REVIEW COMMENTS ON ESF ALCOVE GROUND SUPPORT ANALYSIS AND ESF GROUND SUPPORT—STRUCTURAL STEEL ANALYSIS

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ABSTRACT

A technical review was conducted of the Exploratory Studies Facility (ESF) Alcove Ground Support Analysis (BABEE0000-01717-0200-00001 REV 01C) and ESF Ground Support—Structural Steel Analysis (BABEE0000-01717-0200-00003 REV 00B). The objective of this review is to assess the ability of the support design analysis to meet the ultimate goal of satisfying design and performance objective requirements of 10 CFR Part 60. Concerns raised in this review include

- Numerical analysis to determine the stability of ESF drifts and Alcoves 2, 3, and 4 is based on continuum modeling and does not consider the effect of existing joint sets in the rock mass. The extensive rock mass damage predicted by this analysis under *in situ* and seismic loads is expected to increase if a discontinuum analysis is carried out. Thus, the analysis results presented in these reports may not be representative of the actual conditions.

- Duration of the input shear wave used in dynamic analysis for the stability of ESF drifts and alcoves is unrealistically low and therefore non-conservative.

The ESF construction is nearing completion and its performance will be observed during the next several years of site characterization and testing. Some of the concerns raised in this review can be used by the Nuclear Regulatory Commission to check if the repository design currently being performed by the Department of Energy is acceptable to the staff.

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1 INTRODUCTION

The U.S. Department of Energy (DOE) Title II design package of Exploratory Studies Facility (ESF) Main Drift (Design Package 8A) consists of a series of design analysis documents. These design analysis documents are released by DOE in installments. The design analysis reports reviewed herein include (i) ESF Alcove Ground Support Analysis (BABEE0000-01717-0200-00001 REV 01C) and (ii) ESF Ground Support—Structural Steel Analysis (BABEE0000-01717-0200-00003 REV 00B). In addition to these two design analysis reports, three supporting reports were also reviewed: (i) Drift Design Methodology and Preliminary Application for the Yucca Mountain Site Characterization Project (Hardy and Bauer, 1991); (ii) Fracture Analysis and Rock Quality Designation Estimation for the Yucca Mountain Site Characterization Project (Lin et al., 1993); and (iii) Geotechnical Characterization of the North Ramp of the Exploratory Studies Facility, Volume I of II: Data Summary (Brechtel et al., 1995). Reviews of these three supporting reports are summarized in appendix A.

ESF Alcove Ground Support Analysis is a part of the ESF Title II Main Drift design (Design Package 8A). This report deals with the analysis of the stability of Bow Ridge test alcove (Alcove 2), and two Radial Borehole Tests alcoves (Alcoves 3 and 4) using empirical and analytical methods. Both FLAC3D and 3DEC codes were used in the analysis. No support system was considered in the models analyzed by FLAC3D and 3DEC codes. It was concluded that the effect of 80 and 100 kW/acre of thermal loads would not be significant due to the location of the alcoves relative to the repository horizon. Analysis carried out with *in situ* and seismic loads did not include joints in the rock mass. Significant damage (both tensile and shear failures) was calculated to occur in the roof, floor, sidewalls of the excavations, and in the pillar between the alcoves and the North Ramp. The report on ESF Ground Support—Structural Steel Analysis deals with the analysis, design, and selection of structural steel ground support members and components. The computer program STAAD-III was used for the analysis and American Institute of Steel Construction Specifications (American Institute of Steel Construction, 1989) were used for the design of steel members and components.

The findings of the reviews of two design analysis reports have been documented as general concerns. These concerns are presented in a standard format consistent with previous Nuclear Regulatory Commission (NRC) submittals of concerns provided to the DOE (e.g., Nuclear Regulatory Commission, 1989). This standard format includes objections, comments, and questions.

2 GENERAL CONCERNS

2.1 OBJECTION

There is no objection based on review of the reports on ESF Alcove Ground Support Analysis, and ESF Ground Support—Structural Steel Analysis.

2.2 COMMENTS

Comment 1

Numerical analysis to determine the stability of ESF drifts and Alcoves 2, 3, and 4 in ESF Alcove Ground Support Analysis may not be representative of the actual conditions.

Basis

- Results of 3DEC analysis for Alcoves 3 and 4 presented in figures 43 through 47, 50 through 54, 56 through 61, 64 through 69, and 72 through 110 show significant damaged or failed zones in the roof, floor, sidewall of the excavations, and in the pillar between the North Ramp and the alcoves. The failure from *in situ* stress field is shown in figures 43 through 47, 50 through 54, 56 through 61, 64 through 69, and 72 through 77. Figures 78 through 110 show failure when a shear wave in the form of a sine wave with duration 0.5 s was applied to the model in addition to the *in situ* stress field. In many cases, the pillars between the North Ramp and the alcoves show almost total damage (figures 60, 68, 76, 97, 103, and 109). The analysis presented in this design document for Alcoves 2, 3, and 4 does not include the intended support system. This is also inconsistent with the methodology for ground support analysis developed by Hardy and Bauer (1991 section 8.2). Hardy and Bauer (1991) proposed that uncoupled analysis, as carried out in this design document, is useful "when the ground support system is not expected to modify the deformation response of the rock mass or significantly modify the rock mass strength." They recommended coupled analysis if significant rock yield is expected. Therefore, a coupled analysis should have been carried out to estimate the effectiveness of the support system to control the yielding of the rock mass surrounding the drifts.

- Lin et al. (1993) identified four joint sets in the Tiva Canyon tuff and three joint sets in the Topopah Spring (TS) tuff. Alcove 2 (Bow Ridge fault test alcove) is located in the Tiva Canyon member. Alcove 3 is located near the TCw/PTn (Tiva Canyon welded unit and Paintbrush tuff nonwelded unit) contact. Alcove 4 will be located near the PTn/TSw1 (Paintbrush tuff nonwelded unit and TS tuff welded unit 1) contact. The analysis using 3DEC (3-dimensional Distinct Element Code) and FLAC3D (Fast Lagrangian Analysis of Continua in 3-dimension) does not take into account the effects of these joint sets in the model. The material properties used in the analysis are equivalent material properties estimated from the rock mass classification scheme. Due to the associated averaging at every step of the rock classification process, these estimated values are gross average values for a typical rock mass class without any quantification of the associated uncertainty. Moreover, use of these property values has an inherent assumption—the rock mass is homogeneous and isotropic. Consequently, the very nature of the rock mass as a discontinuum is lost, and the analysis

does not take into account the anisotropy introduced by the prominent joint sets, as identified by Lin et al. (1993). At best, the approach in this report may be considered as an upper-bound of the stability analysis. An analysis with explicit modeling of the joint sets is warranted since the prominent joint sets will control stress distribution and the deformation field. Due to the stringent requirement of 100 yr maintainable life, the analysis needs to use reasonable site parameters to realistically estimate failure scenarios, especially in these cases where the stress distribution is complex, such as at the intersection of two underground excavations inclined at different orientations. Surprisingly, the 3DEC code used in this report for continuum simulation has been developed for this purpose.

- The continuum assumption has a significant impact on stability analysis of the alcoves and the intersections of alcoves and the ESF North Ramp. This assumption only allows bulk failure of the medium and ignores the instability of excavations created by slippage along weakness planes, such as joints. The significance of the impact of this assumption increases when seismic effects on excavation stability are considered (Section 7.6.2.2 in this document). Owen and Scholl (1981) and St. John and Zahrah (1987) considered three modes of damage: fault slip, rock mass failure, and shaking. Excavation damage from shaking is most common and is expressed as slip on joints and fractures with displacement of joint-defined rock blocks. As the analysis presented in this report did not consider the existing joints as discrete entities, the most prevalent form of excavation failure was ignored. Moreover, it has been observed that the joints already stressed close to their limiting strengths are more susceptible to failure (St. John and Zahrah, 1987; Hsiung et al., 1992a,b). Results of a 3DEC analysis for Alcoves 3 and 4 show significant damaged or failed regions in the roof, floor, sidewall of the excavations, and in the pillar between the North Ramp and the alcoves. Consequently, the rock mass modeled in this analysis, especially the immediate roof, floor, sidewall, and the pillar adjoining the North Ramp and the alcoves, will be more vulnerable to failure by seismic waves from earthquakes than that predicted in this continuum-based analysis.

- Bow Ridge fault test alcove support design was carried out assuming Category 3 rock mass quality is best representative of the TCw thermo-mechanical (TM) unit at the Alcove 2 location (Section 4.1 Design Parameters). This is contrary to the definition of rock mass quality class proposed by Hardy and Bauer (1991, pages 5-7). According to their definition, Category 3 implies that 40 percent of the rock mass will have lower quality defined in terms of Q value. It must be realized that the five points associated with five category values in combination describe a curve of variations in rock mass quality. In other words, all five categories are present in the same rock mass and no rock mass is best represented by any one category value. Hardy and Bauer (1991) recommend stability analysis for a given drift should be carried out at all five rock mass quality categories so that the requirement for support systems can be evaluated for a wide variety of rock conditions likely to be encountered. The support system that will be installed ultimately will depend on (i) the level of conservatism that is necessary given the importance and use of the drift, economics, and safety to workers; (ii) opportunities to do maintenance and; (iii) the degree of rock fall or instability that can be tolerated. Stability analysis at all five rock mass quality categories is necessary to bound the support requirement, as suggested by Hardy and Bauer (1991).

- The conclusion made in section 7.6.3 of the document that the frequency of input harmonic shear load has no impact on the stability of openings may not be defensible because the analysis did not include joints.

Recommendations

- In a fractured rock mass, response of the joints will be the controlling factor for stability, both under *in situ* and seismic loads, and should be taken into account in the design analysis. The program used in this analysis (3DEC) is capable of simulating such behavior.

- The design analysis should be carried out for all five rock mass quality categories, as suggested by Hardy and Bauer (1991), to bound the requirements of ground support system whose effectiveness has been established through design analysis.

- As recommended by Hardy and Bauer (1991), coupled analysis using the intended support system should be carried out to establish effectiveness of the support system to meet performance objectives.

Comment 2

Parameters used to define the shear wave in dynamic analysis for the stability for ESF drifts and alcoves are not realistic.

Basis

- This comment is the continuation of the comment made on the parameter values used to define the seismic wave in the analysis presented in Package 2C. Duration and frequency of a seismic event are important parameters affecting stability of underground openings. Such openings are likely to suffer more damage under longer duration shaking or if subjected to repeated episodes of shaking (Hsiung et al., 1992a,b; Kana et al., 1995). St. John and Zahrah (1985) comment that the number of excursions into the nonlinear or failed range experienced by an underground excavation and the surrounding media will control the extent of permanent damage.

- In this analysis of ESF drifts and alcoves and the intersection of the alcoves with the drifts subjected to a dynamic load, the input signal was a sinusoidal wave with a duration of 0.5 s and a frequency of 5, 10, or 20 Hz. The use of such a short-duration harmonic wave for seismic analysis does not appear to be representative of the duration of seismic waves to be expected at Yucca Mountain (YM). It is recognized that the duration and frequency of seismic events vary. However, the duration of 0.5 s used in the analysis seems to be too short compared to the actual observations of earthquakes in the vicinity of YM (Walter, 1993) and may not be conservative.

- The calculated response based on a simulated earthquake wave form input will be different from that using a harmonic input.

Recommendation

A realistic seismic signal with longer duration should be considered in the analysis. Consideration should also be given to using representative seismic motions from the particular region.

2.3 QUESTION

There is no question based on the review of the reports on ESF Alcove Ground Support Analysis and ESF Ground Support—Structural Steel Analysis.

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APPENDICES

REVIEW OF

APPENDIX A DRIFT DESIGN METHODOLOGY AND PRELIMINARY APPLICATION FOR THE YUCCA MOUNTAIN SITE CHARACTERIZATION PROJECT
M.P. Hardy and S.J. Bauer
SAND89-0837, Sandia National Laboratories, Albuquerque, New Mexico, 1991

APPENDIX B FRACTURE ANALYSIS AND ROCK QUALITY DESIGNATION ESTIMATION FOR THE YUCCA MOUNTAIN SITE CHARACTERIZATION PROJECT
M. Lin, M.P. Hardy, and S.J. Bauer
SAND92-0449, Sandia National Laboratories, Albuquerque, New Mexico, 1993

APPENDIX C GEOTECHNICAL CHARACTERIZATION OF THE NORTH RAMP OF THE EXPLORATORY STUDIES FACILITY, VOLUME I OF II: DATA SUMMARY
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SAND95-0488/1, Sandia National Laboratories, Albuquerque, New Mexico, 1995

REVIEW SUMMARY

A technical review was conducted of three reports prepared by Sandia National Laboratories for the Yucca Mountain (YM) Site Characterization Project. These reports are (i) Drift Design Methodology and Preliminary Application for the Yucca Mountain Site Characterization Project (SAND89-0837), (ii) Fracture Analysis and Rock Quality Designation Estimation for the Yucca Mountain Site Characterization Project (SAND92-0449), and (iii) Geotechnical Characterization of the North Ramp of the Exploratory Studies Facility, Volume I of II: Data Summary (SAND95-0488/1). The objectives were to (i) conduct an in-depth review regarding the technical soundness of the methodologies adopted by the Department of Energy (DOE) and their influence on design of the Exploratory Studies Facility (ESF) and the proposed geological repository and (ii) familiarize the Nuclear Regulatory Commission (NRC) and the Center for Nuclear Waste Regulatory Analyses (CNWRA) staffs with the technical approaches adopted by DOE. This appendix presents a summary of each report together with a brief discussion of any potential concerns.

APPENDIX A

DRIFT DESIGN METHODOLOGY AND PRELIMINARY APPLICATION FOR THE YUCCA MOUNTAIN SITE CHARACTERIZATION PROJECT

DRIFT DESIGN METHODOLOGY AND PRELIMINARY APPLICATION FOR THE YUCCA MOUNTAIN SITE CHARACTERIZATION PROJECT

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SUMMARY

This report by Hardy and Bauer (1991) outlines a methodology for the proposed high-level nuclear waste repository at Yucca Mountain (YM) to design drifts at the repository level, including main access and emplacement drifts excavated for waste disposal. The methodology has two parts: a determination phase and a two-step design phase that includes preliminary drift design followed by a detailed drift and support design. In the determination phase, functional requirements of the drifts are defined and criteria and goals for performance are established. A range of appropriate drift sizes and shapes is selected based on functional requirements such as the ability to accommodate mining and waste emplacement equipment, ventilation, and other auxiliary systems. Only general characteristics of the rock mass at the proposed repository horizon are available at this stage.

The proposed method defines variations in rock mass quality by specifying Q or rock mass rating (RMR) values at five different percentage levels. These levels represent the percentage of rock mass better than the specified Q or RMR values. Five percentage levels (90, 70, 40, 20, and 5) have been defined. It must be realized that these five points in combination give the quality of the rock mass. Any single point gives only the percentage of rock better than the particular Q or RMR values. This is similar to specifying median Q or RMR values, in which case 50 percent of the rock mass has better Q or RMR values. Hardy and Bauer (1991) named these percentage levels as Categories 1 through 5, where Category 1 represents 5 percent of rock mass having larger Q or RMR values than the particular value given. Consequently, a particular rock mass will have all five categories representing variation of quality.

Hardy and Bauer (1991) recommend that during the preliminary drift design phase, simplified analysis using heuristic, empirical, and numerical methods be performed to assess the effects of excavation, seismic, and thermal loads. The analysis does not explicitly take into account the ground support to assess whether the drifts are stable with minimum ground support under the expected rock mass conditions and loads. Design of the drifts is then evaluated against performance goals to determine practicality and feasibility of the design. In the detailed analysis phase, performance of the drifts is evaluated in conjunction with the ground support systems using coupled numerical analysis of rock-support interaction and also the effects of seismic and thermal loads. Several alternative rock support designs need to be developed to accommodate the complete range of rock conditions. Temperature of the drift during retrieval, opening stability, and materials that can be used inside the drift dictate the performance goals of drift design. Hardy and Bauer (1991) stated that potential geochemical interactions, corrosion, and usable life should be considered in identifying the materials to be used in the drifts. For example, Portland cement based grouts and concrete may have an adverse effect on water pH and could possibly reduce the waste package containment period (Hardy and Bauer, 1991). During the preclosure period, the major factors affecting worker safety are opening stability and ventilation. Major traffic areas should be designed and supported in such a way to require no major maintenance (i.e., drift to be taken out of service for an extended period of time for maintenance and rehabilitation) throughout the operational life of the proposed repository. Postclosure concerns are the potential for deleterious rock movement and

creation of preferential pathways in the host rock mass affecting waste isolation. Materials used in the drift may have potential for geochemical interactions with the host rock mass.

Hardy and Bauer (1991) stated that stresses from three sources need to be considered both individually and in combination: *in situ*, thermal, and seismic. Empirical methods are used to evaluate the stability of drifts and the requirements of ground support given the range of rock characteristics. Both Q and RMR systems for rock mass classifications are recommended to determine stand-up time (less than one day unsuitable for construction), define range of ground supports, and determine probabilities of encountering each rock category, rock mass strength, and maximum unsupported span. The charts of Hoek (1981) and Schmidt (1987) can be used to identify possible failure modes. Numerical analysis may be carried out to understand the behavior of the rock mass with simple material models and to study alternate drift shapes (Hardy and Bauer, 1991). Effect of thermal load will be incorporated in the model through the coefficient of thermal expansion and possibly temperature dependent material properties. Effect of seismic load will be taken in a pseudo-static way. Generally, the proposed ground support will be added in the model for poorer quality ground after stability has been assessed. On the basis of rock joint characteristics, the specific type of modeling [i.e., equivalent-continuum with elasto-plastic modeling, distinct joint modeling, equivalent-continuum with ubiquitous joint modeling, and discontinuum (discrete element) modeling] will be selected. Numerical analysis will be carried out at critical locations throughout the proposed repository. Comparison of analysis results with requirements and criteria may dictate modification of drift geometry and/or thermal load.

Both uncoupled and coupled analyses have been recommended by Hardy and Bauer (19[illegible]) for ground support analysis. Uncoupled analysis should be sufficient if the ground support system is not expected to significantly modify rock mass response. If significant yielding of the rock mass is expected, coupled analysis is recommended. Final design should be evaluated against all performance goals. The influence of backfill on long-term stability should also be evaluated. The drift design needs to be verified by monitoring during Exploratory Studies Facility (ESF) construction, confirmation testing, and repository construction. An application of the proposed drift design methodology is presented in this report for the Topopah Spring welded unit, lithophysae-poor (TSw2) using preliminary geomechanics data from the site.

CONCERNS

The technical review did not generate any specific comments. However, there are some concerns regarding the application of five rock mass categories at the YM site.

- The word "category" may be a poor choice of word to represent the variation of a particular property value and the overall "quality" of a rock mass. It might have added confusion in ESF Design Packages 2C and 8A by erroneously equating it with the rock mass quality categories in the Q and RMR systems when the meaning of the word was taken in the traditional sense. In the method proposed by Hardy and Bauer (1991), any single category is no better than another in representing the quality of a rock mass. A particular rock mass will have all five categories representing the variation of its quality.

- The methodology proposed by Hardy and Bauer (1991) to take into account the variation of rock mass quality over an extended region is acceptable. The selection of the support system is still carried out using the 38 categories support system recommended by Barton et al., (1974). The Q value for selecting the particular support category is based on what rock quality level the support system of the excavation will be designed. For example, if the excavation is supported at 60 percent

passing level, it is expected that in the rock mass quality will be such that the designed support system will be adequate for 60 percent of the cases. For the remaining 40 percent, the rock quality will be poorer requiring some enhancement of the support system.

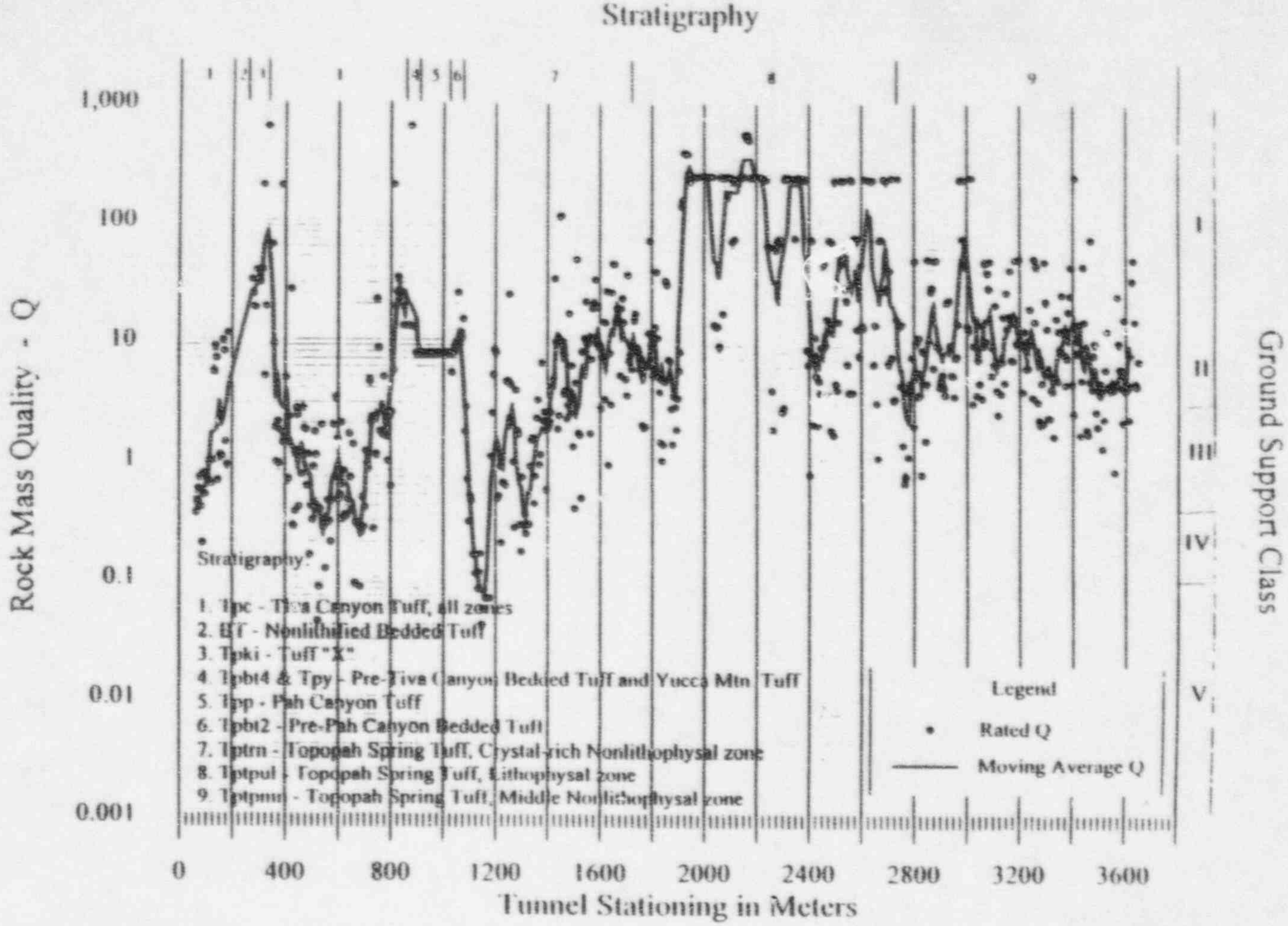
In ESF Design Package 2C and ESF Alcove Ground Support Analysis (Package 8A), the rock mass classification was carried out for each thermo-mechanical (TM) unit by considering only the observed variation of the parameters necessary to estimate Q of that unit. Consequently, rock mass quality of each category varies from one TM unit to another. For example, Category 1 rock masses (5 percent of rock mass has a higher Q value than this Q value) in Tiva Canyon welded unit (TCw), Upper Paintbrush nonwelded unit (PTn), Topopah Spring welded unit, lithophysae-rich (TSw1), and TSw2 units are 0.38, 0.15, 0.24, and 0.30 respectively (Brechtel et al., 1995). Similarly, rock mass quality Category 5 has corresponding Q values in TCw, PTn, TSw1, and TSw2 units equal to 9.14, 3.74, 12.00, and 8.44 respectively. As a result, the meaning of a particular rock mass quality category as defined by Hardy and Bauer (1991) varies from one TM unit to another with different support requirements.

In a recent presentation, Department of Energy (DOE) correlated five categories of ground support (Ground Support Classes I through V) directly with observed Q values rather than the particular Q value of a TM unit so that 60 percent of the rock mass at each TM unit will have higher Q, as done in TS North Ramp Ground Support Scoping Analysis. Ground Support Classes and corresponding values of Q are given in table A-1.

Table A-1. Ground support classes and corresponding values of Q (adopted from Williams, 1996)

Ground Support Class	Q Value	Support Category of Barton et al. (1974)
I	>10	13 to No Support Required
II	3 to 10	17 and 22
III	0.4 to 3	22 and 27
IV	0.1 to 0.4	31
V	<0.1	34 and 38

Figure 1 from Williams (1996) illustrates the relationship along with the measured Q values and moving average. In this figure, different TM units are shown for identification purposes only along the ESF tunnel. This methodology is acceptable and causes less confusion than the previous methodology which created different meanings of rock mass quality in different TM units.



PRELIMINARY

Comparison of Moving Average and Rated Q Values in the North Ramp

Figure 1. Variation of Q values along North Ramp and associated support categories (from Williams, 1996).

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APPENDIX B

FRACTURE ANALYSIS AND ROCK QUALITY DESIGNATION ESTIMATION FOR THE YUCCA MOUNTAIN SITE CHARACTERIZATION PROJECT

FRACTURE ANALYSIS AND ROCK QUALITY DESIGNATION ESTIMATION FOR THE YUCCA MOUNTAIN SITE CHARACTERIZATION PROJECT

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SAND92-0449, Sandia National Laboratories, Albuquerque, New Mexico, 1993

SUMMARY

This report estimates the linear and volumetric fracture frequencies and Rock Quality Designation (RQD) of six thermo-mechanical units of the proposed repository site at Yucca Mountain. These thermal mechanical units are the Tiva Canyon welded unit (TCw); Upper Paintbrush nonwelded unit (PTn); Topopah Spring welded unit, lithophysae-rich (TSw1); Topopah Spring welded unit, lithophysae-poor (TSw2); Topopah Spring welded unit, vitrophyre (TSw3); and Calico Hills and Lower Paintbrush nonwelded unit (CHn1). Orientations of the major joint sets have been estimated only for Tiva Canyon and TS members of the geologic stratigraphy. All analyses are based on existing data from boreholes USW G-1, USW G-3, USW G-4, and UE-25a#1. Surface outcrop data of the TCw member in the vicinity of drill hole USW G-4 and drill hole data were used for estimating fracture roughness.

Average linear fracture frequency (m^{-1}) for each thermal-mechanical unit is as follows: 4.1 for TCw unit, 1.0 for PTn unit, 1.7 for TSw1 unit, 3.0 for TSw2 unit, 2.3 for TSw3 unit, and 0.2 for CHn1 unit. Similarly, the volumetric fracture frequency (m^{-3}) is 20.01 for TCw unit, 9.44 for TSw1 unit, 19.64 for TSw2 unit, 14.34 for TSw3 unit, and 1.6 for CHn1 unit. RQD was not measured directly from the core runs, but was estimated from the available information on Core Index (CI) and number of joints in the cored interval. Orientations of the identified fracture sets are given in table B-1. Average Joint Roughness Coefficient (JRC) at the outcrops of the TCw member was 6.3 with a standard deviation of 3.3. Using drill hole data, JRC ranged from 6 to 12 in welded units and 2 to 8 in nonwelded units. As observed in drill hole UE-25a#1, 15 percent of the fractures in TCw and PTn units contained calcite and 12 percent of the fractures contained clay. In drill hole UE-25a#1, approximately 20 percent of the fractures in the TS member contained calcite.

Table B-1. Orientations of joint sets estimated from oriented core and borehole television surveys

Geologic Member	USW GU-3		USW G-4	
	Strike	Dip	Strike	Dip
Tiva Canyon Member	N18°W-N36°E	85°-90°SW/NE	N-N22°E	65°-90°NW
	N50°W	12°NE	—	—
	—	—	E-W	70°-90°N/S
	—	—	N50°W	70°-90°NE/SW
Topopah Spring Member	N10°W	75°-90°NE/SW	N12°W	80°-90°NE/SW
	N25°E	10°SE	—	—
	N45°E	80°-90°SE/NW	N-N40°E	Not Measured

CONCERN

In-depth review of this document did not reveal any specific comment although there is one general concern regarding the estimated values of RQD. In the absence of measurements from actual cores, RQD was estimated indirectly from CI and the number of joints in the cored interval. It has been observed that the estimated RQD in some core runs is more than 100 percent (appendix C of the report). Errors in the reported values of CI and number of joints may have caused this problem. From the definition of CI, the maximum value of CI can be larger than 100. As pointed out by the authors, 100 is the maximum reported CI value. If any CI value more than 100 was recorded as 100 then it is impossible to estimate the correct JRC value. The Department of Energy recognized this concern with the data and observed that these core logs did not generate data consistent with the requirements for rock mass quality estimate. Consequently, in Exploratory Studies Facility Design Package 2C data from only North Ramp Geotechnical (NRG) holes were used (Civilian Radioactive Waste Management System Management & Operating Contractor, 1994a,b,c,d).

REFERENCES

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APPENDIX C

GEOTECHNICAL CHARACTERIZATION OF THE NORTH RAMP OF THE EXPLORATORY STUDIES FACILITY, VOLUME I OF II: DATA SUMMARY

GEOTECHNICAL CHARACTERIZATION OF THE NORTH RAMP OF THE EXPLORATORY STUDIES FACILITY, VOLUME I OF II: DATA SUMMARY

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SUMMARY

This report presents the results of geological and geotechnical characterization of 11 boreholes drilled along the 2,800 m route of the North Ramp of the Exploratory Studies Facility. Rock property data collected to support the subsurface design included (i) lithologic and rock structural core logs, (ii) cross-sections with stratigraphic and thermal mechanical (TM) units, and (iii) rock mass quality indices. These rock structural and rock mass quality data are summarized in 3 m intervals and grouped according to the TM units.

CONCERNS

No major concerns have been raised during the in-depth technical review that are believed to result in the North Ramp design and construction having an adverse impact on repository performance as specified by 10 CFR Part 60. However, the discussions on the selection of Excavation Support Ratio (ESR) and on rock mass quality categories presented in the review summaries of the previous two reports also apply to this report. Several specific concerns have been generated during in-depth review of the subject report.

- In section 2.4, information is provided on the orientations and dip angles of the 2-3 dominant joint sets, but little is mentioned regarding joint spacings, particularly for the subhorizontal joints/foliation planes, in establishing appropriate rock support.

- In section 5.2, the number of coring-induced fractures represents a significant proportion of the total fractures mapped in the various TM units. For instance, figure 5-7 shows that the drilling induced fractures represent 70, 73, and 62 percent of the total number of fractures identified in the Upper Paintbrush nonwelded unit (PTn), Topopah Spring welded unit lithophysae-rich (TSw1), and Topopah Spring welded unit, lithophysae-poor (TSw2) units respectively. Section 5.2 states that it appears that the compressed-air coring technique is not responsible for the large proportion of lost core and rubble observed in the welded tuffs. It is likely that dry drilling using compressed air results in larger frictional stresses and higher temperatures at the bit-rock interface than would be generated using conventional water drilling, and possibly results in more frequent wedging of the bit. The higher temperatures could create localized TM stresses that might aid in generating coring-induced fractures, especially along weak horizontal foliations identified in the welded tuff. If this is true, then the samples obtained for rock mechanics and other testing might not contain such *in situ* foliation and subsequent test results may not fully represent the rock mechanics properties, since the weaker units would have fractured. It is recommended that a more thorough evaluation of the impact of dry versus wet drilling be done before stating that such dry drilling has no greater effect on core breakage.

- As reported in section 6.1 and again in section 6.7, all rock mechanics testing is accomplished on fully saturated samples. This is done by pressurizing to 10 MPa for one hour, followed by repeated vacuum and resaturation cycles. The reason for this is to eliminate the variability associated with

partial saturation. However, the purpose of conducting the rock mechanics tests is to capture the full range of variability within each of the TM rock units based on the *in situ* conditions present for establishing the range of rock mass classifications and ground support categories. Fully saturating the samples creates conditions that are not representative of those in the field and defeats one of the purposes of conducting more laborious and expensive dry drilling of boreholes in the first place. Finally, the effect of applying a 10 MPa hydrostatic pressure during sample saturation is unclear, especially on those rock samples from units with moderate to low tensile and compressive strengths.

- In section 6.6, the unconfined compressive strengths determined during rock mechanics testing of intact specimens (2 in diameter) in each of the various TM units are considerably different from those determined from triaxial testing of 1 in diameter cores under zero confining pressure. From table 6-1, for example, the mean values for unconfined compressive strength based on all rock specimens tested from the Tiva Canyon welded unit (TCw), TSw1, and TSw2 TM units were 125.1, 56.9, and 178.2 MPa respectively. In contrast, best-fit linear regressions of the triaxial compressive strength data show axial compressive strengths at zero confining stress for the TCw, TSw1, and TSw2 units to be 288.8, 70.5, and 232.6 MPa respectively (table 6-9). Although there is considerable scatter in the triaxial compression test data, these values are considerably larger than those determined from the uniaxial compression tests. It is likely that some scale effects due to the different sample sizes are influencing the test results and need to be taken into account in the interpretation and use of the data, especially since both the uniaxial and triaxial test results (each based on different test sample sizes) are used in developing the overall rock mass mechanical properties. Future triaxial compression tests should, if possible, make use of larger (i.e., 2 in diameter) sample sizes.

- In section 7.2.6, the rock mass rating (RMR) ratings for the five rock mass quality categories based on cumulative frequencies of occurrence of 5, 20, 40, 70, and 90 percent, as listed in table 7-4, do not appear to be consistent with values read at the same cumulative frequencies for each of the TM units from the plots shown in figure 7-5. The RMR values read from figure 7-5 for cumulative frequencies of 5, 20, 40, 70, and 90 percent in many cases are either higher or lower than those RMR values tabulated in table 7-4. As a result, a larger range of RMR values exists for the individual TM units for cumulative frequencies between 5 and 90 percent, perhaps leading to a larger range of relative rock mass category ratings, as listed in table 7-4.