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Civilian Radioactive Waste Management System
Management & Operating Contractor

Waste Isolation Evaluation Comparing Drill and Blast with Mechanical Excavation Techniques

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CIVILIAN RADIOACTIVE WASTE MANAGEMENT SYSTEM
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This waste isolation evaluation was prepared as a quality-affecting activity according to CRWMS M&O QAP-3-1, QAP-3-5 and QAP-3-9. The activities evaluated here, and associated components, have not been assigned a QA classification.

TABLE OF CONTENTS

1. EXECUTIVE SUMMARY

This report presents the postclosure performance evaluation of the potential waste isolation impacts induced by various tunnel excavation techniques. The work reported here was conducted by the Civilian Radioactive Waste Management System (CRWMS) Management and Operating (M&O) Contractor under Work Breakdown Structure (WBS) 1.2.5.4.7 Supporting Calculations for Postclosure Performance Analyses. This Performance Assessment (PA) activity is a part of the "analyses and evaluations of potential impacts on waste isolation of the Exploratory Studies Facility (ESF) and surface-based design, construction, and testing," identified in working Draft 2 of "Plan for FY-1993 Performance Assessment Support," issued on October 20, 1992 by the M&O.

Because the block of rock mass through which the repository drift may be excavated may be used to contain radioactive material, the creation of fractures in the surrounding rock mass induced by the rock excavation processes should be minimized because such fracturing could potentially provide new pathways or reduce the travel time for the transport of radionuclides to the accessible environment through either aqueous or gaseous pathways. Thus, any increase in permeability as a result of created fractures is of potential concern when considering radionuclide containment in geological repositories. In this report, qualitative descriptions of the damage of rock mass around a drift are provided. They are used to assess the possible change of permeability in the rock mass immediately surrounding the drift (very near field).

In this report, the damage of rock mass and the change of the fracture characteristics caused by excavation using the drill and blast method and mechanical excavators (i.e., tunnel boring machine (TBM) or mobile miner) are qualitatively described. The mechanical excavators include tunnel boring machine (TBM) and mobile miner. For an 8-m diameter (about 26 ft) tunnel, the damage of the intact rock associated with the drill and blast method may occur within one or two meters of the excavation limit. Beyond the intact rock damaged zone, a zone of mechanically disturbed rock mass may exist. This disturbed rock mass is characterized by a change in fracture geometry, extent, and characteristics. The mechanically disturbed zone may be defined by a peak particle velocity of about 100 mm/s (about 4 in/s) generated by the excavation operation. Assuming this velocity cutoff, the extent of the mechanically disturbed zone associated with the drill and blast excavation method is about 10 m (about 33 ft) from the excavation limit. Although limited information exists on the rock mass behavior adjacent to mechanical excavators, there is some indication that mechanical excavation techniques induce less intact rock damage in the first two of meters and less rock mass disturbance beyond the damage zone.

The construction water usages of the drill and blast method and the TBM excavation method are also assessed in this report. There are 2.4 m^3 of construction water available to infiltrate the tunnel walls for each meter of tunnel (about 192 gallons per ft) excavated by TBM and 4.5 m^3 per meter (357 gallons per ft) for the drill and blast excavation. The mechanically disturbed zone around a drift excavated by the drill and blast method has significantly higher hydraulic conductivity and drainable porosity than the zone around a drift excavated by the TBM or mobile miner. Although the extent of the mechanically disturbed zone is small compared to the overburden or the underlying rock mass, ignoring the waste isolation capability of the mechanically disturbed zone is expected to have an insignificant effect on the overall waste

isolation capability of the entire repository. Because more construction water trapped in the mechanically disturbed zone created by the drill and blast excavation may change the local hydraulic regime, the waste isolation impacts of such change of hydraulic regime should be analyzed in the future.

2. QUALITY ASSURANCE

The techniques used to excavate the Exploratory Studies Facility may affect natural barriers of a potential repository at Yucca Mountain. These natural barriers are listed in Appendix A of the Q-list [DOE, 1993]. For this reason, this report was prepared as a quality-affecting activity according to CRWMS M&O Quality Administrative Procedure QAP-3-5 "Engineering Calculations and Analyses." No computer code calculations were specifically performed for this evaluation. Simple hand calculations were independently verified and documented according to QAP-3-5.

Some of the data and referenced analyses used in this evaluation may not have been approved for quality-affecting work. The extent and possible effects of non-qualified data and analyses on the evaluations, conclusions, and recommendations of this report have not been specifically determined. However, the conservative assumptions, estimates and methods used in this evaluation were devised to address any reasonable scenario and are believed to bound the potential impacts on waste isolation.

3. INTRODUCTION

The Yucca Mountain Site Characterization Project is studying Yucca Mountain in Nevada as a potential site for a high-level nuclear waste repository. Site characterization includes both surface-based and underground testing. Underground testing is to be facilitated by the construction of an Exploratory Studies Facility (ESF). The ESF Title II Design is currently being conducted by the M&O. The following dimensions for the ESF drift/ramp are described in the ESF Title I Design Summary Report [DOE 1992a]: a 7.6-m (about 25-ft) diameter is being considered for the north and south ESF ramps and the main ESF drift at the Topopah Spring level; a 5.5-m (about 18-ft) drift is being considered for the main drift at the Calico Hills level; and drifts 4.3 m (about 14 ft) high and 5.5 m (about 18 ft) wide are planned as lateral drifts at the Topopah Spring and Calico Hills levels. The M&O performance assessment (PA) team has been tasked to assess the potential waste isolation impacts caused by various excavation methods.

Because the block of rock mass through which the ESF may be excavated may be used to contain radioactive material, the possible creation of fractures in the surrounding rock mass induced by the rock excavation processes must be evaluated. Such fracturing could potentially provide pathways or reduce the travel time for the transport of radionuclides to the accessible environment along either gaseous or aqueous transport paths. Thus, any increase in permeability as a result of created fractures is of potential concern when considering radionuclide containment in geological repositories. One of the primary design goals of the repository is to control the increase in permeability that may be created by the excavation process. The design criterion for the increase in permeability should be based on an assessment of the impact on post-closure waste isolation. The impact on the waste isolation capability of the repository caused by the excavation process must be addressed through comprehensive performance assessment analyses.

Two main factors have to be considered when deciding the method to be used to construct a tunnel, the size and shape of the tunnel and the ground in which it is to be built. In this report, we evaluate the rock mass damage around an 8-m (about 26 ft) diameter tunnel to represent the possible damage associated with excavation of the ESF. In addition, the rock damage assessment in this report is prepared based on a generic ground condition of a highly fractured hard rock mass.

In this report, the damage of the rock mass and the change of the fracture characteristics caused by excavation using the drill and blast method and the mechanical excavators are qualitatively described. The damage of the intact rock and the change of fracture characteristics due to different excavation methods can be assessed. However, there is very little information to assess the change of permeability in the rock mass around an underground opening.

Fenix & Scisson [1986] reported that no scientific data are available to indicate measurable relationships between permeability and blast damage zones, or methods which will separate blast damage effects from the effects of stress relief which accompany all underground excavations. Hence, only a qualitative description of the impact on the rock mass permeability caused by different excavation methods is possible. The qualitative description of the impact is made through the comparison of the peak particle velocities induced by different excavation methods.

As presented in Sections 4 and 5, the peak particle velocity is a widely used parameter by researchers to analyze the rock mass damage caused by excavation or seismic process.

Drill and blast excavation methods have a tendency to create more loosening of the immediate zone that surrounds the excavation. The following discussion by Sinha [1989] describes his comparison of the disturbance caused by drill and blast method and tunnel boring machine methods:

> Methods of excavation such as drill and blast have a tendency to create more loosening of the immediate zone that surrounds the excavation. Drill and blast method creates more loosening and more rock loads than excavation by TBM (tunnel boring machines). The exact quantitative comparison of loosened zone between drill and blast and TBM methods is not possible because the amount of loosening during drill and blast is influenced by several factors such as powder factor, pattern of drilling, sequence of loading, use of delay system, type of explosive used, and characteristics of the rock. The loosened zone during the tunnel boring machine excavation depends on the thrust of the machine, and the type of cutter used, the rotational speed of the tunnel boring machine, and the characteristics of the rock. However, it could be stated that a drill and blast excavation will disturb a zone about three to six times the disturbed zone of a tunnel boring machine. Of course, the disturbed zone during blasting can be reduced by using controlled blasting techniques. A zone of rock which has received a peak particle velocity of more than 4 in/s (100 mm/s) during blasting should be considered to be disturbed. [page 38]

Most of discussion of this paper is focused on the rock damage caused by the drill and blast excavation.

At the present time, very little information about the potential drift sealing and backfill designs are available. The implication of alternative sealing and backfill designs on the waste isolation impact due to different excavation methods are not discussed in this report. In addition, the thermal implications of the impact of different excavation methods on waste isolation requires extensive thermomechanical analyses which are beyond the scope of the present analysis, and are therefore not discussed.

4. ROCK MASS DAMAGE FROM BLASTING

A possible disadvantage of tunnels excavated by drill and blast methods compared with excavation by machine is an increase in overbreak and greater inaccuracy in profiling. Reduction of overbreak, formation of a smooth wall profile and reduction of vibration can be achieved by using explosive cartridges with small diameter, low density and low volume strength. Because no detailed design of the drill and blast operation for excavating the ESF drifts is available, the rock mass damage assessment is performed based on the generalized drill and blast method.

The detonation of an explosive contained in a blasthole initiates several actions which can contribute to the breakage of rock. The gases generated by the explosion produce intense pressures that can reach 10,000 MPa (1.5 million psi) [Fenix & Scisson,1986]. The impact of the high pressure gases against the blasthole wall produces a compressive shock wave which travels radially outward into the rock mass at a velocity of 2000 to 4000 m/s (about 6,000 to 14,000 ft/s) [Fenix & Scisson,1986]. Outside the shattered zone, a zone of new radial fractures can be created, due to the large tangential tensile component of the stress field induced by the compressive shock wave and the reflected tensile shock wave. The most effective action produced by the explosion is not related directly to the shock wave but rather to the release of the expanding gases produced by the explosion. The high pressure gases wedge apart both the previously existing fractures and the newly created fractures, causing them to grow much larger than the rock stress conditions alone would dictate. In addition, the gas pressure moves the broken rock away from the blasthole, and thereby provides an adequate space for expansion of the rock during any subsequent detonations occurring in the same blast round. Therefore, the damage of the rock mass induced by blasting can be discussed in two separate zones. Within the first meter or two (about three to seven feet) from the detonation point, the rock mass is shattered. At a certain distance away from the detonation point, the intact rock is not damaged but the fractures in the rock mass may be affected. The effect of blasting on the fractures outside the shattered zone include fracture aperture change, new fracture formation, and fracture lengthening. The excavation of the mine opening removes material which previously supported the roof strata and creates an unstable condition where the stresses in the rock mass must be redistributed. Production blasts used to excavate the opening may damage the roof strata and thus contribute to cracking and spalling of the roof under the redistributed loads. Generally, several aspects of rock damage such as shock metamorphism, fracture patterns, grain-boundary effects, and physical property changes have been studied to evaluate the effect of blasting on rock mass.

The Bureau of Mines [1973] studied the diametric pulse velocity measurements along preblast and postblast cores to determine the nature and extent of rock damage produced by small (.25 to 2.0-kg) charges of high intensity explosive (C-4) in granite. This study was conducted to provide a better understanding of the nature and extent of rock damage caused by blasting. Pulse velocities from the postblast cores were significantly lower than velocities from the preblast cores and, together with examination of the core logs, provided clear identification of the upper and sometimes the lower limits of the most severe rock damage. These zones ranged from about 0.5 m (1.68 ft) for the 0.25-kg blast to about 1.3 m (4.16 ft) for the 2.0-kg blast. Microfracturing was found to be the principal kind of fabric damage in the portions of the blast-affected rock. The microfracture lengths did not exceed the longest mineral grain with which they were

associated and appeared to be distributed uniformly through the major minerals. Microfracture densities from postblast cores were significantly higher and more variable than densities measured from preblast cores. The microfracture density measurements showed that microscopic damage was present far past the zones of most severe damage as delineated by the pulse velocity logs and core logs. An exponential relationship was observed between the microfracture density levels and the charge weight. Mesofractures (longer fractures that traverse several adjoining grains and are generally visible to the unaided eye) were abundant in the fragmented pieces of core retrieved from the most severely damaged regions. Visible evidence of the extreme blast damage of the 2.0-kg blast was provided by a large crack that extended 4.3 m (about 14 ft) from the collar of the shothole to the edge of the outcrop.

The Bureau of Mines [1973] found that the maximum amplitudes of the high intensity elastic waves associated with routine production blasts were produced by the charge weight fired in the zero delay interval. Although some contribution owing to wave additions from the later delay periods were observed at the more remote gage stations, the high intensity portions of the waveforms of all the periods were separate events. Hence, a reduction of the zero delay charge weight could effectively reduce the maximum vibration amplitudes of the blast. The intense vibrations near the face were attenuated very rapidly as the waves traveled through the mine roof strata and away from the blast zone.

In his experimental studies of explosive effects on cement-sand blocks, Fosse [1968] demonstrated that the shock waves propagated directly into the solid from the explosive charge are attenuated very rapidly. The propagation velocity approaches the speed of sound after the wave travels only two to three charge diameters into the solid. The zone of shock effects thus roughly corresponds to the crushing zone (a few charge diameters wide) observed around the blast borehole. The Bureau of Mines [1970] measured crushed zones produced by small charges of C-4 in the Bellingham granite and found a linear relationship between the intensely damaged crushed zone volume and the charges. The equation $r = 0.12\,W^{1/3}$ predicts the radius of the crushed zone where is the radius of the crushed zone in meters and is the charge weight in kilograms. Although lower in amplitude than the shock waves, the high intensity elastic waves significantly influence rock deterioration by generating and extending cracks. The elastic waves also cause small-scale dislocations, and block slippage along major planes of weakness. The thrust of the borehole gases helps break the rock and also moves it away from the blast zone. Scott et al [1968] believed that the zone of rock affected by blast damage was a few feet thick contrasted with the thickness of the entire low velocity layer which often exceeded 3 m (about 10 ft) and reached a maximum value of 5 m (about 17 ft) at one location. Based on a review of case histories, Case and Kelsall [1987] suggest that the width of the blast damage may vary from approximately 0.3 m (about 1 ft) for cases in which controlled blasting methods (such as smooth blasting) are used to approximately 2.0 m (about 6.5 ft), for cases in which conventional blasting methods are used.

The ground motions observed at a given point are dependent upon the weight of explosive detonated per delay, the distance from the detonation point to the observation point, and the transmission characteristics of the rock mass. Because an acceptable theoretical approach has not

yet been developed for calculating ground motions in rock, the scaling of field measurements is used almost exclusively for predicting ground motions from explosions. To estimate the particle velocity induced by blasting, Lucole and Dowding [1979] provide equations for the components of ground vibration induced by blasting based on data compiled from many construction operations in the state of Illinois. The Bureau of Mines [1972] recorded ground vibrations from blasts used to excavate an exploratory drift at the North American Air Defense Command (NORAD) Complex, Cheyenne Mountain, Colorado. The data were used to develop an empirical equation for prediction of vibration velocities produced by various sizes of tunnel blasts in granite:

$$G = 560\left(\frac{D}{W^{1/3}}\right)^{-2.04} \tag{1}$$

where G is the ground vibration in inches per second, D is the distance from the explosive to the point of measurement in feet, and W is the weight of explosive in pounds. Lucole and Dowding's equation is similar to Equation (1). The granite rock in the NORAD site has an average Young's modulus of 32,000 MPa and an average compressive strength 120 MPa. The Young's modulus and the compressive strength of TSw2 tuff (the repository horizon) are 32700 ± 4600 MPa and 155 ± 59 MPa, respectively [DOE, 1992b].

Leeds, Hill and Jewett, Inc [1974] conducted field investigations of the ground vibrations from blasting during the construction of the Eisenhower Memorial Tunnel. The measured peak particle velocities are generally consistent with the prediction computed using Equation 1. Field measurements of particle velocity at locations less than 80 m (about 262 ft) from the blasting of 4 to 6 kg of explosive collected by Fornaro et al [1993] are also very close to the computed value using Equation 1. Fornaro's measurements also show that vibrations caused by blasting using delay produce a typical vibrogram in which it is almost always possible to distinguish single shots. This suggests that the peak particle velocity is a function of blasting of a single shot.

Holmberg [1979] also proposed that the ground vibration particle velocity due to a blast is a measure of the damage on nearby constructions. He presented a mathematical model similar to Equation 1 to demonstrate the peak particle velocity as a function of the distance to the charge and the linear charge concentration. In the region close to the charge, permanent damage occurs at a given critical level of particle velocity. The model is found to give good agreement with experimental particle velocities in solid rock up to that region close to an extended charge where rock damage or fragmentation occurs. Use of this model enables the zone of permanent deformation or damage in the rock due to the blast to be predicted if the weight of the explosive is known.

5. CRITERIA OF ROCK DAMAGE AND ESTIMATION OF DAMAGE ZONE

The magnitude of structural damage caused by blasting vibrations is a function of the number of cycles or duration of shaking, the ratio of structural frequency to input frequency, and structural damping as well as peak acceleration. In other words, damage potential depends on the structural response to the given ground vibration as well as the susceptibility of the structural components to damage. In practice, particle velocity is commonly employed as a damage index. The maximum amount of explosives which could be detonated in excavations is limited by the allowable particle velocity. If an excessive quantity of explosive is detonated, disturbance or damage to other structures may result. On the other hand, if the limitations on quantity of explosive become overly restrictive, excavation time and construction cost increase.

The susceptibility, i.e., the intensity of ground vibration which can be tolerated, of various structures must be determined before estimates of permissible charge weights can be determined. Studies of vibration damage by the Bureau of Mines [1980] have concentrated on damage to residential type structures. A recommended safe vibration criterion of 50 mm (about 2 inches) per second particle velocity resulted from those studies. The Bureau of Mines [1962] statistically analyzed the vibration measurements and damage correlations made from 124 residential type structures. They also concluded that damage was not observed from blasting vibrations if the particle velocity was below 50 mm (about 2 inches) per second. The Bureau of Mines [1980] indicated that susceptibility of masonry and concrete is about 75 mm (about 3 inches) per second. Langefors and Kihlstrom [1963] give particle velocity criteria for damage to tunnels in rock. A particle velocity of 300 mm (about 12 inches) per second is given as a criterion for the "fall of rock in unlined tunnels" and a particle velocity of 600 mm (about 24 inches) per second is correlated with formation of new cracks in rocks. Holmberg [1979] suggested that the vibration particle velocity 700-1000 mm/s (about 28-39 in/s) is the good criterion to use for prediction of the damage zone and it is valid for charge concentrations that range from 0.2 to 75 kg/m. Sinha [1989] makes the conservative recommendation that a zone of rock which has received a peak particle velocity of more than 100 mm/s (about 4 in/s) during blasting should be considered to be disturbed.

Dowding and Rozen [1978] studied the damage of 71 tunnels caused by earthquake shaking and distortion. The majority of these tunnels were 3 to 6 m (about 10 to 20 ft) in diameter and were located in rock media. Three levels of response were distinguished without regard to geologic media or lining. No damage implies post-shaking inspection revealed no apparent new cracking or falling of stones. Minor damage due to shaking includes fall of stones and formation of new cracks. Damage includes major rock falls, severe cracking, and closure. The three levels of response are stratified with respect to the calculated peak surface motions. There are no reports of even falling stones in unlined tunnels or cracking in lined tunnels up to 200 mm/s (about 8 in/s). Up to 400 mm/s (about 16 in/s) there are only a few incidence of minor cracking in concrete lined tunnels. Between 400 mm/s (about 16 in/s) and 800 mm/s (about 32 in/s), there was only one partial collapse. It was associated with landsliding and was lined with masonry. The above described damage levels of the tunnels are correlated with the particle velocity at ground surface. To provide some basis for engineer's intuition on the subject of the peak particle velocity, Dowding and Rozen's [1978] state that the peak ground surface motion of 200, 400, and 800 mm/s (about 8, 16, and 32 in/s) is about equivalent to the earthquake motion of 0.19g, 0.25g,

and 0.52g, respectively. Extrapolating the above relationships between particle velocity and earthquake motion, the particle velocity of 100 mm/s (about 4 in/s) corresponds to an earthquake motion of about 0.16g.

Hsiung, et al. [1992] conducted a field instrumentation program at the Lucky Friday Mine, Mullan, Idaho, to generate a reliable data set for understanding the key seismic parameters that will affect borehole stability, underground opening stability, and creation of preferential water pathways to connect the emplacement area with perched water zones, neighboring steep hydraulic gradient zones, or the condensation area above the emplacement area. They investigated the mechanical response of a fractured rock mass around deep underground openings subjected to mining-induced seismicity. Transient response of the rock at excavation surfaces as a result of seismic impingement, permanent displacement changes around excavations, and opening closure were monitored. Observations suggested a yield zone around the excavation. This yield zone was evidenced as fractures with wide-gaps. The extent of the yield zone was fairly restricted. The openings at the site responded to seismic events with peak particle velocities as low as 104 mm/s (about 4 in/s) to 134 mm/s (about 5 in/s). However, there were some mine seismic events with peak particle velocities greater than those which did not have any noticeable effect on the openings. The observed rock mass behavior is explained by the authors using the concept of stick-slip on joints or bedding planes within the rock mass.

Depending upon the state of stresses of a joint or bedding plane, different amounts of energy may be needed for the state of stresses to reach the critical strength and thus induce instability, a condition allowing the joint or bedding plane to slip. A repetitive process of joint stick-slip causes an accumulation of joint displacement. The mechanism of joint displacement accumulation is analogous to stress fatigue phenomena commonly observed for natural or artificial materials. Those observations imply that similar or even more damage to an opening may occur due to a number of seismic events with a relatively smaller magnitude, as opposed to the damage due to a single seismic event with a very strong motion (in terms of peak particle velocity). This field instrumentation program was conducted in the rock mass around a tunnel which has a 3.05-m (about 10 ft) square cross section. The 5-anchor extensometers are installed in a number of boreholes in the rock mass around the tunnel. The extensometer data presented by Hsiung, et al. [1992] indicate that more than 1 mm (about ½ in) increase in the total width of fractures within a 1.5 m (about 5 ft) interval along the axis of a borehole in the rock mass around the tunnel may be caused by ground motion of 100 to 200 mm/s (about 4 to 8 in/s) peak particle velocity induced by the mining related seismicity. The change of fracture size was observed to occur in the rock mass at 5 to 6 m (about 16 to 20 ft) into the excavation limit.

Using Equation 1, the particle velocity at 6.4, 4.6, 3.3, and 2.3 m (about 21, 15, 11, and 7.6 ft) from the location of the detonation of 3 kg explosive are 100, 200, 400, 800 mm/s (about 4, 8, 16, and 32 in/s), respectively. The particle velocity at about 10, 7, 5, and 3.5 m (31.6, 22.5, 16.0, and 11.4 ft) from the location of the detonation of 10 kg explosive are 100, 200, 400, 800 mm/s (about 4, 8, 16, and 32 in/s). Although this empirical relationship is based on a granite site, we may use it for the tuff site for the purpose of rough estimation.

The surrounding rock mass contains a number of potential weak planes, each of which is able to withstand a different level of peak particle velocity. The damage zone is generally taken to be the zone in which new cracks are induced or enlarged. However, for the purpose of

estimating the change of permeability of a rock mass, the damage zone should include the zone in which the size of the existing cracks is increased. In this report, a conservative approach is taken. The lower limit of the above described damage criteria, 100 mm/s (about 4 in/s), is considered as the criterion for the change of the fracture characteristics of the rock mass under vibrations. In other words, the zone extended 6 m (about 20 ft) from the tunnel excavation limit is considered as the damage zone if the maximum charge 3 kg per delay in the smooth wall controlled blasting is adopted. For the maximum charge of 10 kg, the damage zone is estimated to be the zone extended 10 m (about 33 ft) from the excavation limit. For an 8-m (about 26 ft) diameter tunnel, the damage zone estimated here is much larger than the disturbed zone defined as half the diameter of the tunnel in many Yucca Mountain reports [NRC, 1986; NRC, 1989; Langkopf, 1987; Case and Kelsall, 1987].

Field measurements of particle velocity at locations 10 to 80 m (about 33 to 262 ft) from the underground face excavated by TBM or high energy hydraulic hammer are reported by Fornaro et al [1993]. The longitudinal peak particle velocity of 0.5 to 5 mm/s (about ¼ to 1 in/s) is reported as an example although the location of the measurement is not identified. This is about the same order of magnitude as the particle velocity measured at more than 10 m (about 33 ft) away from a typical blasting of the underground excavation. Fornaro et al note that the peak particle velocities for blasting vibration and TBM vibration are very close. Therefore, the rock mass located at a large distance away from the excavation limit has about the same amount of ground vibration for excavation done by the drill and blast method and the TBM method. In the rock mass close to the excavation limit, Fornaro et al [1993] stated that in the case when the rock breaking machine is used to replace the blasting, the levels of vibrations can be reduced by more than one order of magnitude.

6. WASTE ISOLATION IN RELATION TO THE ROCK MASS DAMAGE OF DIFFERENT EXCAVATION METHODS

Fenix & Scisson [1986] reported that no scientific data are available to indicate measurable relationships between permeability and blast damage zones, or methods which will separate blast damage from the effects of stress relief which accompany all underground excavations. Sinha [1989] stated that the exact quantitative comparison of loosened zone between drill and blast and TBM methods is not possible because the amount of loosening during drill and blast is influenced by several factors. However, it could be stated that a drill and blast excavation will disturb a zone about three to six times the disturbed zone of a tunnel boring machine [Sinha, 1989]. The disturbed zone caused by blasting can be reduced by using controlled blasting techniques.

As mentioned earlier, in many Yucca Mountain Project documents the mechanically disturbed zone is defined as a rock mass annulus one radius wide around the drift. The definition of the mechanically disturbed zone is based on the combined results of the stress redistribution and the damage induced in the excavation process. For the 8-m (about 26 ft) diameter repository drift, the mechanically disturbed zone is about 4 m (about 13 ft). Most of the cases described in the rock mass damage section of this report have blasting damage zones smaller than 4 m (about 13 ft). However, the size of the damage zone may be much larger than half diameter of the tunnel, if the 100 mm/s (about 4 in/s) damage criterion and the NRC field investigation results [Hsiung, et al, 1992] are used. Hence, the mechanically disturbed zone can only be used as the primary damage zone. Case and Kelsall [1987] estimated the equivalent rock mass permeability in the mechanically disturbed zone is about 80 times the permeability of undamaged rock mass at the Yucca Mountain site.

Because the mechanically disturbed zone has only very limited extent in comparison to the overburden thickness, the damage of rock mass outside the mechanically disturbed zone must be examined in order to evaluate the impact on the waste isolation capability of the repository. Due to the peak particle velocity assessment described above, the rock mass outside the mechanically disturbed zone may respond to the blasting activity induced stress change which may generate secondary phenomenon of fracture aperture change, new fracture formation, and fracture lengthening. Based on the 100 mm/s (about 4 in/s) peak particle velocity as the rock mass disturbance criterion, it is conservatively expected that no secondary rock fracture damage will be encountered at 10 m (about 33 ft) away from the excavation limit (providing the amount of explosive per delay is less than 10 kg). According to the cubic law for laminar flow in fractures, the hydraulic conductivity of a fracture increases eight times, or roughly one order of magnitude, if the width of the fracture is doubled (from $2^3 = 8$). Lin and Hardy (1992) indicate that the fracture size under *in situ* stresses at the potential repository horizon is about 0.01 to 0.02 mm (about 0.0004 to 0.0008 in). The fracture frequency is about 5 to 30 per meter (about 1.5 to 9 per ft) [DOE, 1992b]. Thus the total fracture width within an 1.5 m (about 5 ft) wide rock mass is about 0.075 to 0.9 mm. As described in Section 5, Hsiung, et al [1992] indicate that more than 1mm (about ½ in) increase in the total width of fractures within a 1.5 m (about 5 ft) interval along the axis of a borehole in the rock mass around the tunnel may be caused by ground motion of 100 to 200 mm/s (about 4 to 8 in/s) peak particle velocity induced by the mining related seismicity. Hence, it is conservative to assume that the equivalent permeability of the rock mass from the limit of the mechanically disturbed zone to the limit of the damage zone is one order

of magnitude larger than the permeability of the undisturbed rock mass. The equivalent permeability of the mechanically disturbed zone should be taken as two or three orders magnitude larger than the permeability of the undisturbed rock mass. Using these permeability values as parameters of the underground fluid flow sensitivity study, the waste isolation impacts caused by the drill and blast excavation method may be assessed.

For mechanical excavation, the mechanically disturbed zone results from the excavation process and the stress redistribution. Because the ground vibration generated by the mechanical excavation outside the mechanically disturbed zone is much less than 100 mm/s (about 4 in/s), the damage zone defined by the 100 mm/s (about 4 in/s) particle velocity does not exist.

The principal effect of different excavation techniques is to potentially modify the size and characteristics of the mechanically disturbed zone surrounding the excavation. In many respects, the affect of modifying the size and characteristics of the mechanically disturbed zone is analogous to modifying the size of the underground opening, given there is some relationship between the size of the opening and the size of the mechanically disturbed zone. Therefore, the four possible adverse impacts evaluated with respect to increasing the ramp/drift size [Tsai, 1993] are also relevant to the potential modification in the size of the mechanically disturbed zone. These potential impacts include: (1) increased probability of pre-closure radionuclide releases due to increased instability of the rock mass; (2) increased water quantities at the repository horizon due to the creation of preferential surface water pathways along the entire ramp/drift; (3) increased aqueous radionuclide releases to the accessible environment due to increased groundwater flux through the geosphere; and (4) increased gaseous radionuclide releases to the accessible environment due to increased gas phase flux through the geosphere. The conclusions reached by Tsai [1993] are also germane to the present discussion of the effects of modifying the mechanically disturbed zone, namely (1) the potential pre-closure effects can be mitigated by a stronger rock reinforcement and waste package design; (2) the potential change in fracture flow characteristics is expected to have little or no effect on unsaturated flow properties in units dominated by matrix flow; (3) even if fracture (or episodic) flow is present for units which intersect the mechanically disturbed zone, it is probable that the flow would be attenuated by matrix-dominated capillary suction rather than flowing laterally within the disturbed zone along the ESF ramp/drift axis; (4) the consequences of any net increase in aqueous flux or water saturation at the repository horizon would likely fall within the wide envelope of the uncertain ambient flux (whether thermally perturbed or not); and (5) the effects on gas flow are expected to be small because the extent of the disturbed zone is small in comparison to the plan view size of the repository and the overburden thickness. Therefore, the effects of any possible changes in the extent or characteristics of the mechanically disturbed zone caused by different excavation techniques are expected to be minimal.

7. WASTE ISOLATION IN RELATION TO THE CONSTRUCTION WATER USAGE

The Exploratory Studies Facility Design Requirements (ESFDR) document [DOE, 1992c] defines criteria for limits on water use in terms of acceptable but undetermined increases in the saturation level of the repository horizon bedrock, and criteria for limits on water use underground. It requires the water usage of excavation methods that will limit the potential impacts on the ability of the engineering and natural barrier system at Yucca Mountain to isolate radioactive waste. Most of the water used in underground construction will be removed with the excavated muck and by waste water drainage during construction or by ventilation during the preclosure phase of the ESF and repository operation. Some water will be trapped in the rock mass at the repository.

Dunn and Sobolik [1993] investigated the movement and potential impact on waste isolation of water used or spilled during construction of the underground tunnels for the ESF. They performed computational analyses using a composite fracture/matrix porosity model to simulate the effect of water usage or spillage in the ESF tunnel excavated in the Topopah Spring welded unit (TSw2). The flooding scenario simulates the complete filling of the tunnel with water for a period of one month. After one month has passed, the standing water is removed, and two different sets of calculations continue the analyses. One set estimates the movement of the imbibed water given no evaporation of the water from the tunnel walls into the air (i.e., a no-flow boundary condition). The other set estimates the movement of the imbibed water assuming that air at 90% relative humidity is in contact with the tunnel wall rock. These calculations were carried out to 100 years. The wetting scenario estimates the water imbibition and movement due to keeping the tunnel walls saturated continuously for varying periods of time. Then, the ventilation is applied and the air will draw moisture from the tunnel walls. The air at 90% relative humidity could behave much like a boundary layer, keeping the moisture near the tunnel walls.

Calculations for the extent of water imbibition and movement after the tunnel is filled with water for one month show the maximum extent of water movement is about 3 m (about 10 ft) beyond the side wall and 3 m (about 10 ft) below the floor and above the ceiling. The amount of water imbibed through the tunnel walls in one month is about 2.6 cubic meters of water per meter of tunnel (about 209 gallons per ft of tunnel). Assuming that there is no evaporation of water from the tunnel walls into the tunnel air, the imbibed water moves away from the tunnel in a nearly axisymmetric manner. The region of increased saturation of 0.025 or more extends less than 8 m (about 26 ft) into the wall rock after 100 years. For the case with ventilation, the rock returns to its original moisture content after about 5 months. If the tunnel is ventilated for 100 years the rock continues to dry until about 6.1 cubic meter of water per meter (about 491gallons per ft) of tunnel has been removed. Dunn and Sobolik [1993] compare the amount of water removed by ventilation after the one month flooding scenario and the amount removed immediately after construction with assumed *in situ* parameters as initial conditions. According to the ventilation model used for the analysis, filling a tunnel in TSw2 with water for one month would have a minimal effect on long-term *in situ* moisture content.

The wetting scenario simulates the effects of many saturating and periodic water application activities. The amount of imbibed water and the movement of the water after 1 month of tunnel

wetting are approximately the same as those for the flooding scenario. After 2 years, nearly all of the imbibed water remains within 7 m (about 20 ft) of the tunnel walls. It was concluded that the amount of water that might be added to the rock as a result of 1 month of continuous wetting activities can be drawn out of the rock through ventilation with negligible effects on the long-term *in situ* moisture content. The reason for the similar results of the amount of imbibed water and the movement of the water for both the flooding and wetting scenarios and the negligible effects on the long-term *in situ* moisture content induced by the ventilation is that the model used for the analyses predicts that in the TSw2, capillary forces dominate over gravity-driven flow. Such a prediction is the result of the small hydraulic conductivity assigned to the TSw2 formation. If the hydraulic conductivity of the host rock is large, imbibed water will move away preferentially downward, beyond the effects of moisture movement toward the tunnel walls due to ventilation drying of the rock.

Dunn and Sobolik [1993] performed the same type analyses as described above for the ESF tunnel excavated in the Paintbrush Tuff nonwelded unit (PTn). Since the fracture saturated hydraulic conductivity of PTn is about 35 times of that for TSw2 and the matrix saturated hydraulic conductivity of PTn is about 4 orders of magnitude larger than that for TSw2, the gravity-driven flow is dominated in the PTn. Hence, the imbibed water movement away from the tunnel would not be in a nearly axisymmetric manner and the ventilation cannot draw back most of the imbibed water. After the tunnel is filled with water for 1 month, 56.3 cubic meters of water are absorbed in each meter of tunnel (or about 4,535 gallons per ft of tunnel). The leading edge of water movement is about 10 m (about 33 ft) below the tunnel floor, as opposed to about 4 m (about 13 ft) above the tunnel ceiling. The maximum lateral movement after 1 month is about 5 m (about 16 ft) from the tunnel wall. When a ventilation boundary condition of 90% relative humidity air is imposed, water is drawn out from the rock primarily from the ceiling and the water in the floor continues to infiltrate downward. During the 100-year period, ventilation withdraws only 32 of the 56.3 cubic meters of water absorbed per meter of tunnel (about 2578 gallons of the 4,535 gallons per ft of tunnel) during the water filled period.

The composite porosity model used by Dunn and Sobolik [1993] treats the matrix and the fractures as an equivalent porous medium. The fracture apertures used in their calculations for TSw2 are 4 μm to 6 μm (about 1.6×10^{-4} to 2.4×10^{-4} in), which correspond to a bulk permeability of $3 \times 10^{-16} m^2$ (about $3 \times 10^{-15} ft^2$). Actual fractures within Yucca Mountain could have much different apertures. Montazer et al [1986] and Thordarson [1983] reported bulk permeabilities for Topopah Spring welded tuff in the range of 10^{-13} m^2 to 10^{-11} m^2 (about 10^{-12} ft^2 to 10^{-10} ft^2) which probably represent fracture apertures of at least some of the fractures in the range of 100 μm to 1 mm (about 4×10^{-3} to 4×10^{-2} in). It is known that hydrologic properties from samples within a geologic unit can vary greatly. It is unknown what effect this variation would have on flow. Variations in hydrologic properties in highly conductive and porous regions might have large effects on the vertical and horizontal dispersion of water. If highly conductive regions are vertically connected, the time for the water to reach the water table could be shortened. If highly conductive regions are horizontally connected, lateral dispersion of flow could be enhanced. Buscheck et al [1991] predict that a water front will flow through a 100 μm (about 4×10^{-3} in) fracture through 60 m (about 197 ft) of Topopah Spring welded tuff after 4.5 hours (an average flux of 3.7×10^{-3} m/s) (about 1.2×10^{-2} ft/s). Dunn and Sobolik [1993] re-evaluated their analyses with a bulk permeability of 3×10^{-12} m^2 (about 3×10^{-11} ft^2) in TSw2. Using the cubic law and the fracture properties from the original tunnel

calculations, the fracture property values selected for the additional calculations are fracture porosity = 2.7×10^{-3} and fracture saturated hydraulic conductivity equals to 3.94×10^{-3} m/s (about 1.3×10^{-2} ft/s). According to their calculations, over 8 m^3 of water per meter of tunnel, or about 644 gallons per ft of tunnel, infiltrated the walls over a 4-hour period.

Dunn and Sobolik [1993] estimated that 8 m^3 of water per meter of tunnel (about 650 gallons) of water per linear ft of tunnel are required for the TBM excavation operation and the roof support operation. They assumed 20% (or 130 gallons per linear ft of tunnel) of this water is available to infiltrate the tunnel walls. Through personal communication with Bob Kirk of M&O, it is estimated that 7.5 m^3 of water per meter of tunnel (about 600 gallons per ft) are required for the TBM operation, 4.5 m^3 per meter (about 360 gallons per ft) for bolting, 4.5 m^3 per meter (about 360 gallons per ft) for shotcrete and grouting, and 22.5 m^3 per meter (about 1,800 gallons per ft) for the conveyor dust control. The water for shotcrete and grouting will be bounded within the cement. The dust control water used for the TBM excavation operation is applied directly onto the muck on the conveyor belt. Therefore, we can assume this water will be removed completely from the tunnel mechanically by the conveyor and evaporatively by ventilation. If we assume 20% of water used for TBM operation and bolting are not removed in the mucking operation or through water recovery techniques, there are 2.4 m^3 of water available to infiltrate the tunnel walls for each meter of tunnel (about 192 gallons per ft). Pritchett [1993] reported that 680 m^3 (about 180,000 gallons) of water for drill and blast excavation, 288 m^3 (about 76,000 gallons) for bolting, 382 m^3 (about 101,000 gallons) for miscellaneous use, and 300 m^3 (about 79,000 gallons) for shotcrete and grouting are consumed in the 61 meter (about 200 ft) long starter tunnel of the ESF at Yucca Mountain. Ignoring the water used for shotcrete and grouting and taking 20% of the water used for all other functions, there are 4.5 m^3 of water available to infiltrate the tunnel walls of each meter of tunnel (about 357 gallons per ft) for the drill and blast excavation method.

Hsiung et al [1992] field investigation shows that a change of more than 100 μm (about 4×10^{-3} in) in fracture size may be induced by nearby mining activity (drill and blast). Hence, the hydraulic conductivity of the tunnel's disturbed zone using the drill and blast method is large and the gravity-driven flow dominates capillary flow in this zone. The imbibed water moves into the floor from the tunnel very rapidly and ventilation may not draw back most of the imbibed water. Using Dunn and Sobolik's calculation with a bulk permeability of 3×10^{-12} m^2 (about 3×10^{-11} ft) in the TSw2, all of the 4.5 m^3 per meter (about 357 gallons per ft) available construction water used for the drill and blast excavation method may infiltrate the tunnel walls. Beyond the disturbed zone, if the *in situ* fracture size remains small, the imbibed water will not be drained quickly. Therefore, if waste emplacement tunnels are excavated using the drill and blast method, the increased moisture near the tunnel may have adverse impacts on the waste isolation. For the excavation by TBM, the fractures in the disturbed zone suffer less disturbance and fracture apertures may remain close to their *in situ* values. Therefore, there is not only less construction water available to infiltrate into the tunnel walls for the TBM excavation method, infiltration rates should be lower.

8. CONCLUSION

The potential modification in the size of the mechanically disturbed zone of rock around the excavated drifts caused by different excavation techniques may generate several possible post-closure performance effects. The possible effect on pre- and post-closure performance-related parameters include changes in permeability (in particular the bulk rock mass permeability), gaseous and liquid flux, and rock stability. The magnitude and extent of these effects is uncertain at present. In addition, the consequence of these possible changes in performance-related parameters on the actual performance are also not quantified at present.

Although certain tentative design goals, which have been put forward by Hardy and Bauer [1991], are potentially affected by the choice of excavation technique, there is generally a lack of a performance basis for these design goals. One of the primary goals of repository design is to control the increase in permeability that may be created by the excavation process. This goal is reflected by the design criterion that the change of permeability in the rock mass outside the envelop, defined by a distance equal to the radius of the drift measured from the excavation limit, should not exceed one order of magnitude. This design goal has little proven relevance from a post-closure performance perspective nor do we think that this design goal could be adequately verified in-situ. The impact on the waste isolation capability of the repository caused by the excavation process must be answered through comprehensive performance assessment analyses. It is recommended that these analyses should be conducted along with field investigation during the ESF construction phase. The field investigation method described in Hsiung et al [1992] should be used to assess rock damage caused by the excavation methods.

The mechanically disturbed zone around a drift excavated by the drill and blast method may have a significantly higher permeability than the zone around a drift excavated by the TBM method. If the drill & blast method is used, the mechanically disturbed zone should not be considered as the primary barrier for waste isolation due to its uncertain thermo-hydrologic properties. Compared to the thickness of the overburden or the underlying rock mass of the repository drifts, however, the thicknesses of the mechanically disturbed zone above and below the drifts are expected to be small. Hence, there is an insignificant impact on the overall waste isolation capability of the entire repository by ignoring the waste isolation capability of the mechanically disturbed zone. This statement is made under the assumption, however, that the seals of the mechanically disturbed zone are as effective as the seals of the repository drifts themselves such that any potentially enhanced horizontal movement of underground fluids along the sealed drifts (and their surrounding mechanically disturbed zone) do not cause waste isolation impacts.

The change of permeability in the damaged zone (outside the mechanically disturbed zone) around tunnels excavated by the drill and blast method and by the TBM and other mechanical excavation methods are expected to have only one order of magnitude difference based on the assessment of the particle velocities induced by the excavation. Because the waste isolation capacity of the mechanically disturbed zone around a drift is expected to be negligible, the impacts on waste isolation of the repository induced by the drill and blast excavation is not significantly different from the impact induced by the mechanical excavation techniques. However, more construction water trapped in the mechanically disturbed zone created by the

drill and blast excavation may change the local hydraulic regime. The waste isolation impacts of such change of hydraulic regime are unknown and they should be analyzed in the future.

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