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# YUCCA MOUNTAIN SITE CHARACTERIZATION PROJECT
## STUDY PLAN APPROVAL FORM

Study Plan Number 8.3.1.15.1.7

Study Plan Title In Situ Mechanical Properties

Revision Number 0

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# In Situ Mechanical Properties

# Study Plan 8.3.1.15.1.7

Revision 0
May 10, 1996

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## Abstract

The In Situ Mechanical Properties Study Plan describes a set of in situ experiments to be conducted in the Exploratory Studies Facility (ESF) to evaluate rock mass mechanical properties, along with a detailed rationale for conducting the tests. The information obtained from these tests will be used to: (1) resolve technical issues identified in DOE's site characterization plan (SCP), (2) provide design inputs for repository design, (3) evaluate empirical models of rock mass behavior, and (4) interpret results of ESF thermal-mechanical tests. A series of tests is presented to measure rock mass deformation modulus, compressive strength, joint shear strength, and joint stiffness. Several test configurations are proposed: plate loading tests, borehole jacking tests, prism tests, slot tests, and block tests. Plate loading tests will be used to measure the rock mass load-deformation characteristics and deformation modulus. These measurements will be supplemented with borehole jacking measurements using the Goodman jacking method. Prism tests involve jacking on a test prism cut out of the rib or floor of the test area. Prism tests will be used to assess the rock mass load-deformation characteristics and deformation modulus, along with, uniaxial and confined compressive strength. Slot tests involve internally loading saw-cut slots using flatjacks to determine large-scale joint strength and stiffness along isolated joints. The block test involves loading a relatively large volume of jointed rock under controlled stress boundary conditions. The purpose of the test is to measure rock mass load-deformation characteristics and deformation modulus, along with, joint strength and stiffness measurements.

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# 1.0 INTRODUCTION

## 1.1 Objectives

This study plan describes experiments to measure the mechanical properties of the rock mass at the Exploratory Studies Facility (ESF) being constructed by the Department of Energy (DOE) as part of the Yucca Mountain Site Characterization Project (YMP).[1] These experiments are identified in DOE's site characterization plan (SCP) (DOE, 1987), Section 8.3.1.15.1.7, and are designed to provide information for repository design, model validation, and performance assessment.

This study plan takes into consideration DOE's changing program needs and tries to accommodate the new program objectives of the site characterization program. Recent changes in the program have lead to a significant departure from the comprehensive program laid out in the SCP. The new approach will require less information in the early phases of the program than was planned in the SCP. The most immediate objective of the site characterization program is to develop sufficient information to perform an assessment of the site viability by the 1998-1999 time frame. The development of additional characterization and design information will likely continue in the time period beyond the completion of the viability assessment to support future development of a license application, if the site is deemed viable.

The modified DOE program plan suggests that the data needs from the site activities (as defined in the SCP and other program planning documents) can be divided into two categories:

- data required for performing a viability assessment that includes a comprehensive performance assessment, and
- data required for a license-applicaton repository design.

This study plan takes this prioritization into account. Only limited data on in situ mechanical properties are needed for the viability assessment in 1998. More data will be required to support any subsequent actions such as the license application design, with more extensive testing anticipated to support design and performance confirmation.

This study plan describes experiments and tests that will provide data on rock mechanical properties to meet information requirements in four areas of the repository program:

- Resolution of technical issues
- Inputs for repository design

[1] This work was performed under the auspices of the U.S. Department of Energy, Office of Civilian Radioactive Waste Management, Yucca Mountain Site Characterization Project, under contract DE-AC04-76DP00789.

- Provide data to support evaluation and validation of geomechanical models to be used to predict long-term performance of the repository system
- Interpretation of ESF thermal-mechanical tests.

## 1.2 Technical Issues

The four technical issues (originally identified in the SCP) that require information on rock mass mechanical properties are (1) characteristics of the waste package (postclosure), (2) the configuration of underground facilities (postclosure), (3) seal characteristics, and (4) preclosure design and technical feasibility. Table 1-1 lists the mechanical properties as performance and design parameters, estimates of the goal, and the confidence level needed to define the variability in the parameters that are allowable in the SCP conceptual design.

Performance assessment and design issues that also require information on rock mechanical properties are shown in Figure 1-1. A more detailed logic diagram from the SCP is presented in Figure 1-2 and shows the site data required by specific performance and design issues.

### *Repository Design*

The data needs for repository design were described in *Repository Design Data Needs* (TRW, 1995), which evaluated needs in light of the current project schedule, data availability, and data completeness. This document correlated the needs with specific subsurface facilities and indicated that information on rock mass mechanical properties is required for:

- design of excavated openings (ramps, drifts, ancillary openings, and shafts), and specifically for ramp portals and ground control.
- design of repository closure (seals); specifically joint strength and deformability.

The status of the data needs addressed by this study plan is presented in Table 1-2. The status is presented in terms of whether or not data exists at the level of confidence needed at a given phase of the program. The needs are associated with the two major phases of the repository program leading to licensing: viability assessment, and the license application design (LAD) along with the environmental impact statement(EIS). Three discriptors are used to describe the anticipated level of confidence needed at each phase:

- Substantially Complete (SC): Data at this level are substantially complete and additional analysis or collection is not likely to significantly change the results or conclusions. Data variability (a combination of measurement uncertainty and inherent randomness) e.g., spatial distribution, is reasonably defined.

Table 1-1. Performance and Design Parameters, Tentative Goals, and Characterization parameters for Thermal and Mechanical Properties Program[a] (extracted from SCP)

| Issue requesting parameter (SCP section) | Performance or design parameter | Material type and spatial location [b] | Stratigraphic locations [c] (request) | Tentative goal [d,e] | Needed confidence | Current estimate | Current confidence | Parameter to be measured | Material type tested | SCP activity numbers | Spatial location | Stratigraphic location |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1.11 (8.3.2.2) | Deformation modulus | Rock mass; primary area and extensions | TSw2 | σ ± 0.15σ | High | See Table 6-14 | Low | Deformation modulus | Rock mass | 8.3.1.15.1.7.1 | ESF | TSw2 |
| | | | TSw1 | σ ± 0.15σ | Medium | See Table 6-14 | Low | Deformation modulus | Rock mass | 8.3.1.15.1.7.1 | ESF | TSw1 |
| 4.4 (8.3.2.5) | Deformation modulus | Rock mass; primary area | TSw2 | 11-19 GPa | Medium | 11-19 GPa | Low | Deformation modulus | Rock mass | 8.3.1.15.1.6.3<br>8.3.1.15.1.7.1 | ESF | TSw2 |
| | | | TSw1 | Nonlithophysal: 12-20 GPa | Medium | Nonlithophysal: 12-20 GPa | Low | | | | | |
| | | | | Lithophysal: 4-11 GPa | Medium | Lithophysal: 4-11 GPa | Low | Deformation modulus | Rock mass | 8.3.1.15.1.3.1 | ESF | TSw1 |
| 1.11 (8.3.2.2) | Mechanical properties of fractures | | | | | | | | | | | |
| | Cohesion and coefficient of friction | Fractures; primary area and extensions | Units above TSw1 and below TSw2 | σ ± 0.15σ | Medium | See Table 6-13 | Low | Shear stress at onset of slip | Fractures | 8.3.1.15.1.4.1<br>8.1.1.15.1.4.2 | (g) | (h) |
| | Large-scale joint strength | Fractures; primary area | TSw1, TSw2 | σ ± 0.2σ | High | NS | Low | Shear stress at onset of slip | Fractures | 8.3.1.15.1.4.1<br>8.1.1.15.1.4.2 | (g) | TSw1, TSw2 |
| | Normal and shear stiffnesses | Fractures; primary area and extensions | TSw2 | See Table 6-13 | Medium | NS | Low | Normal and shear stiffnesses | Fractures | 8.3.1.15.1.4.1<br>8.1.1.15.1.4.2 | (g) | TSw2 |
| | | | TSw1, TSw3, CHn1 | See Table 6-13 | Low | NS | Low | Normal and shear stiffnesses | Fractures | 8.3.1.15.1.4.1<br>8.1.1.15.1.4.2 | (g) | (h) |
| 4.4 (8.3.2.5) | Mechanical properties of fractures | | | | | | | | | | | |
| | Cohesion and coefficient of friction | Fractures; primary area | TSw2 | σ ± 0.2σ | Medium | x | Low | Shear stress at onset of slip | Fractures | 8.3.1.15.1.4.1<br>8.1.1.15.1.4.2 | (g) | TSw2 |
| | Normal and shear stiffnesses | Fractures; primary area | TSw2 | NS | Medium | NS | Low | Normal and shear stiffnesses | Fractures | 8.3.1.15.1.4.1<br>8.1.1.15.1.4.2 | (g) | TSw2 |

[a] This table summarizes requirements of both preclosure and post closure issues.
[b] Definitions of the primary area and extensions are provided in Chapter 6 of the SCP, Figure 6-87.
[c] The thermal/mechanical stratigraphy at Yucca Mountain is shown in Figure 2-5 of Chapter 2 of the SCP [CHn = Calico Hills nonwelded unit; PTn = Paintbrush nonwelded unit; TCw = Tiva Canyon welded unit; TSw = Tonopah Spring welded unit; and NS = not specified].
[d] The manner in which tentative goals and levels of confidence are used in planning the characterization program is discussed in the investigation for Section 8.3.1.15.1.
[e] σ = mean value of existing sample group.
[g] Spatial locations will be a combination of new boreholes (location to be determined; see Investigation 8.3.1.4.1 for discussion) and the exploratory shaft and associated underground excavations [ESF = exploratory shaft facility].
[h] Stratigraphic locations will be in the units specified as required by pertinent issues if measurements need to be made. For many parameters for which the required confidence level is low, existing data will be sufficient to satisfy the requirements.

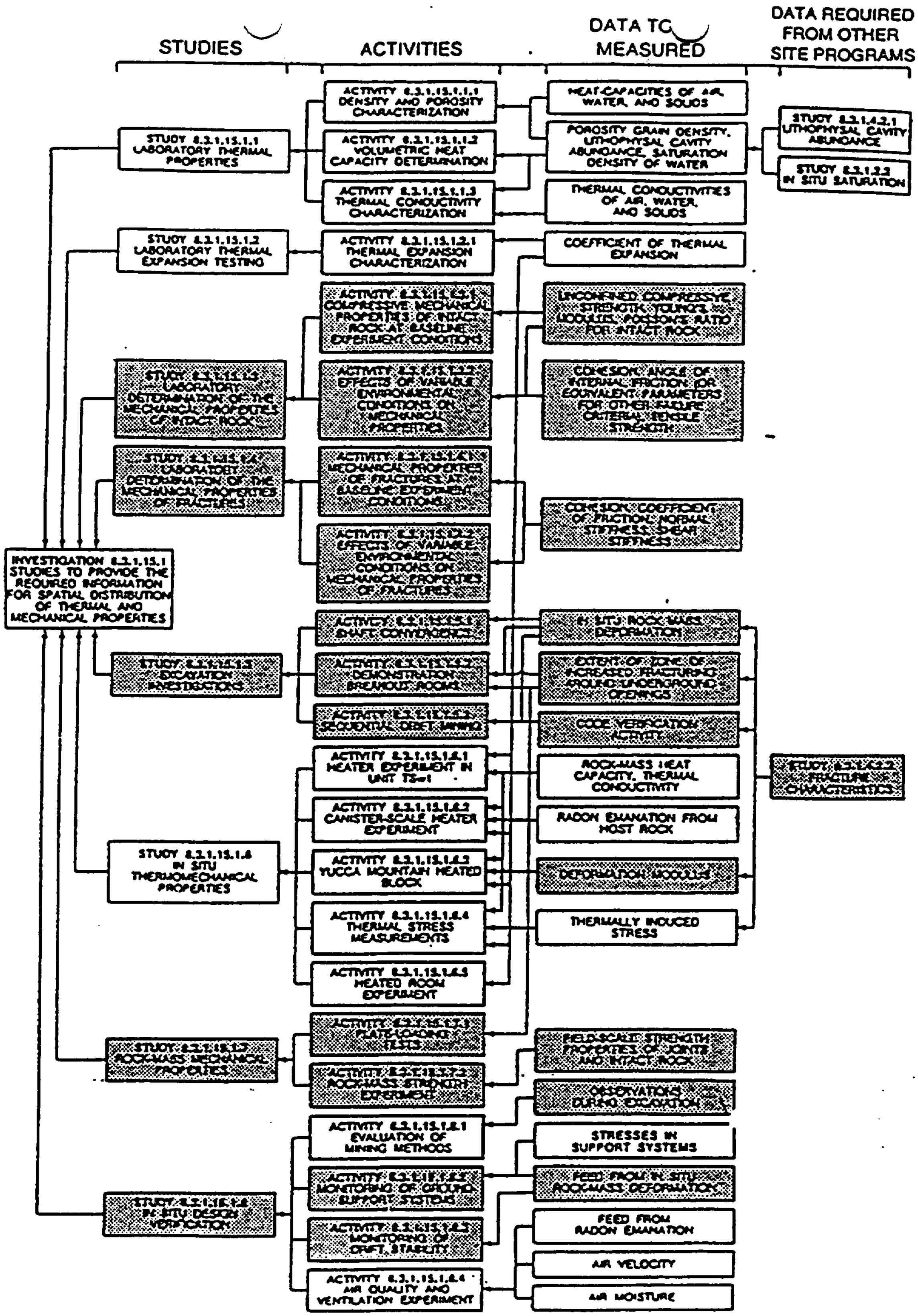



Figure 1-1. SCP logic diagram for investigation 8.3.1.15.1: Spatial Distribution of Thermal Mechanical Rock Properties.

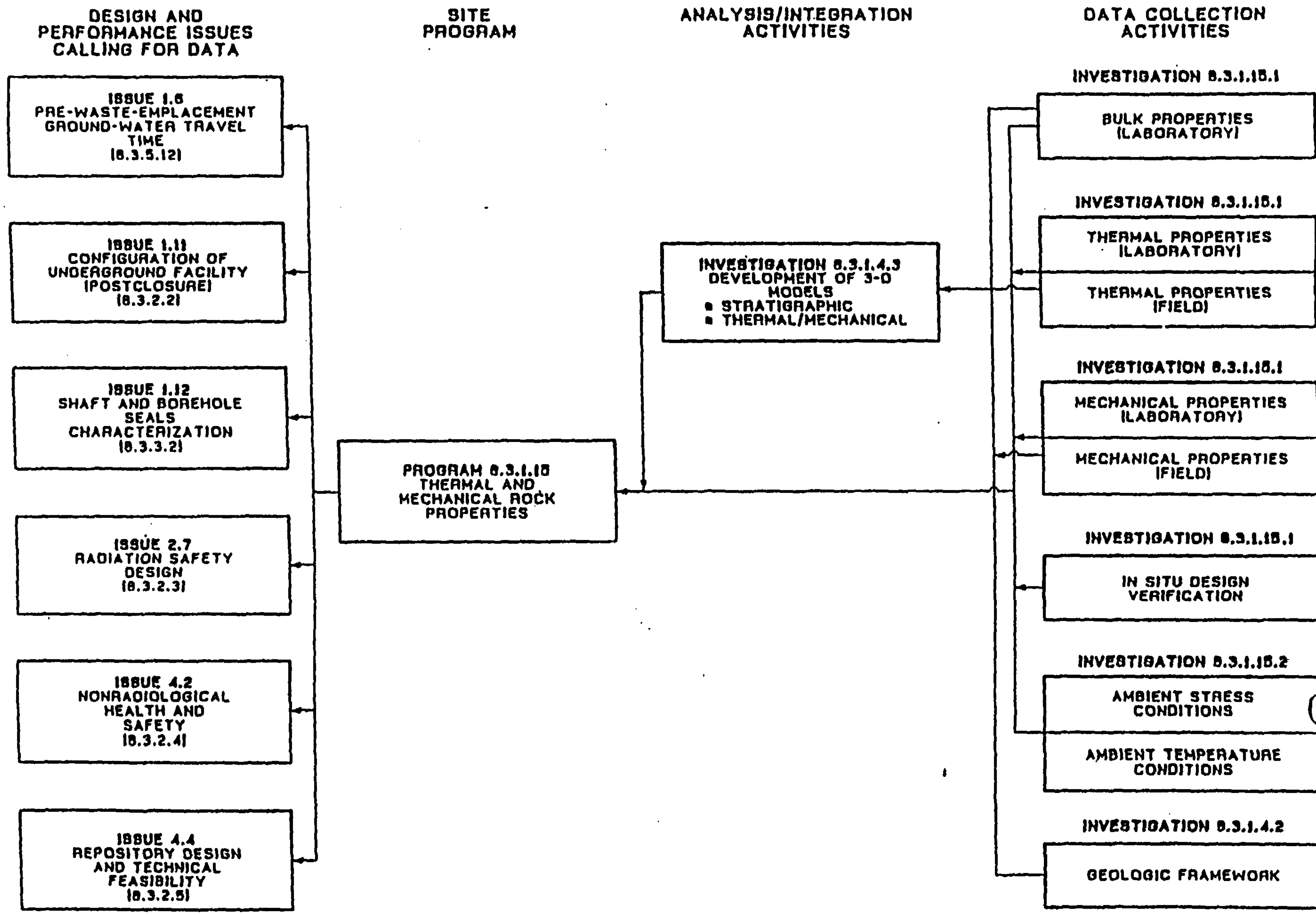



Figure 1-2. Relationship between data acquisition for rock properties and issues requiring the data.

- Bounded (B): Realistic bounding values, with upper and lower extremes identified, have been established for data at this level. Variability is moderately defined.
- Conservative (C): Data are sufficient to estimate credible extreme or worst-case values, conditions, or assumptions. Variability is approximately defined.

The current status is presented in terms of:

- (Y) Yes, data are available at the indicated level.
- (N) No, data or analytical means of adequately estimating the parameter at the indicated level are not available.
- (O) No data are available, but empirical or analytical methods can be used to estimate the parameter. Methods will require verification when data become available.

Data on rock mechanical properties of intact rock are currently judged as adequate for the viability assessment and for the initial phase of LAD. However, rock-mass properties have not been measured. This is the objective of this study. Currently, rock-mass properties can only be estimated from joint properties, rock mass quality index measures, and intact properties. The empirical means of estimating rock-mass properties has not yet been confirmed on a site-specific basis. All properties except joint strength must be bounded for the initial LAD. Rock mass strength, rock mass deformation modulus, and joint deformability must be substantially complete by the LAD. Ultimately, the data collected under this study will contribute to the evaluation and validation of geomechanical models to be used to predict long-term performance of the respository system.

**Table 1-2. Data Needs—Rock Mechanics Properties**

| | Program Phase | | |
|---|---|---|---|
| | 1995-1998 | 1997-? | |
| Data Needs | Viability Assessment | LAD | |
| | | Initial | Final |
| Joint parameters | | | |
| Joint strength | C (Y) | C (Y) | B (N) |
| Joint deformability | B (Y) | B (Y) | SC (N) |
| Rock mechanical properties | | | |
| Rock mass strength | B (O) | B (O) | SC (N) |
| Rock mass tensile strength | B (O) | B (O) | B (O) |
| Rock mass deformability | C (O) | B (O) | SC (N) |

Legend: C = Conservative; B = Bounded; SC = Substantially Complete;
Y = Yes, data are available at this level; N = No, data are not available at this level; O = No data available at this level but empirical estimates can be made.

*Model Validation*

Model validation is specified in both the *Code of Federal Regulations (10 CFR 60)*, and the U.S. DOE Quality Assurance Requirements and Description (QARD) for the Civilian Radioactive Waste Management Program. QARD Supplement III (Scientific Investigation) requires that the use and validation of models of natural phenomena be documented to provide adequate justification for their intended application. It further requires that models be validated by comparing their results with data acquired from laboratory or field experiments or observations.

The tests described in the study plan will contribute to the validation of theoretical models of rock mass deformability. The data developed under this study will be combined with results of other studies (e.g., Excavation Investigations, 8.3.1.15.1.5; In Situ Thermomechanical Properties 8.3.1.15.1.6; and In Situ Design Verification, 8.3.1.15.1.8) to evaluate and validate geomechanical models to be used to predict long-term performance of the repository system.

This study plan will produce data to support the validation of two types of models:

- empirical models that correlate rock mass mechanical properties with parameters of scale or rock mass quality;
- numerical models that perform simulations involving complex shapes, different physical scales, extended time, changes of state, and complex material properties.

Empirical models are being utilized in the design of the ESF to estimate the variation in rock mass properties caused by a variation in rock mass quality. This process was originally outlined by Hardy and Bauer (1991) as part of the YMP drift design methodology. The approach has been adopted for repository design and provides the basis for developing the license-application design.

Examples of empirical models for rock mass strength and rock mass deformation modulus are shown in Figures 1-3 and 1-4. The variation of rock mass deformation modulus with rock mass quality is illustrated by Figure 1-3, which lists equations to estimate modulus from the rock mass quality index (RMR) and compares field test data from the literature with the proposed curve. Data on rock mass quality have been developed in the YMP site drilling program and in the TS North Ramp tunnel, and provide a basis for design. However, YMP site-specific data are needed to verify the adequacy of the predictive equations.

Estimates of rock mass strength derived from qualified laboratory data and rock quality data from the NRG drilling program are shown in Figure 1-4 for the Tiva Canyon Welded (TCw) unit. The estimates are based on empirical criteria proposed by Yudhbir (1983) and Hoek and Brown (1988). The basis of these criteria is highly experiential and there are little field data to confirm their application. They therefore require site-specific testing to verify their use at YMP. Scale dependence of joint shear strength and joint shear stiffness are reported by Bandis et al. (1981) and are the basis for empirical models of joint behavior. Validation of these empirical models that are used to describe rock mass parameters at YMP is also required.

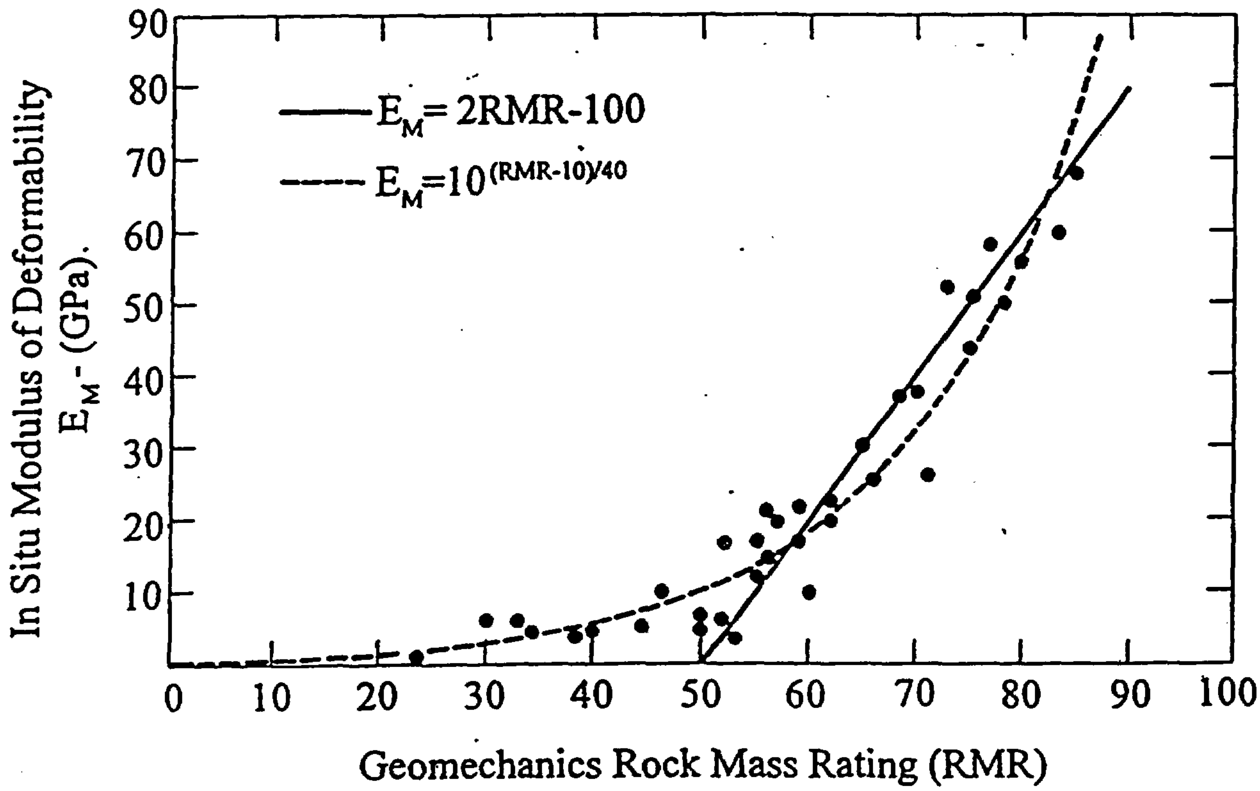



Figure 1-3. Correlation between the in situ modulus of deformation and the geomechanics classification rock mass rating (Bieniawski, 1984).

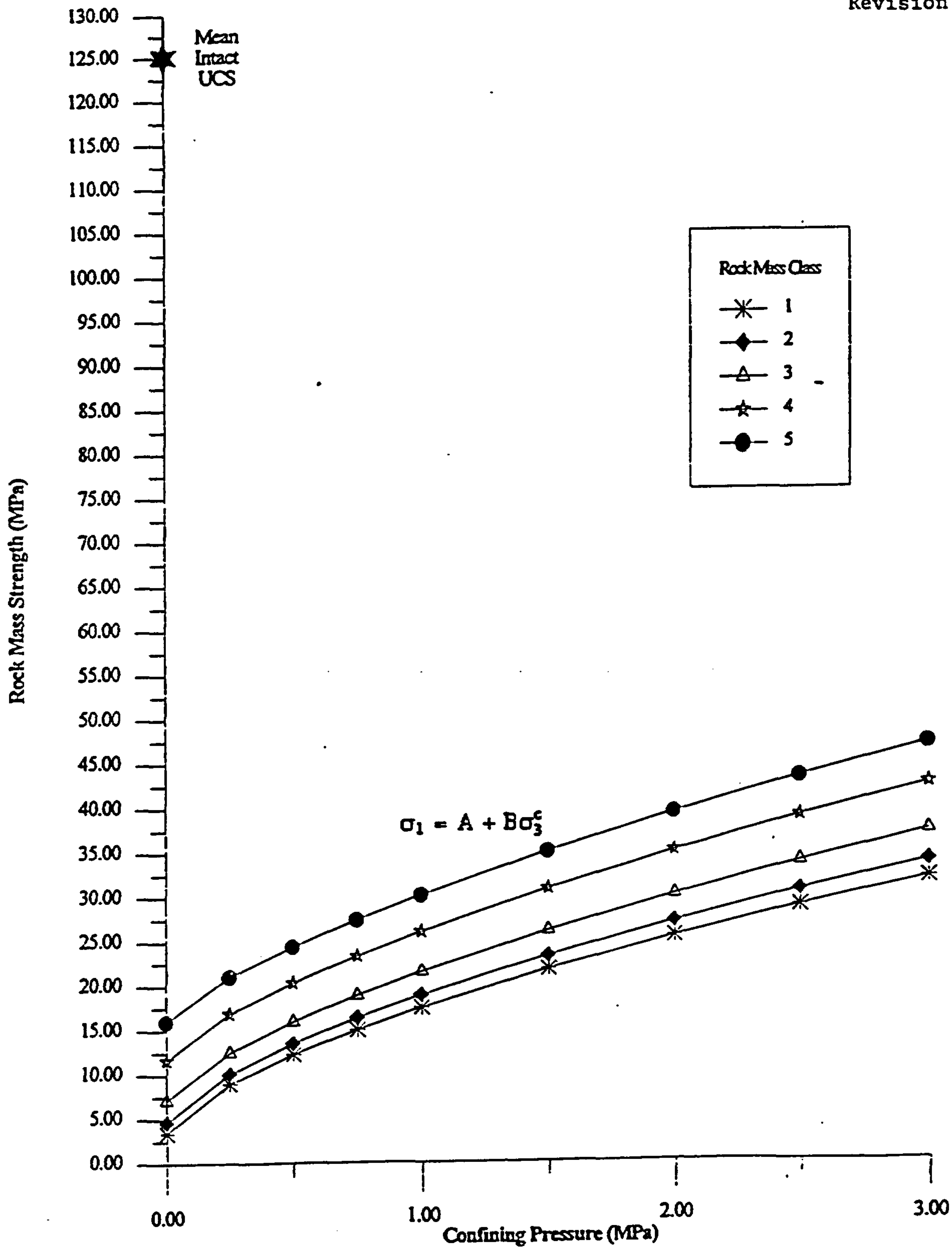



Figure 1-4. Design rock mass strength envelopes for TCw unit—NRG holes (Brechtel et al., 1995).

Numerical models are employed to analyze repository openings under conditions of changing temperature and stress. Figure 1-5 illustrates fundamental material assumptions that divide these models into the basic groups of continuum and discontinuum assumptions. Table 1-3 lists specific computer codes being employed in the YMP and types of mechanical properties input for each model.

**Table 1-3. Some Computer Models Utilized in YMP and Input Requirements**

| Code | Model Type | Input Properties |
|---|---|---|
| ANSYS | Equivalent Continuum | Deformation Moduli, Strength |
| FLAC | Equivalent Continuum | Deformation Moduli, Strength |
| JAC | Equivalent Continuum | Deformation Moduli, Strength |
| JAC | Ubiquitous Joint | Intact rock modulus, Joint Strength, Joint Stiffness, Joint Orientation |
| UDEC | Distinct Joint/Block | Deformation Moduli, Joint Strength, Joint Stiffness, Joint Orientation |
| 3DEC | Distinct Block | Deformation Moduli, Joint Strength, Joint Stiffness, Joint Orientation |
| DDA | Distinct Block | Deformation Moduli, Joint Strength, Joint Stiffness, Joint Orientation |

The model types in Table 1-3 cover the full spectrum of assumptions presented in Figure 1-5. The simplest models require empirical scaling laws and models for the equivalent continuum to define rock mass scale properties for input. The most complicated models incorporate discrete joint structures to approximate the mass scale features, but require joint empirical models to define the input parameter for the joints at excavation scale. Interpretation of the ESF Thermal-Mechanical Tests.

Study plans outlined in the ESF to address rock mechanics properties were organized to produce a data base that provides data for input to the models and for validation of the models, in a series of increasingly complex experiments/studies. This is illustrated in Figure 1-6, which is a proposed validation approach for thermomechanical (thermal-mechanical) models.

The scheduling of tests in the current project plan places emphasis on the thermal-mechanical tests and has resulted in a new thermal testing plan that produces data to meet the requirements of the viability assessment and LAD. However, some testing of mechanical properties at rock mass scale will be required for unambiguous interpretation of the thermal test results. Some of these tests must therefore be accommodated in the early testing plans.

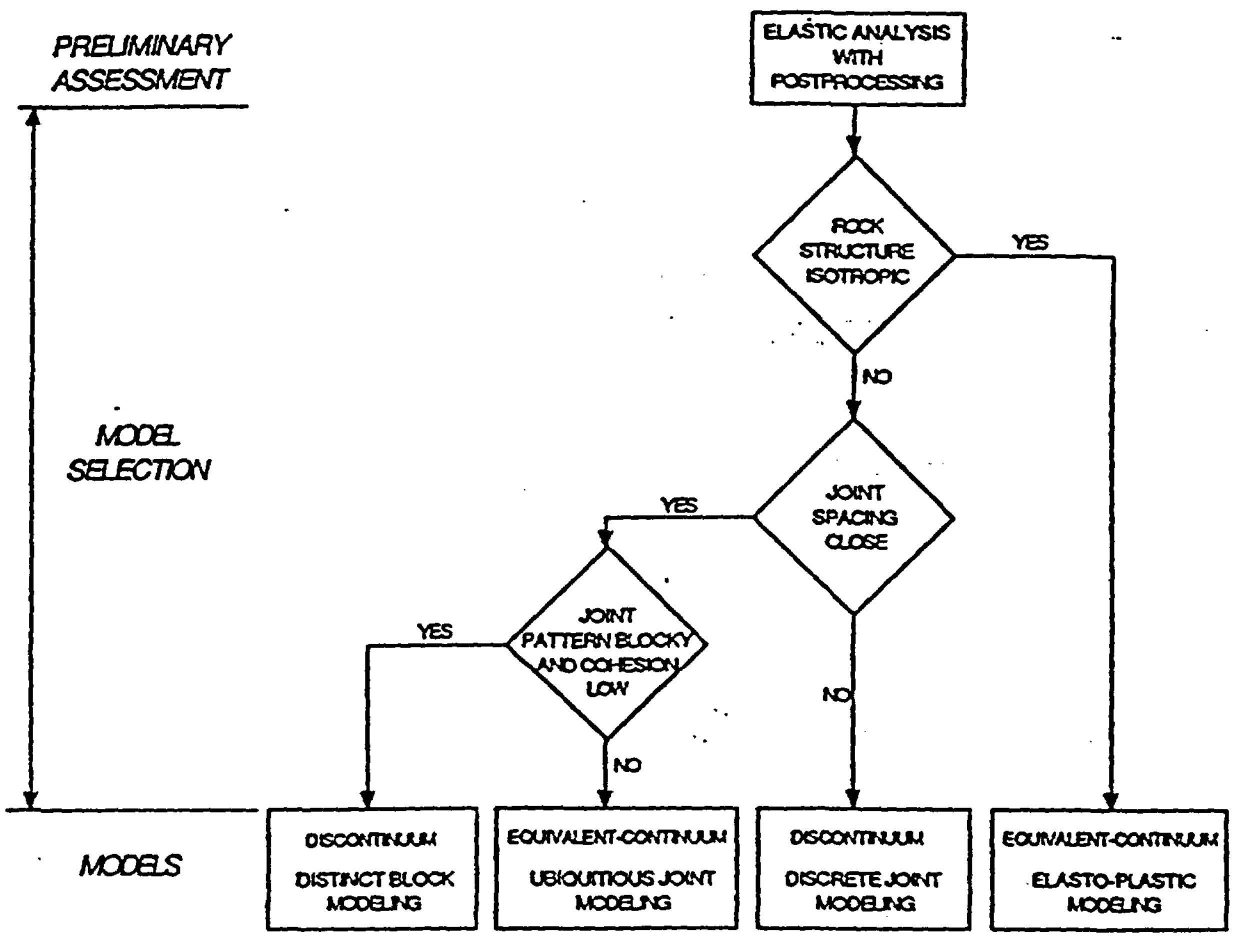



Figure 1-5. Appropriate rock model selection for design analysis (Hardy and Bauer, 1991).

| Study | Test Activities | Thermal Models | Empirical Rock Mass Properties | Theoretical Mechanical Models | Empirical Drift Design Methodologies | Theoretical Thermo-mechanical Models | Empirical Thermo-mechanical Drift Design |
|---|---|---|---|---|---|---|---|
| Soil & Rock Properties Systematic Drilling Program | Core Samples Rock Mass Quality | I | I | | I | | I |
| Study 8.3.1.15.1.1-4 Laboratory Testing | Density and Porosity Characterization | I | I | | I | I | |
| | Volumetric Heat Capacity Determination | I | | | | I | |
| | Thermal Conductivity Characterization | I | | | | I | |
| | Thermal Expansion Characterization | I | | | | I | |
| | Compressive Mechanical Properties of Intact Rock at Baseline Experiment Conditions | | | I | | I | |
| | Effects of Variable Environmental Conditions on Mechanical Properties of Fractures | | | I | | I | |
| | Mechanical Properties of Fractures at Baseline Experiment Conditions | | | I | | I | |
| | Effects of Variable Environmental Conditions on Mechanical Properties of Fractures | | | I | | I | |
| Study 8.3.1.15.1.7 In Situ Mechanical Properties | Plate-Loading Tests | | V | V | | I | |
| | Rock-Mass Strength Experiment | | V | V | | I | |
| | Block Test | | V | V | | I | |
| Study 8.3.1.15.1.5 Excavation Investigations | Shaft Convergence | | | V | | | |
| | Demonstration Breakout Rooms | | | V | | | |
| | Sequential Drift Mining | | | V | | | |
| Study 8.3.1.15.1.8 In Situ Design Verification | Evaluation of Mining Methods | | | | V | | |
| | Monitoring of Ground-Support Systems | | I | | V | | |
| | Monitoring of Drift Stability | | | | V | | |
| Study 8.3.1.15.1.6 In Situ Thermomechanical Properties | Heater Experiment in Unit TSw1 | | | | | V | V |
| | Canister-Scale Heater Experiment | | | | | V | V |
| | Yucca Mountain Heated Block | | | | | V | V |
| | Thermal Stress Measurements | | | | | V | V |
| | Heated Room Experiment | | | | | V | V |

I-Input Data V-Validation Data

Figure 1-6. Flowchart illustrating relationships of rock mechanics study plans to model validation.

# 2.0 SCOPE OF WORK

## 2.1 General Approach

The general approach to defining the scope of testing required to obtain in situ data on mechanical properties is based on the recognition of the dependence of these properties on scale. As proposed by the YMP drift design methodology (Hardy and Bauer, 1991), empirical relationships that allow the incorporation of scale effects have been adopted as the "physical laws" to be verified by the in situ tests. The natural variability of the scaling parameters at Yucca Mountain will be used to define the testing requirements and to extrapolate the results of the limited testing to different parts of the potential repository. The overall approach is based on four key elements:

- *Identify the data needs:* Data needs were identified for the four areas of the repository program discussed in Section 1. These needs will be reevaluated periodically to account for evolution in the repository design.
- *Define methodology to account for scale effects:* Site data on rock mass quality and joint scale and roughness will be used to plan the testing. Final test sites will be based on the spatial variability of these data in ESF excavations in the potential repository horizon.
- *Define specific tests that will produce the required data:* Testing approaches are based on in situ tests described in the literature, standard test methods suggested by ASTM, and test approaches made possible by developments specific to Yucca Mountain.
- *Schedule tests to satisfy data needs on a "just-in-time" basis:* The tests of in situ mechanical properties will be scheduled to meet the data needs and priorities under the different phases of the project.

## 2.2 Key Parameters Measured

The rock mass mechanical properties to be measured by this study include two domains: rock mass-equivalent continuum and large-scale joint surfaces. Rock mass properties incorporate the effects of joint structure as an equivalent continuum, and their variability has typically been defined with respect to the rock mass quality index. Large-scale joint properties are related to the scale of the surface, its roughness/waviness at that scale, and the compressive strength of rock at that scale. The properties to be measured and the variables are listed in Table 2-1.

Table 2-1. Key Parameters Measured in the In Situ Mechanical Properties Studies

| Domain | Property | Variables Investigated | Variability Controlled by |
|---|---|---|---|
| Rock mass | Deformation modulus | Stress | Rock mass quality |
| Rock mass | Strength | Stress | Rock mass quality |
| Joint surface | Shear strength | Normal stress<br>Normal displacement | Surface roughness, length, rock compressive strength |
| Joint surface | Shear stiffness | Normal stress | Surface roughness, length, rock compressive strength |
| Joint surface | Normal stiffness | Shear stress | Surface roughness, length, rock compressive strength |
| Joint surface | Dilation angle | Normal stress | Surface roughness, length, rock compressive strength |

### 2.2.1 Scale Effects in Equivalent Continuum Properties

Scale effects for equivalent continuum properties (rock mass strength, rock mass deformation modulus) have been related to rock mass quality by Yudhbir et al. (1983), Hoek and Brown (1983), Serafim and Periera (1983), and Barton et al. (1980). Both the rock quality indices, Q (Barton et al., 1974) and RMR (Bieniawski, 1979) have been used as the scale variable, to incorporate the effects of inhomogeneities introduced by joints, fractures, weathering, and chemical alteration at the tunnel scale.

Examples of rock mass strength criteria adopted in the YMP drift design methodology (Hardy and Bauer, 1991) are presented in Equations 2-1 and 2-2.

Yudbhir criterion:

$$\sigma_1 = A\sigma_c + B\sigma_c \left\{ \frac{\sigma_3}{\sigma_c} \right\}^{\alpha} \qquad (2\text{-}1)$$

where: $\sigma_1$ = rock mass strength;

$\sigma_3$ = confining stress;

$\sigma_c$ = laboratory-scale intact uniaxial compressive strength;

$\alpha$, $B$ = constants for different rock types, defined by fitting into laboratory data; and

$A = f$(RMR).

Hoek and Brown criterion:

$$\sigma_1 = \sigma_3 + \sqrt{m\sigma_c\sigma_3 + s\sigma_c^1} \qquad (2\text{-}2)$$

where: $\sigma_1$, $\sigma_3$, $\sigma_c$ are identical to Equation 2-1;
$m = f$ (RMR); and
$s = f$ (RMR).

The scale effects are introduced by the terms $A$ and $m$, $s$ in Equations 2-1 and 2-2, respectively. These terms are functions of RMR as shown by Equation 2-3.

$$A = e^{(0.0765\,RMR-7.65)}$$
$$m = m_i e^{(RMR-100)/28} \qquad (2\text{-}3)$$
$$s = e^{(RMR-100)/9}$$

where $m_i$ is the value of $m$ for the intact rock.

The drift design methodology proposed that strengths calculated using Equations 2-1 and 2-2 be averaged and that a composite curve fit be used to produce a power law fit of the form

$$\sigma_1 = a + b\sigma_3^c \qquad (2\text{-}4)$$

where a, b, and c are constants. Predicted rock mass strengths, based on this approach, were presented for the Tiva Canyon welded unit in Figure 1-4, for values of RMR at various levels of cumulative frequency of occurrence.

The physical basis of the criteria in Equations (2-1) and (2-2) is highly experimental and very little data are available to judge their adequacy. The testing program must provide data to verify or qualify their application to YMP.

Price (1986) proposed the following equation for size effects in YMP tuffs:

$$\sigma_c = 1944D^{-0.846} + 69.5 \qquad (2\text{-}5)$$

where: $\sigma_c$ = uniaxial compressive strength (MPa), and
$D$ = sample diameter (mm).

A similar approach has been utilized for the rock mass deformation modulus. Data from field tests and equations relating deformation modulus to RMR were presented by Bieniawski (1984), as shown in Figure 1-3. Another alternative, correlating deformation modulus with the rock mass quality Q, is shown in Figure 2-1. Figures 1-3 and 2-1 present historical data that allow an assessment of the validity of the predictive equations. The scatter in the data provide a basis for estimating the number of tests required to deliver a specified level of confidence. Data collected under this study will be compared with the data and equations shown in the figures to evaluate the applicability of this approach.

### 2.2.2 Scale Effects In Joint Mechanical Properties

Scale effects in joint mechanical properties were described by Bandis et al. (1981) for a unique set of experiments on model joints where similitude was used to extrapolate the results to an 11- to 12-m range. These experiments indicated a high degree of scale dependence in peak shear strength, shear stiffness, normal stiffness, and dilation angle at peak shear displacement.

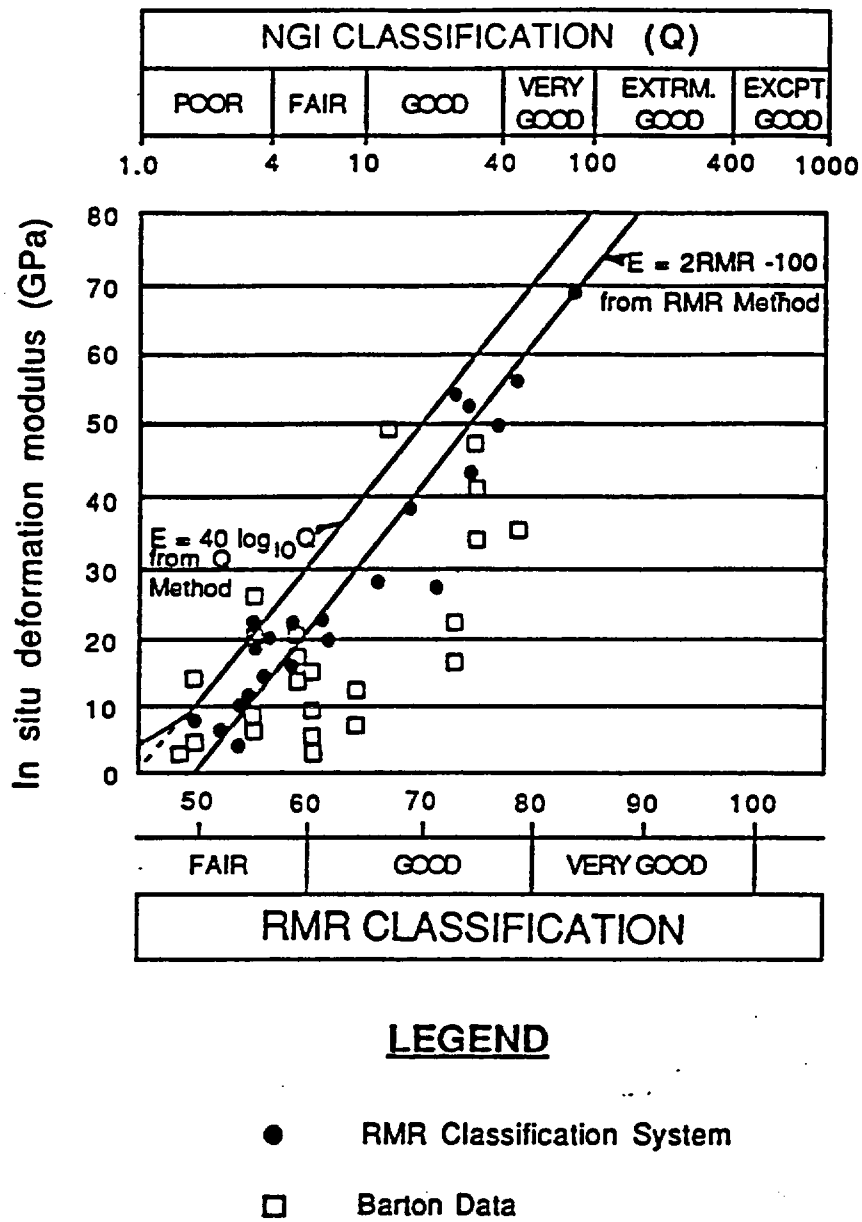



Figure 2-1. Estimation of in situ deformation modulus from rock mass classification methods (Barton et al., 1980).

S. Bandis, A. C. Lumsden and N. R. Barton

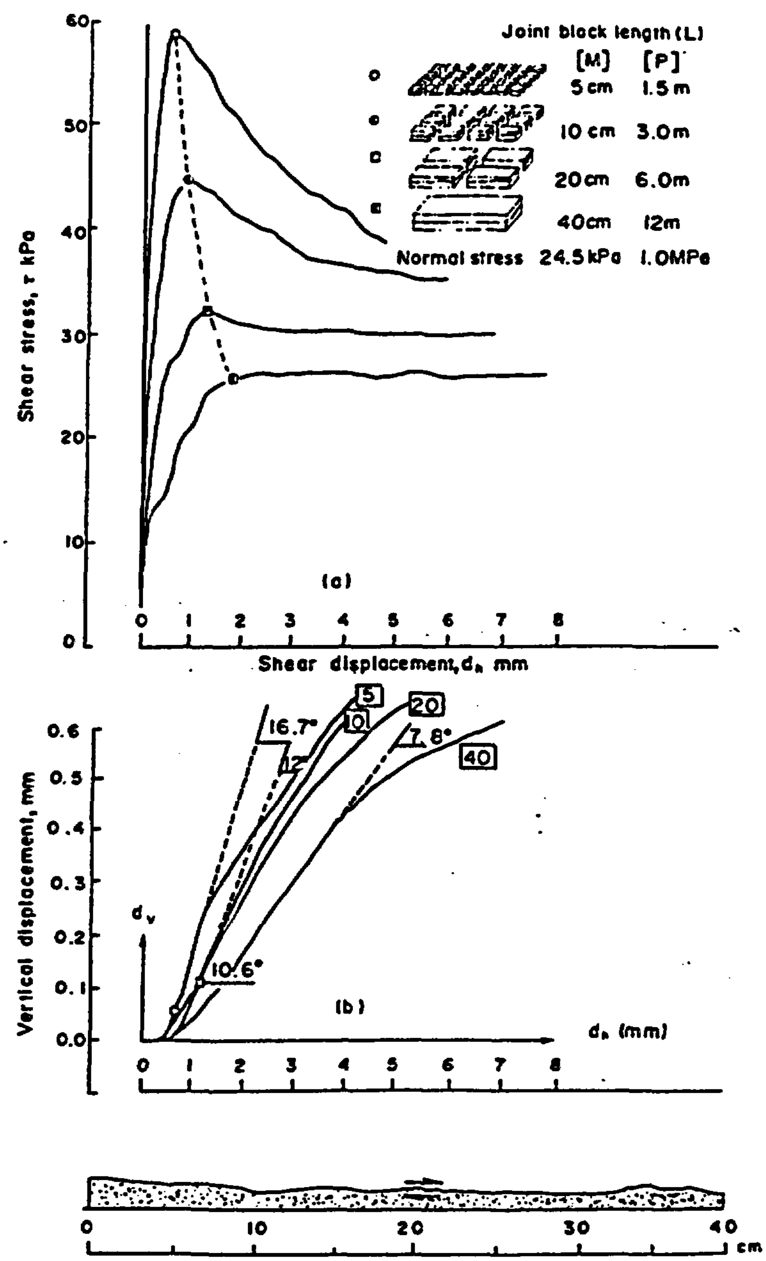



Figure 2-2. Cumulative mean shear stress—shear displacement (a) and dilation (b) curves of *model joints* (Bandis et al., 1981).

This scale dependence was attributed to variations in the surface roughness at different scales and resulted in substantial differences in peak shear strength, as illustrated by Figure 2-2. Peak shear strengths from laboratory tests are shown by the figure to potentially overestimate strengths and shear stiffness at the larger scale by 100%.

One approach to scale dependence is the development of a shear strength criterion by Barton and Choubey (1977) that is based on roughness and basic material strength indices as presented in Equation 2-5:

$$\tau_p = \sigma_n \tan\left[\mathrm{JRC}\log_{10}\left(\frac{\mathrm{JCS}}{\sigma_n}\right) + \phi_r\right] \qquad (2\text{-}5)$$

where: $\tau_p$ = peak shear strength;
$\sigma_n$ = joint normal stress;
JRC = joint roughness coefficient;
JCS = joint surface compressive strength (typically uniaxial compressive strength)
$\phi_r$ = residual angle of friction.

This expression simplified characterization of joint strength by utilizing parameters that could be measured without performing complex shear measurements in the laboratory. Joint roughness coefficient (JRC) was based on standard roughness profiles proposed by Barton and Choubey (1977) and presented in Figure 2-3 for a 10-cm length of joint surface. Surfaces could be visually compared with the reference profiles. JCS could be derived from the easily measured uniaxial compressive strength and $\phi_r$ could be measured on saw-cut surfaces using tilt tests. A great many of these simple measurements could be made.

Both JRC and JCS were shown to be scale dependent by Bandis et al. (1981) because of the mobilization of asperities of different base length and size effects in uniaxial compressive strength. These two effects are illustrated in Figure 2-4 for the model studies by Bandis et al. (1989). Similar effects can be anticipated in the welded tuffs of the potential repository horizon.

Joint deformation is usually characterized by shear and normal stiffness, which have units of pressure divided by displacement (e.g., psi/in.; MPa/mm). Joints characteristically exhibit nonlinear behavior in both properties; however, calculation models typically assume that shear stiffness is bilinear with a postpeak stiffness much lower than prepeak values and that the postpeak is defined by the onset of yield.

Scale effects on shear stiffness were illustrated by Figure 2-2, which indicates that greater displacement is required to reach peak shear stress for larger surfaces. This effect is also shown in Figure 2-5, where scatter in shear stiffness is plotted for discontinuous surfaces from laboratory scale to earthquake fault scale.

Suggested Methods for the Quantitative Description of Discontinuities

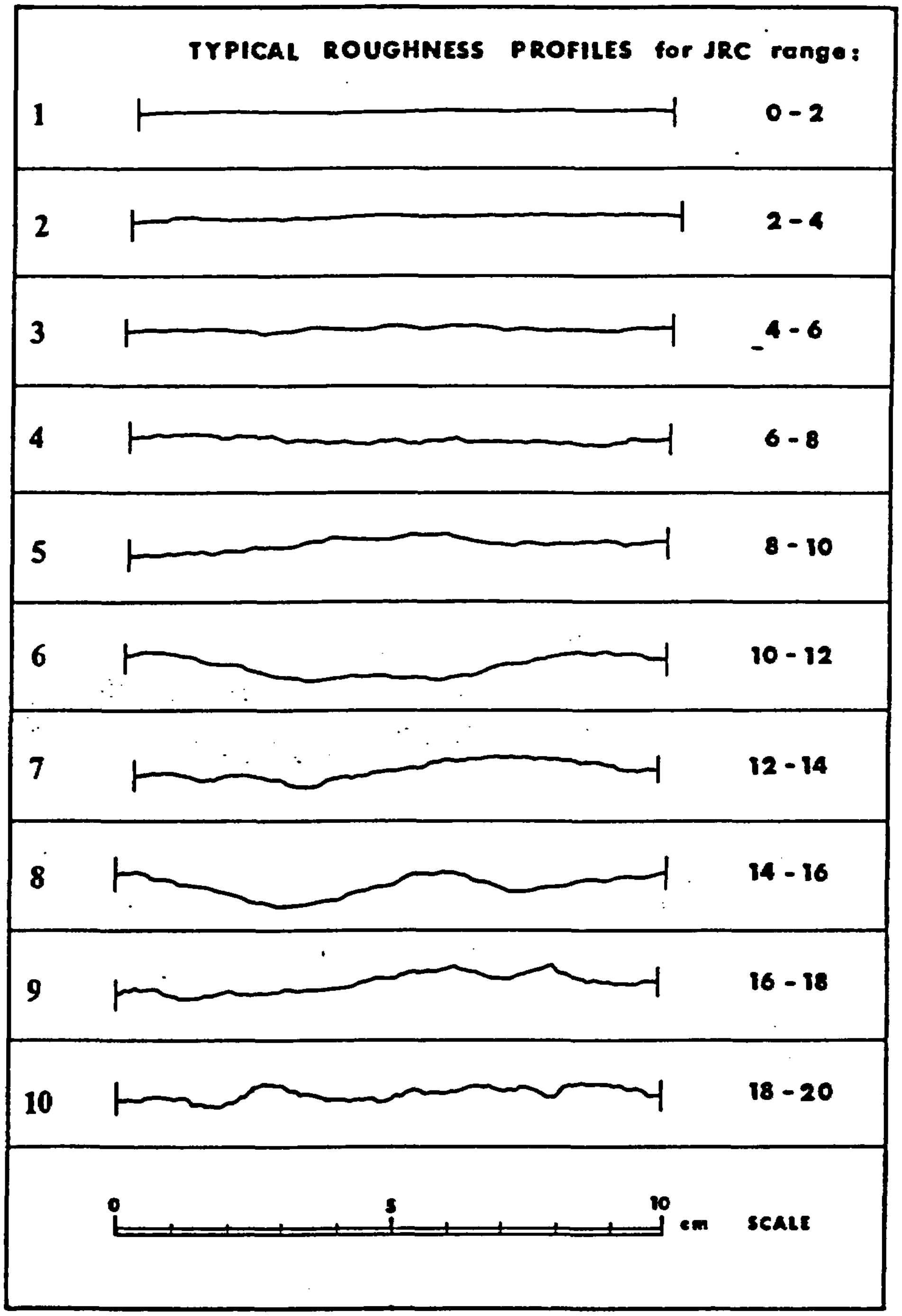



Figure 2-3. Roughness profiles and corresponding range of JRC values associated with each one (Barton and Choubey, 1977).

Scale Effects on the Shear Behaviour of Rock Joints

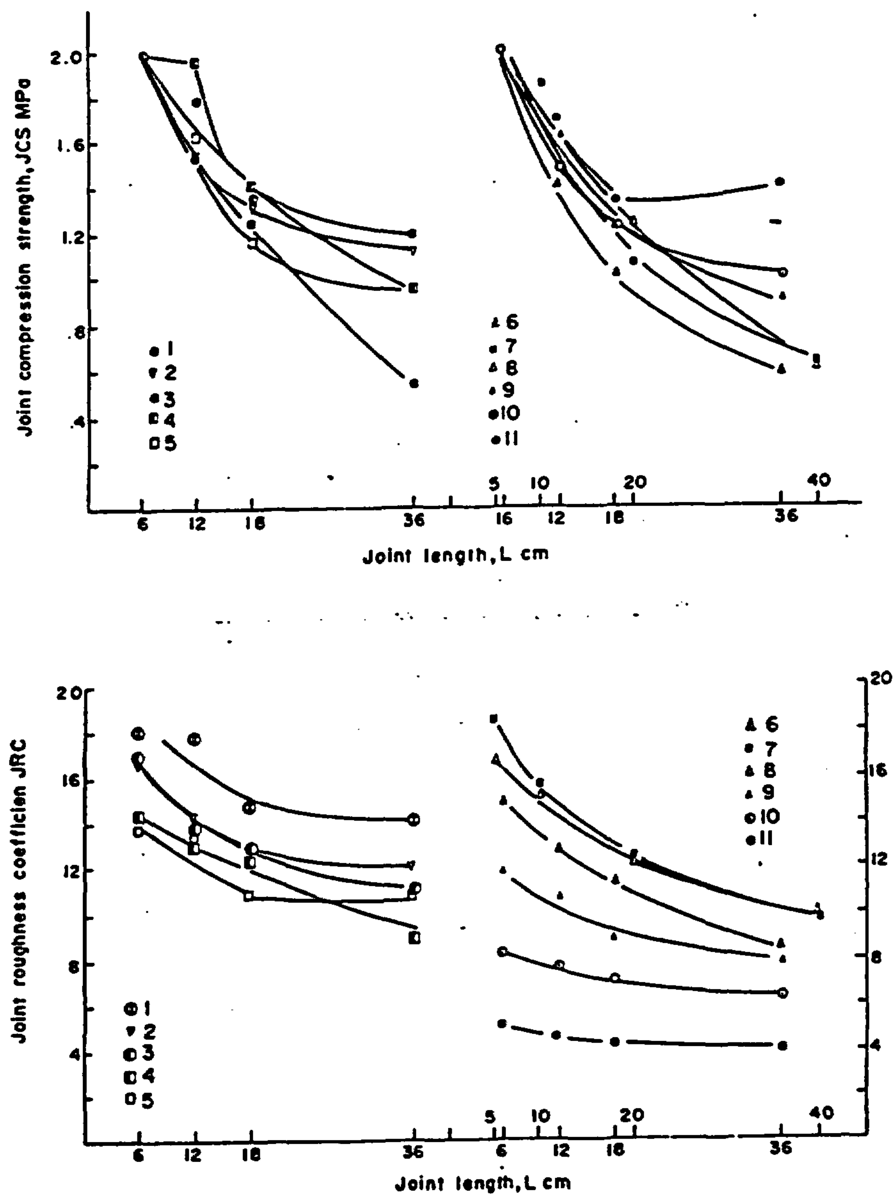



Figure 2-4. Effect of scale on the joint roughness coefficient (JRC). Values of JRC were back-calculated from Equation 2-5 using the scale-corrected values of JCS (Bandis et al., 1981).

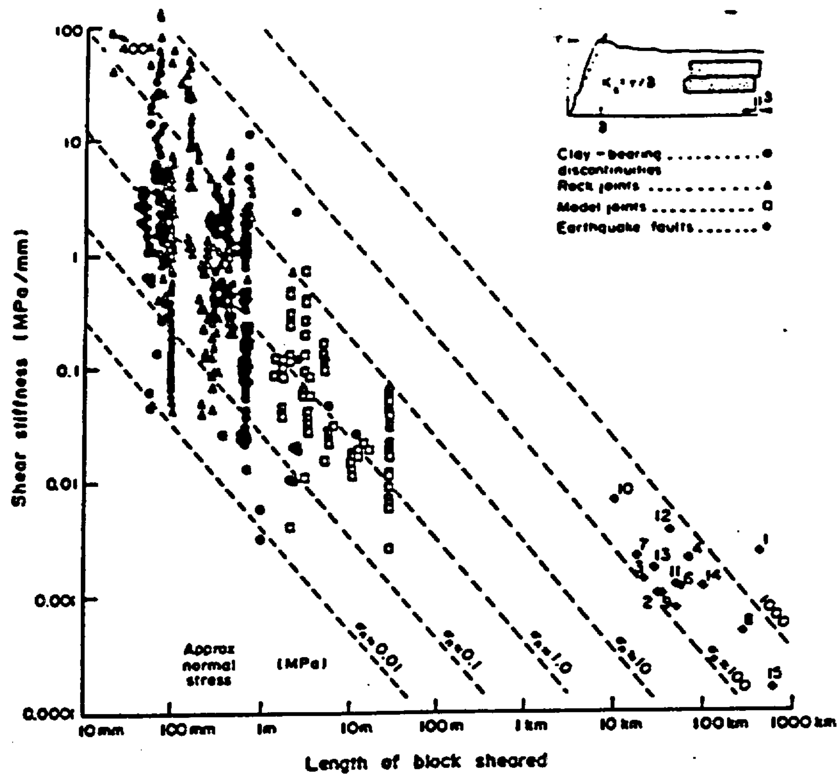



Figure 2-5. Experimental evidence for the scale effect on peak shear stiffness. The normal stress diagonals were tentatively extrapolated from tests at 100-mm size, from the measured effects of scale on JRC, JCS, and $d_h$ in the 100-mm to 1-m size range (Bandis et al., 1983).

Barton and Choubey (1977) suggest the empirical relationship:

$$K_s = \frac{100}{L}\sigma_N \tan\left[\text{JRC}\log_{10}\left(\frac{\text{JCS}}{\sigma_N}\right)+\phi_r\right] \qquad (2\text{-}7)$$

where $K_s$ = joint stiffness;

$L$ = joint length;

$\sigma_N$ = joint normal stress; and

JRC, JCS, and $\phi_r$ are as previously defined.

In this approach, a scale dependence of shear stiffness can be accommodated through the three terms of length (L), JRC, and JCS. Bandis et al. (1983) derived a similar approach for normal stiffness and developed relationships for the maximum joint closure ($M_{max}$) and initial normal stiffness ($K_{ni}$) shown in Equations 2-8 and 2-9.

$$M_{max} = A + B(\text{JRC}) + C\left(\frac{\text{JCS}}{aj}\right)^D \qquad (2\text{-}8)$$

where: A, B, C, and D are constants; and

aj = initial joint aperture = $\frac{\text{JRC}}{5}\left(0.2\frac{\sigma_c}{\text{JCS}}-0.1\right)$

$$K_{ni} = -7.15 + 1.75\ \text{JRC} + 0.02\left(\frac{\text{JCS}}{aj}\right) \qquad (2\text{-}9)$$

Bandis et al (1981) proposed reduction criteria to adjust JRC and JCS for scale effects. However, selecting the JRC for a particular joint surface by visual comparison to the profiles in Figure 2-3 is highly subjective, and recent studies have been performed to utilize quantitative measurements of joint roughness. Tse and Cruden (1979) developed correlations between the Figure 2-3 profiles and the root-mean-square slope of surface topography. Brown and Scholz (1985) proposed that the fractal dimension could characterize not only the roughness but the rate of change of roughness with surface size. Their work indicated that the surface roughness, as measured by root-mean-square asperity height, increased strongly with surface size and showed little tendency to level off at sizes up to 1 m. They also indicated that it was important to quantify the degree of correlation, or mismatch, of the surface. More recently, Hsiuny et al. (1995) suggested that the fractal dimension alone for rock profile characterization is not sufficient, and that the intercept of the power spectral density function is also needed to define a profile curve. Their study of the ten profiles in Figure 2-3 suggests that not all the profiles are representative of the roughness class suggested.

## 2.3 Testing Methodology

### 2.3.1 General

The general methodology used in the rock mass scale mechanical properties tests will be to:

- characterize the variability (including spatial) in the rock mass scaling parameters by making a large number of simple measurements in the ESF excavations and
- evaluate the empirical scaling relationships by doing a limited number of in situ experiments.

This methodology will be integrated with the present schedule and the data needs of the repository design and performance assessment. The initial phases of the testing program will concentrate on bounding the variability of rock mass quality and joint surface characteristics as ESF construction proceeds. The variability in these parameters will provide the basis for qualified data to support design for the viability assessment. These data, used in conjunction with laboratory testing data and the empirical scaling relationships, will provide sufficient basis for conservative (C) and bounded (B) assessments of the rock mass mechanical properties. In situ testing will be performed to further bound the data for the viability assessment and License Application Design (LAD), and to support performance assessment beyond 2004.

### 2.3.2 Numbers of Tests/Measurements

Requirements for precision and accuracy of the data are difficult to establish. The SCP set tentative goals for each data need in terms of the qualitative confidence levels shown in Table 2-2.

**Table 2-2. Qualitative Confidence Levels**

| Qualitative Confidence | Associated Level of Statistical Significance |
|---|---|
| High | 0.05 |
| Medium | 0.10 |
| Low | 0.25 |

Where variability was large, the initial values of the number of tests were set to 35, 10, and 5 for high, medium, and low confidence, respectively. The recent thermal testing plan and the repository design data needs adopt a more general strategy using the definitions of completeness presented in the discussion on repository design in Section 1.2.

The variability of the empirical field scaling parameters described previously will be used to define the rationale for the initial testing plans through LAD. Future definitions of the required level of confidence for performance assessment will be evaluated using the characterization results to determine further testing requirements.

Spatial variability will primarily be evaluated through rock mass quality assessments (Q, RMR) and "index" tests of rock mass stiffness such as borehole jacking and geophysical techniques. While rock mass quality assessments will likely be conducted throughout the underground facility, index tests will likely be conducted in regions that differ significantly from "average" conditions as determined from the rock mass quality assessments. Preliminary testing in support of the In Situ Thermal Testing Program will be evluated to determine the need for and nature of follow-on testing to assess spatial variability.

Rock mass quality data, being obtained as part of the North Ramp construction, will be used to develop planning estimates of the number of tests. Data are being collected as part of the In Situ Design Verification Studies (SCP 8.3.1.15.1.8) using a technical methodology[2] that emphasizes the measurement of quantitative joint characteristics as the basis of determining the rock mass quality indices Q and RMR. A similar set of data was generated (Brechtel et al., 1995) to project rock mass quality variations in the North Ramp on the basis of surface core drilling data. The borehole projections and the tunnel assessments for the TCw thermal-mechanical unit are compared in Figure 2-6 as cumulative frequency of occurrence of RMR. A curve is also plotted for the borehole projections of RMR for the potential repository horizon (Kicker et al., 1995), the Topopah Springs Middle nonlithophysal zone. Based on the SCP tentative goal for rock mass strength of medium confidence, tests should be located so that the maximum RMR is greater than or equal to 66.7 (cumulative frequency = 90%) and the minimum RMR is less than 52.4 (cumulative frequency = 10%) to bound the indicated variability. This would bound the range and at least one intermediate RMR value would be required to confirm the trend of the empirical scaling relationship.

A second variable must be investigated for both rock mass strength and joint mechanical properties, the confining stress or normal stress, respectively. The projected effect of confining pressure was illustrated for rock mass strength in the Topopah Springs Middle nonlithophysal zone in Figure 2-7 for RMR values at 5%, 20%, 40%, 70%, and 90% cumulative frequencies of occurrence (classes 1 to 5), respectively. Tests at three different confining pressures would be required to define confinement effects. Although Figures 2-6 and 2-7 discuss the use of RMR, data will also be collected for the Q-System which is currently used for design of the underground facilities. A similar process would be implemented to measure the joint mechanical properties. Estimated numbers of tests through LAD are listed in Table 2-3.

[2] Sandia National Laboratories, Technical Procedure No. 234: Conducting Rock Mass Quality Assessments.

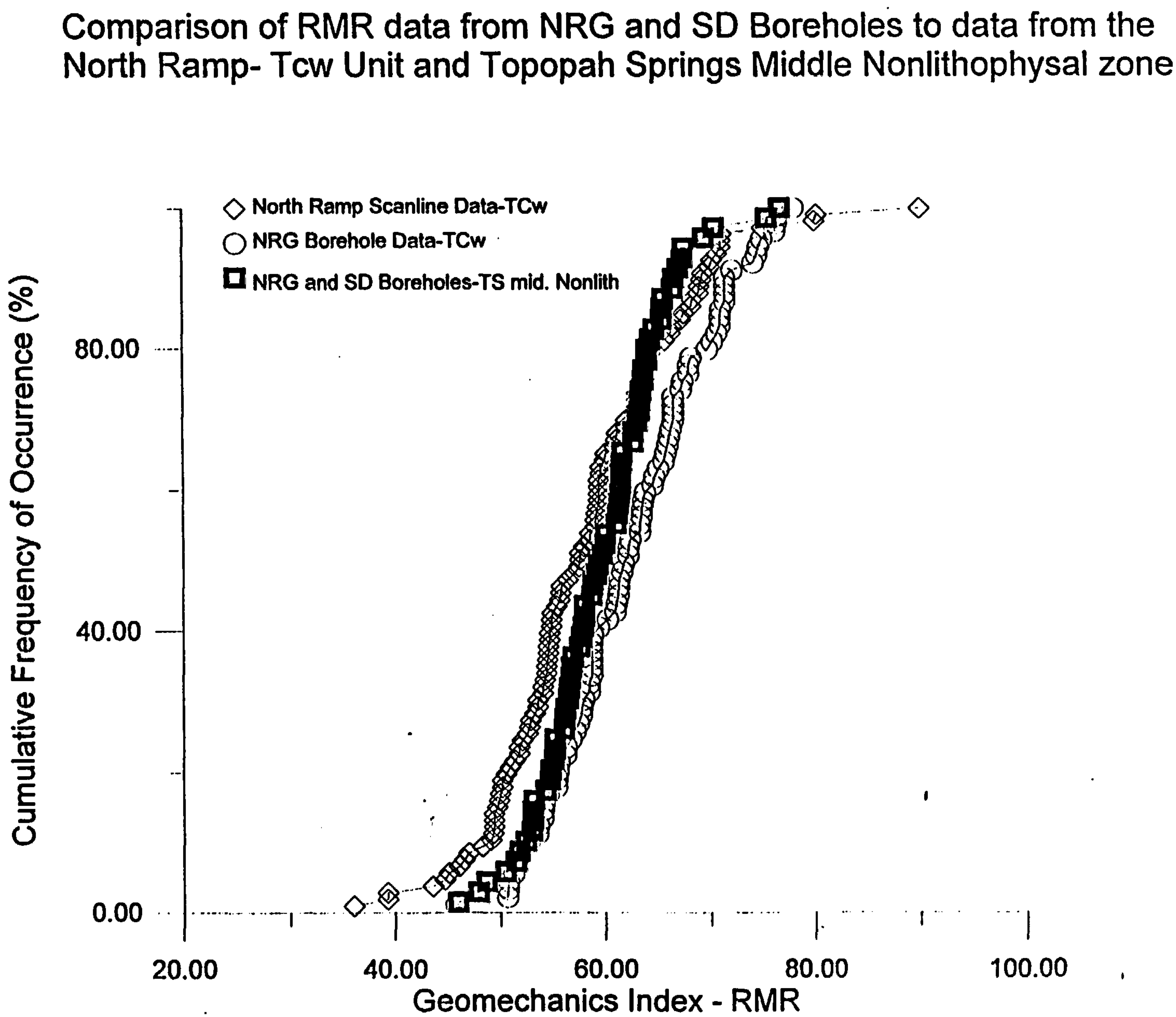



Figure 2-6. Variation of RMR data from YMP characterization program.

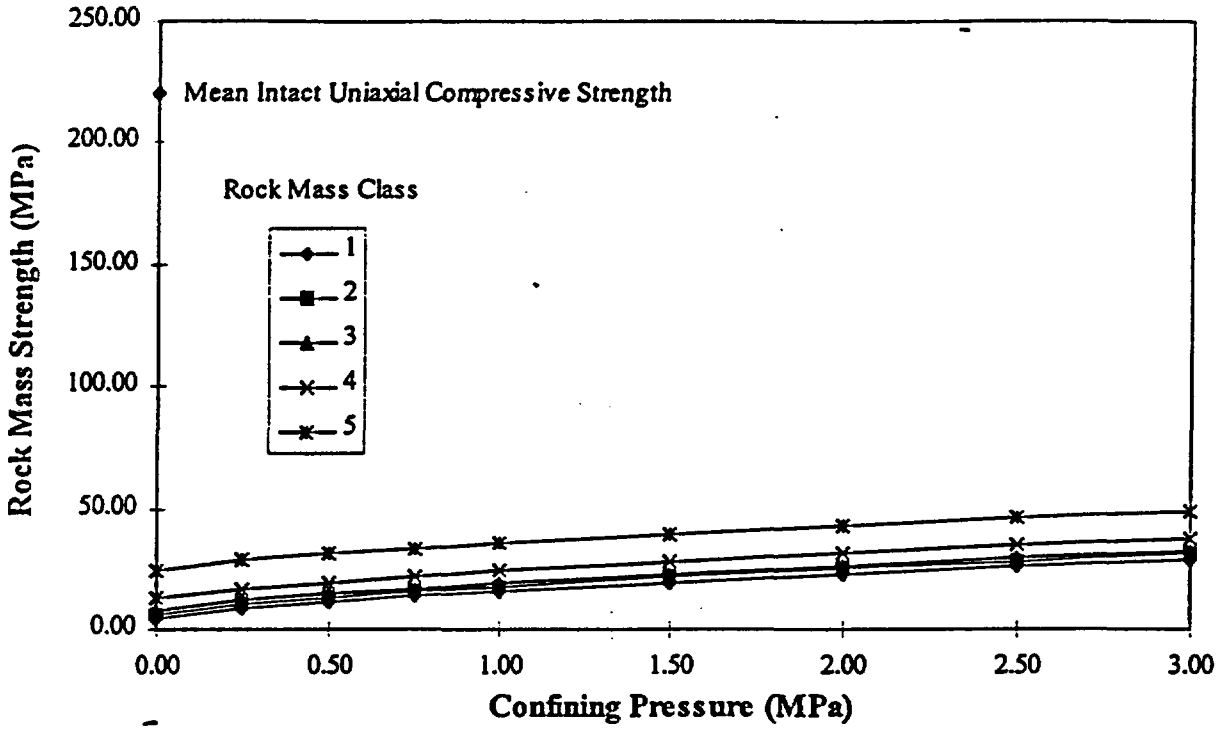



Figure 2-7. Design rock mass strength envelopes for the Topopah Spring Tuff middle nonlithophysal zone—main drift boreholes.

Table 2-3. Projected Numbers of In Situ Tests

| Parameter Measured | Viability Assessment | | | | LAD | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Locations | Test Pressure | Duplicates | Total | Locations | Pressure | Duplicates per Location | Total | Total Tests |
| Rock mass deformation | 2 | - | 3 | 6 | 1 | - | 3 | 3 | 9 |
| Rock mass strength | 2 | 3 | 3 | 18 | 1 | 3 | 3 | 9 | 37 |
| Joint shear strength and stiffness | 2 | 3 | 3 | 18 | 1 | 3 | 3 | 9 | 37 |

### 2.3.3 Types of Testing

In situ rock mechanics testing techniques were reviewed to assess possible methods to be employed in the ESF. The selected tests are listed in Table 2-4 and are discussed in the following subsection.

Table 2-4. Types of Testing

| Domain | Property | Test Type |
|---|---|---|
| Rock mass | Deformation modulus | Plate loading test<br>Borehole jacking test<br>Block test<br>Prism test |
| Rock mass | Uniaxial compressive strength | Prism test |
| Rock mass | Confined compressive strength | Prism test |
| Joint surface | Joint shear strength | Block test<br>Slot test |
| Joint surface | Joint shear stiffness and normal stiffness | Block test<br>Slot test |

#### 2.3.3.1 Plate Loading and Borehole Jacking Tests

The plate loading test will be used as the primary method of obtaining the rock mass deformation modulus. These measurements will be supplemented by borehole jacking measurements using the Goodman jack method. The Goodman jack method only activates a small rock volume so it represents modulus at small-scale. Other data may be derived from block tests and slot tests that will be performed to measure joint mechanical properties. Load deformation response measured by the prism tests may also contribute to the evaluation of deformation modulus. These other tests are discussed in later sections.

Early plate-bearing tests are described for a dam foundation investigation by Rocha et al. (1955). Since that time, plate loading tests have been conducted in various locations around the world in conjunction with large civil construction projects such as underground powerhouses, major tunnel projects, and foundation design of large concrete dams. As a result of this history, various formalized procedures have been established for this test (Misterek et al., 1974; Brown, 1981, ASTM D4394, ASTM D4395; Boyle, 1992).

In the plate loading test, the rock mass under a plate is loaded by jacking opposing walls of an exploratory adit, as illustrated in Figure 2-8. The test results are obtained by monitoring both the jacking pressure and rock deformation under the plate. According to Dodds (1974), other parameters that can sometimes be determined from this test include in situ stress, creep coefficients, rock mass strength, extent of stress relief and/or blast damage, and mode of rock behavior.

Existing ASTM and ISRM procedures will be used in preparing the test procedures. However, the test application for the YMP is sufficiently different from large dam foundations that deviations from these procedures may be needed. Performing the test in a circular, machine-bored opening may change the surface preparation and test interpretation procedures, particularly the models for determining modulus. Selection of the peak test loads will not be based on a design load imposed by an engineered structure, as in a dam foundation, but on the range of stresses expected in the vicinity of underground excavations. These will be based on the results of repository design analyses.

Borehole jacking tests provide a method for making multiple measurements at the sites of the larger, plate-bearing tests. The approach is illustrated in Figure 2-9, which shows a borehole dilatometer, which provides a measurement of deformation modulus by applying a uniform pressure to the borehole wall. Deformation is determined by volumetric measurement of the pressurizing fluid. The Goodman jack is similar in principle but produces a directional measurement by pressing opposing platens against the borehole wall. Multiple measurements can be made in the borehole at various depths. These measurements can evaluate the depth of damage caused by the excavation technique and can be used to improve interpretation of the plate loading tests as well as provide estimates of the spatial variability of rock mass deformation modulus throughout the facility.

### *Test Construction*

Figure 2-8 is a sketch of the plate loading concept for the ESF. Because the ESF is an underground facility, it is assumed that there will be a suitable opposing reaction surface in all cases.

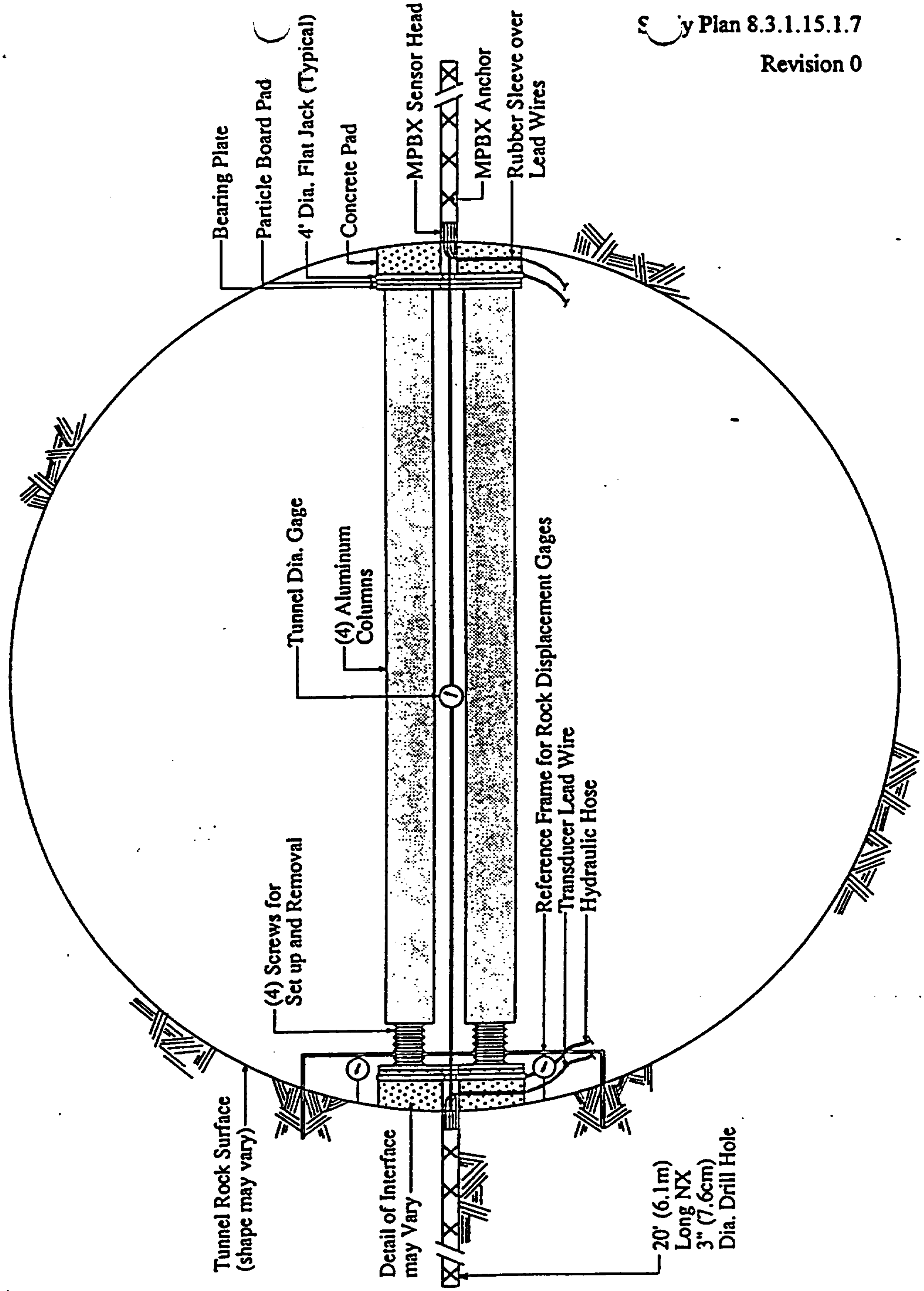
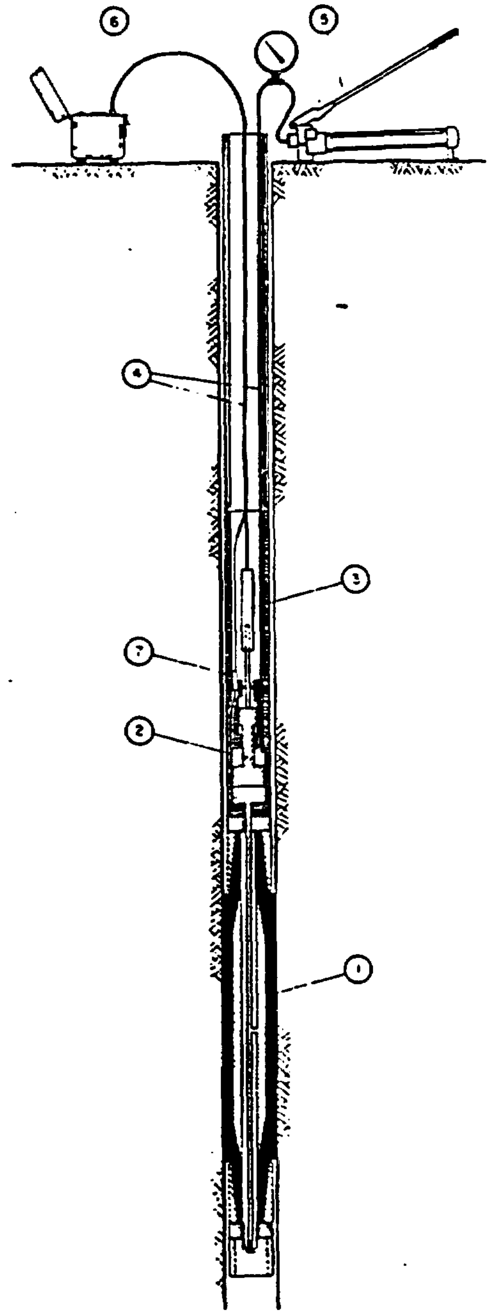



Figure 2-8. Typical setup for performing plate bearing test (not to scale).



1. an inflatable membrane mounted on a steel core.
2. a hydraulic module comprised of a dual piston and cylinder assembly to inflate and deflate the membrane.
3. a measuring module containing a linear transducer which monitors the injected volume.
4. the hydraulic and electrical lead lines.
5. a hydraulic hand pump and pressure gauge.
6. a digital readout.
7. an optional pressure transducer.

Figure 2-9. Schematic illustrating borehole jack for determination of deformation modulus (From Roctest, 1995)

### *Pretest Characterization and Laboratory Testing*

The prepared site will be chosen from rock mass quality assessments based on the desired range (see Section 2.3.2) and then mapped in detail, photographed, and tied into the tunnel survey grid. Core taken from the instrument holes will be logged and photographed. The hole itself will be logged with a borescope or borehole TV camera. Particular note will be taken of fractures that may influence test results. Rock type, orientation and inclination of discontinuities, size of lithophysal cavities, weathering and alteration, and other features will be recorded.

After logging is completed, samples of intact rock will be tested to determine Young's modulus and Poisson's ratio for interpretation of test results. In addition, joint wall compressive strength, joint roughness coefficient, joint geometry, and joint topography will be measured on joints near each experiment location.

### *Instrumentation*

Pressure transducers will be installed on flatjack hydraulic lines and calibrated load cells may be placed in series with the bearing plate. A rod extensometer will be required to measure divergence of the opposing borehole collar anchors. Deformation of the rock surface near the plate will be determined using dial gages or other displacement transducers mounted on a reference frame. Multipoint borehole extensometers (MPBXs) will be used to determine rock displacement at various depths beneath the loaded area. Anchor positions will be carefully selected after reviewing borehole logs using criteria illustrated in Figure 2-10.

It may be desirable to add acoustic emission monitoring to aid test interpretation. Load cells on rock bolts in the vicinity of the tests would also be appropriate.

### *Test Procedure*

Approximately five cycles should be applied, with the value of the peak load as high as practical with the equipment. A typical five-cycle loading sequence is shown in Figure 2-11, with loads applied in 1.4 MPa (200 psi) increments up to 6.0 MPa (1000 psi).

### *Interpretation of Test Results*

Although the plate loading test is relatively standardized, numerous subtleties in the interpretation of test results require discussion. Test results are presented in the form of load-deformation curves. Typical results are shown in Figure 2-12. The principal investigator will be responsible for determining which sections of the curve should be used for modulus determinations and whether secant or tangent values are appropriate. The initial portion of the curve could be concave-down in cases where hardening from in situ stress has occurred (Figure 2-12a), whereas blast damage or other disturbance can result in a concave-up curve (Figure 2-12b).

Because of the test geometry, this slope does not directly define the modulus; however, a modulus can be back-calculated by assuming a physical model. The traditional model is an elastic solution for a uniformly distributed load or displacement over a circular or rectangular area acting on a semi-infinite elastic medium.

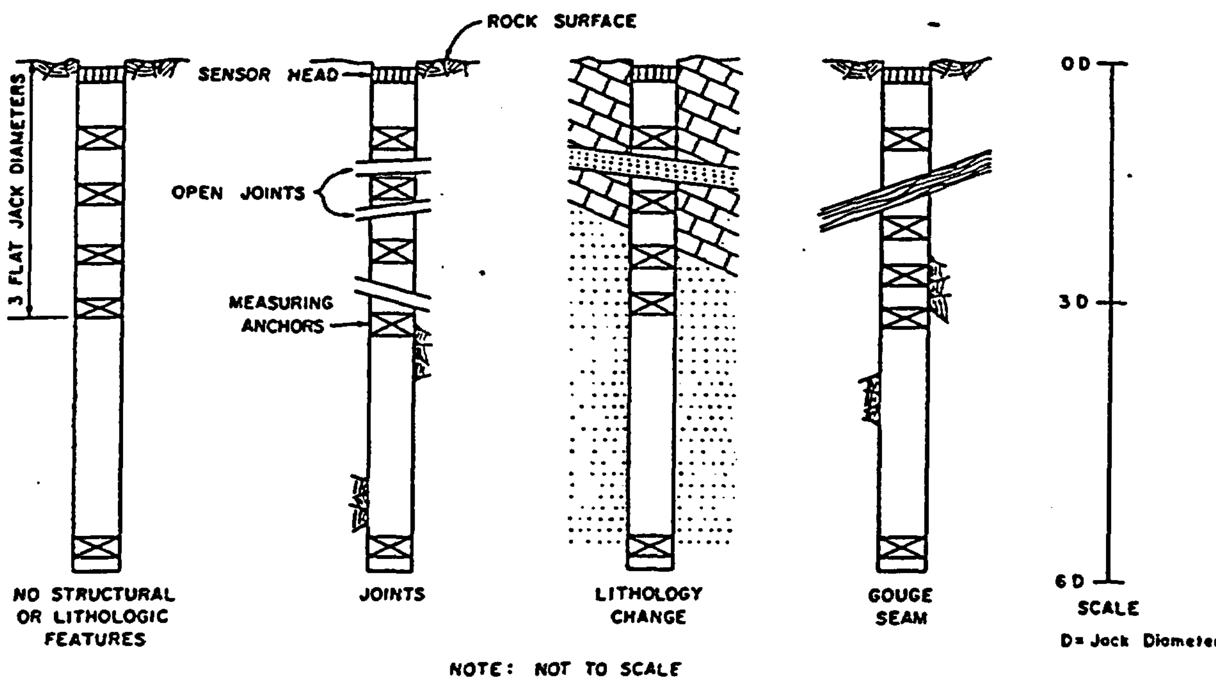



Figure 2-10. Considerations for selecting anchor locations (Brown, 1981:144)

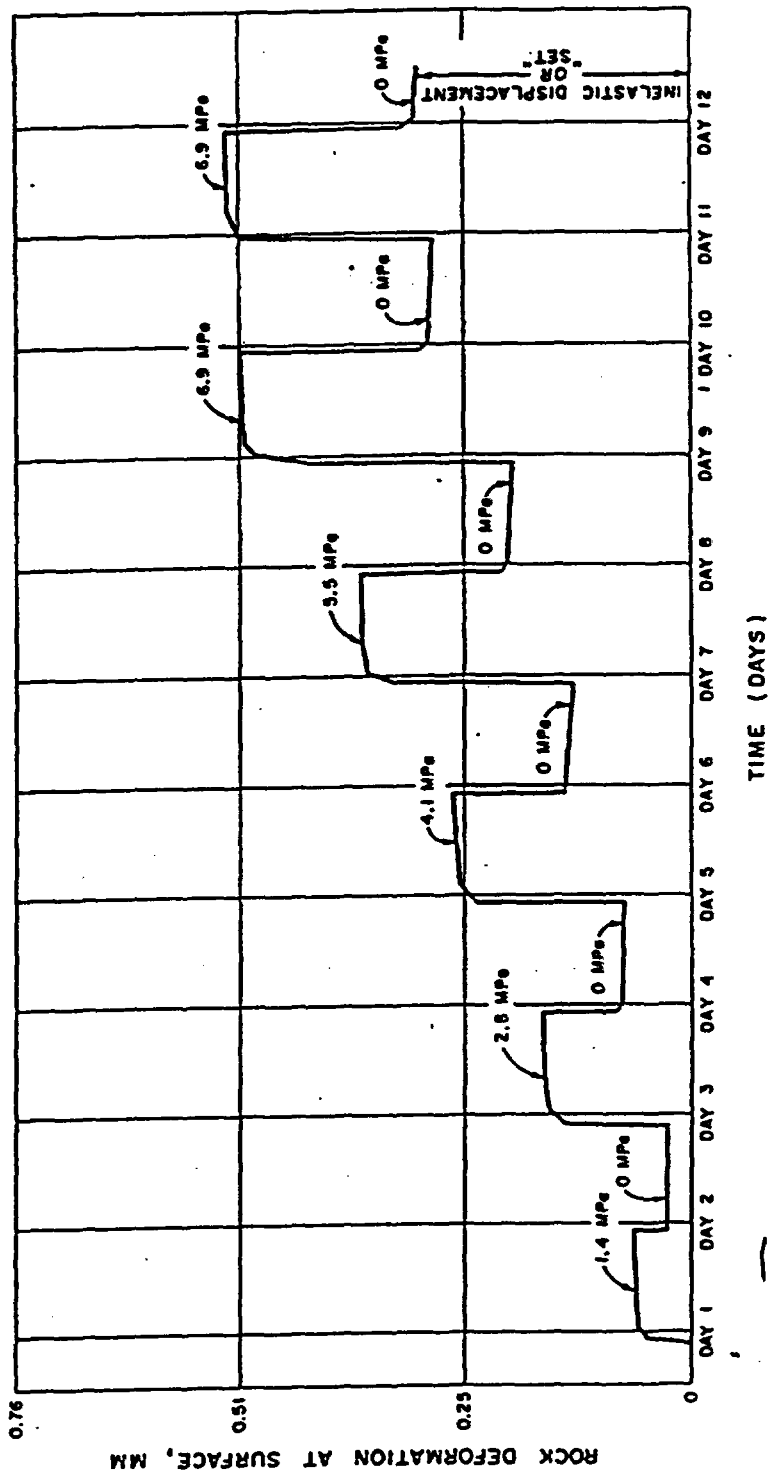
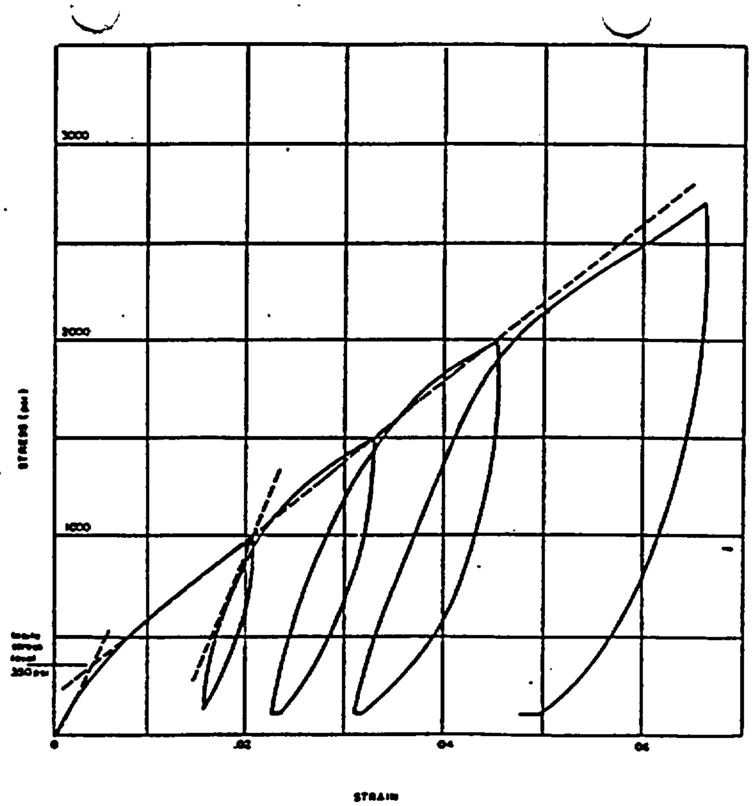



Figure 2-11. Rock deformation at surface versus time: uniaxial jacking test (from Brown, 1981: 146)



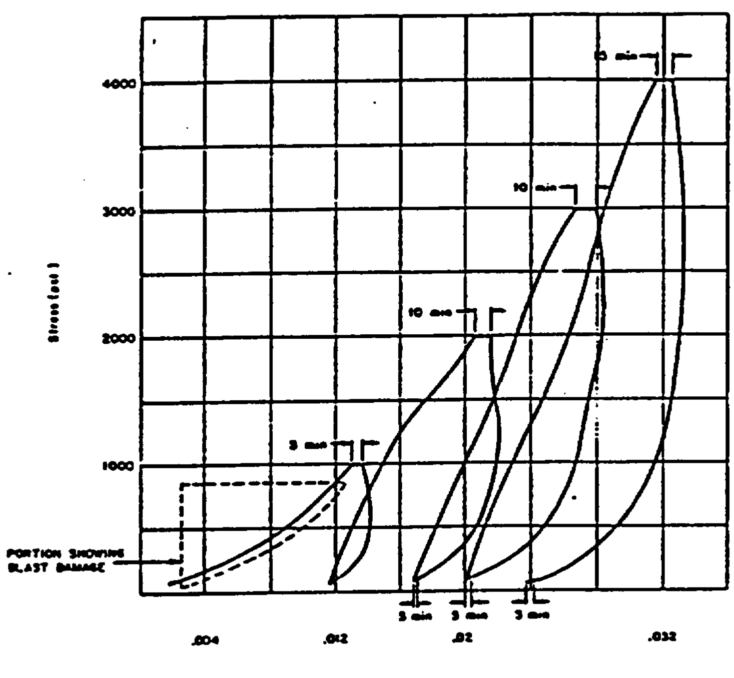




Figure 2-12. (a) Typical curve showing stress history (from Dodds, 1974: 25); (b) typical curve showing blast damage (from Dodds, 1974: 27).

Two variations of this solution are typically used: (1) the rigid plate (constant displacement) and (2) the flexible plate (uniform stress) solution. The appropriate solution must be determined in advance and the test designed accordingly. ASTM has gone so far as to provide two separate procedures for the two models. However, as Jaeger and Cook (1976, p.389) point out: "In practice, the boundary conditions beneath the bearing plate must be between the extremes represented by a constant displacement and a uniform stress, respectively. The stress at the perimeter of a completely rigid bearing plate would be infinite, and the rock must in fact fail".

The modulus can be calculated from the measured displacements of the rock mass under the plates using the theory of elasticity. The general form of the equation for a loaded area is:

$$E = \frac{CP(1-\upsilon^2)B}{w}$$

where: w = average displacement under the plate
C = shape and rigidity factor
B = characteristic dimension of the loaded area
P = magnitude of the uniformly distributed load (pressure)
$\upsilon$ = Poisson's ratio

For a circular loaded area, B is equal to the diameter and for a rectangular loaded area, B is equal to the least dimension.

Table 2-5 lists shape and rigidity factors for a number of cases.

**Table 2-5. Shape and Rigidity Factors $C_d$ for Calculating Settlements of Points on Loaded Areas at the Surface of an Elastic Half-Space**

| Shape | Center | Corner | Middle of Short Side | Middle of Long Side | Average |
|---|---|---|---|---|---|
| Circle | 1.00 | 0.64 | 0.64 | 0.64 | 0.85 |
| Circle (rigid) | 0.79 | 0.79 | 0.79 | 0.79 | 0.79 |
| Square | 1.12 | 0.56 | 0.76 | 0.76 | 0.95 |
| Square (rigid) | 0.99 | 0.99 | 0.99 | 0.99 | 0.99 |
| Rectangle: Length/Width | | | | | |
| 1.5 | 1.36 | 0.67 | 0.89 | 0.97 | 1.15 |
| 2 | 1.52 | 0.76 | 0.98 | 1.12 | 1.30 |
| 3 | 1.78 | 0.88 | 1.11 | 1.35 | 1.52 |
| 5 | 2.10 | 1.05 | 1.27 | 1.68 | 1.83 |
| 10 | 2.53 | 1.26 | 1.49 | 2.12 | 2.25 |
| 100 | 4.00 | 2.00 | 2.20 | 3.60 | 3.70 |
| 1000 | 5.47 | 2.75 | 2.94 | 5.03 | 5.15 |
| 10000 | 6.90 | 3.50 | 3.70 | 6.50 | 6.60 |

(from Winterkorn & Fang, 1975).

The results from the plate loading tests will also be evluated using continuum and/or discrete block numerical models described in this study plan.

Dodds (1974) states "that the equations used in computing the moduli are generally based on elastic theory. These equations assume that the rock mass is an isotropic, homogeneous body with ideal elastic properties. The validity of the computed deformation modulus will depend on the extent to which the in situ rock conforms to the preceding assumptions and how well the actual test condition conforms to the conditions imposed upon the completed structures. An error of 10 percent would be considered excellent and 20 percent acceptable at the present time."

In jointed rock, the rock mass modulus would be expected to be one half or less of the intact rock modulus, depending on the joint characteristics. Care must be taken in interpreting a modulus for design purposes. As Goodman (1980) points out: "almost any departure from conditions assumed will tend to increase the measured displacements so the plate bearing test tends to underestimate the deformation modulus of elasticity. Tests conducted vertically in galleries will usually give yet lower values of the deformation modulus because joints in the roof rock tend to open under gravity". Results from this test will represent lower bounds on the modulus of the rock mass.

### *Potential Impacts on Site*

The plate loading test, when used to determine deformation modulus, is a nondestructive test having a minimal impact on the site. Extensometer holes can be drilled dry or with a minimum amount of water. Rock surface preparation will consist of either chipping and cleaning the rock surface or cutting test surfaces with the SNL hydraulic rock saw system. No heating is planned in conjunction with these tests, although the test can be conducted in adits adjacent to heated areas to examine the effect of temperature on deformation modulus.

### *Simulation of Repository Conditions*

The plate loading test is not designed to simulate repository conditions. However, it can be used to examine the stress dependency of the deformation modulus over the range of stresses expected in the repository. Thermal loading is the mechanism responsible for high stresses in some areas of the repository. At present it is uncertain what the design gross thermal loading will be in the repository. However, it is likely that a wide range of stresses between zero and the rock strength will be encountered at least locally. Accordingly, to be appropriate for design and performance calculations over this range, the peak load should be as near the rock strength as feasible. Plate pressures are limited by available flatjack technology, which is currently limited to 55 MPa (8000 psi) or less in perfectly constrained load situations.

### *Relationship Between Laboratory and Field-Scale Phenomena*

Table 2-6 shows reported values for field/laboratory modulus ratio from a number of projects. It is anticipated that the field modulus will be significantly lower than the lab modulus at Yucca Mountain because of the highly jointed nature of the rock.

**Table 2-6. Field and Laboratory Moduli by Plate-Bearing Test at Major Projects**

| Project (Date) | Type of Rock | No. of Tests | $E_F$ (GP)* | $E_L$ (GPa) | $E_F/E_L$ |
|---|---|---|---|---|---|
| Oroville Dam (1961) | Amphibolite (massive) | 5 | 10.4 | 89.0 | 0.11 |
| Tumut 2 (1962) | Gneiss/granite | 6 | 6.9 | 59.1 | 0.12 |
| Dworshak Dam (1966) | Granite/gneiss (massive) | 24 | 23.5 | 51.7 | 0.45 |
| Tehachapi Tunnel (1967) | Diorite gneiss (fracture) | 4 | 4.8 | 77.9 | 0.06 |
| Crestmore Mine (1966 to 1974) | Marble (blocky) | 2 | 15.0 | 47.5 | 0.31 |
| Gordon Scheme | Quartzite | 8 | 19.0 | 67.0 | 0.28 |
| Churchill Falls (1971) | Gneiss | 10 | 41.5 | 55.0 | 0.75 |
| Waldeck II (1973) | Greywacke | Not known | 5.0 | 20.0 | 0.25 |
| Mica Project (1974) | Quartzite gneiss | 12 | 27.6 | 27.0 | 1.04 |
| LG-2 Project (1976) | Granite (massive) | Not known | 50.0 | 80.0 | 0.62 |
| Elandsberg (1977) | Greywacke | 33 | 39.6 | 73.4 | 0.54 |

* $E_F$: field modulus; $E_L$: laboratory modulus at 50% strength.

### *Test Interference*

Plate loading tests are not expected to directly interfere with any of the other experiments. The test should be 10 meters (32.8 ft) from the nearest test which altered the thermomechanical properties of the rock. Only a small region of rock [approximately 1 to 3 cubic meters (10.8 - 32.3 $ft^3$)] will be directly loaded and the effects of the loading will likely extend a distance of only a few times the width of the area over which the load is applied. No permanent alteration to the local hydrological, chemical, or thermal conditions will result from this test. Testing impedes traffic; therefore, test alcoves should be provided.

### *Impact on ESF Construction*

This test will have an impact on ESF construction. Alcoves will be required, although they will be small. However, a plate loading test will block traffic in adjacent access drifts or tunnels. Accordingly, plate loading tests that are planned in drifts required for access to other experiments will have to be carefully scheduled. In most cases, special alcoves may be required to prevent this type of interference. These alcoves will be approximately 4.6 m (15 ft) tall by 2 m (6.6 ft) wide by 18 m (59 ft) long. The actual cross-section of the alcoves will most likely be the minimum that can be machined. Five or more tests may be performed in each alcove. It is preferable that the excavations be mechanically mined so that the surface damage created approximates the mining damage expected in the proposed repository.

Standard underground facilities for water, air, and electricity for drilling will be used for this test. An uninterrupted power supply and data acquisition system will be needed.

### 2.3.3.2 Prism Test

The prism compression test is an in situ method of testing a medium-to-large volume of rock to failure. The purpose of the test is to assess directly the rock mass uniaxial compressive and confined compressive strength by jacking a prism cut out of the rib or floor of an underground excavation. Load and deformation are monitored, providing deformation modulus as well as compressive strength. The in situ prism compression test has traditionally been a relatively expensive method because each test provides only one value for compressive strength and each test requires extensive preparation. It has therefore not been an efficient method of studying the spatial variability of rock mass strength. Improved methods of sample cutting have been developed for the YMP based on the SNL hydraulic rock sawing system. This advance in sample preparation, coupled with movable test equipment, will facilitate multiple tests.

#### *Test Construction*

The uniaxial compressive strength tests can be constructed in drifts and alcoves located in the ESF. Arrays of samples can be constructed by the SNL rock sawing system to produce multiple test specimens at each location. Figure 2-13 illustrates sample arrays prepared by cutting multiple parallel and perpendicular slots with the rock sawing system. This approach will allow duplicate tests in similar rock conditions. Within the sample array, tests will include both confined and unconfined tests. Confining pressures will be generated by placing flatjacks in the slots. Axial load would be generated by a column jacking system as illustrated in Figure 2-14a, which shows an arrangement similar to that covered by ASTM D4555. More recently, large-scale tests have been reported by Miyaike (1993) and Natau (1991) and are illustrated in Figure 2-14b.

The diamond belt rock sawing technology to produce the sample arrays has been demonstrated at Fran Ridge by cutting the 3 m × 3 m × 4.9 m (10 ft × 10 ft × 16 ft) block for the large block test. Figure 2-15 illustrates the test cutting at Fran Ridge using the SNL hydraulic saw system with a 2-m (6 ft) cutter bar. Figure 2-16 illustrates the production cutting of the large block using the 4.9-m (16 ft) cutter bar. This equipment allows maximum flexibility in freeing test samples within the rock. Techniques were also developed at Fran Ridge to stabilize cut surfaces by injecting expansive foam into 10-mil-thick plastic tubes. These injected foam packages prevented movement of rock on existing joints as the successive slots were cut.

The arrays of samples are shown cut into the floor of the tunnel in Figure 2-13; however, the saw system has the capability to produce similar geometries in the tunnel wall. he injected foam packages could be used to stabilize the lower face of prisms cut in the tunnel wall. In addition, the saw system can be used to cut damaged rock from the tunnel wall, and to produce a flat surface for application of the axial load. Jacking could be conducted between the tunnel walls, rather than roof-to-roof.

Other test geometries could be cut with the rock saw system. In addition, the saw system could be used to control blast damage at experiment locations by precutting slots along the blast round perimeter. These types of mining (topcutting, bottomcutting) have been traditionally practiced in softer rocks, however, the diamond belt cutting technology can allow the technique to be extended to excavation in hard rock.

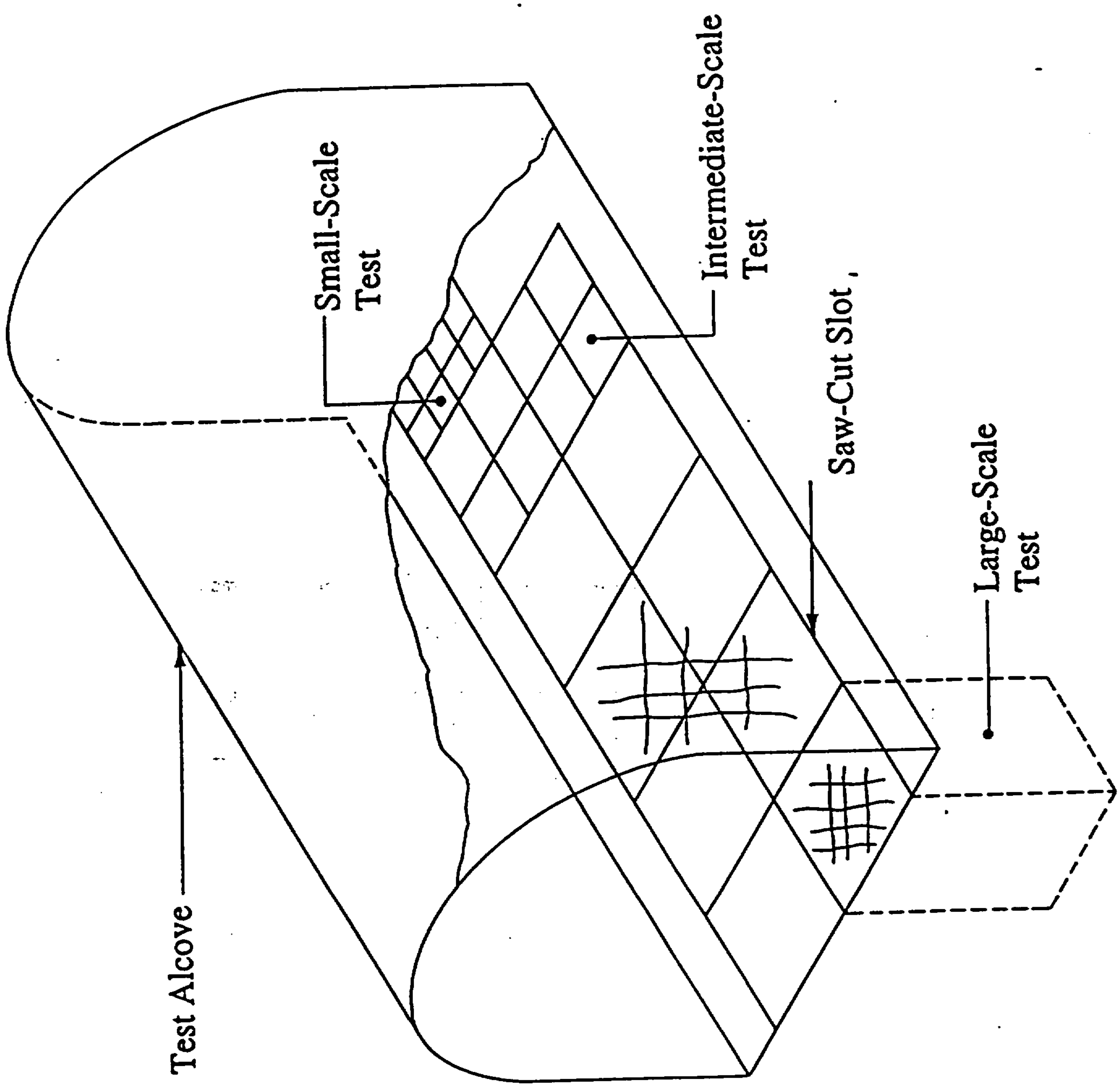



Figure 2-13. Layout of test alcove for vertical tests using diamond saw for specimen preparation.

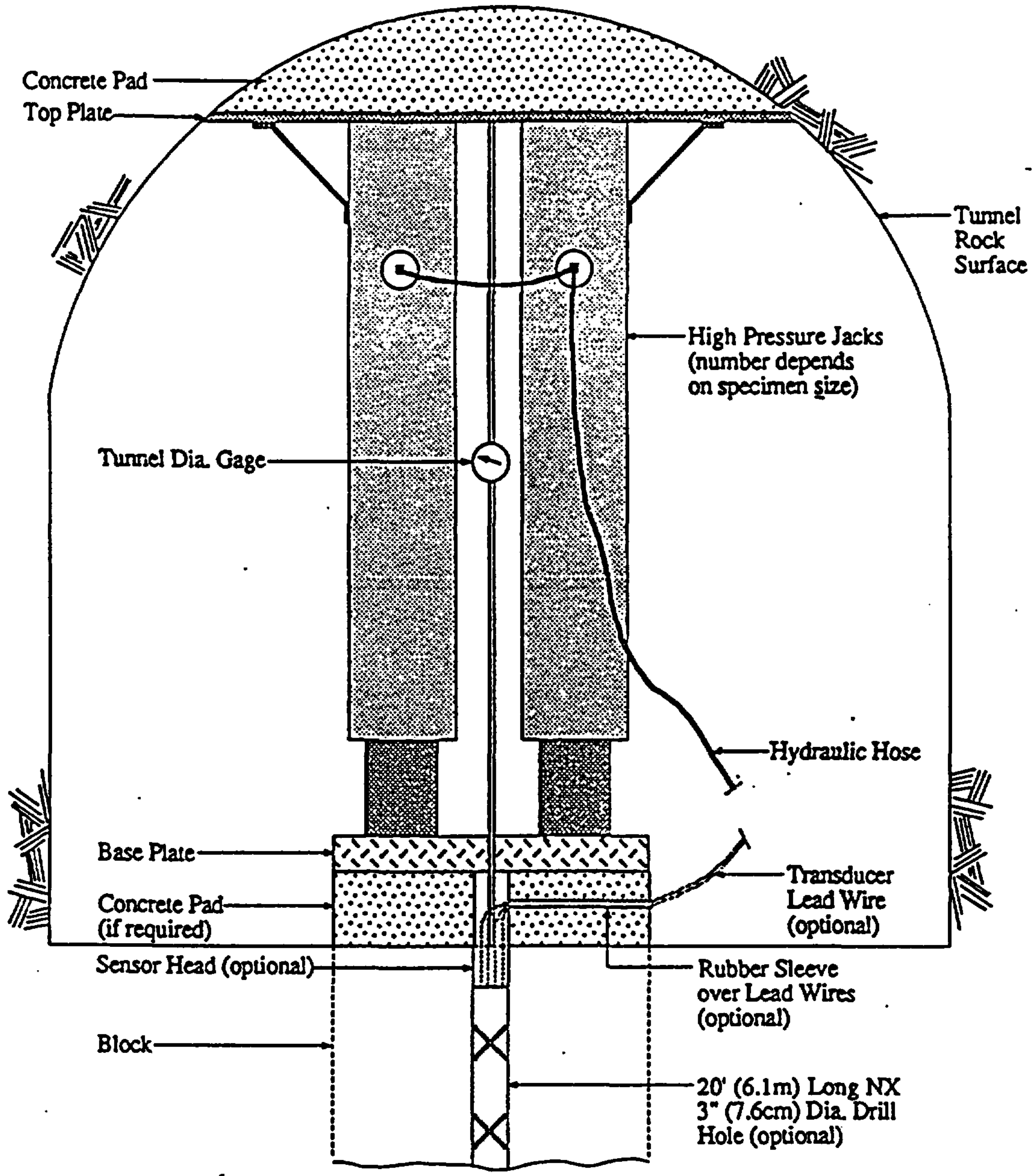
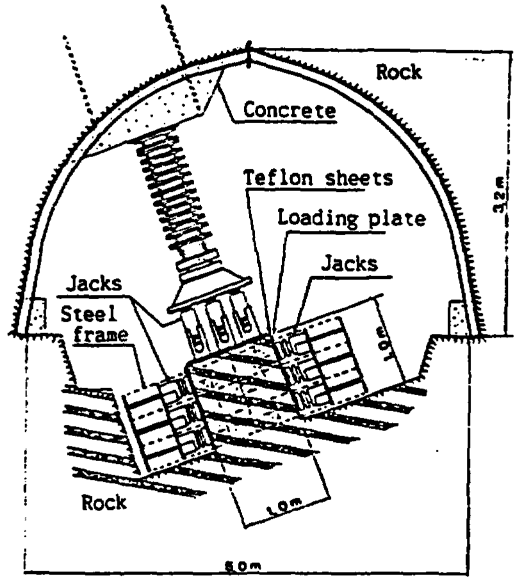
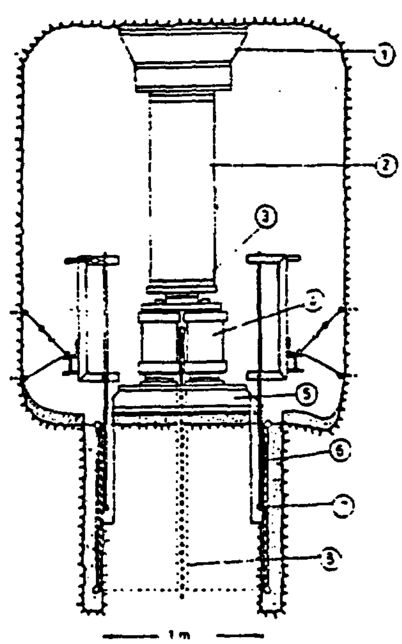



Figure 14a. Configurations of prism compressive strength tests.




Large-scale triaxial test equipment used to measure strength of Miocene sedimentary rocks (Miyaike, 1993)



Large-scale triaxial test equipment reported by Natau (1990)

Figure 14b. Schematics illustrating large-scale confined and compression tests.

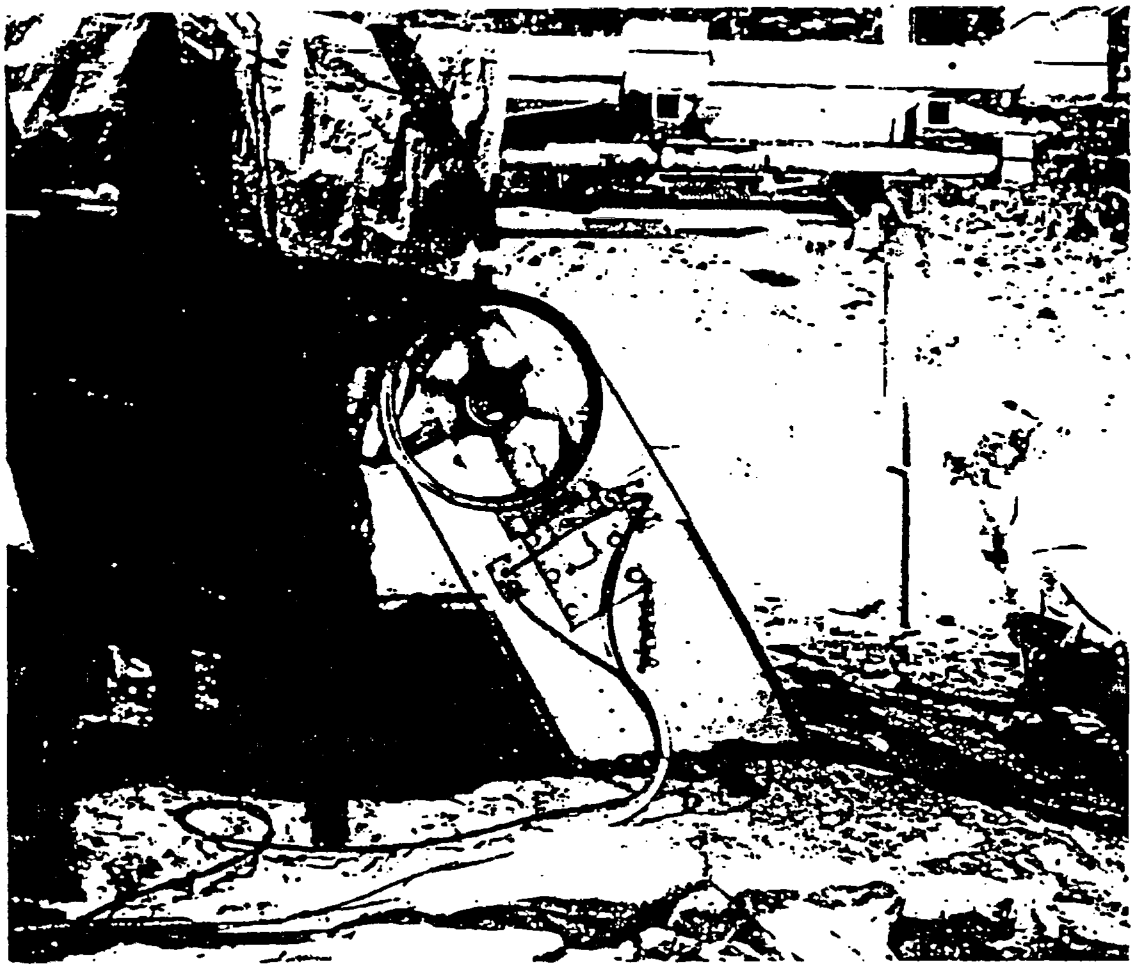


Figure 2-15. SNL hydraulic rock saw system with 2-m (6-ft) cutter bar cutting TSw2 welded tuff at Fran Ridge.

Figure 2-16. Rock saw system with 4.9-m (16-ft) cutter used to produce the large block test at Fran Ridge.

### *Pretest Characterization and Laboratory Testing*

The test alcove or drift wall will first be characterized to determine the optimum location and orientation of the prism array to achieve the test objectives. This will involve detailed mapping of the discontinuities and evaluation of the rock mass.

The selected test sites will be photographed and mapped in detail with the focus on persistence, spacing, condition of joint surfaces, and the presence and size of lithophysal cavities. Video imagery of the joints may be developed to allow characterization of surface roughness. Joint wall compressive strength will be determined using point load tests. Any holes drilled in the block for installing instrumentation as well as the core from these holes will be logged and the holes borescoped. Some intact rock strength and deformability tests will also be conducted on the core. Borehole jacking measurements will be made to increase the data base on the deformation modulus.

### *Instrumentation*

Pressure and displacement instrumentation will be installed to measure rock stress and displacement. Pressure transducers will be installed on each flat jack and the SNL servocontrolled hydraulic pressurization system. The servocontrol feedback loop will be based on displacement control to simulate "stiff" testing conditions, and thereby avoid or reduce violent or indeterminate failure. The pressurization system is based on off-the-shelf servocontrolled pressure regulators that have been bench tested by SNL for this application.

### *Test Procedure*

The basic test procedure will be to continuously monitor applied pressures and displacement while loading the prism, until the hydraulic pressure drops significantly below its peak value or when the specimen disintegrates completely.

### *Interpretation of Test Results*

The uniaxial strength is calculated by dividing the peak load by the original cross section of the specimen. The deformation modulus is calculated directly from the slope of the stress-strain curve. Results from previously reported tests on different-sized specimens are shown in Figure 2-17, and illustrate the scale effect on rock strength.

### *Location and Number of Tests*

Tests will be performed in alcoves and selected areas within the ESF to characterize the effects of rock mass variability on strength, as discussed in Section 2.2. Locations will be selected after geologic mapping to be representative of different rock qualities. Tests will be conducted in the floor and rib of the alcove. The results shown in Figure 2-17 suggest that sample strength reaches a plateau at sample sizes above 1 m, and this plateau would range from 15 to 30% of the intact rock strength based on laboratory-sized samples. Depending on the density of jointing, samples in the 1-2 m size range may not incorporate all the structure features that affect the rock strength at the rock mass scale. The prism tests may, therefore, be representative of the strength of the upper bound of the rock quality assessments where the rock is characterized as intact rock between joints with fairly large spacing. In other areas, the intensity of fracturing may be such

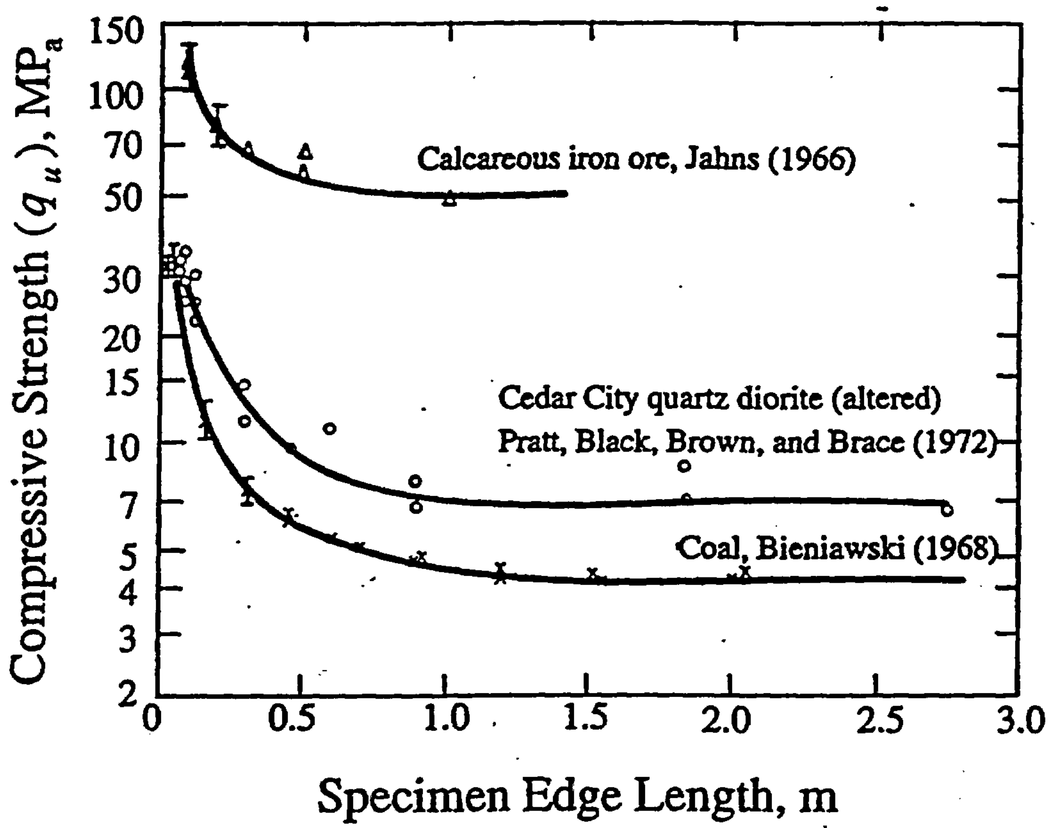



Figure 2-17. Effect of specimen size on unconfined compressive strength (after Bieniawski and Van Heerden, 1975).

that samples in the 1-2 m range contain highly fractured rock and the strength would be representative of the lower bound of rock quality assessments.

Selection of the number of locations and tests will be based upon examination of the structural variability. A strategy will be developed to try to bound the rock mass quality, and evaluate the confining pressure effects in the empirical relationship.

*Potential Impacts on Site*

Potential impacts on the site are expected to be minimal. Holes will be drilled and rock excavated using as little water as possible. No heating is planned in conjunction with these tests. Flatjacks will be filled with water.

*Simulation of Repository Conditions*

The prism tests are not designed to simulate repository conditions. However, they can be used to examine stress dependencies of the deformation modulus over the range of stresses expected in the repository, although this is not the primary objective. Thermal loading is the mechanism responsible for high stresses in some areas of the repository. At present, it is uncertain what the design gross thermal loading will be in the repository. However, it is likely that a wide range of stresses between zero and the rock strength will be encountered, at least locally.

*Relationship Between Laboratory and Field-Scale Phenomena*

It is anticipated that the field compressive strength will be significantly lower than the lab strength at Yucca Mountain because of the highly jointed nature of the rock.

*Test Interference*

Prism tests are not expected to interfere with any of the other experiments. The test should be 10 m (32 ft) from the nearest test that has altered the thermomechanical properties of the rock. Only a small region of rock (up to several cubic meters) will be directly loaded and the effects of the loading will likely extend a distance of only a few times the width of the area over which the load is applied. No permanent alteration to the local hydrological, chemical, or thermal conditions will result from this test. Testing may impede traffic: therefore, test alcoves should be provided.

*Impact on ESF Construction*

This test will have an impact on ESF construction because alcoves will be required; however, they will be small (approximately 3.7 m wide by 3.7 m long by 2.4 m high.). The actual cross section of the alcoves will most likely be the minimum that can be excavated. Five or more tests may be performed in each alcove. It is preferable that the excavations be mechanically mined.

Standard underground facilities for water, air, and electricity for drilling will be used for this test. An uninterrupted power supply and data acquisition system will be needed.

### 2.3.3.3 Slot Test

The slot test is an in situ method of testing a small to medium volume of rock. Historically the test has been used to determine the in situ modulus of deformation by internally jacking a saw-cut slot in the rock using flatjacks; however, its primary purpose here will be to determine large-scale joint strength and stiffness. Variations of the single slot test will be used to isolate joints and apply normal and shear loads to measure joint strength and stiffness.

The principal features of the slot as performed in G-tunnel by Zimmerman et al. (1992a) are shown in Figure 2-18. The test geometry was simple and required a single slot with associated displacement, stress, and pressure instrumentation.

Variations of the slot test will allow fairly simple experiment geometries for testing large joint surfaces in situ using the geometry reported by Swolfs et al. (1981) and illustrated in Figure 2-19. Figure 2-20 illustrates two test geometries that can easily be produced in the tunnel wall using the SNL rock sawing system. By cutting three slots using the geometry of Swolfs et al. (1981), several types of measurements are possible:

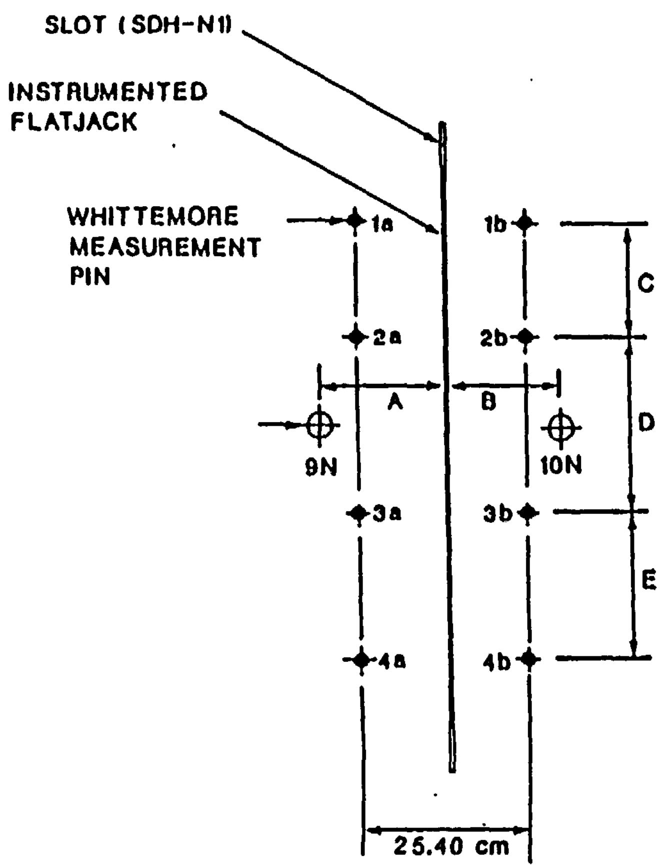
- shear strength at constant normal stress
- direct shear strength with control of the normal stress path
- normal closure versus normal stress at zero shear stress
- normal stiffness at constant shear stress
- shear stiffness at constant normal stress

These tests would provide the typical data derived for rock joints. Joint mismatch that occurs in laboratory testing as sample surfaces are fit back together would be eliminated.

The second slot test, illustrated in Figure 2-20, allows testing the joint strength under conditions more similar to the in situ conditions that exist in the rock mass. In the rock mass, the joint is constrained in the normal direction by the surrounding rock. Shear deformation must take place with zero (or very small) normal displacement (dilation) and the normal stress changes in reaction to the rock constraint in the normal direction. This test requires the monitoring of changes in stress and displacement normal to joint surface.

Displacement measurements before and after the slots are cut can be used to establish the existing stresses in the tunnel wall. The slot tests can therefore characterize the pretest average stress on joints in the tunnel wall, and provide test geometries to perform the large in situ tests.

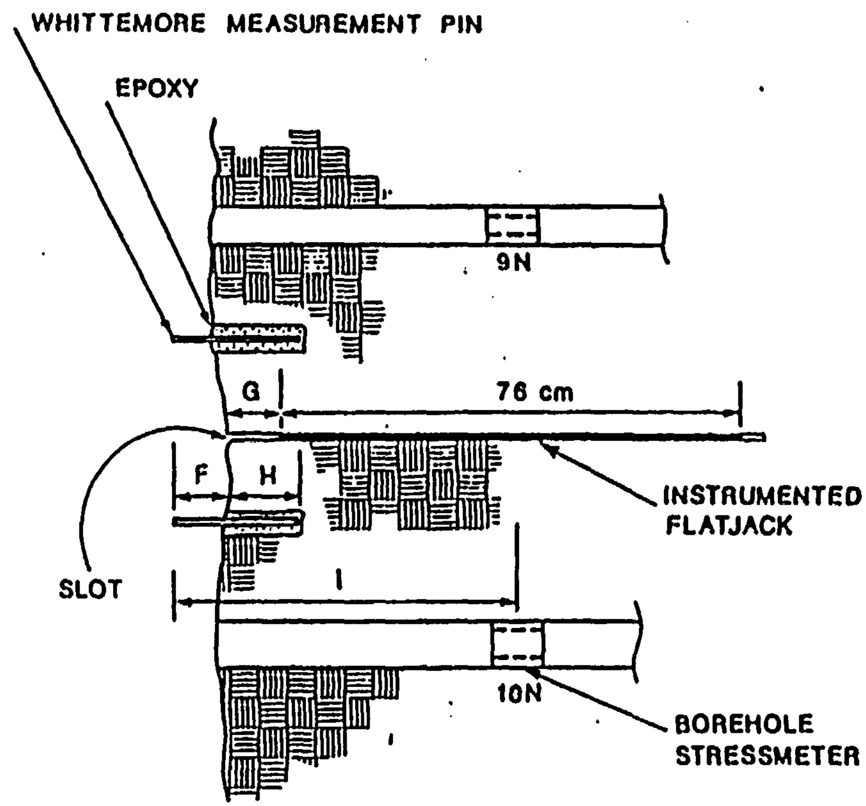
Flatjack pressure, slot dilation, and rock displacement can also be monitored and used to calculate deformation modulus. Because of the experiment geometry, the deformation modulus cannot be directly determined from the load-deformation results but must be calculated, generally, using simple elastic models. If displacement or strain instrumentation is installed prior to cutting the slot, the jacking pressure required to restore the initial conditions provides an indication of in situ stress.




NOTE: DIMENSIONS

| | |
|---|---|
| A-25.40 cm | F-1.27-10.16 cm (Range) |
| B-25.40 cm | G-6.98-12.70 cm (Range) |
| C-16.51 cm | H-12.70-22.86 cm (Range) |
| D- 24.31 cm | I-53.24 cm for 9N |
| E-19.68 cm | 53.08 cm for 10N |

a) Front View




b) Top View

Figure 2-18. Layout showing principal features (from Zimmerman et al, 1992a, p. 5-3) of the slot test.

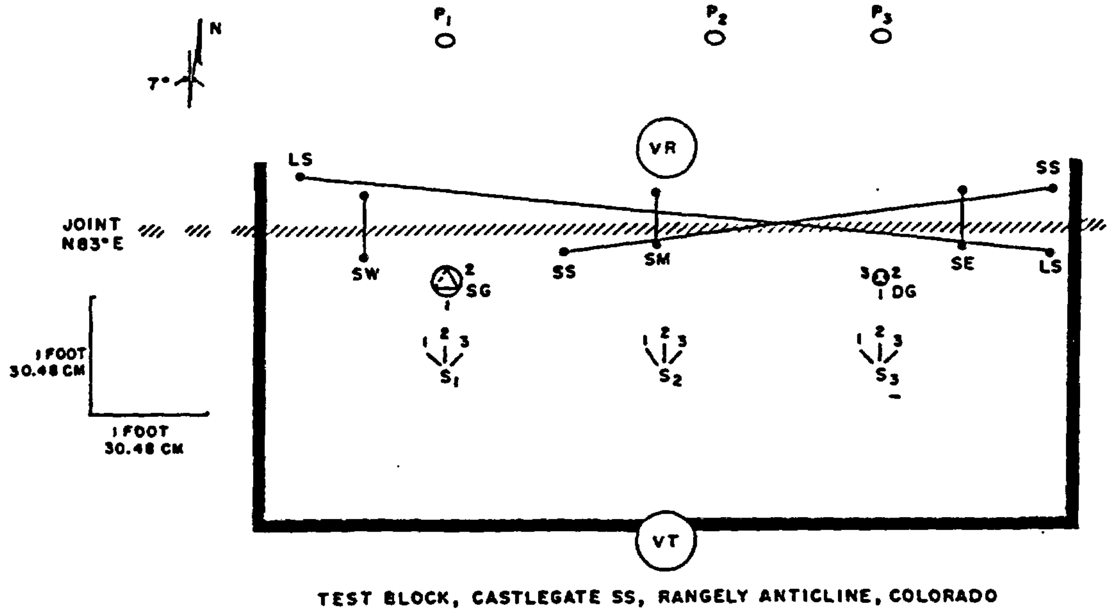



Figure 2-19. Schematic of the top surface of the test block excavated to include portion of the east-west trending joint. Heavy black lines indicate the position of the vertical slots of loading flatjacks. Letters refer to surface and borehole instrumentation identified in the text. (Swolfs et al, 1981).

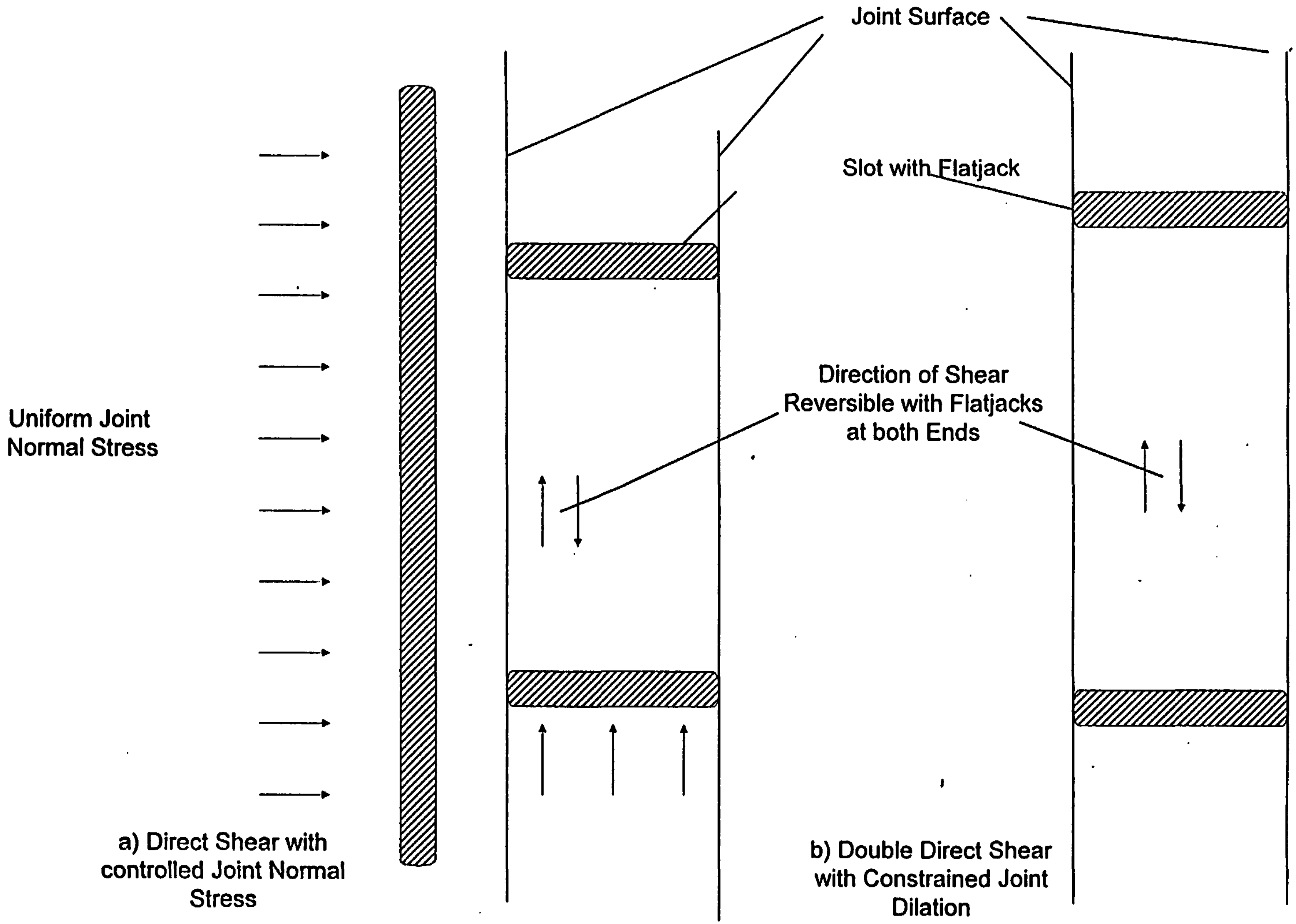



Figure 2-20. Potential slot test configurations for in situ measurements of joint properties.

### *Test Construction*

The slot tests can be conducted in drifts and alcoves located in the ESF. Before excavation of the slot, the rock will be instrumented, to measure strain relief. Flatjacks will be installed in the slots, and testing conducted.

### *Pretest Characterization and Laboratory Testing*

The test alcove or drift wall will first be characterized to determine the optimum location and orientation of slot experiments to achieve test objectives. This will involve detailed mapping of all discontinuities. The selected site will be photographed and mapped in detail, with the focus on persistence, spacing, condition of joint surfaces, and the presence and size of lithophysal cavities. Video images can be made of joint traces to allow determination of joint roughness by pixel mapping. Other joint surfaces will be excavated to allow surface topographic measurements in different directions using surface profilometers. Samples of nearby joints may be taken for laboratory testing as part of this or other investigations. Joint wall compressive strength will be determined using point load testing. Any holes drilled for installing instrumentation, and the core from these holes, will be logged and photographed. Intact rock strength and deformability tests will be conducted on cores, and holes will be tested using a borehole jack to measure deformation modulus. Flatjack slots will receive special attention, both for characterization and to ensure any voids are filled prior to flatjack installation. The slots will be examined using thin copper-impression jacks and shims may be inserted to prevent the flatjacks from deforming into existing voids.

### *Instrumentation*

Instrumentation consists of standard pressure and displacement transducers from which joint and rock mass parameters will be determined. Borehole stress meters will also be used. An important feature of the system will be the flatjacks. A flatjack capacity of up to 69 MPa (10,000 psi) may be required. Zimmerman et al. (1992b) were able to perform tests up to 28 MPa (4100 psi) in welded tuff, although they encountered numerous failures. Flatjack designs were subsequently modified and tested in G-Tunnel to 30 MPa (4350 psi) without flatjack failure (Hansen, 1990). Additional tests using the modified flatjack design were conducted as lower pressures (Finley, 1994). These tests showed the flatjack design capable of displacements exceeding the flatjack thickness.

The jacks may be internally instrumented with deformation gages, which will be used to determine modulus because of their internal location. Pins will be installed for Whittemore gage measurements near the rock surface prior to slot excavation for use in measuring in situ stress. Initial measurements will be required prior to flatjack excavation.

### *Interpretation of Test Results*

Interpretation of the test results will be based upon back-analysis of the scaling criteria described in Section 2.2. In these criteria, the parameters JRC, $\Phi r$, JCS, $a_j$ are developed from evaluation of the data variability by many simple measurements, then the experimental data from the few in situ experiments are used to confirm the criteria. Application of the JRC parameter may be modified by use of surface roughness measures, particularly the fractal dimension and its intercept.

*Location and Number of Tests*

Tests will be performed in alcoves at selected areas within the ESF where favorable joint geometries and other conditions that allow measurement of the samples occur. These favorable locations will also be evaluated with regard to the scaling parameters and their variability. Simple slot tests will be conducted in the rib to measure the induced stresses normal to the joints to be tested.

*Potential Impacts on Site*

Potential impacts on the site are expected to be minimal. Holes will be drilled and slots excavated using as little water as possible by recirculating the cutting water. Flatjack rupture may result in limited amounts of flatjack fluid (distilled water) being released. It is not anticipated that fluids other than water will be required. No heating is planned in conjunction with these tests.

*Simulation of Repository Conditions*

The slot tests are not designed to simulate repository conditions. However, they can be used to examine stress dependencies of the joint properties over the range of stresses expected in the repository. Thermal loading is the mechanism responsible for high stresses in some areas of the repository. At present, the design gross thermal loading in the repository is uncertain. To be appropriate for design and performance calculations over the range of expected conditions, the peak joint normal stress will be based on modeling studies for repository openings.

Peak flatjack pressures will be limited by available flatjack technology, currently projected to be 55 MPa (8000 psi) or less in perfectly constrained load situations.This will be the maximum pressure used as a basis for test design.

*Relationship Between Laboratory and Field-Scale Phenomena*

It is anticipated that the field shear strength and joint stiffness will be significantly lower than the laboratory data because of the scale effects.

*Test Interference*

Slot tests are not expected to directly interfere with any of the other experiments. The test should be 10 m (32 ft) from the nearest test that altered the thermomechanical properties of the rock. Only a small region of rock (several cubic meters) will be directly loaded and the effects of the loading will likely extend a distance of only a few times the width of the area over which the load is applied. No permanent alteration to the local hydrological, chemical, or thermal conditions will result from this test. Testing impedes traffic; therefore test alcoves should be provided.

*Impact on ESF Construction*

This test will have a minor impact on ESF construction because alcoves will be required; however, they will be small (approximately 3.7 m wide by 3.7 m long by 2.4 m high). The actual cross section of the alcoves will most likely be the minimum that can be machined. Five or more tests may be performed in each alcove. It is preferable that the excavations be mechanically mined. Standard underground facilities for water, air, and electricity for drilling will be used for this test. An uninterrupted power supply and data acquisition system will be needed.

### 2.3.3.4 Block Test

The block test, illustrated in Figure 2-21, is an in situ method of testing a relatively large volume of rock. The test evolved out of the simple slot test (Rocha, 1970). The purpose of the test is to study the mechanical response of a volume of jointed rock under controlled stress boundary conditions. As opposed to the plate loading tests, which are one dimensional, the block test provides two-dimensional loading uniformly through the sample. Interpretation of the test results is therefore straightforward. With independent control of the flatjack pressure in each direction, the load path on joint surfaces can be designed for several test types:

- changing shear and normal stress;
- changing shear stress at constant normal stress, and;
- changing shear stress at constant normal closure.

*Test Construction*

The block tests will be constructed in alcoves located in the ESF. After selection of the location and excavation of the test alcove, the block location will be characterized and instrumented. Slots will be excavated on four sides of the block using the rock saw. Flatjacks will be installed in the slots, and the remaining displacement and stress instrumentation installed.

Of these items, the slot excavation merits further discussion. Early block tests utilized difficult and costly techniques for slot cutting that involved drilling. SNL recognized the need for a more efficient cutting method and has utilized the diamond rock saw, shown in Figure 2-15, and illustrated in Figure 2-22. Excavating flatjack slots for a block test requires the ability to cut very planar, precisely oriented orthogonal slots approximately 1.5 to 2 times the block depth. These techniques were demonstrated at Fran Ridge, where slots were excavated to 4.9 m (16 ft) deep on four sides of the large block.

*Pretest Characterization and Laboratory Testing*

The test alcove will first be characterized to determine the optimum location and orientation of the block to achieve test objectives. This work will primarily involve detailed mapping of all discontinuities. The block will be situated to contain one or more vertical joints and will be oriented at 45° to the principal joint direction.

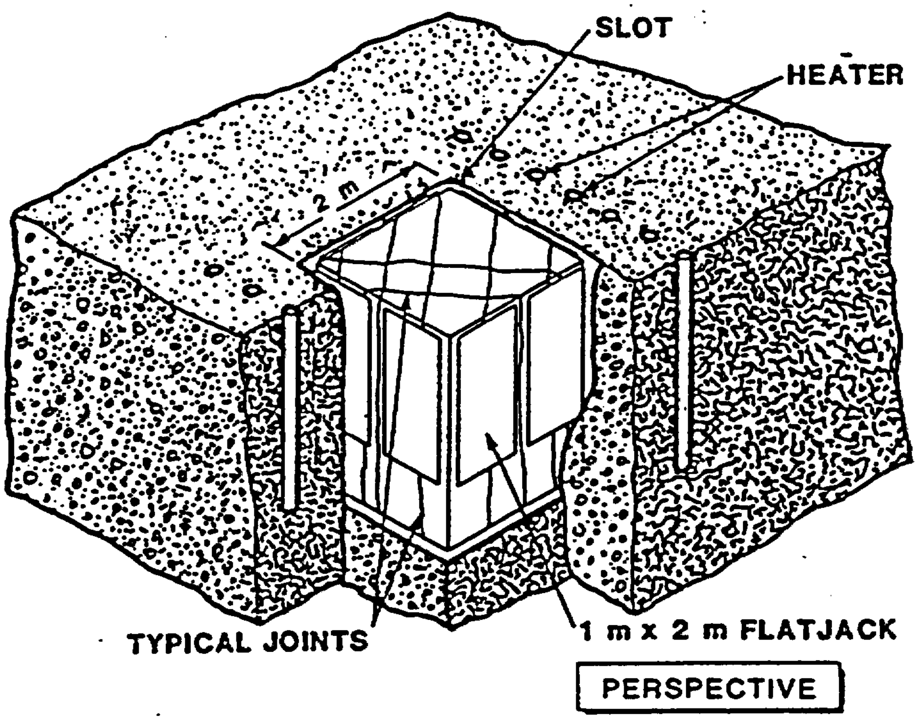



Figure 2-21. Typical block test (note: heaters are not used for ambient testing) (Zimmerman et al., 1984: 283).

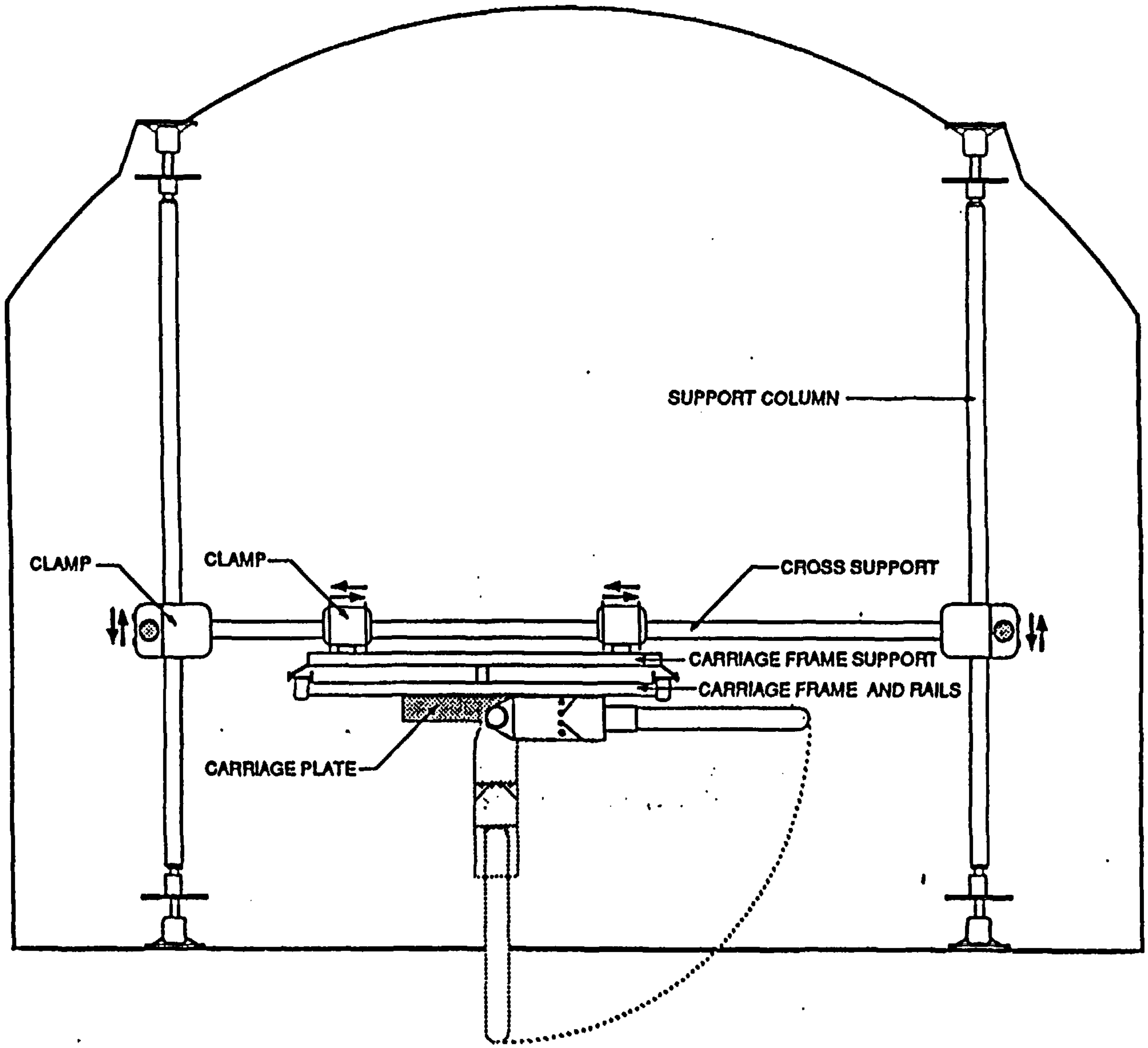



Figure 2-22. Schematic illustrating horizontal configuration of saw for cutting slots in floor (Note: Shape of alcove is conceptual.)

The selected site will be photographed and mapped in detail with the focus on persistence, spacing, the condition of joint surfaces, and the presence and size of lithophysal cavities. Video imagery can be used to characterize joint roughness by pixel analysis. Samples of nearby joints may be taken for laboratory testing as part of this or other investigations. Joint normal and shear response will be determined from these tests. The JCS and JRC will be determined, as well as joint topography by surface profilometer and video imagery. Any holes drilled in the block for installing instrumentation, and the core from these holes, will be logged and photographed. Intact rock strength and deformability tests will be conducted on cores, while holes will be tested using a borehole dilatometer. Flatjack slots will receive special attention, both for characterization of the block and to ensure that voids are filled prior to installation of flatjacks. The slots will be examined using a foil impression, and they will then be filled with a paste grout and cut again.

*Instrumentation*

The primary emphasis will be on pressure and displacement instrumentation from which joint and rock mass parameters will be determined. A critical feature of the system will be the flatjacks. A capacity of up to 69 MPa (10,000 psi) may be required, although this is marginal technology in large (2 m) jacks. Currently available technology can achieve up to 55 MPa (8,000 psi). Some additional testing and development of flatjacks may be required. Details of the SNL servocontrolled flatjack pressurization system are shown in Figure 2-23. Pressure transducers will be installed on each separate flatjack line. A closed-loop hydraulic system with a servocontrolled feedback loop on pressure or displacement will be used to control the test.

Absolute displacement measurements offer numerous advantages but are more difficult and costly. They are justified for validation of discrete block models because they allow separation of displacement, strains, and rotations. However, for determining in situ deformation modulus, joint behavior, and validation of equivalent continuum models, properly deployed relative displacement measurements will be adequate and more easily accomplished. This simplicity may actually improve site characterization if it enables more testing and broadens the statistical data base.

Figure 2-24 shows details of a system for relative displacements. Rigid pins approximately 0.6 m (2 ft) long will be cemented into boreholes. Only the bottom 15 cm (6 in) will be cemented into the hole, and care will be taken to center the pins so they do not touch the borehole wall. These measures, along with careful excavation of the block surface, will ensure that displacement anchors will be located in the stress field produced by the flatjacks. Richardson (1985) and others have shown that these types of displacement measurements must be tilt corrected. Biaxial tilt meters having a high resolution must be placed on each displacement measuring pin. Wire rod extensometers will be installed between pins.

The deployment of displacement instruments will be site-specific and will be determined after each block has been sited and characterized. A typical deployment is shown in Figure 2-24. In addition to lateral displacement measurements, vertical displacements will be measured using an MPBX installed in the center of the block.

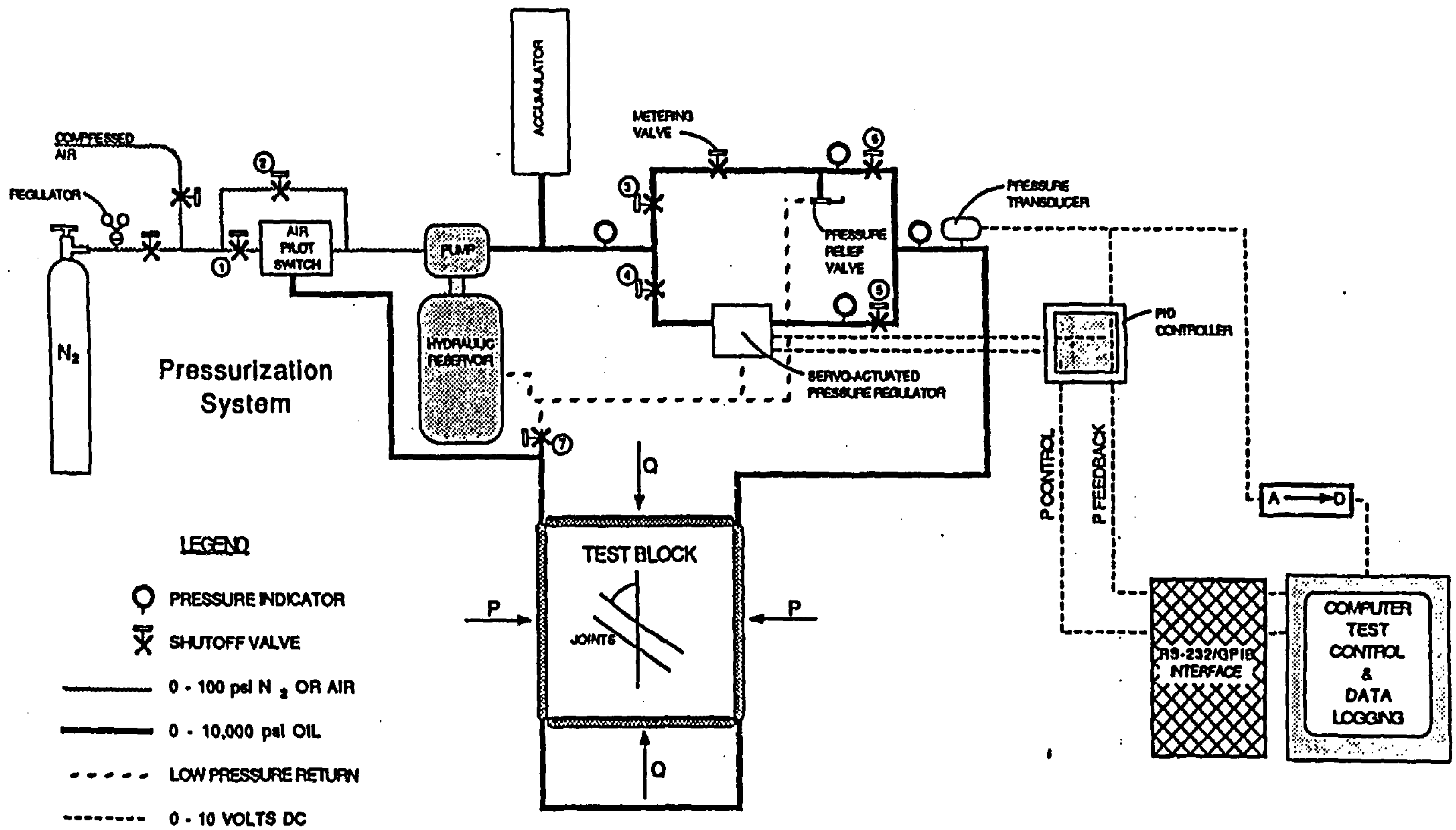
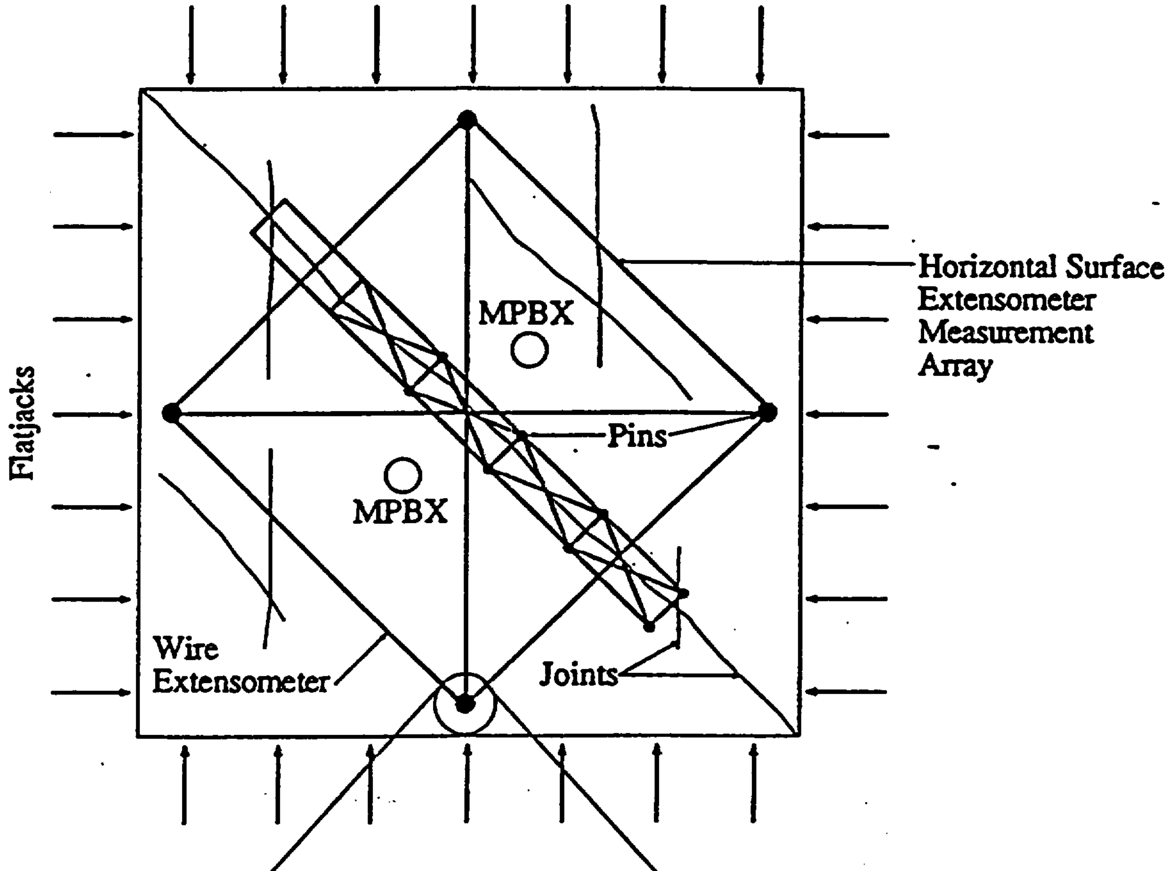
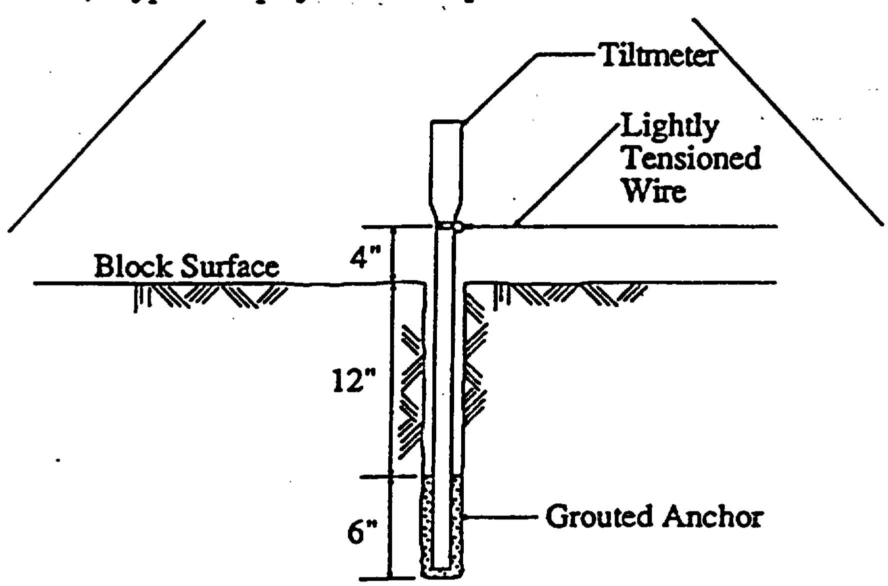



Figure 2-23. Details of flatjack control systems.




a) Typical Deployment of Displacement Instrumentation




b) Section of Displacement Pin

Figure 2-24. Details of system for relative displacements.

*Test Procedure*

The test procedure will be sequenced so that early load cycles have a minimum effect on results from later load cycles. To accomplish operation, equal biaxial loading will be applied first, followed by distortional loading. Load paths should be designed to limit distortion in early tests to the minimum required. A representative loading sequence would be:

1. Apply an equal biaxial preload to the in situ stress level (previously determined by stress relief during slot cutting). Limit pressure differences between jacks during loading.

2. Increase the equal biaxial load in increments up to a predetermined peak stress level, with two or three unload-reload cycles to the preload level. Unload to the preload level.

3. Keeping one pair of jacks at the preload level, increase pressure in the other pair to the predetermined peak stress level. Unload to the preload level.

4. Repeat step 3 in the other direction.

*Interpretation of Test Results*

Displacements will be converted into strains, and flatjack pressures will be converted into stress. Deformation modulus will be calculated from the slope of the stress-strain curve using the broadly spaced corner array. Lateral strain ratio will be calculated from uniaxial tests from the direct and lateral strains using the corner array and the vertical extensometer. Joint properties will be calculated by resolving the applied stresses and joint displacement into normal and shear components along the joint using the closely spaced joint displacement array.

*Location and Number of Tests*

Tests will be performed in alcoves at two or three selected areas within the ESF. Alcove locations will be selected after geologic mapping to be representative of different rock qualities. Block tests will be conducted in the floor or wall, depending on construction techniques.

*Potential Impacts on Site*

Potential impacts on the site are expected to be minimal. Holes will be drilled and slots excavated using the minimum possible water by recirculation of drilling and cutting fluids. Flatjack rupture may result in limited amounts of flatjack fluid (distilled water) being released. No other fluids are anticipated. No heating is planned in conjunction with these tests.

*Simulation of Repository Conditions*

The block test is not designed to simulate repository conditions. However, it can be used to examine stress dependencies on the deformation modulus over the range of stresses expected in the repository. Thermal loading is the mechanism responsible for high stresses in some areas of the repository. At present, it is uncertain what the design gross thermal loading will be in the repository. However, it is likely that a wide range of stresses between zero and the rock strength will be encountered, at least locally. Accordingly, to be appropriate for design and performance calculations over this range, the peak load should be as near the rock strength as feasible. Peak pressures are limited by available flatjack technology, which is currently limited to 55 MPa (8000 psi) or less in perfectly constrained load situations.

*Relationship Between Laboratory and Field-Scale Phenomena*

It is anticipated that the field properties will be significantly lower than the lab properties at Yucca Mountain because of the highly jointed nature of the rock. Zimmerman et al. (1984) measured a lab-to-field modulus ratio of approximately 2. Joint shear strength and stiffness are anticipated to be lower than lab data due to the larger scale.

*Test Interference*

Block tests are not expected to directly interfere with any of the other experiments. The test should be 10 m (32 ft) from the nearest test that altered the thermomechanical properties of the rock. Only a small region of rock (approximately 8 $m^3$) will be directly loaded, and the effects of the loading will likely extend only a few times the width of the area over which the load is applied. No permanent alteration to the local hydrological, chemical, or thermal conditions will result from this test. Testing impedes traffic; therefore, test alcoves should be provided.

*Impact on ESF Construction*

This test will have an impact on ESF construction because alcoves will be required; however, they will be small (approximately 3.7 m wide by 3.7 m long by 2.4 m high.). The actual cross section of the alcoves will most likely be the minimum that can be machined. Several tests may be performed in each alcove. It is preferable that the excavations be mechanically mined.

Standard underground facilities for water, air, and electricity for drilling will be used for this test. An uninterrupted power supply and data acquisition system will be needed.

## 2.4 Sequence of Testing

Initial testing described in this study plan will concentrate on evaluation of the deformation modulus/rock mass stiffness in support of the In Situ Thermal Testing Program. These initial tests will also provide preliminary rock mass data for ESF and repository design evluations. The initial tests include plate loading and borehole jacking. Evaluation of these preliminary tests will guide follow-on testing including block, slot, and prism tests discussed in this study plan. Also, the initial testing will guide the need for and number of in situ tests required to evaluate the spatial variability of measured parameters.

# 3.0 APPLICATION OF RESULTS

## 3.1 Introduction

This section summarizes the application of results in terms of end users or "customers" of the data. Major end uses of data on rock mass properties include waste package and repository design, preclosure and postclosure performance. The results of the in situ tests of mechanical properties requirements must satisfy the basic licensing framework outlined in the SCP (i.e., the issue resolution strategy) and also meet the current needs of the project.

## 3.2 Rock Mass and Joint Mechanical Properties

The rock mass properties of deformation modulus and Poisson's ratio are essential for calculating the stresses induced by excavation and later by thermal expansion. Values for these stresses are required for the viability assessment and LAD. Initially, these properties will be estimated from empirical relationships between intact rock properties and rock mass quality. These empirical models must be validated by some field measurements of rock mass properties at ambient temperature prior to elevated thermal testing. The principal customers for this information are repository design and pre- and postclosure performance assessment.

*Rock Mass Strength*

As with other rock mass properties, rock mass strength is usually estimated by empirical relationships using laboratory measurements and rock mass quality. Very little data exist to support the empirical relationships; therefore, some calibration on a site-specific basis, of the empirical relationship is needed. Principal customers for this information are repository design, and pre- and postclosure performance assessment.

*Fracture Properties*

The rock mass in the repository horizon is highly jointed, with the intact rock blocks having relatively high strength. The fractures or joints will therefore control the bulk of the deformation and structural weaknesses. Fracture properties (normal and shear compliance, shear strength, and cohesion) are essential for estimates of rock mass deformation and strength. Early information from laboratory testing is useful; however, scaling to rock mass is known to result in substantial reductions in strength, especially in highly jointed, strong rock. Some in situ data are required at ambient temperature and prior to testing at elevated temperatures representative of repository conditions. The customers for this information are repository design and preclosure performance assessment.

*Temperature Effects of Rock Mechanical Parameters*

Temperature effects are being investigated in detail in laboratory determinations of the mechanical and fracture properties of intact rock and by in situ thermal tests. The thermal testing at rock mass scale involves measurements of temperature effects to determine if the effects are significant and if the models used incorporate the effects in a reasonable way. However, ambient temperature field measurements are much less complicated and less expensive, and will provide a way to investigate spatial variability throughout the repository area. In this way, they will help extend the results of the few thermal tests planned and increase their credibility. The principal customer for this information is in postclosure performance assessment.

### 3.3 Validation of Rock Mass Models

Thermomechanical and mechanical models are being used to support the design of an ESF and a potential high-level nuclear waste repository at Yucca Mountain, Nevada. These models are used for preclosure design of underground openings such as access drifts and emplacement drifts. The models are also used to resolve postclosure issues relating to performance of the waste canister, disturbance of the hydrogeological properties of the host rock, and assessment of overall system performance. For the ESF and repository design, and for performance assessment, models are used to better understand and quantify phenomena or processes resulting from construction or waste emplacement. If modeling results are used to resolve issues or to support a license application, then the models must be validated.

Validation must demonstrate that the key phenomena, processes, and properties are adequately incorporated or reflected in a model (numerical, empirical). The validation process itself is viewed as having three main components:

- evaluation relative to experimental data obtained from in situ and laboratory tests;
- evaluation relative to empirical evidence and case histories (including natural analogs);
- and evaluation by peer review.

Depending on the particular model, one or more of these components may be applied. For most thermomechanical models, the focus will be on comparisons with the results of specific laboratory and in situ experiments.

In the SCP, performance goals are established as part of the performance allocation process. Thermomechanical models will be used in many instances to assure the licensing agencies and the general public that the proposed repository system can meet those performance goals; therefore model validation is an integral part of the ESF testing program.

Theoretical models are calculation schemes based upon physical laws that predict the coupled or uncoupled thermal-thermomechanical-mechanical response (temperature, displacement, and stress changes) in the rock mass while incorporating the geometric aspects of the underground openings. Empirical models are experience or observation-based correlations between behavior of the excavation and properties of the rock mass. In the planned applications, empirical models described in this study are being utilized in two ways. First, rock mechanical and joint properties, being used as input for the numerical models, are derived from empirical correlations (models) between rock mass characteristics (core strength, fractures, fracture geometry, etc.) and rock mass scale properties (elastic modulus, strength). Second, construction practice (tunnel dimension, rock support, etc.) is derived based on correlations between rock mass characteristics and case histories.

Thermomechanical models will be used in evaluating long-term repository performance to assess the deformation response of the rock mass to changes in temperature. These models are based on coupling of both the thermal and mechanical response of the rock mass. The coupling is necessary because of the expansion of the intact rock as it is heated. Thermal models provide input to the thermomechanical models by estimating the temperature history of the rock mass after the emplacement of waste. Thermal models using simple heat conduction processes have been used extensively and are considered by many to be very reliable predictors of near-field thermal response [see, for example, the discussion in Ubbes et al. (1985), results from the BWIP in situ test program at the Near Surface Test Facility (Hocking et al., 1990), and the results of the spent fuel test at Climax (Montan et al, 1986]. Thus some evidence is already available to support validation of thermal models. Mechanical models are used to predict deformation of the rock resulting from excavation or other changes to environmental conditions. Mechanical models have been developed or adapted to represent most of the different types of rock mass that may be encountered. These rocks range from relatively unfractured nonwelded tuffs to the highly jointed welded tuff at the potential repository horizon. However, the mechanical models have, as yet, been relatively untested at Yucca Mountain because underground access to the various rock units has not been available.

### 3.3.1 Laboratory and Field Experiments to be Used for Thermomechanical Model Validation

The general strategy for developing data to provide input to the models and simulations of repository heat loading for validation of the models is described in the SCP (DOE 1988) through implementation of various study plan activities.

The variability of laboratory data on rock properties and rock mass quality estimates is addressed by the Soil and Rock Properties Study Plan, the Systematic Drilling Program, and the laboratory testing programs. These studies will provide initial estimates of the variation of rock mass quality from core logging and produce laboratory test data using core samples derived from the drilling. The laboratory tests measure variability of the intact material properties (thermal, thermomechanical, mechanical, and joint mechanical) throughout the site. Prototype thermomechanical field tests on a variety of rock types indicate that heat conduction will govern the heat transfer process and that the thermal rock properties (thermal conductivity, heat capacity) are independent of scale (Hocking et al., 1990; Montan et al., 1986; Zimmerman et al., 19??, G-Tunnel Small-Scale Heater Test).

### 3.3.2 Model Validation

The mechanical properties required for theoretical mechanical and thermomechanical models are known to be dependent on local rock structure, rock homogeneity, and the scale of excavation. The scale dependence requires a complex procedure to establish the rock mass scaling parameters and account for rock mass variability. This procedure will be performed by using empirical correlations between rock mass quality and the rock mass mechanical properties as described by Lin et al. (1993) and discussed in Section 2.0 for joint properties. In this strategy, rock mass mechanical properties will be measured by groups of large-scale, ambient-temperature field tests that include plate loading tests, prism tests (rock mass strength tests), slot tests, and block tests. These tests will be sited in areas that cover the range of rock mass qualities and joints to be encountered in the ESF. Because the tests will be based on simple geometries with controlled boundary conditions, they will provide direct data for input to the theoretical models and validation of the empirical correlations with scale.

Validation of numerical models will then be based on a group of ambient-temperature experiments conducted as part of the study plans for in situ mechanical properties and excavation effects. Rock response to perturbations in the stress field will be measured in the ESF using both controlled tests and repository-scale excavations. These tests will provide the basis for more complex constitutive model development.

In situ design verification studies are planned to collect data on rock mass quality and construction methods/ground support performance. These data will be used to evaluate ambient temperature applications of the empirical drift design models based upon the Q and RMR systems. Extrapolation of the empirical design models for thermomechanical applications will be based on the projected changes in stress as a result of thermal loading. These projections of stress change will be developed using the theoretical thermomechanical models. Data for the empirical thermomechanical models will be based on observations of rock failure and ground support performance in the thermomechanical tests.

Projections of the coupled response of the rock using the thermomechanical models will be based on input data for thermal and mechanical properties developed in the laboratory and in situ mechanical properties studies. Coupling of thermal and mechanical response is based on the assumption of laboratory measured thermal expansion for the tuffs. Validation of this model will be based on the results of experiments described in the In Situ Thermomechanical Properties Study Plan. The experiments include a series of tests with increasingly complex boundary conditions. The scale of tests will range from relatively small simulations at the single heater and block scale to a full-scale heated drift test. The heated drift test will include evaluation of ground support performance at high temperatures and may measure perturbation of the stress field with resulting rock failure.

### 3.3.3 Comparison Between Data and Model Predictions

Comparisons between data and model predictions are a key element of the validation exercise. The output from model predicitons used in the comparison can be point comparison, system comparison, or a combination of both. Examples of point data used for comparisons include temperature, displacement, stress, or convergence. Examples of system-based comparisons would be correlation of modeled or predicted stability with field performance using parameters such as frequency and magnitude of rock fallouts, frequency of drift ground support maintenance/rehabilitation, or damage to a container resulting from borehole-rock movement. Traditionally, point comparisons have been used in validation exercises even though it has long been recognized that large scatter in single-point comparison results from measurement errors and large local variability in rock structure and rock properties. To validate their thermal model, Hocking et al. (1990) successfully used plots of measured versus predicted data to determine the correlation coefficient between these two data sets. In this approach, the impact of local divergence of modeled and measured results was weighted into the total goodness of fit.

Empirical models for estimation of rock mass properties can be evaluated by controlled tests such as the block test or by the single heater experiment. The plate loading test will provide data for validation of geomechanical models via comparison of field data with prediction of deformability from laboratory values and characterization data. Empirical models for estimating rock mass or scale properties are based on the fitting to a sequence of data from a point or multiple points.

Because of the history of using numerical modeling for structural design, it is often assumed that point-by-point comparisons of field and modeled data will be an adequate test of the model. This assumption is often valid when dealing with models of structural steel or other engineered materials. However, in rock structures, the variability of rock properties and rock mass conditions is often so great that it makes point-by-point comparisons difficult. An alternative technique is to extrapolate measured data at several points to create a full-field data set. For example, displacement measurements at several points along a drift might be extrapolated to predict the expected displacement everywhere around the drift. This extrapolated displacement field would then be compared with the calculated displacement field. Thus engineering judgments regarding validation can be based on comparisons of the expected range or measured displacement versus predicted range (calculated) displacement over a large volume of rock, not just a few points.

Examples of post-test model-data comparisons are provided by Hocking et al. (1990), Heuze et al. (1992), and Costin (1988, 1990). There are no examples of pretest model analysis results being used exclusively for a validation exercise with good results. Examples of failures of pretest analyses are available, but in most cases, the analysis was completed before adequate site characterization had been completed.

Pretest analyses will normally be conducted as part of the test design process. These analyses are essential to establishing predicted behavior and are used as a basis for setting measuring ranges on instrumentation. After the test is installed and ready to conduct, a final set of pre-test analyses are usually conducted. These analyses can incorporate the as-built geometry and material properties measured at the test location. Prediction of rock-mass behavior are made for expected ranges of material properties estimated from local spatial variability. After the test is conducted, pre-test predicitons will be compared with test results and additional post-test analyses will be conducted to refine the results and provide a more complete understanding of the test.

Two general criteria should be considered when developing conclusions regarding the validity, limitations, and uncertainties of a model:

*Adequacy of model physics*: This criterion generally applies only to theoretical models and addresses this question: Is the physics good enough to predict essential behavior and can important phenomena that may not be incorporated directly in the model, such as scale effects, be accounted for by back-analysis or other calibration so that the model can be used to extrapolate to other cases? The testing strategy for validation of thermomechanical models was designed to address this question by requiring that tests be performed at several scales. The results from tests at different scales can be used along with some additional analysis to determine (1) whether the model is capable of appropriate scaling and (2) the limitations of the scaling capability.

*Prediction of behavior*: This criterion applies to both empirical and theoretical models used for design or performance assessment and addresses the question: Does the model represent the physical world well enough to develop performance goals and to provide means for verifying that a design has met those goals? This criterion indirectly addresses the larger question of what is good enough. For thermomechanical models, the end application will be in the area of design. The strategy used in the SCP (DOE 1988) is to establish performance goals for systems of components to ensure that the potential repository will meet all regulatory requirements. This performance allocation process requires the use of models to assist in establishing goals, for example, for near-field thermal conditions that ensure the design will meet requirements for retrievability and long-term stability of the rock mass.

### 3.3.4 Conclusion

The sequence of tests and monitoring outlined in existing study plans should provide sufficient data for verification of empirical models; for input to theoretical models; and for thermal, mechanical, and thermomechanical model validation exercises. Detailed design and pretest analyses need to be undertaken for all tests to affirm that maximum benefit is derived from the thermomechanical experiments and to ensure that poor pretest analysis, poor characterization, or poor test results do not result in a false negative judgment that invalidates a model.

Validation of some aspects of the thermomechanical models can be accomplished using the tests planned in the SCP (DOE 1988) at the ESF prior to licensing. Other aspects of the models may require additional data to be developed as both the performance confirmation testing and the extensive excavation and monitoring associated with repository construction progress.

# 4.0 SCHEDULE

The in situ mechanical properties tests will be conducted to produce data to support the project milestones. Figure 4-1 illustrates the general schedule of the major milestones of the projects. Tests are projected to begin in FY '97 but specific scheduling will be based on data needs and construction schedule.

Initial tests including plate loading and borehole jacking will provide data to support the In Situ Thermal Testing Program which will, in turn, support the viability assessment. Follow-on testing including block, slot, and prism tests, as well as additional plate loading and borehole jacking will be performed to support initial and final LAD. The locations and-specific schedules for these follow-on tests will be determined following completion of the initial tests in the ESF. Naturally, data collected during the early phases (plate loading and borehole jacking) will support later phases as well.

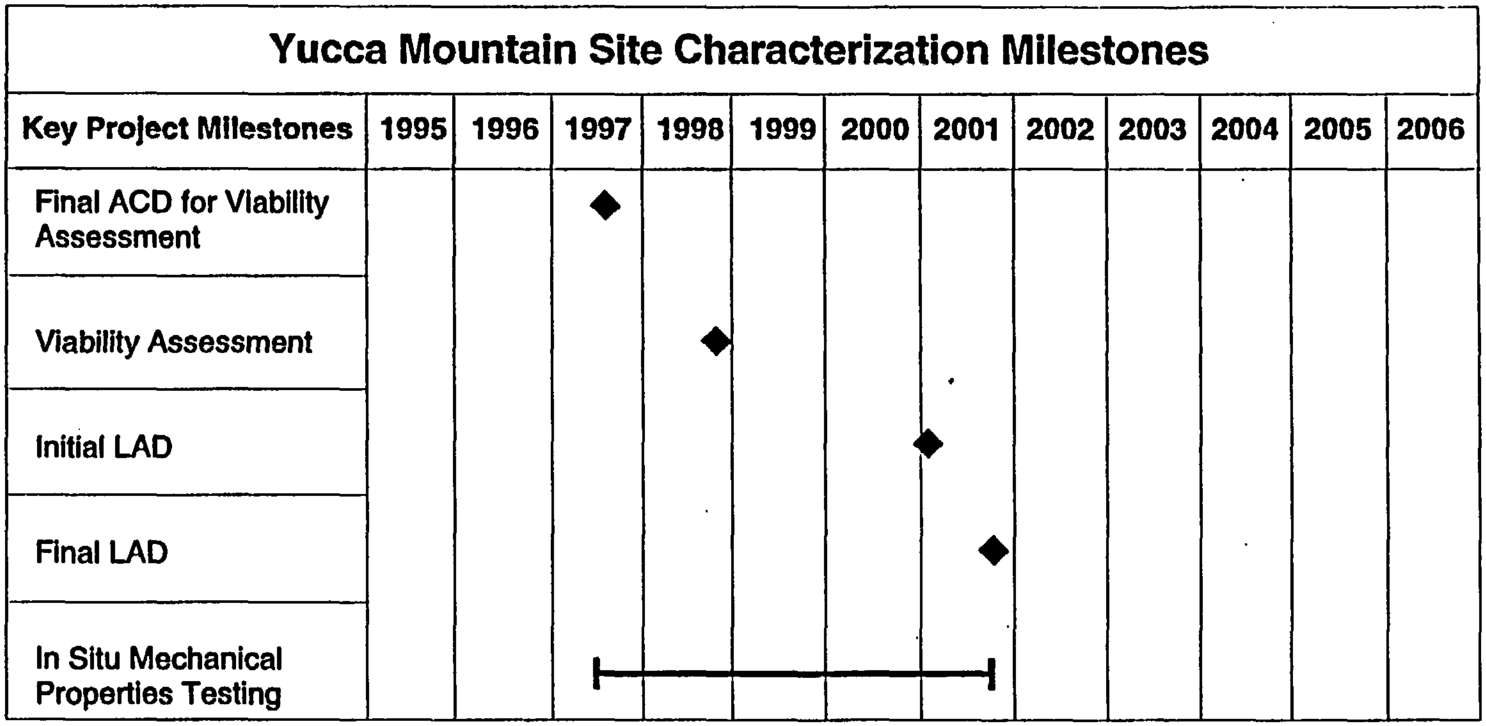



Figure 4-1. Major milestones of the Yucca Mountian site characterization project compared with the testing schedule for in situ mechanical properties.

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