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Ground Control Methodology for Emplacement Drifts

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Civilian Radioactive Waste Management System
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Ground Control Methodology for Emplacement Drifts

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CONTENTS

FIGURES

TABLES

ACRONYMS AND ABBREVIATIONS

3DEC	3-Dimensional Distinct Element Code
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ANSYS	ANSYS Computer Code
ASM	American Society for Metals
ASTM	American Society for Testing and Materials
BSC	Bechtel SAIC Company, LLC.
CRWMS M&O	Civilian Radioactive Waste Management System Management and Operating (Contractor)
DOE	U.S. Department of Energy
DTN	Data Tracking Number
ECRB	Enhanced Characterization of the Repository Block
ESF	Exploratory Studies Facility
ESR	Excavation Support Ratio
FLAC	Fast Lagrangian Analysis of Continua
FLAC3D	3-Dimensional Fast Lagrangian Analysis of Continua
GRC	Ground Reaction Curve
HTOM	High Temperature Operation Mode
LA	License Application
LTOM	Low Temperature Operation Mode
NGI	Norwegian Geotechnical Institute
PFC	Particle Flow Code
PGA	Peak Ground Acceleration
PGD	Peak Ground Displacement
PGV	Peak Ground Velocity
pH	Negative logarithm of Hydrogen Ion Concentration
PSHA	Probabilistic Seismic Hazard Analysis
Q	Rock Mass Quality Index
QARD	Quality Assurance Requirements and Description
RMR	Rock Mass Rating

RQD	Rock Quality Designation
SF	Safety Factor
SR	Site Recommendation
SRF	Stress Reduction Factor
TBM	Tunnel Boring Machine
UDEC	Universal Distinct Element Code
YMP	Yucca Mountain Project

1. INTRODUCTION

1.1 PURPOSE

The purpose of the *Ground Control Methodology for Emplacement Drifts* is to present an acceptable, consistent, systematic approach that can be used for design of the ground support system for the emplacement drifts in the Yucca Mountain repository. This report can provide a basis for design analyses to be used in the development of calculations, drawings, and specifications for License Application (LA) design.

The relationship of the document to the overall design process is shown in Figure 1-1. The overall design process includes three major steps: develop design inputs including identifying requirements, criteria, and scope, and selecting parameters, in parallel with developing design methodologies and computational models; perform design calculations and analyses including preliminary and detailed design calculations and analyses; and evaluate candidate ground support design. The development of this design document is part of Step One in the overall design process.

1.2 SCOPE

The *Ground Control Methodology for Emplacement Drifts* is a design-topic-specific document. This document is developed to a level commensurate with the available design inputs developed at the time of preparation. The document is intended to be a living document, to be revised to the appropriate level of detail as the overall design progresses. The document addresses ground support only, and the invert structure that will support waste packages is not considered part of ground control.

1.3 APPLICABILITY

This report will be applicable to the design of ground control for emplacement drifts. The design methodology for all other non-emplacement openings, such as access mains, exhaust mains, and ventilation shafts, will be addressed in a separate document.

1.4 QUALITY ASSURANCE

The activities addressed in this drift design document are subject to the requirements of the QARD (DOE 2002) since the ground control system for emplacement drifts is classified as QL-2 (YMP 2001, p. A-4). The document was prepared per AP-3.11Q, *Technical Reports*, and its requirements.

All data in this document are presented for illustrative purposes only. A ground control designer is required to use qualified data with data tracking numbers identified. Corroborative information is to be used only if qualified data are not available. It is the designer's responsibility to request that such data be provided in time to support the design.

1.5 REPORT ELEMENTS

This report addresses the key elements that need to be considered in the design process of the ground control system for emplacement drifts. These elements include design requirements and criteria, design basis inputs, rock mass characteristics, preliminary design analysis of ground support, detailed design analysis of ground support, uncertainties, and constructibility and maintenance. How these elements should be addressed in the design is discussed in separate sections of this report. There is no design conclusion to be drawn in this report due to its apparent nature because it provides only the methodology for the design, not the solution of the design, of ground control system for emplacement drifts. Therefore, design assumptions or inputs are not directly used in this report.

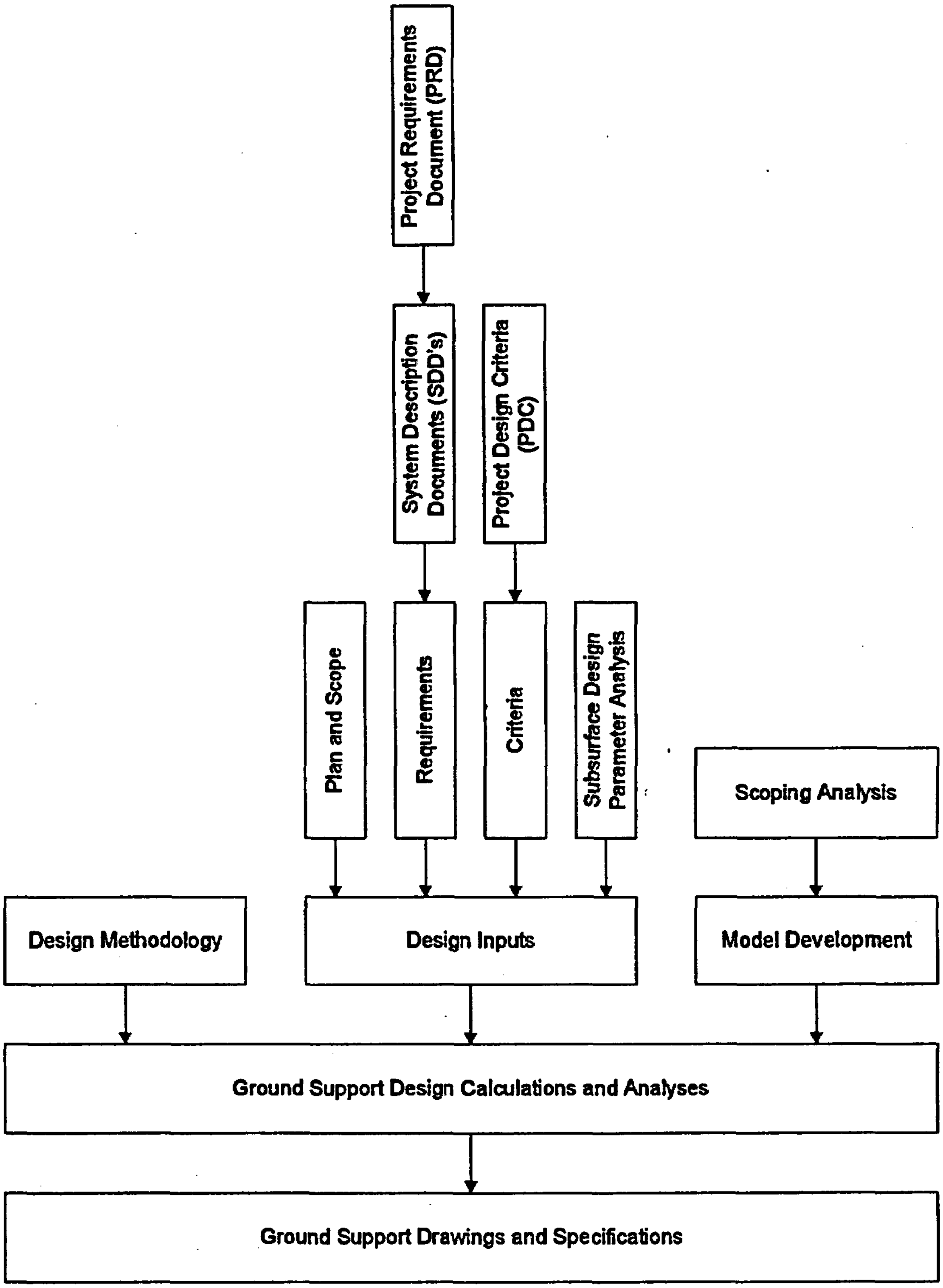


Figure 1-1. Design Process Overview

2. DESIGN REQUIREMENTS AND CRITERIA

This section describes the requirements, criteria, constraints, and codes and standards, applicable to the design of ground support in emplacement drifts.

2.1 REQUIREMENTS AND CRITERIA

2.1.1 Drift Stability

The ground control system shall be designed to ensure the drift stability by minimizing rock deformation and provide for personnel safety and accessibility during construction, operations, maintenance, and retrieval.

2.1.2 Operating Envelope Control

The ground control system shall maintain an operating envelope of 4.9 m (16.1 ft) for emplacement drifts, based on 5,500 mm (18.0 ft) excavated openings (see Section 2.2) plus 200 mm (7.9 in) thick ground control system plus 100 mm (3.9 in) operating clearance (BSC 2001c; 2001d), while allowing for the expected variations in excavated dimensions, and ground support configurations, alignment, and deformation. The inside diameter of the ground support should be large enough to provide a tolerance outside the operating envelope.

2.1.3 Rock Falls

The ground control system shall be designed to provide protection against rock falls, loosening of blocks, and fracturing and surface deterioration of the rock mass surrounding emplacement drifts during the preclosure period to minimize the damage to waste packages. The anticipated size range of rock falls that need to be considered in the ground support design will be determined from an ongoing study of the drift degradation.

2.1.4 Constructibility

Constructibility is a desirable characteristic for the ground control system. The ground control system should be constructible with conventional materials, fabrication methods, and installation means, if viable. This would allow for quality control, effective installation, and adaptation to the range of construction conditions.

2.1.5 Material Acceptability

To ensure waste isolation under preclosure and post closure conditions, the ground support shall use materials having acceptable long-term effects on waste isolation. Acceptability shall be based on the results of evaluations of the potential impacts of the materials proposed on waste isolation. Due to uncertainties associated with the presence of cementitious materials in emplacement drifts and difficulty of resolving these uncertainties for LA design (CRWMS M&O 1999a, Section 9.1.3; CRWMS M&O 1999b, Section 7.2), use of cementitious materials is recommended to be minimized unless they meet acceptable long-term performance requirements for waste isolation.

2.1.6 Operational Life and Maintenance

The ground control system shall maintain its functionality during the operational life of up to 300 years after final waste emplacement (Curry and Loros 2002, PRD-014/T-006, p. 3-89).

Due to uncertainties associated with longevity of ground support materials, periodic inspection and maintenance may be necessary in order to ensure that the function of the ground support to keep the emplacement drift open and stable for the entire preclosure period is met. Due to the complexity and high cost of inspection and maintenance after waste emplacement, development of the strategies used for these programs is very crucial and will be addressed in a separate report. Nevertheless, the ground control system shall be designed to minimize the needs or frequency for periodic maintenance. To achieve this, the ground support system shall be designed for the anticipated bounding cases. The consideration of the bounding cases shall be reflected in the selection of loads and load combinations, material properties, safety factors, and analysis approaches.

2.2 DESIGN CONSTRAINTS

The following design constraints are applicable to the design of the ground control system:

- Drift Spacing: The nominal emplacement drift spacing shall be 81 meters (265.8 ft), drift center line to drift center line (Williams 2002).

- Excavated Diameter: The nominal excavated diameter of emplacement drifts shall be 5.5 meters (18.0 ft) (Williams 2002).

- Design Thermal Load: The ground control system shall be designed for a design thermal load of 1.45 kW/m (1508.4 Btu/hr·ft), averaged over a fully loaded emplacement drift at the time of completion of loading an entire emplacement drift (Williams, 2002).

- Design Seismic Load: The ground control system shall be designed for a design basis seismic load for a QL-2 system.

2.3 APPLICABLE CODES AND STANDARDS

The codes and standards, applicable to the design of ground support, are listed below.

- ACI (American Concrete Institute) 209R-92. 1992. *Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures.*

- ACI 506R-90(95). 1995. *Guide to Shotcrete.*

- ACI 506.2-95. 1995. *Specification for Shotcrete.*

- ACI 349-97. 1998. *Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-97) and Commentary - ACI 349R-97.*

- ACI 318-99. 1999. *Building Code Requirements for Structural Concrete (ACI 318-99) and Commentary (ACI 318R-99).*

- AISC (American Institute of Steel Construction) 1997. *Manual of Steel Construction - Allowable Stress Design,* 9th Edition, 2nd Revision, 2nd Impression.

- ASM (American Society for Metals) 1978. *Properties and Selection: Irons and Steels.* Volume 1 of *Metals Handbook.* 9th Edition. Bardes, B.P., ed.

- ASM International 1990. *Properties and Selection: Irons, Steels, and High-Performance Alloys.* Volume 1 of *Metals Handbook.* 10th Edition.

- ASTM (American Society for Testing and Materials) F 432-95. 1995. *Standard Specification for Roof and Rock Bolts and Accessories.*

- ASTM C 845-96. 1996. *Standard Specification for Expansive Hydraulic Cement.*

- ASTM C 1116-97. 1998. *Standard Specification for Fiber-Reinforced Concrete and Shotcrete.*

- ASTM A 53/A53M-99b. 1999. *Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless.*

- ASTM C 494/C 494M-99a. 1999. *Standard Specification for Chemical Admixtures for Concrete.*

- ASTM A 36/A 36M-00a. 2000. *Standard Specification for Carbon Structural Steel.*

- ASTM A 242/A 242M-00a. 2000. *Standard Specification for High-Strength Low-Alloy Structural Steel.*

- ASTM A 307-00. 2000. *Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength.*

- ASTM C 1240-00. 2000. *Standard Specification for Use of Silica Fume as a Mineral Admixture in Hydraulic-Cement Concrete, Mortar, and Grout.*

- ASTM A 572/A 572M-01. 2001. *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel.*

- ASTM A 276-02. 2002. *Standard Specification for Stainless Steel Bars and Shapes.*

- ASTM A 709/A 709M-01b. 2002. *Standard Specification for Carbon and High-Strength Low-Alloy Structural Steel Shapes, Plates, and Bars and Quenched-and-Tempered Alloy Structural Steel Plates for Bridges.*

- ASTM A 588/ A 588M-00a. 2001. *Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi [345MPa] Minimum Yield Point to 4–in. [100–mm] Thick.*

- ASTM A 1011/A 1011M-02. 2002. *Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability.*

- ASTM A 1022-01. 2002. *Standard Specification for Deformed and Plain Stainless Steel Wire and Welded Wire for Concrete Reinforcement.*

3. DESIGN BASIS INPUTS

This section describes the design basis inputs, including the emplacement drift locations and orientation, host rock, standoff distance from major faults, types and materials of ground support, and loads anticipated.

3.1 EMPLACEMENT DRIFT LOCATION AND ORIENTATION

The repository will be sited at least 200 m (656 ft) deep in volcanic welded tuff to ensure adequate protection from surface events, and about 200 to 300 m (656 to 984 ft) above the regional groundwater level to provide sufficient barrier for waste isolation (DOE 2001, Section 1.4.2.2.2).

The emplacement drifts are orientated at an azimuth of 252 degrees (BSC 2002, Section 5.1.4) to minimize the adverse effects of the dominant rock joint system on the stability of emplacement drift openings.

3.2 EMPLACEMENT DRIFT HOST ROCK

The repository host horizon will contain four rock formations, namely the upper lithophysal rock (Tptpul), the middle non-lithophysal rock (Tptpmn), the lower lithophysal rock (Tptpll), and the lower non-lithophysal rock (Tptpln) (Board et al. 2002, p. viii). The characteristics of the upper and lower lithophysal rocks are similar, and this is also true for the middle and lower non-lithophysal rocks. Hence, these four rock formations are grouped into two types, the lithophysal and the non-lithophysal rocks, for reasons of simplicity.

3.3 STANDOFF FROM MAJOR FAULTS

A standoff of 60 meters (197 ft) from the closest edge of a repository opening to the main trace of any Type I fault zone is required (BSC 2002, Section 7.1.3). This information will be needed in order to evaluate the impact of potential fault displacement on the stability of emplacement drifts.

3.4 MATERIALS OF GROUND SUPPORT

The types of ground support, as well as their materials, recommended for the ground support for emplacement drifts are listed in Table 3-1 (Linden 2002, Section 8). These are all based on the use of steel. In general, the selection of ground support material is a design solution instead of a design input, and should be based on the detailed analysis using the approaches described in this report. However, due to the project-specific requirements for materials used in emplacement drifts on preclosure durability and on postclosure waste isolation, the options for ground support types and materials are very limited. Use of cementitious materials is recommended to be minimized unless future studies on this subject indicate that they meet acceptable long-term impacts on waste isolation.

Table 3-1. Types and Materials of Ground Support

Type of Opening	Final Support
Emplacement drifts (roofs and walls only)	1) Fully-grouted rock bolts with heavy duty welded wire fabric, or 2) Fully-grouted rock bolts with heavy duty welded wire fabric, combined with steel sets as needed

Source: Linden 2002, Section 8.

3.4.1 Preclosure Durability

Steel is the primary material for ground support in emplacement drifts. Steel durability is primarily a function of its resistance to corrosion due to exposure to the chloride content and pH of the rock pore fluid and humid air in the drifts. A viable approach is to provide an adequate sacrificial thickness of steel on surfaces to be exposed to a corrosive environment, provided that such as extra thickness does not lead to a cumbersome structural system (e.g., unusually heavy gauged wire mesh). Other factors that affect longevity of steel include temperature, radiation, and biological effects. A detailed discussion of steel durability during the preclosure period is provided in the *Longevity of Emplacement Drift Ground Support Materials* (BSC 2001b, Section 6.3).

Cement grout is proposed for use to bond rock bolts to rock for rock mass reinforcement. Another function of cement grout is to limit water percolation and provide corrosion protection for steel rock bolts. In order for fully grouted rock bolts to function as required during the preclosure period, the grout should act as both corrosion and hydraulic barriers, in addition to its desired mechanical bonding capacity. Factors that affect longevity of cement grout include permeability, sulfate resistance, carbonation, thermal, radiation, and biological effects. A detailed discussion of cement grout durability during the preclosure period is provided in the *Longevity of Emplacement Drift Ground Support Materials* (BSC 2001b, Section 6.4).

3.4.2 Postclosure Acceptability

Materials selected for ground support must have acceptable long-term effects on waste isolation. Two aspects of waste isolation that could be affected by the characteristics of the materials used for ground support are: 1) potential corrosion of waste packages and 2) increased mobility of radionuclides. Of primary concern is the pH of any water flowing through or from the concrete.

Use of steel in emplacement drifts has little adverse impact on the postclosure repository performance (CRWMS M&O 1997a, Section 7.2.3), and hence is acceptable.

The concerns associated with the use of cementitious materials in emplacement drifts are related to repository postclosure performance. Among them, the primary issue is whether water influenced by the cement or concrete (in its solid form before closure and in a broken form some time after closure) would cause an unacceptable increase in the mobility of the radionuclides present in the wastes. Though there is not sufficient evidence or data currently available to justify or disqualify the use of cementitious materials in terms of their impact on the postclosure waste isolation (CRWMS M&O 1999a, Appendix B.5), uncertainties associated with the issue

are high. In order for the project to select the ground support for emplacement drifts based primarily on the use of cementitious materials, additional study on the issue is needed.

3.5 DESIGN LOADS

The ground control system shall be designed for the appropriate combinations of the major loads defined below. Some loads, such as construction and operation loads, which may need to be considered in the future detailed design, are not discussed in this report for the LA design.

3.5.1 In Situ Stress Loads

In situ rock stress is the stress existing prior to drift excavation. Excavation will cause the stress around the opening to be re-distributed. Thereafter, some ground relaxation will take place, depending on the relaxation characteristics of the rock, the length of the time interval between excavation and support placement, and the stiffness of the support. As a result, the opening dimensions will change, as the new equilibrium state between the rock and the ground support is attained.

Installation of the ground support will likely result in installation stresses (jacking, grouting, etc.) After installation, additional relaxation of the surrounding rock will further stress the ground support. Pre-emplacement stress in the ground support is the stress existing just before emplacement of waste in the drift at that location. This stress level will depend on installation stresses and the additional stress transferred from the rock to the support, a function of rock relaxation characteristics, time of installation, and support stiffness. Appropriate empirical relationships and analytically established relationships can be used to estimate values of the pre-emplacement rock load to be used for design of ground support at a given location.

The in situ stress state at the repository horizon will vary from location to location. For the initial state of stress, the vertical stress (σ_v) at some point caused by the overburden weight is generally given as (Goodman 1980, Eq. 4.1)

$$\sigma_v = -\sum_{i=1}^{n} \rho_i g h_i \qquad \text{(Eq. 3-1)}$$

where
ρ_i = average bulk density of the ith layer of rock mass, kg/m^3
h_i = thickness of the ith layer of rock mass above an opening, m
g = gravitational acceleration, m/s^2
n = total number of overlaying layers of rock mass, dimensionless

Assuming that lateral displacements are prevented, linear elasticity theory predicts that the horizontal stress (σ_h) is determined by (Goodman 1980, Eq. 4.2 and p. 101)

$$\sigma_h = \frac{\upsilon}{1-\upsilon}\sigma_v \qquad \text{(Eq. 3-2)}$$

where υ = Poisson's ratio, dimensionless

This formula is derived from the assumption that gravity is suddenly applied to an elastic mass of material in which lateral movement is prevented. This condition hardly ever applies in practice due to repeated tectonic movements, overburden removal, material failure, and locked-in stresses due to localized geologic heterogeneous conditions and faulting. Studies on the Yucca Mountain Project have estimated the relationship between horizontal and vertical stresses (DTN: MO0007RIB00077.000). The in situ stress measurements by hydraulic fracturing in three test boreholes drilled in the ESF have shown that the minimum and maximum K_o values ($K_o=\sigma_h/\sigma_v$) are about 0.34 and 0.91, respectively (DTN: MO0007RIB00077.000, Tables 4 and 5). Therefore, the lower and upper bound K_o values of 0.3 and 1.0 are recommended for use in ground support design.

Table 3-2 provides a summary of the recommended parameter values related to in situ stress loads.

Table 3-2. Recommended Parameter Values Related to In Situ Stress Loads

Parameter		Value
Initial Vertical Stress (MPa)		To be calculated using Eq. 3-1
Initial Horizontal-to-Vertical Stress Ratio (K_o)	Minimum	0.3
	Maximum	1.0

3.5.2 Thermal Loads

The thermal loads are associated with the thermal expansion of the rock mass and the ground support materials that accompanies temperature increases caused by the thermal energy released from the waste packages. In response to the expansion, the confinement of the rock mass and the interaction between the rock and the ground support produce stresses in the rock mass and in the ground support materials. The thermally induced stresses are generated after waste packages are emplaced in the emplacement drifts.

The thermal stresses at any location depend on the proximity and timing of waste emplacement, the rate of waste heat generation, the age of the waste, packaging and emplacement configuration, the rate and duration of continuous ventilation, and the thermomechanical properties of the rock mass and the ground support materials. Hence, thermal loads are time-dependent. Since the ground support are designed to maintain the stability of repository drifts only in the preclosure period, the thermal loads are considered up to 300 years, the duration of the preclosure period following the final waste emplacement (Curry and Loros 2002, PRD-014/T-006, p. 3-89).

A design thermal load of 1.45 kW/m (1508.4 Btu/hr·ft) in terms of the initial linear heat power generated by waste packages is considered for the LA design (Williams, 2002). This thermal load corresponds to the high-temperature operation mode (HTOM). Continuous ventilation at 15 m^3/s (530 ft^3/s) for a duration of 50 years following the final waste emplacement is featured in this operation mode. Since the repository operation beyond the 50 years is not specified, it is assumed that the continuous ventilation will be operated for the entire period of preclosure in case this period exceeds 50 years.

In addition to the HTOM, a low temperature operation mode (LTOM) was also considered in the Site Recommendation (SR) design (BSC 2001e, Section 6.3.3). The LTOM used an initial linear heat load of 1.0 kW/m (1040.3 Btu/hr·ft), together with a 50-year continuous ventilation at an air flow rate of 15 m^3/s (530 ft^3/s), followed by a 250-year natural ventilation with an air flow rate of 3 m^3/s (106 ft^3/s) for the first 50 years and 1.5 m^3/s (53 ft^3/s) for the remaining 200 years. This LTOM is not considered for the LA design.

A design constraint related to thermal load is the limit of drift wall temperature of 96°C (205°F) during the preclosure period (Williams, 2002). This constraint can be used as the upper bound of thermal load in design calculations.

3.5.3 Seismic Loads

A seismic event associated with either an earthquake or an underground nuclear weapons testing event generates elastic waves that propagate outward from the source. Body waves may be classified as P (dilational or compressional) waves that consist of alternating compression and tension in the transmission medium or S (shear or distortional) waves that consist of oscillating shears. Distortional ground motions are typically resolved into S_H waves with horizontal particle motions and S_V waves with motions in a vertical plane. Both shear motions are orthogonal to the incident wave direction. Particle motions resulting from S_V waves are in a vertical plane but not necessarily in a vertical direction. P waves have an inherently higher velocity of propagation than S waves and will always arrive first at the drift location.

The elastic waves from a seismic event induce transient stresses and strains as well as shaking and vibratory motions in the rock mass and any embedded structures such as tunnels and their lining or reinforcement systems. The effects of ground motions on the rock mass and the underground structures will depend on a number of parameters, including the physical properties of the rock and the underground structures, and the amplitude, frequency, and duration of the ground motions.

In contrast to surface structures, underground structures are constrained by the surrounding medium and do not move independently of the surrounding rock. In reality, the underground structures display significantly greater degrees of redundancy due to the confinement from the ground compared to surface structures, which are generally unsupported above their foundation. Therefore, for underground openings, the surrounding rock acts as a means of support to the engineered components such as steel sets or rock bolts during a seismic event. This implies that the design of ground support related to seismic loads requires a ground-structure interaction analysis.

Since the seismically induced ground motions are transient, a fully dynamic analysis is generally required when the seismic loads are considered. Computational efforts associated with a dynamic analysis are significant. Providing that dominant wavelengths of a seismically induced ground motion are much larger than the drift sizes, seismic effects may be assessed based on quasi-static methods because dynamic amplification will be small if the wavelengths are at least eight times the drift diameters (Hendron and Fernandez 1983, p. 160). In general, a minimum wavelength for both seismic events and underground nuclear weapon testing events is about 100 m (Hardy and Bauer 1991, Section 5.1.3), or about eighteen times the emplacement drift

diameter. Therefore, the application of quasi-static analyses is limited to assessing the seismic effects on drift stability and ground support when the sizes of rock falls are not the problem of interest of an analysis since it is difficult, using the quasi-static approach, to quantify the seismic effects on rock falls.

3.5.3.1 Ground Motion Description

Seismically induced ground motions are defined in terms of horizontal and vertical response spectra for application at repository depths. These frequency-domain design ground motions will be augmented by time histories that are consistent with the earthquakes identified to be controlling the seismic hazard at the reference probabilities of exceedance. For underground openings at the repository host horizon, ground motions due to body waves [compression/tension (P) and shear (S)] will be considered; shear (S) waves, in particular, are the leading cause of structural damage. For design, the potential effects of propagation orientation (angle of incidence) should be considered.

As an alternative to frequency domain analysis, an appropriate real or synthetic time history for motion at repository level may be used.

Depending on the method selected, a seismic analysis usually requires the information of peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD), frequency and duration of ground motions, and time histories of acceleration, velocity, and displacement.

For a fully dynamic approach, the complete site-specific time histories of design basis ground motions are required. Total duration of time histories generated for any particular mean annual probability is fairly long in terms of dynamic time. The strong motion portion of these time histories, which covers 5 to 95 percent energy levels, however, is quite short in many cases. The effects caused by vibratory ground motions beyond the strong motion portion are, in general, insignificant. Therefore, it is recommended the 5 to 95 percent energy portion of a strong motion time history be used for the design analysis.

For a quasi-static approach, the PGA values from the site-specific design basis ground motions are required. The PGA values are converted into additional body forces, much like extra gravitational forces, and are instantly applied to the rock mass under consideration.

3.5.3.2 Repetitive Seismic Loading

A design concern is the repetitive seismic loading scenario for preclosure design (i.e., the number of repetitions of the design earthquake) (YMP 1997, Section 3.3.2.2). A related concern is the experimental evidence that joints weaken during repetitive forward and reverse shearing motion (YMP 1997, Section 3.3.3.2) (i.e, a decrease in shear strength and dilation during reverse shearing.) This effect might become cumulative with multiple earthquakes, especially when jointed rock mass is considered.

3.5.4 Load Combinations

The design analysis of ground support for repository drifts should consider the following combinations of loads:

- In situ
- In situ plus seismic
- In situ plus thermal
- In situ plus thermal plus seismic

For linear response in the rock mass and the ground support, the stresses from each load source can be combined using the superposition principle. If any component of the system responds nonlinearly, an approach based on a step sequence of loading should be used.

3.5.5 Safety Factors

The safety factor (SF) is defined as the assumed material strength divided by the computed stress. It is difficult to determine the SF for an underground opening since the SF for the rock mass can only be calculated on a point by point basis and accepted methods for integrating such information to obtain a single value representative of the opening do not exist. Hence, focus of the design for the underground opening is on the engineered components intended to ensure stability.

For the design of underground ground support components, there is no universally accepted standard for selecting the SF. The practice used in the design of structural steel and concrete should serve as guidance. The selected SF for the design of ground support components should also reflect the variability of the materials used with respect to strength and other physical properties, the nature of the loads, uncertainties associated with the long-term service requirements, and the consequence which might result from failure.

The SF of an underground structural steel or concrete component is judged by comparing the maximum computed stress to the allowable stress of the material. In design of structural steel components, the maximum allowable stress is specified as 60 percent of the yield strength of the steel based on the guideline of the American Institute of Steel Construction (AISC) (1997, p. 5-46). This number gives a SF of 1.7. For design of structural concrete or shotcrete members, the allowable stress for flexure is 45 percent of the specified compressive strength at 28 days using the working stress theory for reinforced concrete beams (ACI 318-99 1999, Appendix A.3.1). This is a SF of 2.2.

The SF values recommended for the design of ground support components for the emplacement drifts are listed in Table 3-3 (Hardy and Bauer 1991, Table 4-3). The steel refers to the steel in steel sets, rock bolts, plates, straps, channels, or shotcrete/concrete reinforcement and any structural steel that might be specified. The SF is higher for shotcrete/concrete components because of the higher variability in the strength of concrete as compared to steel. The SF is lower for the load combination with seismic loads due to the following two factors: (1) the

seismic loads are dynamic in nature and the dynamic strength of a material is generally higher than a "static" strength; and (2) the earthquakes that result in the seismic loads might occur at a very low frequency, and use of a relatively low SF is appropriate for such low frequency events to avoid overdesign.

The SFs recommended in this report reflect neither the requirements for long-term service life of ground support components nor the uncertainties associated with the determination of magnitude of loads, such as seismic, and load combinations. The longevity of ground support can be considered during the selection of materials and material specifications, while the uncertainties of loads can be addressed by using the bounding cases in design analyses.

Table 3-3. Recommended Safety Factors for Design of Ground Support Components

Load Type	Emplacement Drifts	
	Steel	Concrete/Shotcrete
In Situ + Thermal	1.7	2.3
In Situ + Thermal + Seismic	1.4	1.8

Source: Hardy and Bauer 1991, Table 4-3.

4. ROCK MASS CHARACTERISTICS

The characteristics of a rock mass define its key parameters such as deformation modulus, strength properties, and failure mechanisms under loads. Lithophysal and non-lithophysal rocks may behave differently under similar loading conditions. Their characteristics need to be defined differently.

4.1 LITHOPHYSAL ROCKS

Lithophysal rocks contain many random voids with various sizes varying from about a few centimeters to over a meter. Characteristics of the lithophysal rocks are strongly affected by the presence of these voids, and their sizes and distributions. In addition, the lithophysal rocks also contain abundant natural, narrowly-spaced, short length fractures that interconnect lithophysae. Traditionally used empirical approaches based on rock mass classification systems cannot reasonably represent these unique features of the lithophysal rocks and hence are not applicable. A site-specific constitutive model is desirable.

To develop the site-specific constitutive model, sufficient data collected from laboratory and field tests are needed. At present, however, data from tests on this type of rock are limited. Even though additional tests are planned or ongoing, it is not feasible to develop a general statistically-based site-specific approach. This kind of approach usually requires data collected from sufficient number of tests based on samples large enough to capture the basic deformation and failure mechanism of lithophysal rocks with large cavities. Therefore, numerical tests based on computer codes, such as the Particle Flow Code (PFC), can be used to supplement the testing program to examine the effects of parameters such as lithophysal porosity, shape, size, and distribution on the rock mass failure and deformation mechanisms. A constitutive model can then be developed to represent the lithophysal rocks for subsequent ground support design efforts. The study on the calibration of a constitutive model using PFC is an ongoing project, and details on this will be provided in the future by other project documents.

Once the site-specific constitutive model is defined, the deformation and strength parameters of lithophysal rocks, such as deformation modulus, cohesion, friction angle, and dilation angle, can be derived. These parameter values represent an equivalent homogeneous and continuum medium for the lithophysal rocks, and can be applied to numerical models using FLAC or UDEC to analyze the behavior of emplacement drifts subjected to in-situ, thermal, and seismic loads.

The choice of a numerical method, either a continuum-based code such as FLAC or a discontinuum-based code such as UDEC, for an analysis depends on the objective of the analysis. If the analysis is to estimate the size of rock falls, a discontinuum approach based on UDEC should be used, while for evaluation of the overall stability of an emplacement drift and the performance of ground support with a limited interest in the size of rock falls, a continuum approach using FLAC is appropriate.

4.2 NON-LITHOPHYSAL ROCKS

The non-lithophysal rocks are typical strong, hard volcanics that are fractured with various degrees of extensity. There are three dominant fracture sets, two subvertical and one

subhorizontal, with additional random fractures of moderate dip. The fractures are non-persistent in nature with mean spacings ranging from a few centimeters to over several meters, and mean trace lengths of about a few meters (DTN: GS990408314224.001). Characteristics (or behavior) of the non-lithophysal rocks are primarily influenced by the joint patterns and strength, not by the characteristics of the rock blocks formed by the joint patterns.

For the non-lithophysal rock, widely used empirical approaches based on rock mass classification systems are applicable for initial estimation of rock support. These approaches can provide not only an estimate of the rock mass properties and failure mechanisms, but also a preliminary assessment of the needs for ground support. For the analysis of the response of ground support under thermal and seismic loading, either a two- or three-dimensional discontinuum numerical approach will be required.

To fully characterize the non-lithophysal rocks, mechanical properties of joints, such as cohesion, frictional angle, and dilation angle, are also required. These properties can be obtained primarily from field and laboratory tests. Though a large number of test data is available for use, additional laboratory tests are planned or ongoing to collect more data in order to build sufficient confidence in the data selected for ground support design. Additionally, field mapping of fractures is used for empirical estimates of joint shear response.

Once required data are collected, a representative joint model needs to be developed. This joint model should include dominant joint sets that are expected to have significant impact on the stability of emplacement drifts and the performance of ground support. The representative joint model can be generated using either the UDEC, 3DEC, or FracMAN computer code, depending on the conditions to be evaluated. The joint model can then be included in numerical analysis based on UDEC or 3DEC for assessment of the stability of emplacement drifts and the performance of ground support.

4.3 RECOMMENDED CONSTITUTIVE MODELS FOR DESIGN

Based on the discussion provided in Sections 4.1 and 4.2, the constitutive models for the design of ground support for emplacement drifts in both lithophysal and non-lithophysal rocks are recommended as listed in Table 4-1. Since the study on the characteristics of the lithophysal rock is still ongoing, recommendation of the constitutive model of this rock will be provided in the future. For the non-lithophysal rock, use of the Mohr-Coulomb yield criterion for both joints and rock blocks formed by the joint patterns is appropriate because it is widely accepted in the areas of mining and geotechnical engineering. In addition to the Mohr-Coulomb yield criterion, the Barton-Bandis joint model is also recommended for joints since this model can describe in more detail the effects of surface roughness on joint deformation and strength.

Table 4-1. Recommended Constitutive Models for Different Types of Rock

Type of Rock		Constitutive Model
Lithophysal Rock		To be determined
Non-lithophysal Rock	Rock Blocks	Mohr-Coulomb Yield Criterion
	Joints	Mohr-Coulomb Yield Criterion/Barton-Bandis Joint Model

5. PRELIMINARY DESIGN ANALYSIS OF GROUND SUPPORT

Both empirical and numerical methods may be employed in the preliminary design analysis. The empirical methods are primarily tools for assessing the needs for initial ground support of emplacement drifts. They may also be used to develop preliminary estimates of final ground support system(s) to be used. Design issues such as personnel safety, constructibility, and geologic mapping requirements should be factored into the design of the ground support system at this stage. Then, with the aid of computer modeling, the stability of the unsupported opening should be further assessed and the recommended ground support system analyzed. Applicable thermal and seismic loads should be considered in the design in addition to the in situ loading conditions. Based on empirical estimates, design issues, and computer modeling results, the final ground support system should be developed. The ground support system recommended should either cover most ground conditions or else be adaptable to varying ground conditions. The selection or adaptation of a particular approach or support should be coordinated with the A/E representative in the field and documented in accordance with applicable procedures as required.

Use of the empirical and numerical methods for assessing the stability of unsupported drifts and needs of ground support are presented in Sections 5.1 and 5.2, respectively.

5.1 EMPIRICAL METHODS

As stated above, empirical methods are commonly used in the preliminary design of ground support, especially when numerical analyses cannot sufficiently or accurately account for all the variables needed to capture the non-uniform, complex behavior of the rock mass. Also, empirical methods may account for geologic complexity which will not be known until excavation or advance drilling is conducted. Empirical methods are also used when sufficient prior experience already exists and/or for projects where the cost of numerical analyses is not warranted. The application of empirical methods is, however, typically limited to estimates of ground support needs under in situ stress loading conditions. To appropriately assess the stability of unsupported drifts and their needs for ground support under combined in situ, thermal, and seismic loads, numerical methods should be used.

Empirical methods are particularly applicable to the non-lithophysal rocks because of their characteristics primarily being controlled by the fracture patterns and strength, as mentioned in Section 4.2.

The empirical methods for ground support selection are developed by relating the key parameters that affect stability to the support used successfully in existing tunnels and caverns. The methods are used in design to accomplish the following functions:

- Quantify rock quality by an index that combines different characteristics of a rock mass.
- Assess stability of a given opening span, either by indicating the estimated standup time or by indicating for what span ground support is required.
- Conduct preliminary assessment of ground support requirements for a given size opening.

- Estimate rock mass mechanical properties such as rock mass modulus or strength.

The empirical methods used in the preliminary design analysis should be adjusted to account for experience and data accumulated during construction of the Exploratory Studies Facility (ESF) and the Enhanced Characterization of the Repository Block Drift (ECRB) (CRWMS M&O 1997b; 1997c).

The following empirical ground support selection methods can be used, as appropriate:

- Rock Mass Rating (RMR) classification system (also referred to as the Geomechanics Classification System) (Bieniawski 1989)
- Rock Mass Quality (Q) system of Norwegian Geotechnical Institute (NGI) (Barton, Lien, and Lunde 1974)

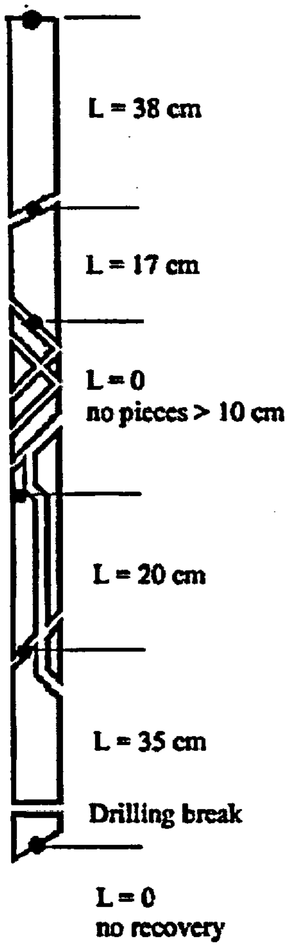
The RMR system is useful for obtaining estimates of elasticity and strength parameters for a rock mass and for estimating stand-up time for underground openings. For selection of ground support components, however, the method gives guidance only for rock bolts, shotcrete, and steel sets for 10-meter wide horseshoe shaped tunnels.

The Q system is much more sensitive and flexible than the other procedure, covers a much wider range of conditions and is based on a broader range of documented experiences. It can be adapted to give estimates of elasticity and strength parameters and stand-up times by correlation of Q and RMR values. Thus, this should be the primary system used in designing initial support.

An important component of the RMR and Q rock mass classification systems is the Rock Quality Designation index (RQD), developed by Deere (Deere and Deere 1988). The following three subsections discuss the application of RQD, RMR and Q systems in detail.

5.1.1 RQD Index

The RQD index provides a quantitative estimate of rock mass quality from drill core logs. It is defined as the percentage of intact core pieces longer than 100 mm (4 inches) in the total length of core from a given core run. The core should preferably be 54.7 mm or 2.15 inches in diameter and should be drilled with a double-tube core barrel (Deere and Deere 1988). The correct procedures for measurement of the length of core pieces and the calculation of RQD are summarized in Figure 5-1.



Total length of core run = 200 cms

$$RQD = \frac{\Sigma \text{ Length of core pieces} > 10 \text{ cm length}}{\text{Total length of core run}} \times 100$$

$$RQD = \frac{38 + 17 + 20 + 35}{200} \times 100 = 55\,\%$$

Source: After Deere and Deere 1988, Figure 1.

Figure 5-1. Procedure for Measurement and Calculation of RQD

When no core is available but discontinuity traces are visible in surface exposures or exploration adits, the RQD may be estimated from the number of discontinuities per unit volume. The relationship for clay-free rock masses suggested by Palmström (Hoek et al. 2000, Eq. 4.1, p. 30) is:

$$RQD = 115 - 3.3J_v \qquad \text{(Eq. 5-1)}$$

where J_v = sum of the number of joints per unit length for all joint sets known as the volumetric joint count

RQD is a directionally-dependent parameter and its value may change significantly, depending upon the borehole orientation. The use of the volumetric joint count can be quite useful in reducing this directional dependence. RQD is intended to represent the rock mass quality in situ. When using diamond drill core, care must be taken to ensure that fractures, which have been caused by handling or the drilling process, are not accounted for when determining the value of

RQD. When using Palmström's relationship for exposure mapping, blast induced fractures should not be included when estimating J_v.

The use of RQD for the repository ground support design is primarily as a component of the RMR and Q rock mass classification systems to be covered below.

5.1.2 RMR System

The RMR system was developed by Bieniawski (1989). This engineering classification of rock masses, especially evolved for rock engineering applications, provides a general rock mass rating (RMR) increasing with rock quality from 0 to 100. It is based upon the following six parameters:

- strength of the rock
- drill core quality or RQD
- joint and fracture spacing
- joint conditions
- ground water conditions
- orientation of joints

These parameters not only are measurable in the field but can also be obtained from borings. Joints are the major factor in this classification system; four of the six parameters (RQD, joint spacing, joint conditions, and orientation of joints) are related to joint characteristics. Increments of rock mass rating corresponding to each parameter are summed to determine RMR.

The RMR values for various rock units at the repository host horizon are generally available from the ESF and ECRB. Hence, the rock mass quality for each rock unit considered can be judged based on the guidelines provided in Table 5-1 (Bieniawski 1989, Tables 4.1 and 4.2). The stand-up time and the maximum stable rock span for a given RMR can be estimated using Figure 5-2 (Bieniawski 1989, Figure 4.2). Recommendation for the excavation scheme and initial rock support can be made by following the guidelines presented in Table 5-2 (Bieniawski 1989, Table 4.4).

Details on how to apply the RMR classification system to the preliminary design of ground support in repository emplacement drifts can be found in the *Engineering Rock Mass Classifications* (Bieniawski 1989).

Table 5-1. RMR System

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/(Major principal σ)	0	< 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25	< 15

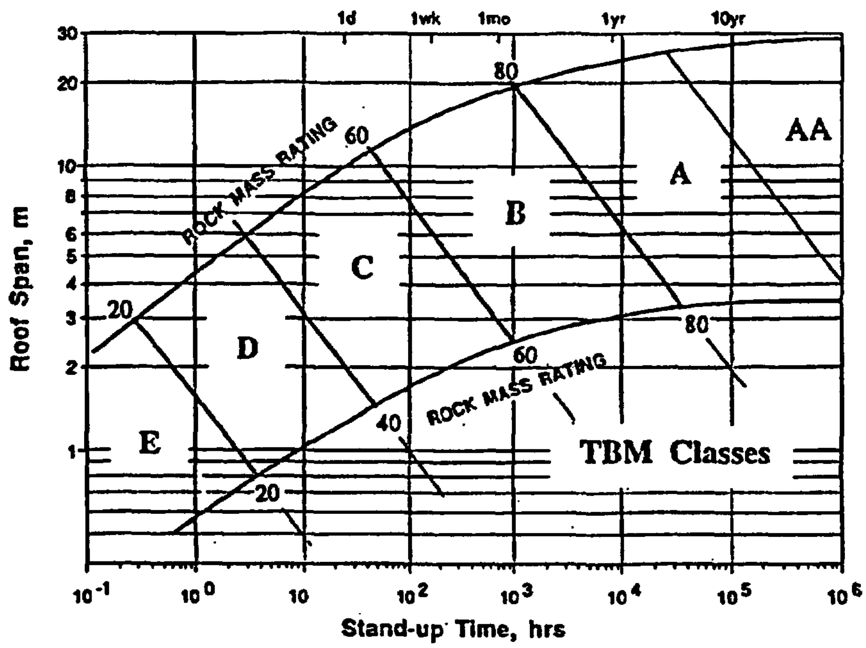
E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

Discontinuity length (persistence) Rating	< 1 m 6	1 - 3 m 4	3 - 10 m 2	10 - 20 m 1	> 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1 - 1.0 mm 4	1 - 5 mm 1	> 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard filling < 5 mm 4	Hard filling > 5 mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0
Weathering Ratings	Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
Fair	Unfavourable	Fair	

Source: Bieniawski 1989, Tables 4.1 and 4.2.



Source: Bieniawski 1989, Figure 4.2.

Figure 5-2. Geomechanics Classification of Rock Masses for TBM Applications

Table 5-2. RMR System

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
I - Very good rock *RMR*: 81-100	Full face, 3 m advance.	Generally no support required except spot bolting.		
II - Good rock *RMR*: 61-80	Full face, 1-1.5 m advance. Complete support 20 m from face.	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.	50 mm in crown where required.	None.
III - Fair rock *RMR*: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5 - 2 m in crown and walls with wire mesh in crown.	50-100 mm in crown and 30 mm in sides.	None.
IV - Poor rock *RMR*: 21-40	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.	100-150 mm in crown and 100 mm in sides.	Light to medium ribs spaced 1.5 m where required.
V – Very poor rock *RMR*: < 20	Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.	150-200 mm in crown, 150 mm in sides, and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

Source: Bieniawski 1989, Table 4.4.

5.1.3 Q System

The Q system, developed in Norway by Barton, Lien, and Lunde (1974), provides for the design of rock support for tunnels and large underground chambers. The system utilizes the following six factors:

- RQD
- Number of joint sets
- Joint roughness
- Joint alteration
- Joint water condition
- Stress condition

The factors are combined in the following way to determine the rock mass quality (Q) as,

$$Q = \left(\frac{RQD}{J_n}\right)\left(\frac{J_r}{J_a}\right)\left(\frac{J_w}{SRF}\right) \qquad \text{(Eq. 5-2)}$$

where
RQD = rock quality designation
J_n = joint set number
J_r = joint roughness number
J_a = joint alteration number
J_w = joint water reduction factor
SRF = stress reduction factor (dependent on loading conditions)

The three ratios in the equation - RQD/Jn, Jr/Ja, and Jw/SRF - represent block size, minimum inter-block shear strength, and active stress, respectively. Table 5-3 gives the classification of individual parameters used to obtain the Index Q for a rock mass.

The Q index is used with the Equivalent Dimension, defined as the largest of span, diameter, and height divided by the excavation support ratio (ESR). ESR is roughly analogous to the inverse of the factor of safety used in engineering design. The ESR reflects the degree of safety and ground support required for an excavation as determined by the purpose, presence of machinery, personnel, etc., to meet safety requirements. In essence, the safety factor of an opening can be increased by reducing the ESR value. The ESR values for various underground openings can be estimated based on Barton et al. (1974, Table 7). As recommended by Hardy and Bauer (1991, Section 12.7.1) an ESR value of 1.3 is appropriate for the Yucca Mountain repository design.

The Equivalent Dimension is plotted against Q on the design chart (Figure 5-3) to determine the required rock support category (After Barton 2002, Figure 1). Thermal or seismic loads can be included in an implicit way, by increasing the stress reduction factor, thereby requiring a higher degree of support.

Table 5-3. Q System Description and Ratings

DESCRIPTION	VALUE	NOTES	
1. ROCK QUALITY DESIGNATION	*RQD*		
A. Very poor	0 - 25	1. Where *RQD* is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q.	
B. Poor	25 - 50		
C. Fair	50 - 75		
D. Good	75 - 90	2. *RQD* intervals of 5, i.e. 100, 95, 90 etc. are sufficiently accurate.	
E. Excellent	90 - 100		
2. JOINT SET NUMBER	J_n		
A. Massive, no or few joints	0.5 - 1.0		
B. One joint set	2		
C. One joint set plus random	3		
D. Two joint sets	4		
E. Two joint sets plus random	6		
F. Three joint sets	9	1. For intersections use $(3.0 \times J_n)$	
G. Three joint sets plus random	12		
H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc.	15	2. For portals use $(2.0 \times J_n)$	
J. Crushed rock, earthlike	20		
3. JOINT ROUGHNESS NUMBER	J_r		
a. Rock wall contact			
b. Rock wall contact before 10 cm shear			
A. Discontinuous joints	4		
B. Rough and irregular, undulating	3		
C. Smooth undulating	2		
D. Slickensided undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.	
E. Rough or irregular, planar	1.5		
F. Smooth, planar	1.0		
G. Slickensided, planar	0.5	2. $J_r = 0.5$ can be used for planar, slickensided joints having lineations, provided that the lineations are oriented for minimum strength.	
c. No rock wall contact when sheared			
H. Zones containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)		
J. Sandy, gravely or crushed zone thick enough to prevent rock wall contact	1.0 (nominal)		
4. JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
a. Rock wall contact			
A. Tightly healed, hard, non-softening, impermeable filling	0.75		1. Values of ϕr, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
B. Unaltered joint walls, surface staining only	1.0	25 - 35	
C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25 - 30	
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	20 - 25	
E. Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 - 2 mm or less)	4.0	8 - 16	

Source: After Barton et al 1974.

Table 5-3. Q System Description and Ratings (Continued)

DESCRIPTION	VALUE	NOTES	
4. JOINT ALTERATION NUMBER	J_a	ϕ_r degrees (approx.)	
b. Rock wall contact before 10 cm shear			
F. Sandy particles, clay-free, disintegrating rock etc.	4.0	25 - 30	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick)	6.0	16 - 24	
H. Medium or low over-consolidation, softening clay mineral fillings (continuous < 5 mm thick)	8.0	12 - 16	
J. Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water.	8.0 - 12.0	6 - 12	
c. No rock wall contact when sheared			
K. Zones or bands of disintegrated or crushed	6.0		
L. rock and clay (see G, H and J for clay	8.0		
M. conditions)	8.0 - 12.0	6 - 24	
N. Zones or bands of silty- or sandy-clay, small clay fraction, non-softening	5.0		
O. Thick continuous zones or bands of clay	10.0 - 13.0		
P. & R. (see G.H and J for clay conditions)	6.0 - 24.0		
5. JOINT WATER REDUCTION	J_w	approx. water pressure (kgf/cm^2)	
A. Dry excavation or minor inflow i.e. < 5 l/m locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates; increase J_w if drainage installed.
D. Large inflow or high pressure	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure	0.1 - 0.05	> 10	
6. STRESS REDUCTION FACTOR		*SRF*	
a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated			
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth)		10.0	1. Reduce these values of *SRF* by 25 - 50% but only if the relevant shear zones influence do not intersect the excavation
B. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth < 50 m)		5.0	
C. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth > 50 m)		2.5	
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)		7.5	
E. Single shear zone in competent rock (clay free). (depth of excavation < 50 m)		5.0	
F. Single shear zone in competent rock (clay free). (depth of excavation > 50 m)		2.5	
G. Loose open joints, heavily jointed or 'sugar cube', (any depth)		5.0	

Source: After Barton et al 1974.

Table 5-3. Q System Description and Ratings (Continued)

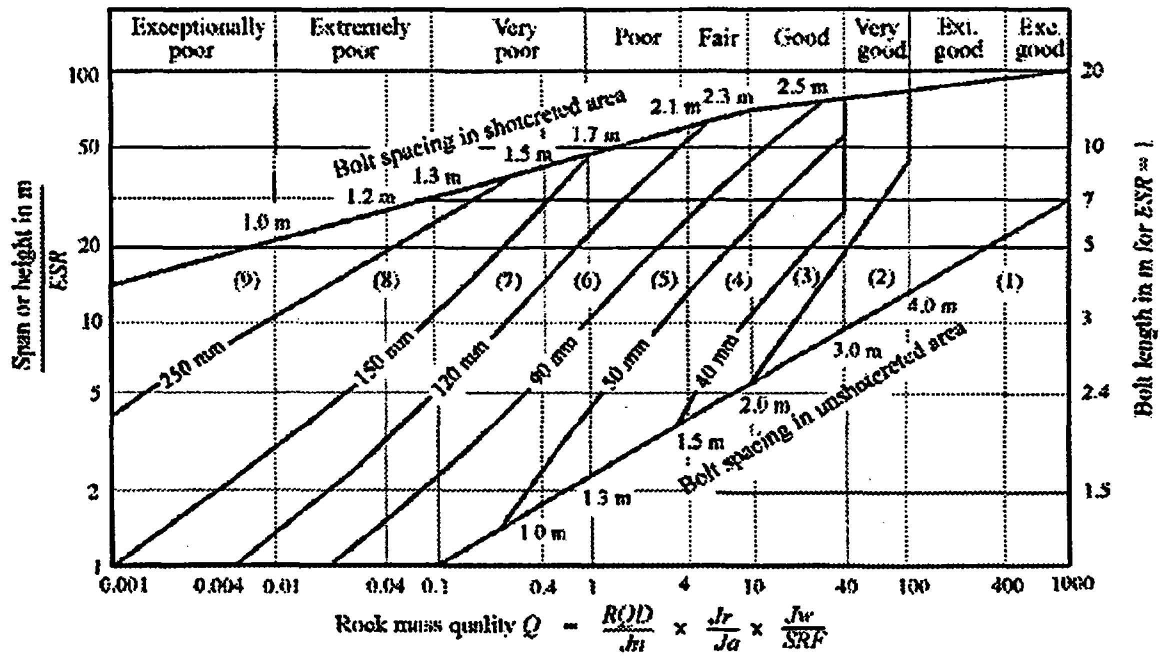
DESCRIPTION	VALUE			NOTES
6. STRESS REDUCTION FACTOR			*SRF*	
b. Competent rock, rock stress problems				
	σ_c/σ_1	$\sigma_t\sigma_1$		2. For strongly anisotropic virgin stress field (if measured): when $5 \le \sigma_1/\sigma_3 \le 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
H. Low stress, near surface	> 200	> 13	2.5	
J. Medium stress	200 - 10	13 - 0.66	1.0	
K. High stress, very tight structure (usually favourable to stability, may be unfavourable to wall stability)	10 - 5	0.66 - 0.33	0.5 - 2	
L. Mild rockburst (massive rock)	5 - 2.5	0.33 - 0.16	5 - 10	
M. Heavy rockburst (massive rock)	< 2.5	< 0.16	10 - 20	3. Few case records available where depth of crown below surface is less than span width. Suggest *SRF* increase from 2.5 to 5 for such cases (see H).
c. Squeezing rock, plastic flow of incompetent rock under influence of high rock pressure				
N. Mild squeezing rock pressure			5 - 10	
O. Heavy squeezing rock pressure			10 - 20	
d. Swelling rock, chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure			5 - 10	
R. Heavy swelling rock pressure			10 - 15	

ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass Quality (Q), the following guidelines should be followed in addition to the notes listed in the tables:

1. When borehole core is unavailable, *RQD* can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relationship can be used to convert this number to *RQD* for the case of clay free rock masses: $RQD = 115 - 3.3\,J_v$ (approx.), where J_v = total number of joints per m^3 ($0 < RQD < 100$ for $35 > J_v > 4.5$).
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed, these parallel 'joints' should obviously be counted as a complete joint set. However, if there are few 'joints' visible, or if only occasional breaks in the core are due to these features, then it will be more appropriate to count them as 'random' joints when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of J_r/J_a is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating Q. The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.
4. When a rock mass contains clay, the factor *SRF* appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to the present and future in situ conditions. A very conservative estimate of the strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

Source: After Barton et al 1974.



REINFORCEMENT CATEGORIES
1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting with 40-100 mm unreinforced shotcrete
5) Fibre reinforced shotcrete, 50 - 90 mm, and bolting
6) Fibre reinforced shotcrete, 90 - 120 mm, and bolting
7) Fibre reinforced shotcrete, 120 - 150 mm, and bolting
8) Fibre reinforced shotcrete, > 150 mm, with reinforced ribs of shotcrete and bolting
9) Cast concrete lining

Source: After Barton 2002, Figure 1.

Figure 5-3. Estimated Ground Support Categories Based on Q Index

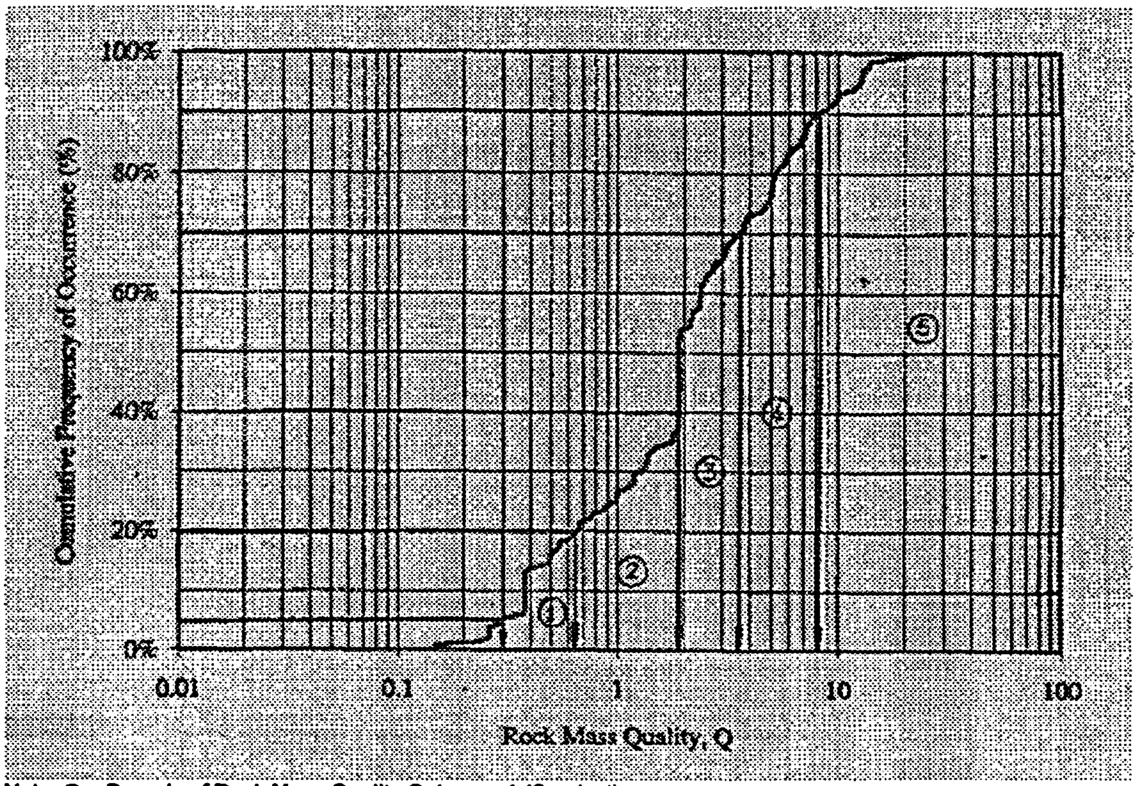
5.1.4 Rock Quality Categories

A given Q (or RMR) value is an indicator of rock quality at a specific location in the rock mass. For a length of tunnel where the variation in Q values is small, the minimum value may be used for determination of rock support required. In other cases, Q values should be plotted against cumulative frequency of occurrence, as shown in Figure 5-4. This type of plot gives an approximation of the distribution of rock quality values to be expected throughout the repository host horizon. In addition the plot can be used to estimate the frequency at which various types of ground support will be required. Following the *Drift Design Methodology and Preliminary Application for the Yucca Mountain Site Characterization Project* (Hardy and Bauer 1991, Section 12), rock quality categories 1 through 5 can be defined by projecting the cumulative frequencies of occurrence of 5, 20, 40, 70 and 90 percent, respectively, to the curve and then to the Q-value axis to give bounding values for the quality categories. In the case shown in Figure 5-4, the bounds and their interpretations are provided in Table 5-4.

This gives a reasonable breakdown of the distribution of quality levels, though a different breakdown could be used as well. Obviously, this overall distribution does not identify conditions by location, but it can be used for planning and estimating purposes.

Once the rock quality category is determined, the rock mass properties, such as deformation modulus, corresponding to each category, can be estimated based on the Q value.

The rock quality categories are particularly useful for the non-lithophysal rocks. Primary application of the concept of rock quality categories is to define the bounding values of rock mass properties of the non-lithophysal rocks. With these bounding values, the associated ground support can be estimated. The selected single type of ground support should be able to accommodate all rock conditions anticipated.



Note: ① = Bounds of Rock Mass Quality Category 1 (See text).

Figure 5-4. Cumulative Distribution of Rock Mass Quality (Q) Values for Typical Tuff

Table 5-4. Rock Mass Quality Categories and Their Bounding Values

Rock Mass Quality Category	Cumulative Frequencies of Occurrence	Bounding Q Values		Interpretation of Limits
		Lower Bound	Upper Bound	
1	5	0.30	0.65	20% minus 5% = 15% of the rock has quality between 0.30 and 0.65
2	20	0.65	1.91	40% minus 20% = 20% of the rock has quality between 0.65 and 1.91
3	40	1.91	3.75	70% minus 40% = 30% of the rock has quality between 1.91 and 3.75
4	70	3.75	8.44	90% minus 70% = 20% of the rock has quality between 3.75 and 8.44
5	90	8.44	∞	100% minus 90% = 10% of the rock has quality > 8.44

5.2 NUMERICAL METHODS

Evaluation of the behavior of rock mass and stability of drifts often relies on numerical methods due primarily to the complex nature of the loads encountered. Closed-form solutions to most of the problems to be considered are either non-existent or too idealized. Empirical methods discussed in Section 5.1 are generally limited to underground openings subjected to in situ stress load. They may only provide a preliminary assessment of the conditions.

Use of numerical methods to simulate the behavior of unsupported emplacement drifts is generally considered as one of the important steps during the analysis process for ground support design for this project. Usually, the presence of ground support, such as rock bolts or steel sets, in a drift excavated in a hard or competent rock has a minimal impact on the predicted deformation and stress of rock mass in the vicinity of a drift. Modeling an unsupported drift is valuable for understanding the fundamental behavior of the drift subjected to various loading conditions without the effects of incorporating the ground support into the model.

As mentioned in Section 3.2, the emplacement drifts will be excavated in mainly two different types of rocks, lithophysal and non-lithophysal. The drift response to anticipated loads is expected to depend on the characteristics of rock where the drifts are located. The approaches used for design analysis should reflect this difference. For this reason, the numerical methods for drifts in lithophysal and non-lithophysal rocks will be discussed in the following two separate subsections.

5.2.1 Analysis for Drifts in Lithophysal Rocks

The application of numerical methods to assessing the stability of repository drifts in the lithophysal rock depends on the rock mass structure, material properties, and failure mechanisms. As discussed in Section 4.1, the characteristics of this type of rocks are primarily controlled by the presence of lithophysae and the short fractures interconnecting lithophysae, particularly in the lower lithophysal unit. Joint sets with a spacing of over two meters also exist, but are not felt to be a key factor that governs the deformation and failure mechanism. It is, therefore, appropriate to use an equivalent two-dimensional continuum approach based on equivalent rock mass properties derived from a constitutive model representing the lithophysal

rocks. If quantitative analysis of rock falls in emplacement drifts is performed, a discontinuum approach is required since the continuum approach will not generate the information needed.

In the equivalent continuum approach, the rock structural features such as lithophysae and short-length fractures are not modeled explicitly. The effects of these features are reflected in the material properties and failure mechanisms defined. The behavior predicted based on the numerical models using these properties is considered equivalent to what would be predicted otherwise using the models that would simulate these features explicitly and in great details.

To enhance the confidence in the numerical approach used, a comparative analysis based on the discontinuum approach is recommended. The purpose of this comparative analysis is to examine whether the predicted results from both continuum and discontinuum approaches are similar or equivalent. If not, further investigation on the approaches and input data is required.

Numerical codes available to model the repository drifts in the lithophysal rocks include FLAC and UDEC. FLAC can be used for continuum modeling, while UDEC can be used for discontinuum calculations. Both FLAC and UDEC are for two-dimensional analysis. Two-dimensional models incorporate the appropriate phenomena of the emplacement drifts. This is because the length of these drifts is relatively long compared to the dimensions of their cross-section, and the effects of any out-of-plane joints, perpendicular to the drift, are negligible. The behavior captured by a unit length of a drift that is modeled in the two-dimensional calculations is representative for the entire drift.

5.2.2 Analysis for Drifts in Non-lithophysal Rocks

A primary feature of the non-lithophysal rocks is having fracture sets with a mean spacing of about a few centimeters to several meters, depending on the non-persistent joint set. To appropriately model the behavior of a jointed rock mass, a discontinuum approach is recommended. With the discontinuum approach, interaction between joints and rock blocks which are bounded by joints can be simulated explicitly and in great details. The characteristics of joints, such as joint geometry and frequency, are the direct inputs to numerical models.

Numerical codes available for use include UDEC and 3DEC. UDEC can be used only for two-dimensional modeling, while 3DEC can be used for both two- and three-dimensional modeling. Due to the presence of non-persistent joints in the non-lithophysal rocks, use of the three-dimensional code of 3DEC appears more appropriate, especially when the effects of out-of-plane joints, such as in a study of rock falls, are analyzed. Two-dimensional models, which assume that joints are continuous in the axial direction of a tunnel, may be used as an alternative if the effects of rock falls are not the primary problem of interest of an analysis.

The analyses with the discontinuum approach are usually more complex and time-consuming than those with the equivalent continuum approach, especially if the joint spacings are too small to practically discretize the joints. The equivalent continuum approach may be used as an alternative for the jointed rock mass. The effects of major joint sets can be accounted for by including some ubiquitous joints in the equivalent continuum models.

5.2.3 Analysis Procedures

The procedures for selecting appropriate numerical models for assessing the stability of unsupported drifts are summarized in the flow diagram shown in Figure 5-5. Once a numerical model is selected, the remaining modeling efforts are generally similar, regardless of equivalent continuum or discontinuum modeling, and two-dimensional or three-dimensional modeling. The steps involved in numerical modeling include model construction, load application, and result interpretation. Some of these are described below.

5.2.3.1 Model Construction

The following information is required in order to develop a numerical model:

- Drift geometry, including drift dimensions and drift spacing
- Joint geometry, including spacings, trace lengths, dip angles and directions
- Material properties, including thermal and mechanical
- Initial conditions
- Boundary conditions
- Loading conditions

Determination of what model dimensions, for example lateral and vertical dimensions for a two-dimensional model, are used, depends on the drift geometry, the type of numerical models selected (continuum or discontinuum), and loading conditions (in situ, thermal, or seismic). A general guideline for selecting the model dimensions is provided in Table 5-5. These are recommended based on the general rule-of-thumb used by numerical modelers and experiences gained from numerical analyses related to ground support design for the Yucca Mountain project. Use of a greater vertical dimension in the thermal-only analysis is required in order to minimize the effects of assumed boundary conditions on the model upper and lower boundaries on the temperature distributions in rock. Variations from these values can be justified if specific modeling requirements should be met or computational efficiency is desired, as long as sufficient accuracy is guaranteed.

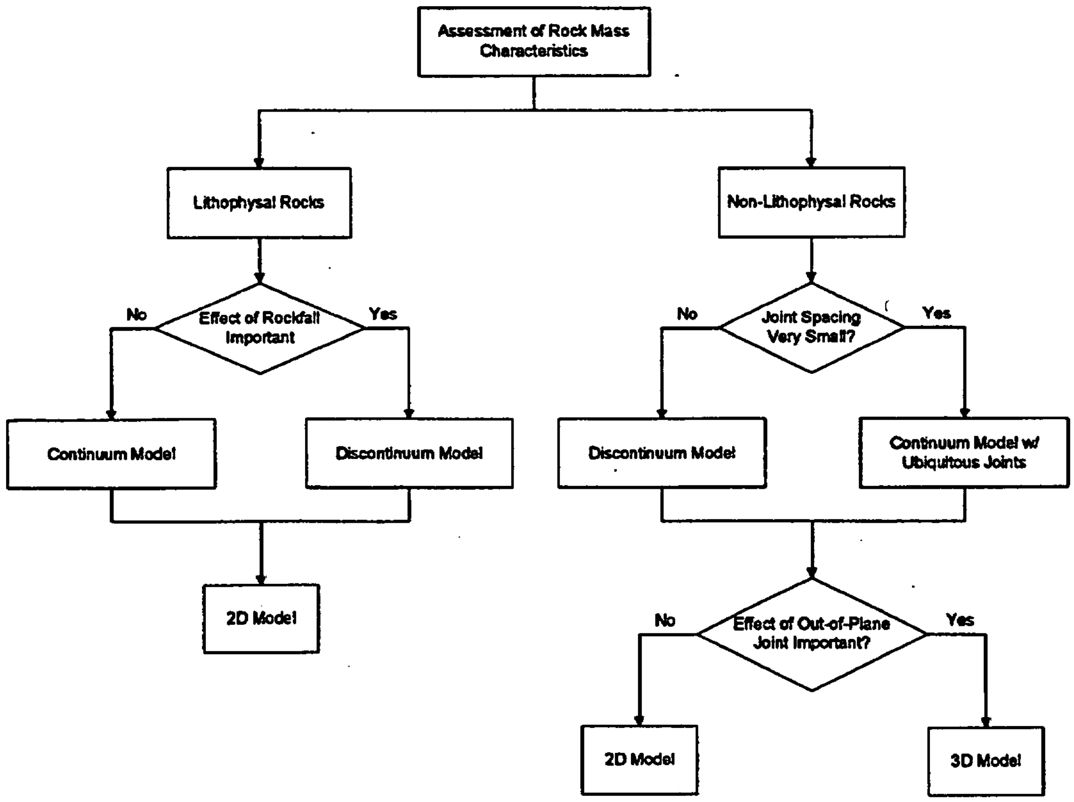


Figure 5-5. Flow Diagram for Numerical Model Selection

Table 5-5. Recommended Dimension Values for Numerical Models

Type of Model		Loading Condition: In Situ Stress, Thermomechanical, or Seismic (Condition II)	Loading Condition: Thermal Only (Condition I)
Continuum	2D	Lateral: 1 × drift spacing Vertical: 100 m (328 ft)	Lateral: 1 × drift spacing Vertical: Drift overburden above + about 300 m (984 ft) below
	3D	Lateral: 1 × drift spacing Vertical: 100 m (328 ft) Axial: To be determined by modeler	Lateral: 1 × drift spacing Vertical: Drift overburden above + about 300 m (984 ft) below Axial: To be determined by modeler
Discontinuum	2D	Lateral: 1 × drift spacing Vertical: 100 m (328 ft)	Lateral: 1 × drift spacing Vertical: Drift overburden + about 300 m (984 ft) below
	3D	Lateral: 1 × drift spacing Vertical: 100 m (328 ft) Axial: To be determined by modeler	Lateral: 1 × drift spacing Vertical: Drift overburden above + about 300 m (984 ft) below Axial: To be determined by modeler

5.2.3.2 Application of In Situ Stress Loads

The effect of in situ stress loads on the behavior of unsupported drifts can be examined by applying the body force and boundary stress on the numerical models constructed. As specified in Table 3-2, the initial vertical stress is equal to 10 MPa (1450 psi) at the center of emplacement drifts, while the lower and upper bounds of the horizontal-to-vertical stress ratio are 0.2 and 1.0, respectively. These in situ stress values will determine the body force components applied to the models. The stresses applied on the upper boundary of the models can be calculated based on the assigned stress at the drift center and the bulk density of the rock mass, using the following equations:

$$p_v = p_0 - \sum_{i=1}^{n} \rho_i g h_i \qquad \text{(Eq. 5-3)}$$

where
where	p_v	=	vertical component of the upper boundary stress, MPa
	p_0	=	vertical component of in situ stress at the drift center, MPa
	ρ_i	=	bulk density of the *i*th layer of rock mass, kg/m^3
	g	=	gravitational acceleration, m/s^2
	h_i	=	thickness of the *i*th layer of rock mass (*i* goes from the upper boundary to the drift center), m

and

$$p_h = k_o p_v \qquad \text{(Eq. 5-4)}$$

where	p_h	=	horizontal component of the upper boundary stress, MPa
	k_o	=	horizontal-to-vertical stress ratio

The approach described here is applicable to continuum and discontinuum, and two- and three-dimensional models.

5.2.3.3 Application of Thermal Loads

Calculations and application of thermal loads require the knowledge of initial rock temperatures, heat generation rates of waste packages, thermal management measures such as ventilation, and rock thermal properties.

Initial rock temperatures are to be calculated using the average rock temperature at the ground surface and the vertical thermal gradients within the rock.

As mentioned in Sections 2.2 and 3.5.2, use of only the HTOM condition is required for the LA design. The HTOM involves an initial linear heat load of 1.45 kW/m (1508.4 Btu/hr·ft), together with a 50-year continuous ventilation at an air flow rate of 15 m^3/s (530 ft^3/s).

There are two ways to apply the thermal loads to a thermomechanical calculation. One is to use a single model for both the thermal and thermomechanical analyses, and the other is to employ two different models for the thermal and the thermomechanical analyses.

Approach One. In the first approach, the vertical dimension of the model is controlled by the thermal calculation, and hence the size of the model is fairly large. The effects of waste package heating and ventilation cooling are modeled explicitly. Heat transfer mechanisms considered include conduction, convection, and radiation. Conductive heat flow occurs within the waste package, and within the surrounding rock whenever there is a thermal gradient. Convective heat transfer occurs between the waste package surface and the ventilating air as well as between the drift wall and the air. Electromagnetic radiation heat transfer occurs directly between the waste package surface and the drift wall.

According to Fourier's law of heat conduction, the general three-dimensional heat conduction equation for a drift can be expressed in Cartesian coordinates as (Holman 1997, Equation 1-3, p. 5):

$$\frac{\partial}{\partial x}\left(k\frac{\partial T}{\partial x}\right)+\frac{\partial}{\partial y}\left(k\frac{\partial T}{\partial y}\right)+\frac{\partial}{\partial z}\left(k\frac{\partial T}{\partial z}\right)+q'''=\rho c\frac{\partial T}{\partial t} \quad \text{(Eq. 5-5)}$$

where
T = temperature, K
t = time, s
k = thermal conductivity, W/m·K
ρ = density, kg/m^3
q''' = heat generation rate per unit volume, W/m^3
c = specific heat, J/kg·K

Fourier's law of heat conduction is usually embedded in the computer code selected such as ANSYS. When a temperature gradient exists in a medium, such as rock or a waste package canister, a heat or energy transfer from the high-temperature region to the low-temperature region is through conduction. The temperature changes caused by conduction are calculated in the code with Fourier's law.

For an air-ventilated drift, the overall effect of convection can be evaluated using Newton's law of cooling (Holman 1997, Equation 1-8, p. 12):

$$q = hA(T_w - T_a) \quad \text{(Eq. 5-6)}$$

where
q = heat flow rate, W
h = convection heat transfer coefficient, W/m^2·K
A = convection surface area, m^2
T_w = drift wall or waste package surface temperature, K
T_a = ventilation air temperature, K

Convection heat transfer occurs at the interface of a solid and a fluid due to a temperature gradient between these two media. In most computer codes, convection heat transfer is treated as a boundary condition. Hence, a convection heat transfer coefficient (h) and a fluid temperature (T_a) are required as inputs.

The heat from the waste packages to the drift wall is transferred mainly through thermal radiation. In the calculations, the waste packages can be assumed to be completely enclosed by the drift wall, so the total radiant exchange can be calculated using the following equation based on the Stefan–Boltzmann law (Holman 1997, Equations 1-9, 1-10, and 1-11, pp. 13-14):

$$q = F_{\varepsilon} F_G \sigma A \left(T_p^4 - T_w^4\right) \qquad \text{(Eq. 5-7)}$$

where

	q	=	heat flow rate, W
	F_{ε}	=	emissivity function, dimensionless
	F_G	=	geometric view factor function, dimensionless
	σ	=	Stefan-Boltzmann constant equal to 5.669×10^{-8} W/m^2·K^4 (0.1714×10^{-8} Btu/hr·ft^2·R^4)
	A	=	radiation surface area, m^2
	T_p	=	absolute temperature of the waste package surface, K
	T_w	=	absolute temperature of the drift wall, K

The Stefan–Boltzmann law is embedded in the computer code selected. In contrast to the mechanisms of conduction and convection, where heat transfer through a material medium is involved, electromagnetic radiant heat exchange occurs without involvement of a material medium. Since the heat transfer due to radiation varies with the fourth power of the surface's absolute temperature, the thermal calculation is highly nonlinear. In a thermal analysis, the radiation heat transfer can be handled by assigning radiation surfaces which involve the radiation heat transfer. The temperature changes contributed by radiation due to a thermal gradient are calculated by the computer code based on the Stefan–Boltzmann law (Equation 5-7).

Since the heat decay from waste packages varies with time, a transient analysis is required for the thermal analysis. In the transient analysis, time-dependent temperatures of the rock are to be calculated for a preclosure period of up to 300 years following final waste emplacement (Curry and Loros 2002, PRD-014/T-006, p. 3-89).

Once the time-dependent temperatures are determined, thermally-induced strains in the rock can be calculated using the following equations

$$\varepsilon_{ii} = \beta \Delta T \qquad \text{(Eq. 5-8)}$$

where

	ε_{ii}	=	normal strain components (i=1,2,3), dimensionless
	β	=	thermal expansion coefficient, 1/K
	ΔT	=	temperature change, K

Since the temperature changes vary with time, the thermally-induced strains and stresses are also time-dependent. A fully-coupled transient thermomechanical analysis is very time consuming, and usually unnecessary. Instead, a quasi-static approach is recommended. With the quasi-static approach, the temperature changes up to the time of interest are first calculated, and then the thermal mode is "turned off" and the mechanical mode is "turned on". These are followed by a thermomechanical analysis to calculate the thermal stresses and strains caused by the temperature changes up to the time of interest modeled. Once the thermal stresses and strains are

determined, the thermal mode is "turned on" again and the mechanical mode is "turned off" to continue the calculations for temperature changes until the next desired time of interest is reached. This process is continued until the time of 300 years is reached.

Due to the size of the thermal model required for this Approach One, it may take a fairly long time to conduct a complete thermal and thermomechanical calculation. Therefore, the second approach, to be described below, is recommended.

Approach Two. In this approach, the temperature calculation for each thermal analysis will be conducted under two conditions, as follows: For Condition I the thermal analysis procedure used in Approach One is employed. This analysis gives the time-dependent distributions of temperatures of Condition II for a period of 300 years. For Condition II the vertical dimension is 100 m, as specified in Table 5-5, and the distributions of temperatures from Condition I are used as inputs to the model for Condition II. The time-dependent temperature inputs can be either the temperatures on the model boundaries or those at the each node. Using this approach it is possible to evaluate the time-dependent near-field temperature distributions within the rock. The latter calculation is necessary to enable the coupled thermomechanical analyses to accurately reflect the near-field stress and displacement distributions affected by the temperature changes.

Though the thermal analysis with this approach is conducted twice, one for Condition I and the other for Condition II, overall computational efforts may still be much less than that for Approach One. The model dimensions for Condition II are much smaller than those for Condition I. This will result in fewer elements, better mesh refinement and aspect ratios in the model, and more importantly the accuracy of results will be improved with a better quality of elements constructed.

The approaches outlined above can be applied to both continuum and discontinuum models. Use of Approach Two is actually more appropriate for the discontinuum model because efforts for constructing joints in a large model can be reduced significantly.

5.2.3.3.1 Modeling of Forced or Natural Ventilation

Evaluation of heat exchange in a ventilated drift is a very complex three-dimensional, time-dependent, and coupled heat and fluid flow problem. The thermal analysis using the ANSYS computer code for example can only handle heat transfer without fluid flow, at least not directly. Convection due to forced ventilation is treated as a boundary condition, which is different from many computational fluid dynamics (CFD) codes. A complete analysis requires the use of a CFD code like FLUENT. However, modeling a transient and coupled heat and mass transfer problem with a CFD code is very time-consuming, and is often not feasible.

To overcome this obstacle, an alternative approach, called the ANSYS ventilation model, is developed by using ANSYS for heat transfer involving conduction, convection, and radiation, and the Excel spreadsheet for fluid energy balance involving convection only. A detailed description on how to use the ANSYS ventilation model for calculating the ventilation efficiencies and temperature distributions is provided in the *Thermal Management Analysis for Lower-Temperature Designs* (BSC 2001a, Sections 6.1.3, 6.2.3.1, and 6.2.3.3).

5.2.3.3.2 Calculation of Convection Heat Transfer Coefficient

A convection heat transfer coefficient is required in the ventilation model. Its value depends on the air flow rate, drift configuration, and heat load (temperature gradient near a convection surface). A simple approach that is based on a correlation for turbulent flow in a smooth annulus is described in the *Thermal Management Analysis for Lower-Temperature Designs* (BSC 2001a, Section 6.2.3.3), and is considered applicable to the design analysis for ground support.

Accurate determination of the effect of ventilation relies on how accurately the convection heat transfer coefficient is evaluated in the ventilation model. Many correlations are available for use. Care should be taken in selection of a correlation since most of them were developed for specific conditions.

It is generally conservative to use a correlation that is applicable to forced convection only when evaluating the convection heat transfer coefficient for flow caused by forced ventilation. The secondary phenomenon, natural convection, usually exists during forced ventilation, and may contribute significantly to cooling on waste packages and drift rock. Ignoring its effect results in a lower convection heat transfer coefficient and a higher temperature.

Uncertainties associated with the use of any correlation are high. To assess the uncertainties, data from the laboratory ventilation tests should be analyzed in evaluation of the convection heat transfer coefficients for the design.

5.2.3.4 Application of Seismic Loads

This section describes design calculation approaches for evaluating the effects of seismic loads on the stability of unsupported emplacement drifts subject to vibratory ground motions. Typical seismic effects and modes of response of the jointed rock mass to seismic loads are discussed.

As mentioned in Section 5, design of underground openings in jointed rock generally involves a combination of 1) empirical methods, which are experience-based rules summarized in design charts, and 2) analytical methods, which are theory-based procedures based on solid mechanics and rock mechanics principles. The empirical techniques have been developed for underground design for static loading conditions but do not incorporate seismic loading. To more accurately account for thermal or dynamic effects, analytical methods are used. Thus, only analytical methods are discussed here.

The interaction of a seismic wave with an underground opening depends on the ratio of the wavelength to the maximum span of the opening. For large ratios and relatively long ground motion duration, the transient ground motion caused by seismic waves produces basically quasi-static loading. For small ratios and relatively short ground motion duration, the loading is dynamic. Both loading conditions are applicable to the repository host horizon and are taken into account for design analysis. In light of the complexities involved, analytical methods that include extensive numerical modeling are used for examining both quasi-static and fully dynamic loading conditions.

It will be necessary to perform dynamic analyses for all cases where the wavelength of the seismic waves with significant energy content is less than eight times the excavation diameter

(Hardy and Bauer 1993, p. 213). Some engineering judgment is required in applying this criterion because dynamic analysis is not necessary if the energy of the high-frequency waves is low.

A jointed rock mass will be more susceptible to the damaging effects of seismic load than an intact rock mass. Because one of the repository host rocks, the non-lithophysal units, contains many vertical and subvertical joints, it is important to determine the influence of these joints on the seismic response of the rock mass. The effects of joints can be incorporated in continuum models, or joints can be modeled directly using a discrete block model.

Regardless of the model used (i.e., continuum or discontinuum), the strength of the rock mass or the joints is represented by a Mohr-Coulomb yield criterion. Strength and modulus, although dynamic properties, typically are determined from static tests because dynamic tests are more difficult to run, especially on jointed rock. However, the use of static strength and modulus values in dynamic analyses is conservative because the static values typically are lower than the dynamic values; i.e., calculated deformations are greater when smaller (less stiff) moduli are used.

The following types of parameters are input to the distinct-block models:

- Joint orientations (dip angle and direction)
- Joint spacing with both continuous and discontinuous planar joint patterns accounted for
- Joint shear stiffness and joint normal stiffness
- Joint cohesion, friction angle, and dilation angle
- Joint with Mohr-Coulomb yield or continuously yielding (i.e., strain-softening) mode.

Lower bound values for joint strengths apply to weaker than expected rock masses, while upper bound values approach those of an unjointed rock mass model (i.e., a continuum).

These joint strength values apply to both initial direction and reverse shear modes. A number of laboratory tests on jointed rock samples have revealed that joints behave differently during initial direction and reverse shearing motion (YMP 1997, Section 3.3.3.2; Hsiung et al. 1994; Souley et al. 1995). Experimental evidence points out that decreases in joint shear strength and dilation effects during reverse shearing may adversely affect rock mass strength. Because joint behavior could apply to the repository horizon rock mass, which will undergo cycles of thermal and seismic loading, lower bound (residual) joint strength and stiffness values can be used as conservative values in the jointed rock analysis.

Additional details for quasi-static and dynamic approaches are presented below.

<u>Quasi-Static Approach</u> (YMP 1997, p. 3-19 and 3-21). The most important step in the quasi-static approach is to implement the equivalent loads that correspond to the design basis ground motion. For the numerical models listed, the quasi-static loads are represented either as body

forces and displacements acting at nodal points or as boundary pressures and displacements acting at model boundaries.

After correctly specifying the loads, key elements for conducting a quasi-static seismic analysis are appropriate constitutive models and suitable material properties. Both the continuum and discontinuum models can be applied in the repository design. The application and utility of either type of model depends on the nature of the rock mass and the scale of the problem of interest.

For design of ground support in emplacement drifts, the decision to model joints discretely depends on the joint spacings relative to the dimensions of the underground opening and, to some extent, on knowledge of the joint pattern, continuity, and mechanical properties of the jointed rock mass. As for most geotechnical material, the variation of material properties of the jointed rock mass is large. The analysis should therefore include a range of material properties, and the material properties used should consider the scale effect appropriate for the physical size of the material being analyzed.

In general, the quasi-static design analysis procedure involves:

- Adding the equivalent PGA as a static gravity load to the existing in situ and thermal-induced loads in such a way that the least favorable directions of the incident P wave and S wave are accounted for, especially with respect to the orientation of joints and the alignment of underground openings.

- Evaluating the impact of the combined seismic, in situ, and thermal loads on the underground structures using appropriate constitutive models (with no ground support) to assess the extent of regions of rock yield, joint slip, and rock block movements.

- Selecting ground support components, numerically representing those components as realistically as possible in the models, and performing numerical simulation to demonstrate the appropriateness of the ground support systems in terms of limiting and stabilizing yielding rock and joints; i.e., in limiting deformation to acceptable values. This step is not considered in the preliminary design analysis.

<u>Dynamic Approach.</u> The dynamic analysis approach is similar to the quasi-static method in that the methodology involves analysis of unsupported openings to assess damage to the host rock, then selection of ground support components and reanalysis to assess loads on the ground support components. The analysis loads and material properties differ between the quasi-static and dynamic methods, as discussed below.

The use of dynamic analysis algorithms differs from quasi-static analysis in that mass-acceleration terms are included in the equilibrium equation, and rate-dependent properties are sometimes considered. Dynamic seismic loads are exerted at the equilibrated state after the static in situ and thermal loads have been applied. The procedure for dynamic analysis for repository openings involves the following:

- Analysis of the in situ equilibrium state for the excavation.

- Thermomechanical analysis that considers the effects of heat from the emplaced waste. (Omit this step for the load combination without thermal load.)

- Dynamic simulation of seismic waves traveling through the rock mass surrounding the underground opening.

Seismic loads are numerically approximated and are applied to the models being analyzed. Seismic waves are characterized by their amplitude values (acceleration, velocity, displacement or stress), wave type (P or S), duration, frequency, and propagation direction with respect to the emplacement drifts, in conjunction with depth attenuation and the damping effect from the rock mass.

The site-specific seismic ground motions should be used in the design. They are represented by a ground acceleration history or velocity history. These will be received from the probabilistic seismic modeling that is based on the Probabilistic Seismic Hazard Analysis (PSHA) hazard levels.

For analysis of deep underground structures, non-reflecting and free-field boundaries can be used to simulate the surrounding infinite medium. For confirmation of this approach, benchmarking the non-reflecting boundaries using a simple harmonic wave propagating in an infinite medium can be conducted before performing a full-scale dynamic analysis (YMP 1997, p. 3-22).

Construction of the mesh used in the model needs to account for boundary effects due to loading from excavation, thermal changes, and seismic events. In general, a mesh boundary three to five diameters away from the opening is sufficient to model the static loading due to excavation. For the boundary to be as realistic as possible, seismic loading requires a larger distance between mesh boundaries.

Quiet (or absorbing) boundary conditions are used at the bottom and top of the model during the dynamic analysis to prevent the outwardly propagating waves from reflecting back into the model at those boundaries. Free-field boundaries are combined with quiet boundary conditions on the vertical lateral boundaries. This type of boundary condition allows propagation of incoming waves vertically upwards without distortion due to presence of quiet boundaries, i.e., the energy of incoming waves is not dissipated at the lateral boundaries, while seismic energy reflected from structures internal to the model is dissipated at these boundaries.

Seismic loads are imposed on the model after equilibrium has been reached under both the in situ stress field and subsequent thermal loadings generated by emplaced waste packages. Therefore, the initial velocity for each grid point before the application of dynamic loads is zero. The seismic loads are usually applied at the bottom boundary of the model in the form of velocity, acceleration, or stress boundary conditions. Stress (both normal and shear tractions) boundary conditions are recommended because they are consistent with quiet boundaries. In case velocity boundary conditions are used, both P- and S-plane waves are applied. Since seismic loads are time-dependent, time histories of stress or velocity are required. The time histories of stress are calculated based on the P- and S-plane waves using Equation 5-9 (Itasca Consulting Group 2002, FLAC Version 4.0, *Optional Features*, Equations 3.12 and 3.13). The factor of two (2) accounts for energy dissipated by the quiet boundaries.

$$t_n = 2\rho C_p v_n$$
$$t_s = 2\rho C_s v_s \qquad \text{(Eq. 5-9)}$$

where	t_n	=	normal traction, Pa
	t_s	=	shear traction, Pa
	ρ	=	rock mass density, kg/m^3
	C_p	=	speed of P-wave propagation through the rock, m/s
	C_s	=	speed of S-wave propagation through the rock, m/s
	v_n	=	normal particle velocity history, m/s
	v_s	=	shear particle velocity history, m/s

C_p and C_s are given by (Itasca Consulting Group 2002, FLAC Version 4.0, *Optional Features*, Equations 3.14 and 3.15)

$$C_p = \sqrt{\frac{K + 4G/3}{\rho}} \qquad \text{(Eq. 5-10)}$$

$$C_s = \sqrt{\frac{G}{\rho}} \qquad \text{(Eq. 5-11)}$$

where	K	=	bulk modulus of the rock mass, N/m^2
	G	=	shear modulus of the rock mass, N/m^2

5.3 PRELIMINARY ASSESSMENT OF NEEDS FOR GROUND SUPPORT

The results from the empirical analysis discussed in Section 5.1 and numerical analysis for unsupported drifts addressed in Section 5.2 can be used for preliminary assessment of needs for ground support. The needs for ground support should be based on the predicted stability conditions and the evaluation of desired functions that installed ground support is anticipated to achieve.

6. DETAILED DESIGN ANALYSIS OF GROUND SUPPORT

The final phase of the ground support design process involves the detailed evaluation of performance of ground support selected in the preliminary design phase. The primary tools for the detailed analysis are the numerical methods. Incorporation of ground support in the numerical analysis is required.

6.1 GROUND SUPPORT FUNCTIONS AND FAILURE MECHANISMS

6.1.1 Ground Support Functions

The basic role of ground support in emplacement drifts is to ensure safety and stability during the preclosure period. This requires limiting rock deformations and providing protection from deterioration or weathering of the host rock. To limit deformations, the ground support may be required to reinforce the rock mass, to pin joints or blocks together, to suspend rock loads, or to support potential loose (or yielding) rock by providing structural load transfer to the floor. In some unfavorable ground conditions, the ground support components might be required to yield to accommodate large rock deformations.

One of the key functional requirements for ground support is to protect waste packages from damages caused by potential rock falls over a long time period. The ground support methods selected will depend on the sizes of rock falls anticipated and the service life specified. Additionally, ground support methods also depend on the rock conditions. For example, rock bolts with welded wire fabric may be appropriate for use in the emplacement drifts in the non-lithophysal rocks, while a continuous support method like shotcrete may work better in the lithophysal rocks because it can hold small pieces of rock in place.

Different from conventional design, the ground support installed in emplacement drifts should be functional for an extended service life with minimum planned inspection and maintenance since the access to the drifts after waste emplacement is limited. To maintain this functionality, a conservative design of the ground support system is required. Longevity is also an important factor to be considered in selecting the ground support materials.

6.1.2 Ground Support Failure Mechanisms

To properly model ground support, identification of the principal factors that control the performance of ground support, such as structural failure mechanisms, is necessary. The potential failure mechanisms of various types of ground support are identified below and some of them are listed in the *Drift Design Methodology and Preliminary Application for the Yucca Mountain Site Characterization Project* (Hardy and Bauer 1991, Section 8.1.5). These failure mechanisms should also be taken into account in material selection of ground support components.

<u>Fully-grouted Rock Bolts</u>

- Corrosion of bolt

- Debonding of bolt-grout or grout-rock interface
- Anchorage failure, including plate bending, plate pullthrough, and corrosion of bolt or plate
- Overstress of bolt
- Creep/relaxation of bolt tension
- Bolt bending failure because of block rotations

<u>Wire Mesh/Straps</u>

- Failure at bolt/mesh interface
- Tension failure of steel mesh
- Puncturing of mesh by block fallout
- Corrosion of steel mesh

<u>Steel Sets</u>

- Corrosion of steel
- Yielding/rupture due to excessive compression
- Buckling instability due to differential block movement or rock squeezing

<u>Concrete Lining/Shotcrete</u>

- Tensile cracking
- Debonding between rock and concrete/shotcrete
- Shear failure due to excessive compression
- Shear punching
- Buckling instability due to differential block movement
- Dehydration, chemical degradation
- Corrosion of reinforcing fiber or mesh
- Debonding from reinforcement
- Degradation due to elevated temperatures

6.2 IN SITU STRESS LOADING AND GROUND REACTION CURVES

As discussed in Section 3.5, various loading conditions, including in-situ, thermal, and seismic, are anticipated for ground support in emplacement drifts. With the numerical methods, these loading conditions can be easily modeled in the analysis.

In situ stress load on the ground support is a result of excavation. Its effects will depend on the timing of support installation relative to excavation, stiffness of the support, and time-dependent characteristics of the rock mass. These effects are often determined by the interaction between the rock and the ground support. The interaction of the rock deformation and loads applied to the rock support/reinforcement system can be illustrated by the ground reaction curve shown in Figure 6-1. In this figure, the rock behaves elastically initially, but then deforms inelastically, so the ground reaction curve A-B is nonlinear. In unstable ground, the ground reaction curve may not intersect the zero pressure line (as indicated by the dotted ground reaction curve), indicating the need for a rock support system.

The timing of support installation usually determines the magnitude of load on the ground support. Initial ground support, such as temporary rock bolts or shotcrete, is typically installed near the tunnel face right following the excavation, and are thus subjected to most of the rock deformation accompanying the advance of the face. Stress relief for initial ground support will be little. Final ground support, such as fully-grouted rock bolts or steel sets, are installed well after excavation, especially with a two-pass construction scheme. The final ground support may experience little or no impact from in situ stresses. Stress relief for final ground support will be great. This is also illustrated in Figure 6-1. If the support is installed after Point B, no load is developed in the support. If the support is installed near the face, for example, after deformation represented by Point C, then as the face advances, the tunnel perimeter will further deform, loading the support. If installed later, for example, at Point E, the load transferred to the support will be less.

The stiffness of the ground support also affects the loading in the support because stiff support, such as concrete lining, will assume high loads with little ground deformation. Softer, more flexible support, such as rock bolts or thin shotcrete layers, do not assume high loads, but may allow more deformation in the rock, possibly leading to unstable conditions. This is illustrated in Figure 6-1 by lines C-D, C-G, and C-H, where C-H represents the most flexible support of the three.

The load estimated from the GRC for ground support only reflects the effects of in situ stress. To fully examine the performance of ground support under combined in situ stress, thermal, and seismic loading conditions, numerical analysis is required.

Use of the ground reaction curve (GRC) in the detailed design analysis for ground support can serve two purposes: (1) to estimate the ground relaxation prior to installation of ground support; (2) to determine the pressure caused by rock deformation on ground support.

The estimated percentage (*n* percent) of ground relaxation will be used in numerical models to account for the effects of excavation sequence and timing of ground support installation. This approach assumes that the rock will deform during excavation under the action of *n* percent of in

situ stress before the ground support is installed. The ground support and rock interaction occurs during the remaining (100-*n*) percent of the in situ stress. Table 6-1 lists recommended percentages (*n* percent) of the ground relaxation before the installation of final ground support. These values are suggested based on engineering judgment. They are lower than those recommended in the *Drift Design Methodology and Preliminary Application for the Yucca Mountain Site Characterization Project* (Hardy and Bauer 1991, Table 8-1) for the reason of conservatism.

In case the final ground support analyzed also serves as the initial ground support, the recommended percentage values of ground relaxation, as listed in Table 6-1, should not be used in numerical models since the initial ground support is generally installed right behind a TBM. The percent of excavation-induced deformation prior to installation of the initial ground support is relatively low. Therefore, a lower percentage value of ground relaxation should be used. For conservatism, a zero percent of ground relaxation is recommended for the initial ground support.

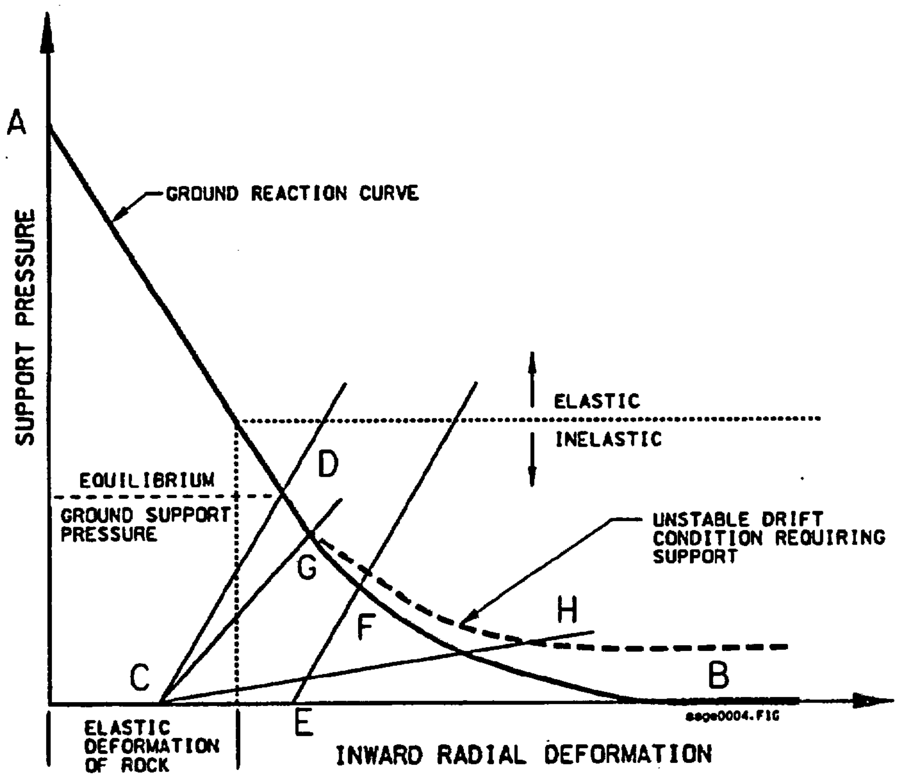


Figure 6-1. Interaction of Ground Reaction Curve and Ground Support System

Table 6-1. Recommended Percentage (%) of Ground Relaxation Prior to Ground Support Installation

Type of Final Ground Support	Emplacement Drifts
Fully-grouted Rock Bolts	60
Steel Sets	60
Shotcrete, if used	60
Cast-in-place Concrete Lining, if used	80

6.3 NUMERICAL ANALYSIS PROCEDURES

Unlike for aboveground structures, calculations in the design of ground support used for underground structures in rock usually require the consideration of interaction between the support and the rock mass. Two approaches are widely used. One is based on the ground reaction curve, as outlined in Figure 6-1, and the other on a fully coupled numerical analysis using computer codes such as FLAC or UDEC. For the rock mass with strong discontinuity, such as the non-lithophysal rocks, the effects of movement of rock blocks on ground support are important. The latter approach can capture these effects in more details, and is thus preferred. For the rock mass with less discontinuity, such as the lithophysal rocks, use of the former approach is appropriate as long as the characteristics of the rock mass and the ground reaction curve can be correctly defined. In the following, focus will be on the approaches based on numerical analyses.

The approaches presented in Section 5.2.3 for unsupported emplacement drifts in terms of how to construct a model, set up boundary conditions, and calculate the effects of in situ stress, thermal, and seismic loads are applicable to supported drifts. Discussion of these approaches will not be repeated in this section. The unique part of modeling supported drifts is how to incorporate ground support components into numerical models and how to analyze the results related to the response of the ground support subjected to combined in situ, thermal, and seismic loads.

The following four subsections will discuss the calculation methods for each type of potential candidate ground support separately since each has unique features and requires different approaches to modeling. As mentioned in Section 3.4, the currently recommended types of ground support are all based on the use of steel. Shotcrete or concrete lining is not recommended due to the uncertainties related to postclosure waste isolation. However, for the purpose of completeness of the discussion, the design methods for shotcrete and concrete are also provided.

6.3.1 Fully-Grouted Rock Bolts

Fully-grouted rock bolts with welded wire mesh could be used as final ground support. These rock bolts can be installed either tensioned with mechanical anchoring or untensioned and without mechanical anchoring. Grout will bond threaded steel bar to the rock, providing reinforcement to the rock mass. These rock bolts also hold welded wire mesh in place.

Grouting in rock bolt system serves two purposes:

- It bonds a bolt to the rock, making the bolt an integral part of the rock mass. This improves the interlocking of the individual blocks in the rock mass, and results in a significant improvement in the properties of the rock mass.

- It protects the bolt against corrosion, maintaining its long-term function and performance. This is extremely important since ground support installed in emplacement drifts is required to function with minimum planned maintenance for up to 300 years after final waste emplacement (Curry and Loros 2002, PRD-014/T-006, p. 3-89).

A rock bolt fully grouted with cement will bond to the rock to reinforce the rock mass along its length. The fully-grouted rock bolt is judged to fail when either the bond yields or the rock bolt material itself yields. Of these two strength factors, the grout bonding (shear) strength is considered more critical to the performance of the rock bolt system when subjected to thermal loading because of the difference in the coefficients of thermal expansion of the bolt steel and the grout material. This difference may result in a shear stress at the bolt/grout interface that could be high enough to exceed the grout shear strength.

Fully-grouted rock bolts will interact with the rock mass under loads, and act as reinforcement to the rock mass. Hence, simulation of the interaction between the rock bolts and the rock mass is important when modeling the rock bolts in order to correctly understand their behavior. Depending on the rock mass characteristics, rock bolts can be modeled with computer codes based on either the continuum approach, such as FLAC or FLAC3D, for the lithophysal rocks, or discontinuum approach, such as UDEC or 3DEC, for the non-lithophysal rocks. With these computer codes, the interaction between the rock bolts and the rock mass can be easily simulated.

Rock bolts are modeled with one-dimensional cable elements. The cable element is defined by two nodes, together with the cross-sectional area and the material properties. The cable is divided into a number of segments that are grouted along its length to provide bonding to the rock. The cable element is an axial member, meaning that only the uniaxial resistance, compression or tension, is taken into account. The axial stiffness of the cable elements is described in terms of the bolt cross-sectional area and Young's modulus of the bolt material. Both a tensile yield-force limit and a compressive yield-force limit are assigned to the cable. Once the tensile or compressive limits are reached, no higher cable forces can develop and the bolt is considered to yield.

The presence of grout is modeled implicitly, represented by its cohesive strength and stress-dependent frictional resistance. The maximum shear force that can be developed in the grout, per length of element, is a function of the cohesive strength of the grout and the stress-dependent frictional resistance of the grout. The relation used to determine the maximum shear force is given as follows (Itasca Consulting Group 2002, FLAC Version 4.0, *Structural Elements*, Equation 1.14):

$$\frac{F_s^{max}}{L} = S_{bond} + p' \times \tan(S_{friction}) \times perimeter \qquad \text{(Eq. 5-12)}$$

where F_s^{max} = maximum shear force along a bolt, N

L	=	length of a bolt, m
S_{bond}	=	intrinsic shear strength or cohesion, N/m
p'	=	mean effective confining stress normal to the element, N/m^2
$S_{friction}$	=	friction angle, degrees
perimeter	=	exposed perimeter of the element, m

For the bond stiffness and strength of grout, the following expressions are used based on Itasca Consulting Group (2002, FLAC Version 4.0, *Structural Elements*, Equations 1.19 and 1.21):

$$K_{bond} \cong \frac{2\pi G}{10 \ \ln(1+2t/D)} \qquad \text{(Eq. 5-13)}$$

and

$$S_{bond} = \pi(D+2t)\tau_{peak} \qquad \text{(Eq. 5-14)}$$

where			
	K_{bond}	=	grout bond stiffness, N/m/m
	G	=	grout shear modulus, N/m^2, $G=E/[2(1+\upsilon)]$
	t	=	grout annulus thickness, m
	D	=	rock bolt diameter, m
	S_{bond}	=	grout cohesive (shear) strength per meter of bolt, N/m
	τ_{peak}	=	grout peak shear strength, N/m^2
	E	=	grout modulus of elasticity, N/m^2
	υ	=	Poisson's ratio, dimensionless

Since fully-grouted rock bolts are installed at a fixed spacing (interval) along the axial direction of a drift, the mechanical properties such as the modulus of elasticity and the strength of the bolts should be scaled by dividing the spacing in a two-dimensional calculation. Axial forces from the calculation are then multiplied by the spacing to obtain the actual loads.

6.3.2 Steel Sets

Steel sets may be used, if needed, as the final ground support in emplacement drifts. They are placed as full circular rings. Tie rods with pipe spacers are used to ensure a fixed spacing between two adjacent steel sets, and to securely tie them together. These rods are arranged so that each distinct beam of a steel set ring is bolted firmly to the corresponding beam of the adjacent set. This means that the entire lining could stand by itself without being wedged against the rock.

Steel sets installed will neither bond nor conform perfectly to the bored surface of the opening. The joint connection between two steel set segments may also allow some relative displacement. Areas with gaps or no contact between the steel sets and the rock exist. These factors will alter the load condition on the steel sets, producing lower stresses than for the perfect contact condition. Therefore, an approach which takes into account the gaps between the steel sets and the rock is proposed for modeling the behavior of steel sets under thermal loading conditions.

The interface or gaps between the steel sets and the rock surface can be represented by the two-dimensional circumferential point-to-surface contact elements which are available in the ANSYS code. The interface may separate the steel sets from the rock or maintain physical contact between them, depending on the loading condition. It may also allow the steel sets and the rock to slide relative to each other. Hence, the contact elements are capable of supporting compression in the direction normal (perpendicular) to the surfaces and shear in the tangential direction. Behavior of the circumferential contact elements is governed by their stiffness and frictional resistance.

An initial "gap" is assigned to the contact elements, representing an initial mismatch between the steel set ring and the rock surface. The initial "gap" is not uniform along the circular ring, varying from the greatest at the crown to zero at the invert. The only physical property of the contact elements is the coefficient of friction. Its value depends on the types of materials which form the interface. For numerical analysis purposes, two numerical parameters, called normal contact stiffness (k_n) and sticking contact stiffness (k_t) are also associated with the contact elements. The values of these parameters are related to the stiffness of the system being modeled and determine the amount of the so-called "penetration" in the normal (perpendicular) direction and the relative displacement (sliding) between two surfaces.

It is noted that introduction of the circumferential interface is intended to simulate the interaction between the steel sets and the rock more realistically. No perfect fit will be achieved during excavation. Any small degree of mismatch will yield gaps between the steel sets and the rock. When the steel sets and the rock are heated by emplaced waste, they will expand without resistance initially, owing to the presence of these gaps. With the increase in temperature and the decrease in the size of gaps, the gaps may be closed completely under combined loading conditions, depending on the magnitude of the thermal load or temperature increase and the size of the mismatch.

The approach which takes into account the gaps between the steel sets and the rock, as discussed above, is not limited to the application of ANSYS. Any other code, such as FLAC, which contains interface elements, can also be used. Refer to the user's manuals of these codes for details on how to simulate the interface between the steel sets and the rock under thermal loading conditions.

Steel sets are represented by a lining ring with two-dimensional beam elements. A beam element is defined by two nodes, together with the cross-sectional area, the area moment of inertia, and the material properties. The beam ring is subdivided into a number of beam segments. With the beam elements, the bending resistance of the lining is considered. The results are the thrust and bending moment within the beam ring. In modeling the steel sets, the mechanical properties such as the modulus of elasticity of steel are scaled by dividing by the spacing of the sets along the drift. Thrust and bending moment outputs from the model are then multiplied by the spacing to obtain the actual loads.

6.3.3 Cast-in-place Concrete Lining

Cast-in-place concrete lining is commonly used in underground projects such as road or railway tunnels, due to its flexibility and durability. It is usually installed after the entire drift has been

excavated. Hence initial ground support is required prior to its installation, and geologic mapping along the full length of the drift can be accommodated. Cast-in-place concrete lining usually requires the use of a two-pass construction scheme.

Concrete lining is usually bonded to the rock surface since contact grouting and grouting of voids behind the lining during installation are performed. It is modeled as a lining ring with two-dimensional beam bonded to the drift surface, which is different from the approach used for modeling steel sets. The beam element is defined by the cross-sectional area, the area moment of inertia, and the material properties. The beam ring is subdivided into a number of beam segments. With the beam elements, the bending resistance of the lining is considered. The results are the thrust and bending moment within the beam ring. In modeling the concrete lining, the mechanical properties such as the modulus of elasticity of concrete are not scaled since it is installed continuously along the axial direction of a drift. Thrust and bending moment outputs from the model are the actual loads.

In case concrete is expected to experience significant increases in temperature, creep-induced stress-relaxation at elevated temperature may need to be considered in the analysis. The creep of concrete is strain that increases with time under a constant load (ACI 209R-92 1992). Because this strain can be several times as large as the initial strain on loading, creep is of considerable importance in structures. Creep is also beneficial in another sense: if a concrete structure is deformed by external movement, stress increases in the concrete due to the imposed deformation will subsequently be relaxed due to creep of the concrete, thus reducing the need for a compensating increase in strength. Further discussion of the beneficial stress relieving aspects of creep is provided by Neville (Neville 1996, p. 474) and Ross (Ross 1958). The ACI code recognizes that creep can be considered when the reduction in stresses from other loads (generally tensile stresses) due to thermal stress is computed (ACI 209R-92 1992, Section 2.5.6). Richardson (1990, p. 6-2) recognizes that creep may be beneficial in reducing lining stresses. It should be recognized that creep of concrete may also result in a reduction in strength, which is detrimental to the function of concrete.

Creep is a time-dependent phenomenon, and a transient analysis is required to model the creep. This type of analysis involves incremental calculations and iterations, and is thus time-consuming. Many creep equations that relate the creep strain to stress and temperature are available for use in some computer codes such as ANSYS. To accurately account for the creep of concrete at elevated temperatures, further investigation is necessary to evaluate and establish the creep characteristics of the concrete to be considered under the anticipated conditions.

6.3.4 Fiber-Reinforced Shotcrete

As an alternative to cast-in-place concrete lining, fiber-reinforced shotcrete may be used for final ground support. Shotcrete sprayed upon freshly excavated rock fills open cracks and crevices in the rock surface immediately following excavation. This prevents the loosening of rock pieces from the crown and walls. For the emplacement drifts excavated in lithophysal rocks such as the Tptpll unit, which is heavily fractured with short length fractures between lithophysae, ideal long-term ground support would be continuously-placed fiber-reinforced shotcrete together with fully-grouted rock bolts. Shotcrete would provide a retention function.

For shotcrete applied as a final ground support, steel fiber is usually added to shotcrete in order to increase its toughness and flexibility. The reinforced shotcrete can retain strength even after it has cracked. Typically the fiber in the shotcrete is used as a replacement for wire mesh.

Shotcrete is relatively thin and flexible, and can thus deform with the rock over time. If the thickness of shotcrete is increased significantly, the lining will act in a manner similar to that of a standard concrete lining. Because of this feature, modeling of shotcrete is essentially the same as that of concrete lining. Refer to Section 6.3.3 for details.

6.3.5 Summary of Numerical Analysis Procedures

The steps followed in the detailed design analysis for ground support in emplacement drifts are summarized as follows:

- Step #1: Run a thermal only model to determine time histories of rock temperatures, and average rock temperatures on the boundaries of a model to be used in the subsequent thermal/mechanical analysis.
- Step #2: Run the model for an unsupported drift subjected to in situ stress load until equilibrium is reached. Make sure that time histories of displacements at the drift crown, springline, and invert following excavation are recorded.
- Step #3: Determine the number of steps (cycles) corresponding to the percentage of ground relaxation of the total displacement at the point of interest (crown, springline, or invert) under the given in situ stress loading condition. This information can be found in the recorded time histories of displacements.
- Step #4: Rerun the model by cycling the number of steps determined in Step #3 following excavation.
- Step #5: Apply ground support selected, and cycle until equilibrium is reached.
- Step #6: Apply temperature boundary conditions determined in Step #1. Perform thermal analysis by turning thermal mode on and mechanical mode off until the predetermined time of interest is reached.
- Step #7: Perform thermomechanical analysis by turning mechanical mode on and thermal mode off until equilibrium is reached.
- Step #8: Repeat Steps #6 and #7 until the time of ground support service period considered is reached.
- Step #9: Restore the results from Step #7, which are corresponding to the time of interest for seismic effects. Apply seismic load and run the model for the dynamic time associated with the seismic load applied. Following the dynamic time, continue the model run until equilibrium is reached.

A flowchart that summarizes the numerical analysis process for detailed design analysis is illustrated in Figure 6-2.

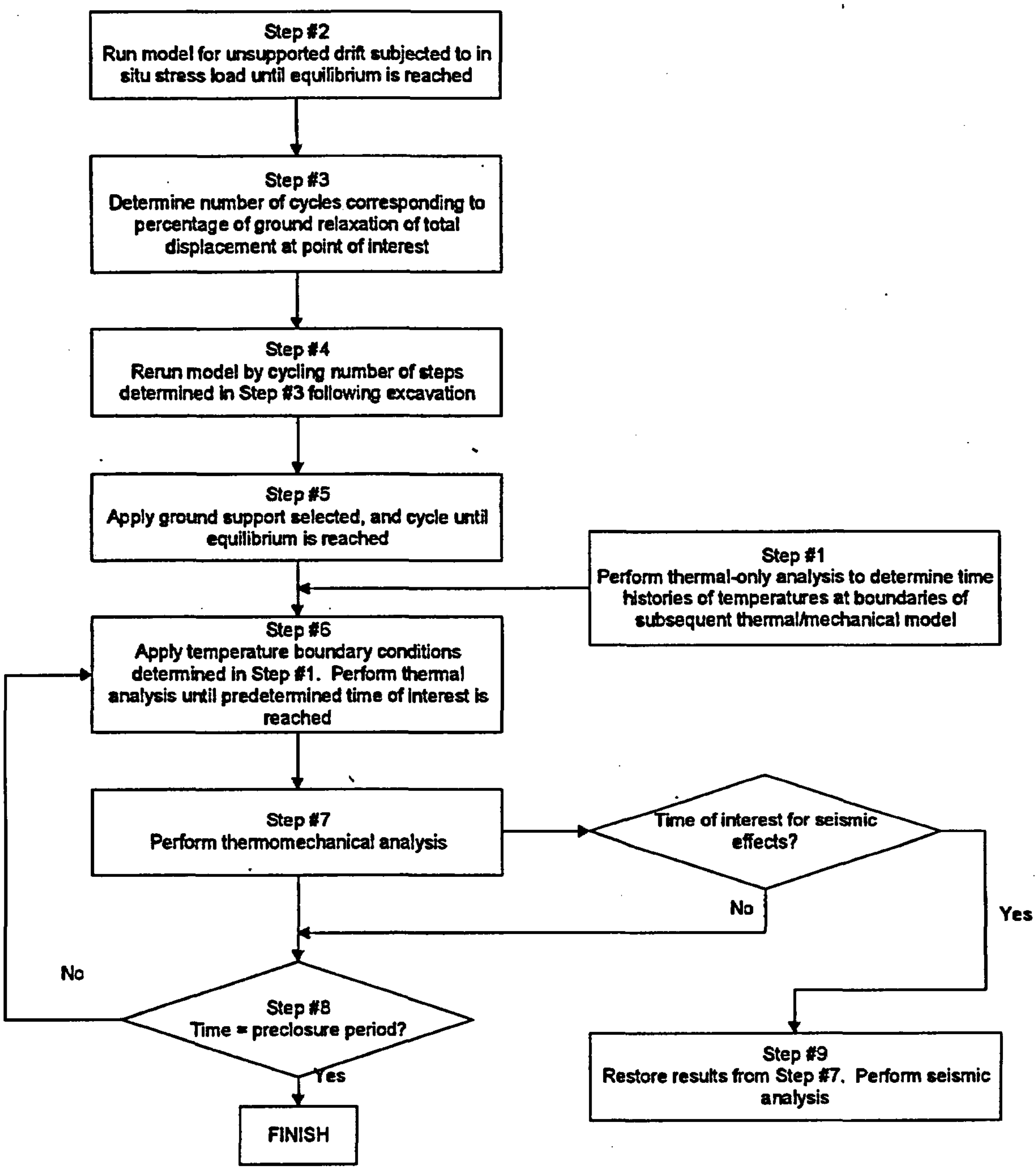


Figure 6-2. Flowchart for Numerical Analysis Process

7. UNCERTAINTY CONSIDERATIONS

Uncertainties associated with the predictions of performance of emplacement drifts and ground support should be addressed in design analyses. These uncertainties are primarily related to the design parameter values used and the modeling approaches selected.

7.1 UNCERTAINTIES OF DESIGN INPUTS

Rock mass properties vary from location to location. It is not feasible, and most of the time unnecessary, to consider all variations of the properties in design analyses.

As a general guideline, uncertainties associated with the design parameter values can be accounted for by using the bounding values. For example, in determining the magnitude of in-situ horizontal stress, the horizontal-to-vertical stress values of 0.2 and 1.0 are recommended since they represent the bounding horizontal stress levels anticipated at the emplacement drift horizon, as discussed in Section 3.5.1.

In case the bounding values or the range of property values are not known, a deviation from a specific given value may be assumed to evaluate the effect of variation on the predicted results. This deviation should be able to cover the anticipated range of variations of the property value being considered.

7.2 UNCERTAINTIES OF MODELING APPROACHES

Each numerical model involves some degree of simplifications and idealizations in order to feasibly and practically simulate reality. These simplifications and idealizations will certainly result in uncertainties in prediction of the system performance. Due to the complex nature of rock mass and ground support system and loading conditions anticipated, and limit in resources, it is very difficult to accurately quantify these uncertainties.

The "best" way to address these uncertainties is to conduct sensitivity studies that examine the effects of model variations for the same problem analyzed. For example, in simulating an emplacement drift excavated in lithophysal rock, both equivalent continuum and discontinuum approaches may be used. The results from these two approaches are then compared. This comparison should allow a qualitative evaluation of uncertainties associated with the use of either approach. This kind of sensitivity studies may be very broad and time-consuming since issues related to the uncertainties of modeling approaches may cover many areas. It is recommended that they be performed in a separate analysis. This analysis should be able to address various uncertainty issues related to the modeling approaches in a very subjective way. Once it is completed, any subsequent design calculations can cite it as a reference to avoid additional efforts for addressing relevant uncertainties.

8. CONSTRUCTIBILITY AND MAINTENANCE CONSIDERATIONS

8.1 CONSTRUCTIBILITY

Constructibility of ground support may have an impact on the design. This impact is usually reflected on the requirements of ground support dimensions, excavation scheme and installation sequence, and ground relaxation. Since the ground support dimensions and ground relaxation are directly related to the design, constructibility and design impact are important factors to be considered in the design of ground support.

Some of the constructibility-related issues that may impact the design of ground support are list below.

Fully-grouted Rock Bolts. Unless mechanically anchored, fully-grouted rock bolts will not take loads until grout is hardened and bonding between the bolt and the rock is established. To bore a hole, insert a bolt, apply grout, and wait for hardening of grout will take longer than to install a regular bolt. Fully-grouted rock bolts with mechanical anchoring can also serve as initial ground support. They are sometimes grouted in the second pass in order to maintain a high excavation rate. For use of fully-grouted rock bolts in the lithophysal rocks, potential grout loss into lithophysae will be an issue. It will also be very difficult to ensure quality. Use of fully-grouted rock bolts can accommodate the operation of field mapping.

Steel Sets. Depending on the surface profile and jacking loads, the "gap" or mismatch between the steel set ring and the rock surface varies. As discussed in Section 6.3.2, the approach for modeling steel sets subjected to thermal loads is valid only if the gap exists. In addition, results based on this approach also depend on the size of the gap assumed. Additional issues associated with the use of steel sets as the final ground support include how the sets are properly jacked in place and whether the sets can be used in the lithophysal rocks to prevent small pieces of rock falling out between sets. These issues have a direct impact on the design. Use of steel sets can also accommodate the operation of field mapping.

Cast-in-place Concrete Lining. Cast-in-place concrete lining is installed in the second pass of construction. Initial ground support is required. Field mapping, if required, is conducted before cast-in-place concrete lining is installed.

Fiber Reinforced Shotcrete. If shotcrete is applied during a drift boring, it can serve as initial ground support. Field mapping, if required, should be completed before shotcrete is applied. In this case, other initial ground support is needed.

8.2 MAINTENANCE

Emplacement drifts will be designed to minimize or eliminate planned maintenance, for a preclosure life of up to 300 years after final waste emplacement (Curry and Loros 2002, PRD-014/T-006, p. 3-89). Key factors that affect the design and determine whether there is a need for periodic maintenance of ground support installed in emplacement drifts are steel corrosion and rock falls.

Corrosion of steel will depend on material properties and the in-drift environment anticipated. Details on the drift environment and steel corrosion rate during the preclosure period are addressed in the *Longevity of Emplacement Drift Ground Support Materials* (BSC 2001b).

Rock falls are directly related to the stability of emplacement drifts. The function of ground support is to maintain the drift stability and prevent rock falls throughout the preclosure period. If rock falls are anticipated due to ground support material deterioration from corrosion or fatigue, the design should be modified to prevent such rock falls from happening. For example, stainless steel or other types of corrosion resistant materials may be used for ground support components in emplacement drifts in order to minimize potential damage induced by corrosion. Or, if rock falls are primarily due to lack of adequate support, the design should also be changed to make sure that the ground support system is functional under the worst condition anticipated.

The design of ground support for a service life of over 300 years is unprecedented. Consequently, it is uncertain that the designed ground support will last for the anticipated service period. Strategies for regular inspection and maintenance should be developed. Since any maintenance or repair operation will require that the affected emplacement drift will first be rapidly cooled, and then cleared of all waste packages to allow access for personnel and equipment, it can be expected that considerable time and effort will be involved in any maintenance operation involving an emplacement drift. The ground support for emplacement drifts should be designed to be as robust as possible with the objective of minimizing frequency of maintenance.

9. DESIGN METHODOLOGY OVERVIEW

The major steps of the overall design methodology are summarized in this section. The design methodology describes methods and procedures for the design of ground support for emplacement drifts. The overall outline of the methodology, given in the flow chart in Figure 9-1, begins with the development of inputs including requirements and criteria for design, shows the general path for the calculation of initial and final ground support, and illustrates the iterative nature of the process of ground support design. The steps in the methodology are summarized below.

<u>Step One – Develop Design Inputs</u>

- **Requirements/Criteria.** Regulatory requirements that govern the design and performance of ground control for the repository are identified. Requirements for emplacement drift design are typically incorporated in program and project requirements documents. On this basis, functional criteria and performance goals are defined, including designing to minimize maintenance throughout the required service life, to maintain operational envelope, and to prevent rock falls. The necessary criteria for service life, maintenance, and loads are also defined. In addition, the appropriate ranges of site and materials parameters are determined.

- **Site Data.** Once requirements are identified and evaluated, site characteristics defined, and certain aspects of construction taken into consideration, repository layout analyses are performed to establish the needed range of drift sizes and shapes. Appropriate excavation methods are identified, and specific code and performance criteria (safety factors, and allowable stresses, strain, and deformations, etc.) are developed.

<u>Step Two – Perform Analysis</u>

- **Preliminary Design Analysis.** Stability of unsupported drifts is evaluated based on either empirical or numerical approach or both. Both continuum and discontinuum models may be needed to evaluate the effects of joints. Results of this evaluation will be used to determine potential rock mass behavior modes, nature of drift deformation over the expected range of rock conditions and properties, and the needs for initial and final ground support. The application of empirical methods will include experience obtained from the design, excavation, and ground support of the ESF tunnel and ECRB Cross Drift, including grouping rock conditions into discrete categories. Loads from in situ stress, waste emplacement, and seismic events will be incorporated in the analysis.

- **Detailed Design Analysis.** Supported drifts are analyzed for in situ stress load using the ground reaction curves and for combined in situ, thermal, and seismic loading conditions using numerical methods. For the latter case, both continuum and discontinuum models may be needed to evaluate the effects of joints. Models developed will also reflect the variations of behavior modes for different types of ground support. Results of the analyses will be used to evaluate the interaction of rock mass and ground support and the performance of ground support components for the anticipated range of rock conditions and properties, temperatures, and seismically induced ground motions.

Step Three – Evaluate Candidate Designs

- **Performance under Loads.** The predicted performance of each candidate ground support design is reviewed in regard to the requirements, goals, and criteria established in Step One. The results from Step Two for a particular design are evaluated against the established structural acceptance criteria, and other requirements, and the design determined to be either acceptable or to require modification.

- **Constructibility and Maintenance.** The candidate designs are evaluated on the basis of the results of constructibility and maintenance analyses, including the evaluation of the effects of construction and operational loads, and the assessment of potential maintenance needs. Results from these analyses are compared with the established structural acceptance criteria and the special requirements on maintenance, and the design determined to be either acceptable or to require modification.

Due to the complexity of various types of characteristics of rock mass anticipated, the methodology presented in this document may not be suitable for a particular condition. The designer will have the option to modify the methodology to best account for the specific conditions. With more test data available, improved knowledge about the rock, and design evolution, revision of this methodology in the future is possible.

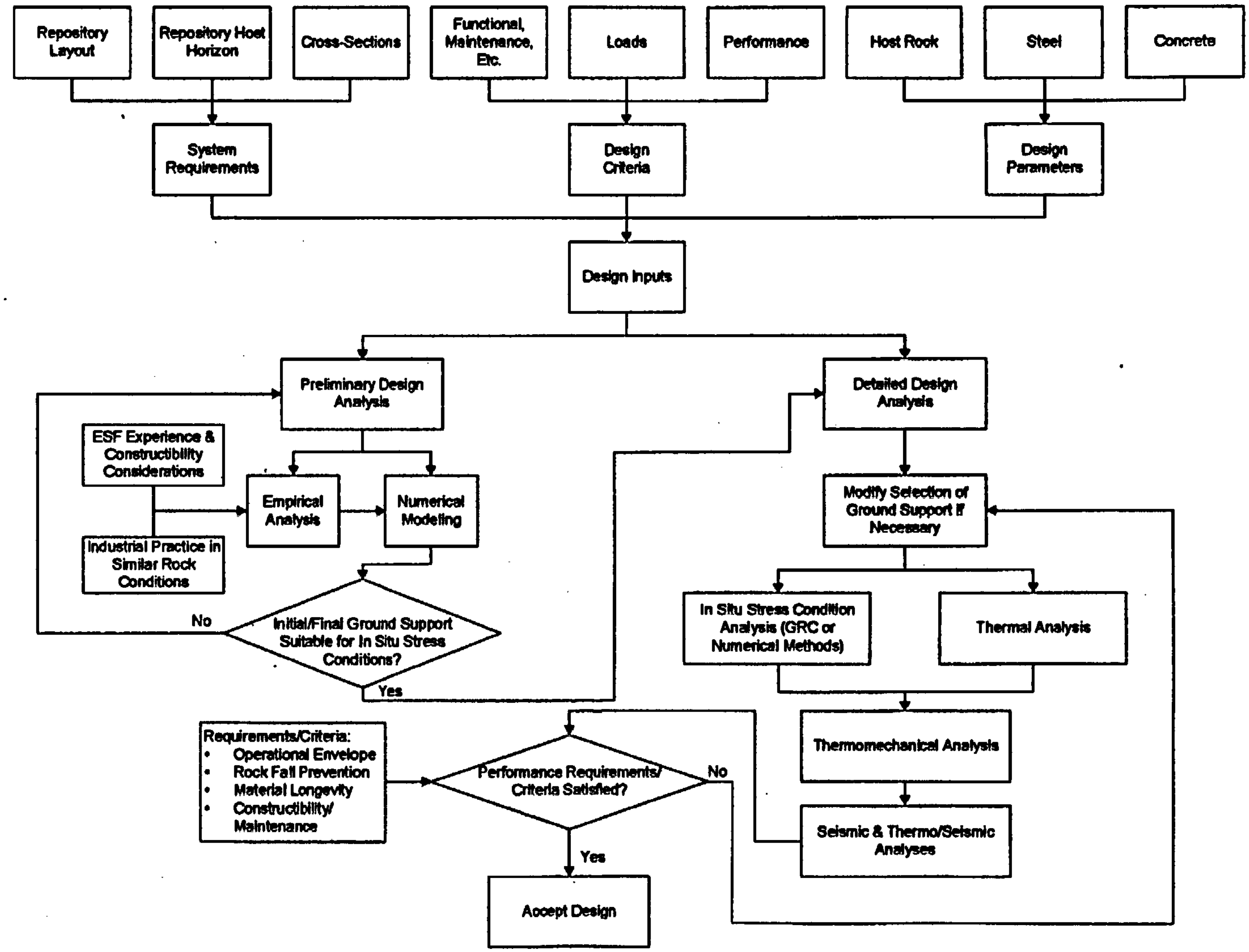


Figure 9-1. Flowchart for Ground Support Design Methodology

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