RAI Volume 2, Chapter 2.1.1.1, First Set, Number 1, Supplementary Questions: During a teleconference on February 23, 2009, the NRC Staff asked the DOE four supplementary questions:

(a) i Identify the location of language that shows that uniformity of soil classification implies shear strength. (The NRC Staff referenced page 2, paragraphs 4 and 5 of the original response to support this question.)

ii Identify the location of the information that supports the position that the required shear strength is conservative. (The NRC Staff referenced page 3, paragraph 2 of the original response.)

(b) Identify the location of information that supports the statement that the occurrence of cemented and uncemented alluvium is random. (The NRC Staff referenced page 3, paragraph 4 of the original response.)

(c) Identify the location of information regarding compressibility of very low density tuff. (The NRC Staff referenced page 3, paragraphs 4 and 5 of the original response and page 1.1-125 of the SAR.)

(d) Identify the location of the basis for the conclusion that the allowable bearing pressure for a mat foundation is very high under static conditions and the full range of seismic events. (The NRC Staff referenced page 4, paragraph 1 of the original response.)

1. SUPPLEMENTAL RESPONSE

(a) i *Identify the location of language which shows that uniformity of soil classification implies shear strength.*

Uniformity of soil classification does not imply shear strength. Shear strength was determined based on testing as outlined in Section I.2.2.1 of BSC 2002c from Section 1.10 of the SAR.[1] The resulting strength determination from tested material was applied to like materials, based on material classification. Based on subsurface exploration results, the alluvium is classified mainly as poorly-graded gravel (GP), poorly-graded gravel with silt (GP-GM), well-graded gravel with silt (GW-GM), and poorly-graded sand with gravel (SP)g containing varying amounts of boulders and cobbles throughout the surface Geologic Repository Operations Area (GROA). As discussed further in the paragraphs below, the tested material was classified similarly demonstrating that it is representative of site conditions. Therefore, it is appropriate to apply the shear strength determined from the tested material to the alluvium throughout the surface GROA.

Results from drilling and test pit excavations indicate that alluvial material is consolidated and cemented. When taken as a whole, the material encountered in Midway Valley is laterally discontinuous and layered. However, when taken at the scale of the surface facilities that are categorized as "Important to Safety," it can be interpreted as a homogeneous material with common properties that allow for use of single design parameters (e.g., a friction angle of 39°).

As shown in Figure 6.4.2-42 of *Supplemental Earthquake Ground Motion Input for a Geological Repository at Yucca Mountain, NV*[2], shear wave velocity measurements made within the alluvium throughout the surface facility areas, varied from about 1,200 fps near the surface to about 2,700 fps at a maximum depth of about 130 feet, indicating that the alluvium encountered at the site is composed of strong, relatively incompressible material as noted in Section I.2.2.1 of BSC 2002c.[3]

(a) ii *Identify the location of the information that supports the position that the required shear strength is conservative.*

The development of shear strength is addressed in Section I.2.2.1 in BSC 2002c.[4] First, a conservative approach was selected to determine the strength of the alluvium. Second, the determination of alluvium strength conservatively did not take into account the presence of cementation.

The strength of the alluvium was initially evaluated using two approaches: (1) correlations between measured shear-wave velocity and standard penetration test (SPT) blow count, with the blow counts then correlated to shear strength, and (2) correlations between relative density and shear strength. The second approach was chosen for design since it provided a lower, more conservative shear strength.

Shear strength of soil is a combination of cohesion and internal friction between soil particles. For predominantly granular material, the internal friction generally controls the strength, as the cohesion is relatively minor and generally neglected. The alluvial materials underlying the surface facilities area are granular in nature with varying amounts of cementation. Cementation acts as an additional cohesive-like strength source for the alluvium. However, in determining the shear strength of the alluvium in Section I.2.2.1 of BSC 2002c,[5] cementation and its contribution to the shear strength of the alluvium was not considered. This approach to determining the shear strength is conservative since the design shear strength of the alluvium only considers inter-particle friction.

(b) *Identify the location of information that shows that the occurrence of cemented and uncemented alluvium is random.*

As discussed in (a) i, at the scale of the surface facilities categorized as "Important to Safety," the alluvium behaves in a homogeneous manner.

Lithologic and engineering descriptions of the alluvium and surficial deposits from natural exposures, test pits, trenches, and boreholes typically record qualitative estimates of the amount of cementation or the amount of reactivity to dilute hydrochloric acid, and some have quantitative amounts of cementation (a partial reference list includes summary reports such as *Evaluation of the Location and Recency of Faulting Near Prospective Surface Facilities in Midway Valley, Nye County, Nevada*;[6] *Geotechnical Data for a Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project*;[7] *Quaternary Paleoseismology and Stratigraphy of the Yucca Mountain Area, Nevada*;[8] and *Technical Report: Geotechnical Data for a Geologic Repository at Yucca Mountain,*

Nevada[9]). These documents do not represent a systematic compilation of cementation; however, they indicate that cementation can vary from uncemented to well-developed cementation on small scales within specific beds, and the development of cementation can vary on larger scales within units such as those in the Quaternary.[10,11] Although some units have the tendency to be cemented, there are vertical and lateral variations in the amount of cementation within the unit. These descriptions indicate that (1) uncemented or poorly cemented sediments tend to form the host material for the localized and cemented sediments, (2) locally, the moderately to well-cemented sediments can vary vertically on the scale of one foot to several feet, and (3) by tracing caliche horizons in the test pits, they can be several tens of feet in length. On a scale consistent with the size of the buildings that are categorized as "Important to Safety," that is several hundreds of feet across, there will likely be areas with minimal cementation and local areas (both vertically and laterally) with moderately to well-developed cementation; however, these patterns are variable vertically and laterally.

(c) *Identify the location of information regarding compressibility of very low density tuff.*

Because the soils beneath the surface facilities are granular in nature, the compressibility of the soils due to foundation loads can be determined utilizing elastic theory. To estimate the elastic modulus for use in the compressibility or settlement determination, the measured low-strain shear wave velocity is converted to shear modulus, which in turn is converted to elastic modulus as described in Section I.2.3 of BSC 2002c[12] and Appendix B, Section B6.7 of *Supplemental Soils Report*.[13] The resulting moduli for the granular soils beneath the site are then used to determine structure load settlement behavior due to external loading.[14] Note that a higher modulus results in a stiffer behavior and less compressability. The same procedure is used for determining bedrock moduli (including tuff) as described in Section I.3.3 of BSC 2002c.[15] Therefore, shear wave velocity can be used to compare the compressibility of the alluvium and the underlying tuff.

SAR Figure 1.1-146 shows that "very low density tuff" has laboratory-measured shear wave velocities of 2,000 fps and higher. In addition, Figure 6.5-61 of *Technical Report: Geotechnical Data for a Geologic Repository at Yucca Mountain, Nevada*[16] shows the results of two laboratory-measured shear wave velocity measurements made of the Rainier Mesa Tuff and the ignimbrite of Comb Peak (Tpki). The figure presents a comparison of the shear wave velocity ranges determined with Special Analysis of Surface Waves testing in the Exploratory Studies Facility (ESF) tunnel with the shear-wave velocities measured in "Free-Free unconfined resonant column tests" in the laboratory. For the Rainier Mesa Tuff, both shear-wave velocities are equal to or greater than 1,400 fps. For the ignimbrite of Comb Peak (Tpki), both shear-wave velocities are equal to or greater than 2,800 fps. The average field-measured shear wave velocity profiles for all tuffs encountered at the site, as presented in Figure 6.4.2-37 of *Supplemental Earthquake Ground Motion Input for a Geological Repository at Yucca Mountain, NV*,[17] show all velocities greater than 2,500 fps. By comparison, shear wave velocity measurements made within the alluvium varied from about 1,200 fps near the surface to about 2,700 fps at a maximum depth of about 130 feet.[18] Based on these comparisons, the "very low density tuffs" and the tuffs directly underlying the alluvium at the surface facilities have shear wave velocities equal to or greater than the alluvium,[19] and the compressibility of the tuff is therefore less than or equal to that of the alluvium.

Tuffs whose total unit weight ranged from 60 pcf to 92 pcf were identified as "very low density" in LA SAR Figure 1.1-146 based on laboratory measurements on core samples from lithostratigraphic units below the Tiva Canyon Tuff. SAR Section 1.1.5.3.2.6.2, "Test Program on ESF Specimens," specifically addresses tests done on samples from the ESF. It should be noted that the four "very low density" samples are from the Yucca Mountain and Pah Canyon Tuffs, and the pre-Yucca Mountain and pre-Pah Canyon bedded tuffs that occur between the Tiva Canyon and Topopah Spring Tuffs. Therefore, the evaluations of these specific lithostratigraphic units do not directly contribute to evaluations of properties of the Rainier Mesa Tuff, the pre-Rainier Mesa bedded tuff, and the ignimbrite of Comb Peak (also referred to as "tuff unit 'x'" (Tpki)) that directly underlie the alluvium in Midway Valley.

(d) *Identify the location of the basis for the conclusion that the allowable bearing pressure for a mat foundation is very high under static conditions and the full range of seismic events.*

The basis for the determination of the ultimate bearing capacity (q_{ult}) of alluvium is presented in Attachment IV of BSC 2002c[20] and follows the same methodology as presented in Appendix B of the *Supplemental Soils Report*.[21] The resulting estimates of ultimate bearing capacity for different mat sizes founded on alluvium are presented in Table IV-3 of BSC 2002c.[22] The mat foundation sizes for the Canister Receipt and Closure Facility structures, the largest of the surface facility structures, are about 260 ft wide by 420 ft long. The smallest structure, the Initial Handling Facility, is about 75 ft wide by 142 ft long. Calculated ultimate bearing capacities for various foundation dimensions, presented in Table IV-3 of BSC 2002c,[23] show that ultimate bearing capacity for equivalent footprint area foundations are greater than about 200 ksf. Allowable bearing capacities of 50 ksf and 10 ksf were chosen for use in design evaluations for the seismic design basis ground motion-2 and normal loading cases, respectively.[24] Static foundation pressures for the surface facility structures are on the order of 3 to 5 ksf.[25] Temporary loading due to seismic shaking is larger, but is less than the allowable bearing capacity for alluvium.

2. COMMITMENTS TO NRC

None.

3. DESCRIPTION OF PROPOSED LA CHANGE

None.

4. REFERENCES

[1] BSC (Bechtel SAIC Company) 2002. *Soils Report for North Portal Area, Yucca Mountain Project*. 100-00C-WRP0-00100-000-000. Las Vegas, Nevada: Bechtel SAIC Company. ACC: MOL.20021015.0323; ENG.20050823.0018.

[2] BSC 2008. *Supplemental Earthquake Ground Motion Input for a Geologic Repository at Yucca Mountain, NV*. MDL-MGR-GS-000007 REV 00 ACN 02. Las Vegas, Nevada: Bechtel SAIC Company. ACC: DOC.20080221.0001; DOC.20080314.0001; DOC.20080328.0001.

[3] BSC 2002, *Soils Report for North Portal Area, Yucca Mountain Project*.

[4] Ibid.

[5] Ibid.

[6] Swan, F.H.; Wesling, J.R.; Angell, M.M.; Thomas, A.P.; Whitney, J.W.; and Gibson, J.D. 2001. *Evaluation of the Location and Recency of Faulting Near Prospective Surface Facilities in Midway Valley, Nye County, Nevada*. Open-File Report 01-55. Denver, Colorado: U.S. Geological Survey.

[7] BSC 2002. *Geotechnical Data for a Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project*. ANL-MGR-GE-000003 REV 00. Las Vegas, Nevada: Bechtel SAIC Company. ACC: MOL.20021004.0078.

[8] Keefer, W.R.; Whitney, J.W.; and Taylor, E.M., eds. 2004. *Quaternary Paleoseismology and Stratigraphy of the Yucca Mountain Area, Nevada*. Professional Paper 1689. Reston, Virginia: U.S. Geological Survey. ACC: MOL.20050512.0077.

[9] SNL (Sandia National Laboratories) 2008. *Technical Report: Geotechnical Data for a Geologic Repository at Yucca Mountain, Nevada*. TDR-MGR-GE-000010 REV 00. Las Vegas, Nevada: Sandia National Laboratories. ACC: DOC.20080206.0001.

[10] Swan et al. 2001, *Evaluation of the Location and Recency of Faulting Near Prospective Surface Facilities in Midway Valley, Nye County, Nevada*.

[11] Keefer et al., eds. 2004. *Quaternary Paleoseismology and Stratigraphy of the Yucca Mountain Area, Nevada*.

[12] BSC 2002, *Soils Report for North Portal Area, Yucca Mountain Project*.

[13] BSC 2008. *Supplemental Soils Report*. 100-S0C-CY00-00100-000-00E CACN 001. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20080828.0016; ENG.20081117.0004.

[14] Ibid., Appendix B.

[15] BSC 2002, *Soils Report for North Portal Area, Yucca Mountain Project*.

[16]SNL 2008, *Technical Report: Geotechnical Data for a Geologic Repository at Yucca Mountain, Nevada*.

[17] BSC 2008, *Supplemental Earthquake Ground Motion Input for a Geologic Repository at Yucca Mountain, NV*.

[18] Ibid., Figure 6.4.2-42.

[19] BSC 2002, *Soils Report for North Portal Area, Yucca Mountain Project*.

[20] Ibid.

[21] BSC 2008, *Supplemental Soils Report*.

[22] BSC 2002, *Soils Report for North Portal Area, Yucca Mountain Project*.

[23] Ibid.

[24] BSC 2008, *Supplemental Soils Report*, Section 7.1.2 and Appendix B, page B-42.

[25] Ibid., Table 1-1.

RAI Volume 2, Chapter 2.1.1.1, First Set, Number 2, Supplementary Questions: During a teleconference on February 23, 2009, the NRC Staff asked DOE two supplementary questions:

(a) Identify the location of the analysis that shows the 2H:1V slopes in the alluvium will be stable under static and full range of seismic loading conditions;

(b) Identify the location of information showing the terraced layout for the geometric configurations of slopes and the associated stability analysis.

1. SUPPLEMENTAL RESPONSE

(a) *Identify the location of the analysis that shows the 2H:1V slopes in the alluvium will be stable under static and full range of seismic loading conditions*;

Section 14 of *Soils Report for North Portal Area, Yucca Mountain Project*[1] addresses permanent slopes. Grading of the slopes near the aging pads and the access road to the aging pads and the associated analysis of the slopes for stability are considered to be aspects of detailed design that will be completed in the future. The permanent slopes, which as indicated in SAR section 1.1.5.3.2.1 will be no steeper than 2H:1V, will be evaluated as described in the original response to show that they remain stable, including the load of the site transporter, under normal operating conditions and for the design basis ground motion-2 ground motion.

(b) *Identify the location of information showing the terraced layout for the geometric configurations of slopes and the associated stability analysis*;

Figures 1 and 2, which are annotated versions of SAR Figures 1.2.7-3 and 4, show the general arrangement of the Aging Facility areas 17P and 17R. The figures have been annotated to show a control point for each pad and the associated elevation. Each individual pad is sloped slightly down from west to east at 1/8 in. per 10 ft. The change in control point elevations in Area 17P from north to south is 4.5 ft; the change in control point elevations in Area 17R from north to south is 3 ft for each of the two sets of pads.

Figure 3 is a site plan, not included in the SAR, that shows the Aging Facility areas and the existing topographic contours. This area is within the surface geologic repository operations area and within the flood protection system (see SAR Figure 1.2.2-7). Grading of the existing topography and associated stability analysis are considered to be aspects of detailed design and are scheduled for future completion.

2. COMMITMENTS TO NRC

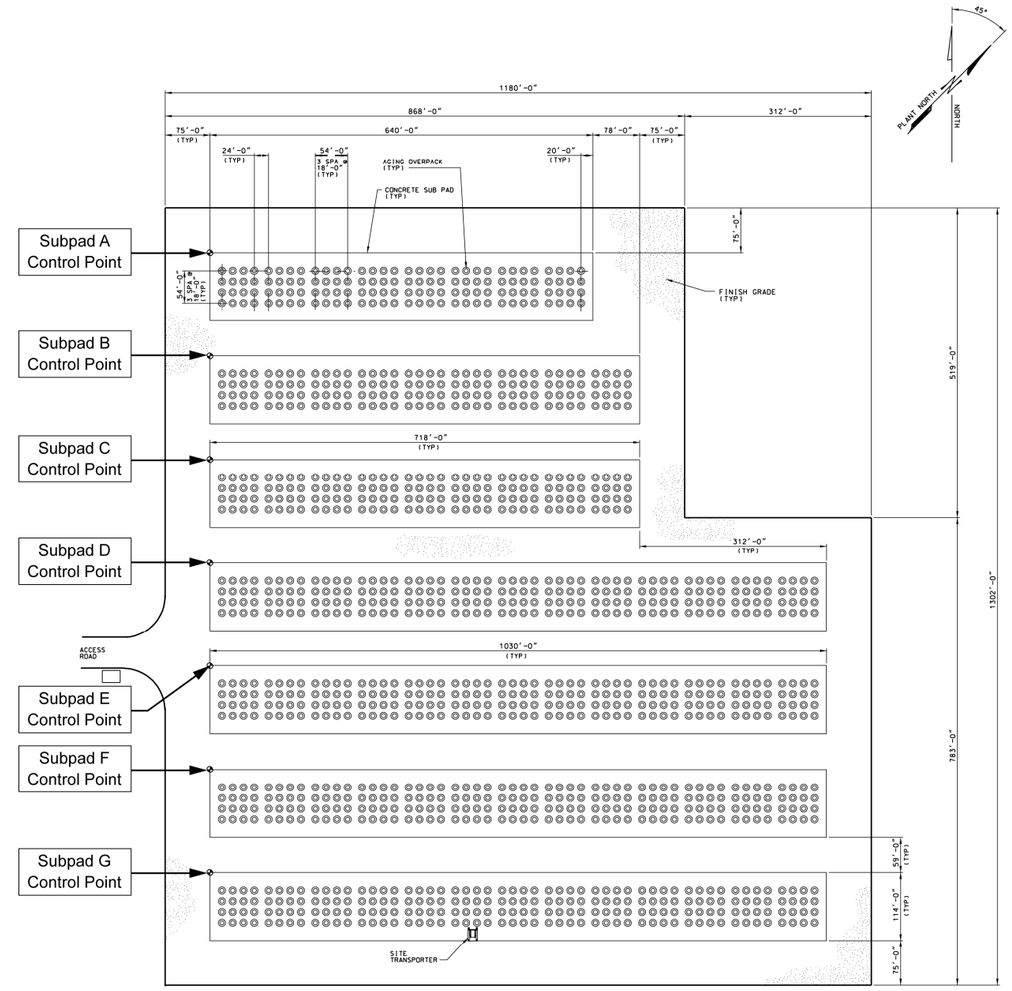
None.

3. DESCRIPTION OF PROPOSED LA CHANGE

None.

4. REFERENCES

[1] BSC (Bechtel SAIC Company) 2002. *Soils Report for North Portal Area, Yucca Mountain Project*. 100-00C-WRP0-00100-000-000 ECN 1. Las Vegas, Nevada: Bechtel SAIC Company. ACC: MOL.20021015.0323; ENG.20050823.0018.



Locations of Subpad Control Points			
Location	Plant North Coordinate (ft)	Plant East Coordinate (ft)	Elevation (ft)
Subpad A Control Point	24,862.00	16,984.00	3,805.00
Subpad B Control Point	24,740.00	17,106.00	3,804.25
Subpad C Control Point	24,617.00	17,229.00	3,803.50
Subpad D Control Point	24,495.00	17,351.00	3,802.75
Subpad E Control Point	24,373.00	17,473.00	3,802.00
Subpad F Control Point	24,250.00	17,596.00	3,801.25
Subpad G Control Point	24,128.00	17,718.00	3,800.50

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Figure 1. Aging Pad 17P Plan

Note: Control Points are located at the Plant Northwest corner of their related subpad. Concrete subpads are to be sloped down 1/8 in. per 10 ft. from west to east

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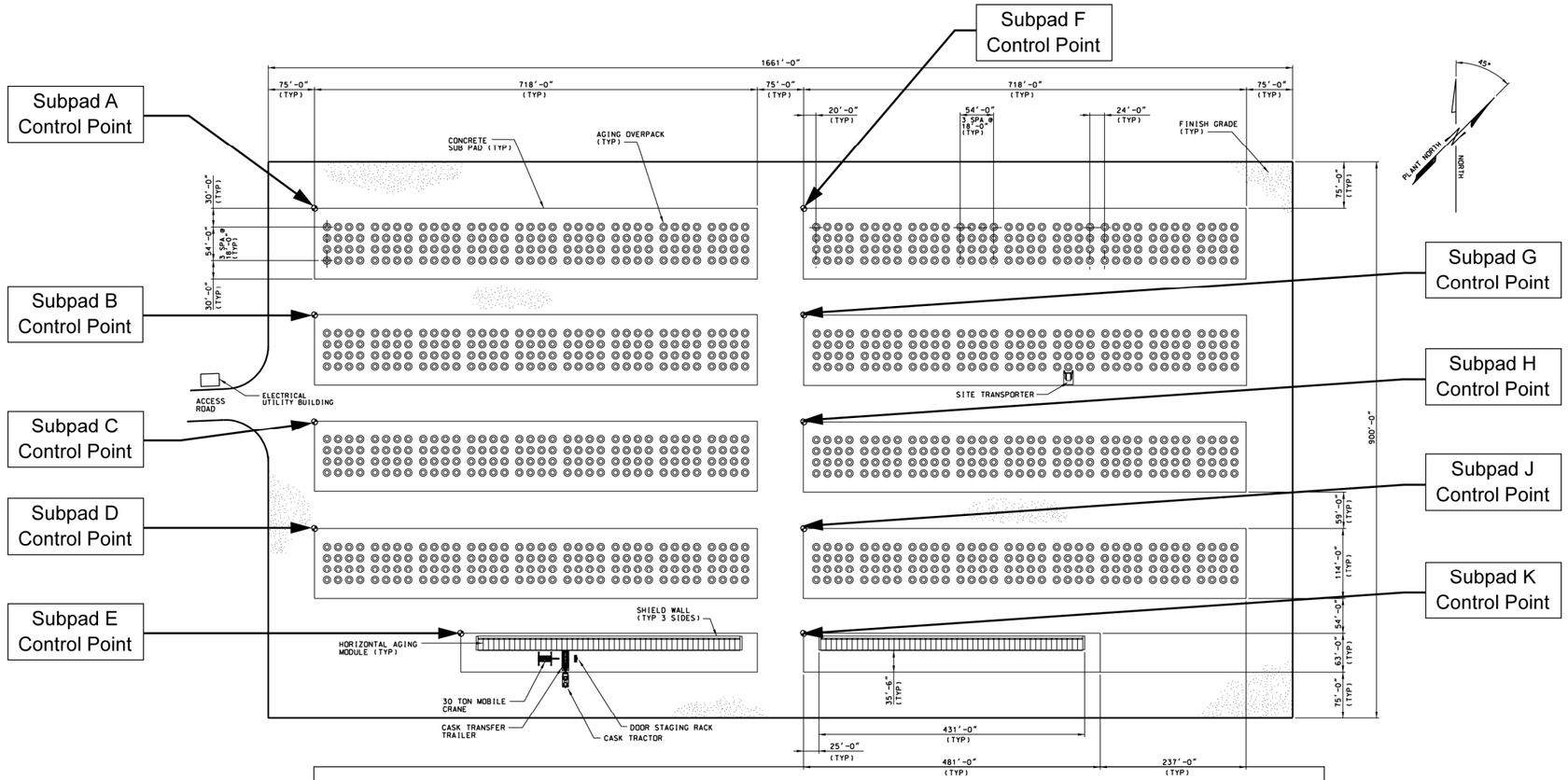

Locations of Subpad Control Points			
Location	Plant North Coordinate (ft)	Plant East Coordinate (ft)	Elevation (ft)
Subpad A Control Point	23,229.00	17,982.00	3,752.50
Subpad B Control Point	23,107.00	18,104.00	3,751.75
Subpad C Control Point	22,985.00	18,226.00	3,751.00
Subpad D Control Point	22,862.00	18,349.00	3,750.25
Subpad E Control Point	22,909.00	18,637.00	3,749.50
Subpad F Control Point	23,790.00	18,543.00	3,750.75
Subpad G Control Point	23,668.00	18,666.00	3,750.00
Subpad H Control Point	23,545.00	18,788.00	3,749.25
Subpad J Control Point	23,423.00	18,910.00	3,748.50
Subpad K Control Point	23,302.00	19.030.00	3,747.75

Figure 2. Aging Pad 17R Plan

Note: Control Points are located at the Plant Northwest corner of their related subpads. Concrete subpads are to be sloped down 1/8 in. per 10 ft. from west to east.

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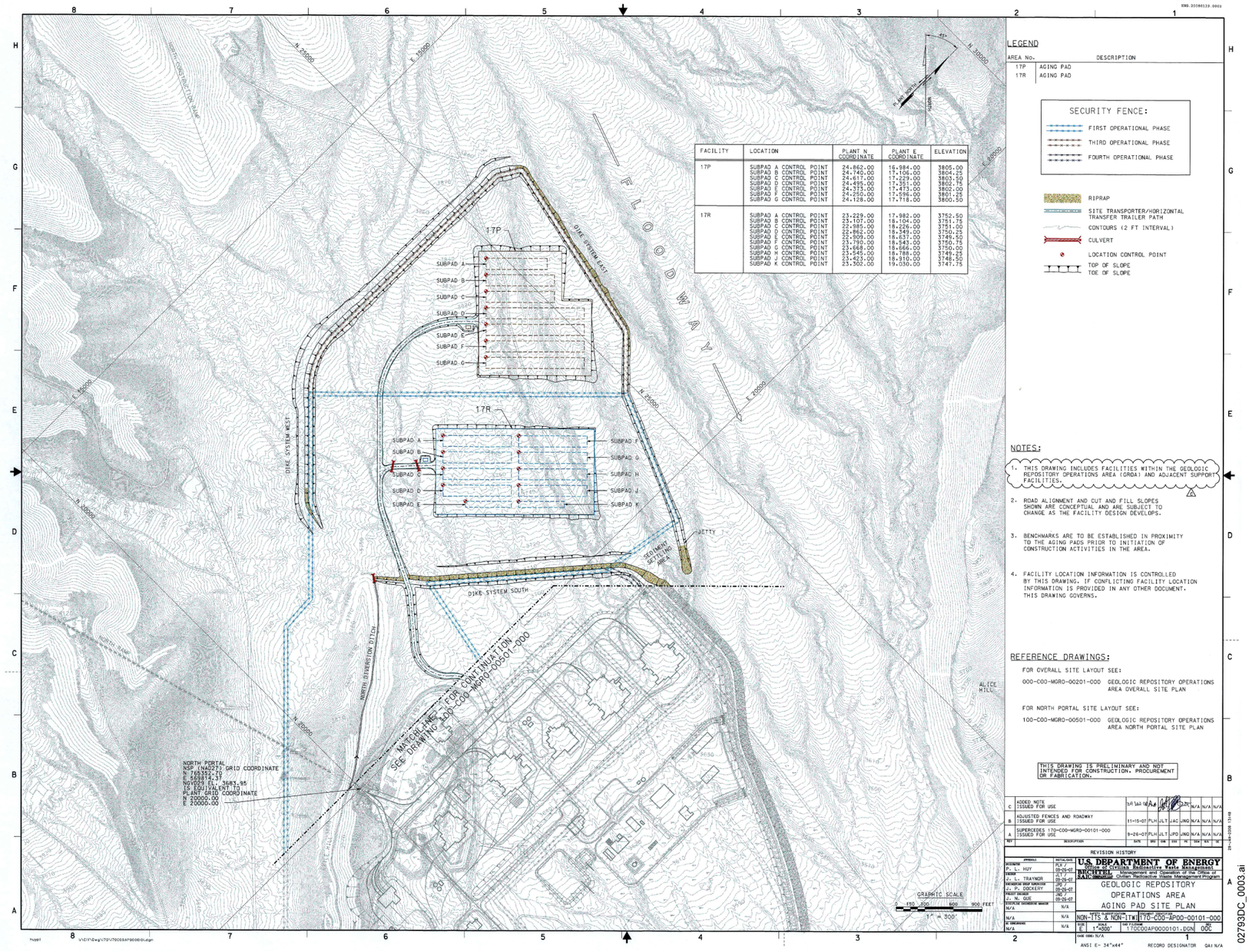

Figure 3. Aging Site Plan

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RAI Volume 2, Chapter 2.1.1.1, First Set, Number 4, Supplementary Questions: During a teleconference on February 23, 2009, the NRC Staff asked DOE one supplementary question:

Identify the location of the analyses of the geotechnical drilling program for the surface facilities.

1. SUPPLEMENTAL RESPONSE

The analyses of the geotechnical drilling program for the surface facilities are reported in the following six documents:

1. *Soils Report for the North Portal Area, Yucca Mountain Project*.[1]
2. *Update of Geotechnical Boreholes and Test Pit Program through 2007 in Midway Valley, Nevada.*[2]
3. *Supplemental Soils Report*.[3]
4. *Geotechnical Data for a Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project*.[4]
5. *Technical Report: Geotechnical Data for a Geologic Repository at Yucca Mountain, Nevada*.[5]
6. *Supplemental Earthquake Ground Motion Input for a Geologic Repository at Yucca Mountain*, *NV*.[6]

Documents 1 and 2 above are enclosed with this supplemental response. Documents 3, 4, 5, and 6 above were submitted with the initial submittal of the Yucca Mountain License Application.

SAR Section 1.1.5 provides a historical description of the geotechnical drilling program which was conducted in a number of phases over several years. The SAR summarizes the site investigations and the results from the above documents. Recent geologic and geotechnical data collected at the site are being reviewed for impacts; however, to date the results reported in the SAR remain valid.

The response to RAI 2.2.1.1.1, Second Set, Number 1, will be submitted by March 30, 2009. The trace of the Bow Ridge Fault shown in SAR Figure 1.1-59 considers the data from the recent geotechnical drilling. The characterization of the Bow Ridge Fault and the associated fault displacement and vibratory ground motion hazards are addressed in the response to RAI 2.2.1.1.1, First Set, Number 10. The characterization of the other faults in Midway Valley will be provided in the March 30, 2009 submittal, along with six geologic cross sections in the vicinity of the surface facilities and the aging pads and an updated geologic map at appropriate scales.

2. COMMITMENTS TO NRC

None.

3. DESCRIPTION OF PROPOSED LA CHANGE

None.

4. REFERENCES

[1] BSC (Bechtel SAIC Company) 2002. *Soils Report for North Portal Area, Yucca Mountain Project*. 100-00C-WRP0-00100-000-000 ECN 1. Las Vegas, Nevada: Bechtel SAIC Company. ACC: MOL.20021015.0323; ENG.20050823.0018.

[2] Buesch, D.C. and Lung, R.C. 2008. *Update of Geotechnical Borehole and Test Pit Program through 2007 in Midway Valley, Nevada*. Denver, Colorado: U.S. Geological Survey. ACC: LLR.20080121.0080.

[3] BSC 2008. *Supplemental Soils Report*. 100-S0C-CY00-00100-000-00E CACN 001. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20080828.0016; ENG.20081117.0004.

[4] BSC 2002. *Geotechnical Data for a Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project*. ANL-MGR-GE-000003 REV 00. Las Vegas, Nevada: Bechtel SAIC Company. ACC: MOL.20021004.0078.

[5] SNL (Sandia National Laboratories) 2008. *Technical Report: Geotechnical Data for a Geologic Repository at Yucca Mountain, Nevada*. TDR-MGR-GE-000010 REV 00. Las Vegas, Nevada: Sandia National Laboratories. ACC: DOC.20080206.0001.

[6] BSC 2008. *Supplemental Earthquake Ground Motion Input for a Geologic Repository at Yucca Mountain, NV*. MDL-MGR-GS-000007 REV 00 ACN 02. Las Vegas, Nevada: Bechtel SAIC Company. ACC: DOC.20080221.0001; DOC.20080314.0001; DOC.20080328.0001.

RAI Volume 2, Chapter 2.1.1.1, First Set, Number 8, Supplementary Question: During a teleconference on February 23, 2009, the NRC Staff asked the DOE one supplementary question:

Explain how equations 1, 2, and 3, provided in the initial response, correspond with the text description of the calculations.

1. SUPPLEMENTAL RESPONSE

This supplemental response explains how equations 1, 2, and 3, provided in the initial response, correspond with the text description of the calculations.

Equation 1

Two approaches are used to condition ground motions with low mean annual probabilities of exceedance, resulting in a single, conditioned hazard curve for each ground motion measure (e.g., peak ground velocity (PGV), peak ground acceleration (PGA), 1-Hz spectral acceleration). Equation 1 is used to implement the extreme-stress-drop conditioning approach. This approach uses an assessment of the probability distribution for extreme stress drop to characterize the extreme ground motions that might occur at Yucca Mountain. As discussed in the initial response, Equation 1 takes the form:

$$P[x > x']^{C_{ESD}} = P[x_m > x' | \Delta\sigma_m] P[x > x'] \qquad \text{(Equation 1)}$$

in which $P[x > x']^{C_{ESD}}$ is the extreme-stress-drop conditioned hazard for the PSHA reference rock outcrop for ground motion x, $P[x_m > x' | \Delta\sigma_m]$ is the complementary cumulative distribution function for extreme ground motion x_m given extreme stress drop $\Delta\sigma_m$, and $P[x > x']$ is the unconditioned hazard for the PSHA reference rock outcrop.

The assumed extreme-stress-drop distribution is represented by six cases and Equation 1 is used to condition the unconditioned PSHA reference rock outcrop hazard for each case. The cases comprise combinations of three extreme stress drop values (150, 400, and 1100 bars) and two moment magnitudes (6.0 and 6.5). The three extreme stress drop values, with weights of 0.2, 0.6, and 0.2, respectively, are used to represent the lognormal distribution for extreme stress drop. The moment magnitude values are based on hazard deaggregation. For each combination of extreme stress drop and moment magnitude, ground motion modeling that incorporates aleatory variability results in a distribution of peak ground acceleration (PGA) that is characterized by its lognormal mean and a σ_{ln} of 0.15. Each of these distributions forms the basis for the complementary cumulative distribution function, $P[x_m > x' | \Delta\sigma_m]$ in Equation 1, that is used to condition the unconditioned hazard for the PSHA reference rock outcrop. This process results in six conditioned hazard curves. To produce the final conditioned hazard curve, the annual probabilities of exceedance for the six cases are averaged using weights of 0.2, 0.6, and 0.2, respectively for the extreme stress drop values and weights of 0.5 for each moment magnitude. The weighted averaging of the conditioned hazard curves for the six cases is not represented in Equation 1.

Equation 2

As described in the initial response, Equation 2 is used to implement the second component of the overall ground motion conditioning process. This approach conditions ground motion based on an analysis that establishes a level of ground motion that has not been experienced at Yucca Mountain. To develop ground motions for preclosure analyses, an initial implementation of this approach (see Equation 3 below) was updated to condition the entire response spectrum and incorporate aleatory variability. As provided in the initial response, the updated version of the shear-strain-threshold conditioning approach was given by Equation 2 and had the form:

$$P[x > x']^{C_{SST}} = \sum_i P[\gamma_m > \gamma \mid x_i] P[x_i] \qquad \text{(Equation 2)}$$

Note that while ground motion conditioning was implemented as intended based on the calculated distribution of cyclic shear strain (γ), Equation 2 was inadvertently composed such that the first term within the summation incorrectly indicates that the distribution is on shear-strain threshold (γ_m).

A corrected version of the equation that reflects the intent and actual implementation of shear-strain-threshold conditioning is provided here:

$$P[x > x']^{C_{SST}} = \sum_i P[\gamma < \gamma_m \mid x_i] P[x_i] \qquad \text{(Equation 2a)}$$

in which $P[x > x']^{C_{SST}}$ is the shear-strain conditioned ground motion hazard at the PSHA reference rock outcrop for ground motion x, $P[\gamma < \gamma_m \mid x_i]$ is the cumulative distribution function for cyclic shear strains at the repository waste emplacement level (γ) given ground motion x_i at the PSHA reference rock outcrop with γ_m the non-exceeded shear strain threshold value, and $P[x_i]$ is the probability of ground motion x_i at the PSHA reference rock outcrop. The correction reverses the inequality in the first term within the summation to make it clear that the distribution is for cyclic shear strain, not shear strain threshold. The summation is over the range (i) of ground motion values defining the hazard curve. Summation takes place specifically from the annual probability of exceedance for x' to the annual probability of exceedance for the largest value of x for which the hazard curve is defined.

In Equation 2a, the cumulative distribution function is defined in terms of strain (γ) conditional on ground motion level (x_i) because no widespread seismic-related damage to the lithophysal units has been observed during site investigations, indicating a lack of exceedance of a shear strain threshold. Ground motion at the PSHA reference rock outcrop results in a distribution of shear strain at the repository waste emplacement level because of aleatory variability in the properties of site materials. For a given ground motion level, x_i, the distribution of shear strain at the repository waste emplacement level is characterized by its lognormal mean and $\sigma_{\ln}$, which form the basis for the cumulative distribution function used in Equation 2a.

Similar to Equation 1, Equation 2a does not include the weighted averaging of conditioned hazard curves for the cases used to represent the shear-strain threshold distribution. The uniform distribution used to characterize the shear-strain threshold in the updated shear-strain-threshold approach used for preclosure ground motions is represented by two values: 0.09% and 0.25%. The conditioning also takes into account epistemic uncertainty in tuff dynamic material properties and deterministic variability in seismic velocity for the repository block. Equation 2a is implemented for each combination of shear-strain threshold (0.09%, 0.25%), velocity profile ("stiff," "soft"), and dynamic material property curve set ("upper mean tuff," "lower mean tuff") resulting in eight cases. Shear-strain threshold values and velocity profiles are weighted equally. The upper mean tuff and lower mean tuff results are weighted 0.7 and 0.3, respectively. Equation 2a is used to determine a conditioned hazard curve for each of the eight combinations of shear-strain threshold, velocity profile, and dynamic material property curve set. These conditioned hazard curves are then averaged using appropriate weights to produce the final shear-strain-threshold conditioned hazard curve. The weighted averaging of the conditioned hazard curves for the eight cases is not represented in Equation 2a.

Equation 3

As discussed in the initial response, the original implementation of the shear-strain-threshold conditioning approach used for postclosure analyses differs from the updated implementation used for preclosure analyses. In the original implementation, the conditioning was applied to the hazard curve for horizontal PGV at the repository waste emplacement level rather than at the PSHA reference rock outcrop. As provided in the initial response, Equation 3 is used to implement the original version of shear-strain-threshold conditioning. It takes the form:

$$P[y > y']^C = P[y_m > y \mid \gamma_m]P[y > y'] \qquad \text{(Equation 3)}$$

in which y is horizontal PGV at the repository waste emplacement level (taken as equivalent to PGV at the nominal depth of the Tptpll), $P[y > y']^C$ is conditioned horizontal PGV hazard, $P[y_m > y \mid \gamma_m]$ is the complementary cumulative distribution function for the probability that the bounding horizontal PGV value is greater than y, given the shear-strain-threshold distribution γ_m, and $P[y > y']$ is the unconditioned horizontal PGV hazard at the repository waste emplacement level. In preparing this supplemental response, it was noted that the prime (′) on y in the complementary cumulative distribution function term was inadvertently omitted in the initial response. The corrected equation is:

$$P[y > y']^C = P[y_m > y' \mid \gamma_m]P[y > y'] \qquad \text{(Equation 3a)}$$

Cases representing a high and low response frequency range and epistemic uncertainty in velocity profile (Profile 1, Profile 2) and dynamic material property curve set ("upper mean tuff," "lower mean tuff") are evaluated separately using Equation 3a. This results in eight conditioned hazard curves whose annual probabilities of exceedance are then averaged with equal weight to produce the final conditioned hazard curve. The averaging of the conditioned hazard curves for the eight cases is not represented in Equation 3a.

2. COMMITMENTS TO NRC

None.

3. DESCRIPTION OF PROPOSED LA CHANGE

None.