIAL HANDLING FACILITY

1.1.1 Floor and Roof Slabs

The analysis and design of steel and concrete components of the floor and roof slabs for the Canister Receipt and Closure Facility (CRCF) and Wet Handling Facility (WHF) are contained in the following calculations, which are included as part of this response:

1. *CRCF Diaphragm Design* (BSC 2007a)
2. *CRCF Interior Structural Steel Design* (BSC 2007b)
3. *Wet Handling Facility (WHF) Concrete Slab and Diaphragm Design* (BSC 2007c)
4. *Wet Handling Facility Structural Steel Framing* (BSC 2007d)

The analysis and design of steel and concrete components of the floor and roof slabs for the Initial Handling Facility (IHF) are contained in the following calculations:

1. *Initial Handling Facility (IHF): Concrete Structure Design* (BSC 2007e)
2. *IHF Steel Structure Seismic Analysis and Steel Member Design* (BSC 2008a).

1.1.2 Joints and Structural Discontinuities

Section 21.5 of ACI 349-01/349R-01, *Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-01) and Commentary (ACI 349R-01)* and Section 6.2.2 of ASCE/SEI 43-05, *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities*, provide specific requirements for the design of joints. The requirements are specific to the design of joints in frames (e.g., beam-column joints and slab-wall moment frames) that resist seismic lateral loading by out-of-plane bending. The typical reinforced concrete important to safety facilities (e.g., the CRCF, WHF, and Receipt Facility (RF)) are not slab-wall moment frames, but are low-rise shear wall structures that resist lateral load with shear walls and diaphragms in both directions. However, although not strictly required by code, transverse hoops or crossties will be provided at roof slab-shear wall and floor slab-pool wall joints to provide confinement of the reinforcement, as shown in the typical sections in SAR Figures 1.2.2-1 through 1.2.2-3.

The joint details (e.g., seismic hook and tension splice lengths, size and spacing of crossties) are considered aspects of the detailed structural design for construction and will be designed in accordance with code requirements. Additionally, local stress concentrations around discontinuities (e.g., openings and penetrations) will be evaluated and detailed accordingly. Likewise, any areas that potentially exhibit frame behavior will be further evaluated to ensure that lateral loads can be transferred to perpendicular shear walls through diaphragm action or will be designed and detailed as a frame in accordance with the code requirements mentioned earlier.

1.2 CRCF

The design of the reinforced concrete shear walls for the CRCF is contained in the following calculation included as part of this response:

1. *CRCF Shear Wall Design* (BSC 2007f).

1.3 RECEIPT FACILITY

These analyses and design methods for the steel and concrete components of the RF are similar to the methods implemented for the CRCF.

1.3.1 Floor and Roof Slabs

The analysis and design of steel and concrete components of the floor and roof slabs for the RF are contained in the following calculations included as part of this response:

1. *Receipt Facility Concrete Diaphragm Design* (BSC 2008b)

2. *Receipt Facility (RF) Structural Steel Framing Design* (BSC 2007g).

1.3.2 Joints and Structural Discontinuities

The design of shear wall-slab joints and structural discontinuities is discussed in Section 1.1.2 of this response.

1.3.3 Shear Walls

The design of the reinforced concrete shear walls for the RF is contained in the following calculation included as part of this response:

1. *Receipt Facility (RF) Shear Wall Design* (BSC 2007h).

1.4 WHF

The shear wall design methodology for the WHF is similar to the method implemented for the CRCF.

1.4.1 Pool Sloshing

The pool sloshing evaluations for the WHF are contained in the following calculations included as part of this response. These calculations determine the water pressures imposed on the pool walls based on the 2004 seismic ground motions and provide a comparison of the accelerations between the two that demonstrates that the use of the 2004 ground motion results in the pool wall design bounds the 2007 ground motion results:

1. *WHF Pool Sloshing Evaluation* (BSC 2007i)

2. *Comparison of WHF Fuel Pool Sloshing for the 2004 Versus 2007 Seismic Input Ground Motions for the Wet Handling Facility (WHF)* (BSC 2008c).

1.4.2 Pool-Foundation Joints

The analysis and design of pool-foundation joints are discussed in Section 1.1.2 of this response.

1.4.3 Shear Walls

The design of the reinforced concrete shear walls for the WHF is contained in the following calculation included as part of this response:

1. *WHF Shear Wall Design* (BSC 2007j).

1.5 IHF

1.5.1 Design Loads

The design loads for the IHF are contained in the following calculation included as part of this response:

1. *Initial Handling Facility (IHF): Design Loads for the Steel and Concrete Structures* (BSC 2007k).

1.5.2 Crane Load Case Input and Output Files

The crane load case input and output files for the IHF are contained in Attachment D of the following calculation included as part of this response:

1. *IHF Steel Structure Seismic Analysis and Steel Member Design* (BSC 2008a).

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/SEI 43-05. 2005. *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities*. Reston, Virginia: American Society of Civil Engineers. TIC: 257275.

BSC (Bechtel SAIC Company) 2007a. *CRCF Diaphragm Design*. 060-DBC-CR00-00300-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070320.0005.

BSC 2007b. *CRCF Interior Structural Steel Design*. 060-SSC-CR00-00300-000-00B. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070817.0002; ENG.20080516.0001.

BSC 2007c. *Wet Handling Facility (WHF) Concrete Slab and Diaphragm Design*. 050-DBC-WH00-00100-000-00B. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070921.0008.

BSC 2007d. *Wet Handling Facility Structural Steel Framing*. 050-SSC-WH00-00100-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070510.0006; ENG.20070802.0012.

BSC 2007e. *Initial Handling Facility (IHF): Concrete Structure Design*. 51A-DBC-IH00-00100-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071205.0021; ENG.20080303.0025.

BSC 2007f. *CRCF Shear Wall Design*. 060-DBC-CR00-00100-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070301.0008.

BSC 2007g. *Receipt Facility (RF) Structural Steel Framing Design*. 200-SSC-RF00-00100-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070515.0012.

BSC 2007h. *Receipt Facility (RF) Shear Wall Design*. 200-DBC-RF00-00100-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070322.0011.

BSC 2007i. *WHF Pool Sloshing Evaluation*. 050-SYC-WH00-00400-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070227.0007.

BSC 2007j. *WHF Shear Wall Design*. 050-DBC-WH00-00200-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20070524.0006.

BSC 2007k. *Initial Handling Facility (IHF): Design Loads for the Steel and Concrete Structures*. 51A-SYC-IH00-00700-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071106.0008.

BSC 2008a. *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; ENG.20080303.0026.

BSC 2008b. *Receipt Facility Concrete Diaphragm Design*. 200-DBC-RF00-00200-000-00C. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20080129.0008.

BSC 2008c. *Comparison of WHF Fuel Pool Sloshing for the 2004 Versus 2007 Seismic Input Ground Motions for the Wet Handling Facility (WHF)*. 050-SYC-WH00-01500-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20080131.0005; ENG.20080304.0015.

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

> The analysis and design of the ITS buildings are based on one seismic load combination (e.g., BSC 2007ba, Appendix A; and BSC 2007af, Section 6.1), instead of the set of load combinations indicated in SAR Section 1.2.2.1.9.2. Thus, provide structural analysis and design for the ITS buildings that include the load combinations presented in SAR Section 1.2.2.1.9.2. Alternatively, demonstrate that the load combination(s) considered in the analysis and design bound the structural response of the ITS buildings.

1. RESPONSE

The load combinations presented in SAR Section 1.2.2.1.9.2 are based on the combinations given in ACI 349-01/349R-01, *Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-01) and Commentary (ACI 349R-01)* and ANSI/AISC N690-1994, *American National Standard Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities*. The load combinations are intended to comprehensively include all the load sources that may impact the nuclear safety related structures. Depending on the use of a particular facility, some of the individual load sources are either not applicable to the design of the structure or are bounded by other load sources in the controlling load combination.

For the design of the important to safety (ITS) buildings, the following loads fall into this category:

- Ash load, A: Loads due to volcanic ash fall are considered as roof live loads, L_r. The typical roof live load of 40 pounds per square foot (psf) bounds the expected ash load of 21 psf.

- Snow and ice loads, S_N: Loads due to the accumulation of snow are compared to roof live load, L_r, for use in the load combinations. The typical roof live load of 40 psf bounds the typical base snow load of 12 psf.

- Fluid loads, F: Loads due to the weight and pressure of fluids are excluded in the analysis and design of the Canister Receipt and Closure Facility (CRCF), Initial Handling Facility and Receipt Facility because these facilities are located above the potential flood plain and do not contain a source of significant internal fluid loads. The analysis and design of the Wet Handling Facility (WHF) subgrade structure includes the fluid loads from the pool in the load combinations used.

- Lateral earth pressure loads, H: Loads due to lateral earth pressure are excluded in the analysis and design of the CRCF, Initial Handling Facility, and Receipt Facility because these facilities do not have any major subgrade portions. The analysis and design of the WHF subgrade structure includes the lateral earth pressure loads in the load combinations used.

- Wind loads, W and W_t: Loads due to normal wind, W, and tornado wind, W_t, are compared to earthquake loads, E, for use in the load combinations. Using the CRCF as a representative example, the resulting base shears due to wind and tornado wind loads are less than 10% of the base shear due to earthquake loads. Similarly, the local out-of-plane forces on the external walls due to wind and tornado loads are less than half of the seismic out-of-plane forces. Thus, the earthquake loads bound the wind and tornado loads.

- Operating pipe reaction loads, R_o: Loads due to pipe support reactions are minimal because the ITS facilities do not contain a significant amount of piping or any high energy piping systems. The analysis and design of the ITS concrete facilities include a distributed equipment dead load of 100 psf on the floor slabs and 10 psf on the roof slabs, and a live load of 100 psf on the floor slabs and 40 psf on the roof slabs that envelope any reactions due to R_o.

- Thermal loads, T_o and T_a: Loads due to normal operating thermal conditions, T_o, are minimal because the temperature difference for normal operating conditions is approximately 30°F between day and night. The expansion/contraction of the facilities that results from the temperature differential are accommodated within expected construction tolerances. Loads due to event sequence thermal conditions, T_a, are excluded because the ITS facilities do not contain high energy systems.

Removing the non-applicable and non-controlling design loads, the typical primary load combinations for the ITS concrete facilities become:

$$U = 1.4D + 1.7L + 1.7L_r + 1.4F + 1.7H$$

$$U = D + L + L_r + E + F + H$$

Note: F and H loads are only applicable to the WHF subgrade structure. All variables provided in the above equations are defined in SAR Section 1.2.2.1.9.1.

The second load combination that includes earthquake loads controls over the first load combination that represents a factored gravity load case because the design seismic ground motion results in significant earthquake loads.

The use of the seismic load combination considered in the typical analysis bounds the structural response of the ITS facilities. Therefore, the resulting design satisfies the load combinations presented in SAR Section 1.2.2.1.9.2.

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.

ANSI/AISC N690-1994. *American National Standard Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities*. Chicago, Illinois: American Institute of Steel Construction. TIC: 252734.

RAI Volume 2, Chapter 2.1.1.7, Eleventh Set, Number 3:

> Justify using 25 percent of the live load for seismic load combinations, instead of the factors proposed in the load combinations presented in SAR Section 1.2.2.1.9.2.
>
> Basis: According to DOE (BSC 2007ba, Section 8.3.1), the 25 percent factor for live load was obtained from International Building Code (ICC, 2006; Section 1617.5.1). This IBC section, however, refers to the effective seismic weight, not to load combinations.

1. RESPONSE

In the determination of seismic forces on structures, ASCE 4-98, *Seismic Analysis of Safety-Related Nuclear Structures and Commentary*, and ASCE/SEI 43-05, *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities*, state that the applicable portion of the design live load that is considered present during a seismic event shall be included in the effective seismic mass. The *International Building Code 2000, with Errata to the 2000 International Building Code* (ICC 2003) and ASCE 7-98, *Minimum Design Loads for Buildings and Other Structures*, provide a factor of 25% as the part of live load to be considered in the effective seismic mass.

The load combinations presented in SAR Section 1.2.2.1.9.2 are based on the combinations given in ACI 349-01/349R-01, *Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-01) and Commentary (ACI 349R-01)* and ANSI/AISC N690-1994, *American National Standard Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities*. These consensus design codes do not address applicable portions of load sources in regards to the load combinations. However, it is reasonable to extrapolate that if only a portion of a load is considered present during an earthquake, then the identical portion is present in combination with other load sources. This approach provides consistency in the application of loads.

Consideration of the full live load factor in the load combinations in SAR Section 1.2.2.1.9.2 does not impact the overall design of the important to safety structures because the magnitude of the live loads is relatively small compared to the other load sources, and significant margin exists in the design. Using the Canister Receipt and Closure Facility as a representative example, and considering the full live load factor in the applicable load combinations, the total demand increases by less than 5% for the diaphragm impacted the most. The slightly increased demand is still significantly less than the capacity. Therefore, the design satisfies the load combinations presented in SAR Section 1.2.2.1.9.2.

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.

ANSI/AISC N690-1994. *American National Standard Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities*. Chicago, Illinois: American Institute of Steel Construction. TIC: 252734.

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

ASCE 7-98. 2000. *Minimum Design Loads for Buildings and Other Structures*. Revision of ANSI/ASCE 7-95. Reston, Virginia: American Society of Civil Engineers. TIC: 247427.

ASCE/SEI 43-05. 2005. *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities*. Reston, Virginia: American Society of Civil Engineers. TIC: 257275.

ICC (International Code Council) 2003. *International Building Code 2000, with Errata to the 2000 International Building Code*. Falls Church, Virginia: International Code Council. TIC: 251054; 257198.

RAI Volume 2, Chapter 2.1.1.7, Eleventh Set, Number 4:

Provide technical basis to support the percent of critical structural damping values used to analyze the seismic response of reinforced concrete ITS buildings, which appear to be inconsistent with the expected demand-to-capacity (D/C) ratios of the structural components. In addition, provide the rationale for using the damping recommendations of ASCE 43-05 (2005b), instead of NUREG-0800 (NRC, 2007a).

Basis: For reinforced concrete structures, DOE has selected the percentage of structural damping value, ξ, as a function of the expected D/C ratio (BSC, 2007ba, Section 7.2.4.2; and ASCE, 2005b, Section 3.4.3). According to ASCE (2005b), $\xi = 4$ percent is recommended when D/C ratios are expected to be 0.5 or smaller, and $\xi = 7$ percent for D/C ratios between 0.5 and 1.0. For analysis involving Design Basis Ground Motion-2 (DBGM-2) seismic events, DOE has used $\xi = 7$ percent. However, DOE indicates that the D/C ratio should be less than 0.5-0.6 for SSCs designed to DBGM-2 (BSC, 2007ba; Section 8.4). Thus, for D/C ratios less than 0.5, ξ would be 4 percent, according to ASCE (2005b).

Moreover, NUREG-0800 (NRC, 2007a; Section 3.7.1) indicates that ξ should be in accordance with Regulatory Guide 1.61 (NRC, 2007b), which is based on NUREG/CR-6919 (Morante, 2006; Section 3.1.3). These guidelines state that ξ = 4 percent should be used when the expected D/C ratios are 0.8 or smaller.

1. RESPONSE

The seismic response analyses of important to safety (ITS) surface facilities use critical damping values that are in accordance with Section 3.4.3 of ASCE/SEI 43-05, *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities*, and are consistent with the provisions of NUREG/CR-6919, *Recommendations for Revision of Seismic Damping Values in Regulatory Guide 1.61* (Morante 2006), and Regulatory Guide 1.61, *Damping Values for Seismic Design of Nuclear Power Plants* as recommended by NUREG-0800. Per SAR Section 1.2.2.1.6.3.1.3 and SAR Table 1.2.2-9, the damping values provided in ASCE/SEI 43-05 are an acceptable alternative to Regulatory Guide 1.61.

1.1 DAMPING VALUES USED IN THE ANALYSES FOR DEVELOPMENT OF IN-STRUCTURE RESPONSE SPECTRA

The direct-integration time history analyses used to generate in-structure response spectra use the Response Level 1 critical damping value $\xi = 4\%$ that is consistent with the demand to capacity ratio limits in ASCE/SEI 43-05 and is identical to the damping value specified in Table 2 of Regulatory Guide 1.61 for reinforced concrete structures.

The in-structure response spectra that will be used for the design of ITS subsystems and components are developed from the time history analyses of the lumped-mass multiple-stick models using the direct integration method. Table 3-3 of ASCE/SEI 43-05 provides the estimated damping response level for use in the generation of in-structure response spectra based on the demand to capacity ratio. Response Level 1 is represented by a demand to capacity ratio ≤ 0.5 that is consistent with the demand to capacity ratio limits recommended in Section 8.4 of *Seismic Analysis and Design Approach Document* (BSC 2007) for design of structures to design basis ground motion (DBGM)-2 seismic loads. In accordance with Tables 3-2 and 3-4 of ASCE/SEI 43-05, the Response Level 1 critical damping value of 4% is used in time history analyses to represent the dissipation of energy in the reinforced concrete structures. Table 2 of Regulatory Guide 1.61 specifies a value of critical damping $\xi = 4\%$ for reinforced concrete structures that is identical to the Response Level 1 damping value used in the analyses for generation of in-structure response spectra. Section 1.2 of Regulatory Guide 1.61 specifies that use of the damping values in Table 2 for generation of in-structure response spectra is acceptable.

1.2 DAMPING VALUES USED IN THE ANALYSES FOR THE DESIGN OF THE REINFORCED CONCRETE STRUCTURES

The design of reinforced concrete structural members is based on seismic demands obtained from response spectrum analyses using the Response Level 2 critical damping ratio $\xi = 7\%$ for DBGM-2 seismic loads. The use of the damping values for structural evaluations of reinforced concrete members is consistent with the provisions of ASCE/SEI 43-05 and Section 1.2 of Regulatory Guide 1.61.

The reinforced concrete structural members of the ITS facilities are designed using the results from the response spectrum analyses of the lumped-mass multiple-stick models under DBGM-2 seismic loads. In accordance with Tables 3-2 and 3-4 of ASCE/SEI 43-05, the Response Level 2 critical damping value of 7% is used for evaluation of seismic-induced forces and moments in reinforced concrete structural members by elastic analysis at Limit State D. Table 1 of Regulatory Guide 1.61 specifies a value of critical damping $\xi = 7\%$ for reinforced concrete structures that is identical to the Response Level 2 damping value used in the analyses for design of the ITS structural members. Section 1.2 of Regulatory Guide 1.61 specifies that use of the damping values in Table 1 may be inconsistent with the predicted structural response level for design that is significantly below code stress limits. However, Section 1.2 further states that for structural evaluation, this is not a concern because the stresses resulting from damping-compatible structural response will still be less than the code stress limits.

Therefore, the percent of critical structural damping values used to analyze the seismic response of the ITS reinforced concrete facilities for design and generation of in-structure response spectra are appropriate. The ASCE/SEI 43-05 damping values used are consistent with Regulatory Guide 1.61 as recommended by NUREG-0800.

2. COMMITMENTS TO NRC

None.

3. DESCRIPTION OF PROPOSED LA CHANGE

None.

4. REFERENCES

ASCE/SEI 43-05. 2005. *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities*. Reston, Virginia: American Society of Civil Engineers. TIC: 257275.

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

Morante, R.J. 2006. *Recommendations for Revision of Seismic Damping Values in Regulatory Guide 1.61*. NUREG/CR-6919. Washington, D.C.: U.S. Nuclear Regulatory Commission. ACC: MOL.20070417.0050.

Regulatory Guide 1.61, Rev. 1. 2007. *Damping Values for Seismic Design of Nuclear Power Plants*. Washington, D.C.: U.S. Nuclear Regulatory Commission. ACC: MOL.20070926.0078.

RAI Volume 2, Chapter 2.1.1.7, Eleventh Set, Number 5:

Justify using a percentage of structural damping, ξ, of 7 percent for the Initial Handling Facility (IHF) steel frame system.

Basis: A damping value of ξ = 7 percent is recommended by ASCE (2005b) and Regulatory Guide 1.61 (NRCb, 2007), which is based on NUREG/CR-6919 (Morante, 2006). This damping value, however, is specified for containment structures, containment internal structures, and other seismic category I structures. Demonstrate that these specifications can be applied to the steel frames of the IHF facility, while most of the steel frame structures usually report ξ values lower than 5 percent (e.g., Satake et al, 2003).

Moreover, DOE specified ξ = 7 percent for bearing-bolted steel structures for ITS response level 2, which corresponds to a D/C ratio between 0.5 and 1.0 (BSC, 2008am; Section 4.3). The analysis results involving DBGM-2 load combinations, however, show that most steel components have D/C ratios lower than 0.5 (BSC 2008am; Section 7.1.4).

1. RESPONSE

The seismic response analyses for the steel framed portion of the Initial Handling Facility (IHF) use critical damping values that are in accordance with Section 3.4.3 of ASCE/SEI 43-05, *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities*, and are consistent with the provisions of NUREG/CR-6919, *Recommendations for Revision of Seismic Damping Values in Regulatory Guide 1.61* (Morante 2006), and Regulatory Guide 1.61, *Damping Values for Seismic Design of Nuclear Power Plants*.

1.1 DAMPING VALUES USED IN THE ANALYSES FOR DEVELOPMENT OF IN-STRUCTURE RESPONSE SPECTRA

The direct integration time history analyses used to generate in-structure response spectra use the Response Level 1 critical damping value ξ = 4% that is consistent with the demand to capacity ratio limits in ASCE/SEI 43-05 and is lower (more conservative) than the damping value ξ = 5% specified in Table 2 of Regulatory Guide 1.61 for bearing-bolted steel structures.

The in-structure response spectra that will be used for design of IHF important to safety subsystems and components are developed from time history analyses of the finite element model for the steel frame structure using the direct integration method. Table 3-3 of ASCE/SEI 43-05 provides the estimated damping response level for use in the generation of in-structure response spectra based on the demand to capacity ratio. Response Level 1 is represented by a demand to capacity ratio ≤ 0.5 that is consistent with the demand to capacity ratio limits recommended in Section 8.4 of *Seismic Analysis and Design Approach Document* (BSC 2007) for design of structures to design basis ground motion (DBGM)-2 seismic loads. In accordance with Tables 3-2 and 3-4 of ASCE/SEI 43-05, the Response Level 1 critical damping value of 4% is used in the time history analyses to represent the dissipation of energy in the IHF steel structure. Table 2 of Regulatory Guide 1.61 specifies a value of critical damping ξ = 5% for bolted steel structures with bearing connections that is higher (less conservative) than the Response Level 1 damping value ξ = 4% used in the analyses for generation of in-structure response spectra. Section 1.2 of Regulatory Guide 1.61 specifies that use of the damping values in Table 2 for generation of in-structure response spectra is acceptable.

1.2 DAMPING VALUES USED IN THE ANALYSES FOR DESIGN OF THE INITIAL HANDLING FACILITY STEEL STRUCTURE

The design of the IHF structural steel members is based on seismic demands obtained from response spectra analyses using the Response Level 2 critical damping ratio ξ = 7% for DBGM-2 seismic loads. The use of the damping value for structural evaluations of the steel members is consistent with the provisions of ASCE/SEI 43-05, and Section 1.2 of Regulatory Guide 1.61.

The structural members of the IHF steel structure are designed using the results from the response spectrum analyses of the finite element model under DBGM-2 seismic loads. In accordance with Tables 3-2 and 3-4 of ASCE/SEI 43-05, the Response Level 2 critical damping value of 7% is used for evaluation of seismic-induced forces and moments in bearing-bolted steel structural members by elastic analysis at Limit State D. Table 1 of Regulatory Guide 1.61 specifies a value of critical damping ξ = 7% for bolted steel structures with bearing connections that is identical to the Response Level 2 damping value used in the analyses for design of the structural members. Section 1.2 of Regulatory Guide 1.61 specifies that use of the damping values in Table 1 may be inconsistent with the predicted structural response level for a design that is significantly below code stress limits. However, Section 1.2 further states that for structural evaluation, this is not a concern because the stresses resulting from damping-compatible structural response will still be less than the code stress limits.

1.3 EVALUATION OF DAMPING STUDY

Damping Evaluation Using Full-Scale Data of Buildings in Japan (Satake et al. 2003) presents a damping evaluation study based on full scale data collected from measurements performed on 137 steel framed commercial and residential buildings in Japan. Most of the data used for the study has been obtained from vibration tests with small amplitudes of the measured response. The study provides empirical formulas for prediction of damping ratios based on their dependence on the natural frequency of the structure and the amplitude of the dynamic response where the damping ratio increases proportionally to the natural frequency of the building and the amplitude of the dynamic response.

Per Table 6.6.3 of *IHF Steel Structure Seismic Analysis and Steel Member Design* (BSC 2008), the modal analysis of the IHF steel structure yields natural frequencies that range from approximately 2.3 to 3.8 Hz in the horizontal directions depending on the crane position and direction considered. Per equation (5a) of the damping evaluation (Satake et al. 2003), the natural frequencies of vibration in the horizontal directions correspond to first mode damping values ranging from 3% to 5%. These damping values are only applicable for small amplitude dynamic response. The damping values for larger amplitude responses increase proportionally to the non-dimensional amplitude of the response (x/H) as shown in equations (6) and (9) of the damping evaluation (Satake et al. 2003). However, the non-dimensional amplitude of the response is limited to 2×10^{-5}. In contrast, the IHF steel structure will experience significantly higher response amplitudes under DBGM-2 seismic excitation: conservatively using the minimum displacement from Table 7.1.11 of *IHF Steel Structure Seismic Analysis and Steel Member Design* (BSC 2008) of 0.97 inches at the roof elevation (building height) of 105 ft yields a displacement amplitude response of $0.97/(105 \times 12) = 7.7 \times 10^{-4}$, or more than an order of magnitude greater than the 2×10^{-5} limit. As such, the prediction of first mode damping ratios in the damping evaluation (Satake et al. 2003) are not applicable to the important to safety facilities like the IHF steel structure that are subjected to high amplitude dynamic response.

Therefore, the percent of critical structural damping values used to analyze the seismic response of the IHF bolted steel structure for design (7%) and generation of in-structure response spectra (4%) are appropriate and in accordance with the damping values specified in ASCE/SEI 43-05 and consistent with the values specified by Regulatory Guide 1.61.

2. COMMITMENTS TO NRC

None.

3. DESCRIPTION OF PROPOSED LA CHANGE

None.

4. REFERENCES

ASCE/SEI 43-05. 2005. *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities.* Reston, Virginia: American Society of Civil Engineers. TIC: 257275.

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

BSC 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; ENG.20080303.0026.

Morante, R.J. 2006. *Recommendations for Revision of Seismic Damping Values in Regulatory Guide 1.61*. NUREG/CR-6919. Washington, D.C.: U.S. Nuclear Regulatory Commission. ACC: MOL.20070417.0050.

Regulatory Guide 1.61, Rev. 1. 2007. *Damping Values for Seismic Design of Nuclear Power Plants*. Washington, D.C.: U.S. Nuclear Regulatory Commission. ACC: MOL.20070926.0078.

Satake, N.; Suda, K.; Arakawa, T.; Sasaki, A.; and Tamura, Y. 2003. “Damping Evaluation Using Full-Scale Data of Buildings in Japan.” *Journal of Structural Engineering*, Vol. 129, 470-477. Reston, Virginia: American Society of Civil Engineers.

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> Provide technical bases to demonstrate that the design response spectra presented in BSC (2007ba; Section 6.3.1) properly account for peak broadening of the spectra, as specified in BSC (2007ba; Section 7.3.2.2).

1. RESPONSE

Section 6.3.1 of the *Seismic Analysis and Design Approach Document* (BSC 2007) presents the design basis ground motion response spectra. Section 7.3.2.2 of the *Seismic Analysis and Design Approach Document* (BSC 2007) provides the methodology for generation of in-structure response spectra. Peak broadening as defined in Section 7.3.2.2 (BSC 2007) is not applicable to the development of the ground motion response spectra. It was applied to the development of the in-structure response spectra.

The development of the site-specific ground motion and design spectra is described in *Supplemental Earthquake Ground Motion Input for a Geologic Repository at Yucca Mountain, NV* (BSC 2008) and summarized in SAR Section 1.1.5. Epistemic (knowledge) uncertainty, aleatory (random) variability, and deterministic variability are incorporated in the generation of the ground motions resulting in enveloped hazard curves that provide conservatism. Further clarification on the treatment of uncertainty and variability in the site response modeling is provided in the response to RAI 2.2.1.1.1-005.

The in-structure response spectra for the important to safety facilities are developed by direct integration time history analysis. In accordance with Regulatory Guide 1.122, *Development of Floor Design Response Spectra for Seismic Design of Floor-Supported Equipment or Components*, and ASCE 4-98, *Seismic Analysis of Safety-Related Nuclear Structures and Commentary*, the raw data are smoothed (enveloped) and peaks of the raw data are broadened by ±15% to account for uncertainty in the structural and soil properties.

Therefore, the development of both the design ground response spectra and the in-structure response spectra properly account for uncertainty.

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

BSC 2008. *Supplemental Earthquake Ground Motion Input for a Geologic Repository at Yucca Mountain, NV*. MDL-MGR-GS-000007 REV 00. Las Vegas, Nevada: Bechtel SAIC Company. ACC: DOC.20080221.0001; DOC.20080314.0001; DOC.20080328.0001.

Regulatory Guide 1.122, Rev. 1. 1978. *Development of Floor Design Response Spectra for Seismic Design of Floor-Supported Equipment or Components*. Washington, D.C.: U.S. Nuclear Regulatory Commission. TIC: 2787.