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Numerical Prediction of Basalt Response for Near-Surface Test Facility Heater Tests #1 and #2

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NUMERICAL PREDICTION OF BASALT
RESPONSE FOR NEAR-SURFACE
TEST FACILITY HEATER
TESTS #1 AND #2

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ABSTRACT

This report details the numerical predictions undertaken by Dames and Moore for Rockwell Hanford Operations' Basalt Waste Isolation Project. Predictions are made for the temperatures, stresses, strains and displacements in the basalt around Full-Scale Heater Tests #1 and #2 at the Near-Surface Test Facility using the finite element code DAMSWEL. The rock around the main heaters was modeled using an axisymmetric idealization in which deformational properties were transversely isotropic with a bilinear stress/strain relationship which was independent of temperature. The selection of the input parameters represents an engineering assessment of their values based on the results of laboratory tests and in situ measurements. The predictive modeling analysis, using the best information available as of April 1980, was completed prior to test startup. Additional information on geology, geological characterization, rock-mass characterization, laboratory properties, and field properties of basalt is being acquired on a regular basis as part of the overall Near-Surface Test Facility test program. An assessment of the effect of additions to the data base upon the predictive modeling and test analysis shall be made on a periodic basis.

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CONTENTS

1.0	Introduction	1
2.0	Conclusions	2
3.0	Full-Scale Heater Tests #1 and #2	3
3.1	Introduction	3
3.1.1	Full-Scale Electric Heater Test #1	4
3.1.2	Full-Scale Electric Heater Test #2	4
3.2	Geological Characterization	8
3.2.1	Rock-Mass Characterization	10
3.2.2	Goodman Jack Results	19
3.3	Laboratory Material Properties	20
3.3.1	Thermal Properties	20
3.3.2	Mechanical Properties	22
4.0	Numerical Model	22
4.1	Rock-Mass Properties	22
4.1.1	Thermal Properties	29
4.1.2	Mechanical Properties	30
4.2	Geometric Idealization	38
4.2.1	Thermal Loads	39
4.2.2	Physical Boundaries	43
4.3	Numerical Formulation.	43
4.3.1	Incremental Tangent Stiffness Approach.	49
4.3.2	Iterative Approach.	49
4.3.3	Test Examples	53
4.4	Checks On Accuracy of Model	54
5.0	Predictive Results	59
5.1	Full-Scale Heater Test #1	59
5.2	Full-Scale Heater Test #2	64
5.3	Predictions of Instrument Response	78
	References.	91
	Appendix.	94
FIGURES:		
3.1	Diagrammatic Illustration of FS#1 with Eight Peripheral Heaters.	5
3.2	Diagram of Electric Heaters (a) Central and (b) Peripheral.	6
3.3	A Plan View of the Test Layout Showing Heater and Instrument Hole Locations	7
3.4	A Plan View of a Main Heater and Instrumentation Layout for FS#2	9
3.5	Cross Section of Extensometer Room, Looking South	12
3.6	Full-Scale Heater Test #2, Looking East	13

3.7	Full-Scale Heater Test #1, Looking East	14
3.8	Thermal Conductivity of Pomona Basalt as a Function of Temperature	21
3.9	Relative Enthalpy ($H_T - H_{298\text{ K}}$) for Basalt as a Function of Temperature	23
3.10	Specific Heat (C_p) for Basalt as a Function of Temperature	24
3.11	Coefficient of Thermal Expansion Versus Temperature	25
3.12	Elastic Properties of Intact Cores of Pomona Basalt (a) Young's Modulus and (b) Poisson's Ratio	26
3.13	Young's Modulus (E) Versus Temperature.	27
4.1	Normal Loading of a Fracture in Basalt.	31
4.2	Correlation Between In Situ Modulus of Deformation and Geomechanics Classification Rock-Mass Rating (RMR).	32
4.3	Rock-Mass Deformation Modulus Reduction Factor as a Function of Rock Quality and Minimum Principal Stress	33
4.4	Physical Limits on Anisotropic Material Parameters.	36
4.5	Three-Dimensional Mesh Used for Analysis of FS#1.	40
4.6	Vertical Sections Passing (a) Through a Peripheral Heater and (b) Between Two Neighboring Heaters; (c) Temperature Contours on a Vertical Circumferential Section ($^{\circ}\text{C}$)	41
4.7	Temperatures at FS#1 Mid-height for Asymmetric Thermal Loading	42
4.8	Part of the Three-Dimensional Mesh Used by FINEL	44
4.9	(a) Two-Dimensional Mesh With Insert Element and (b) Alternate Insert for Analysis of Extensometer Room Effects	45
4.10	Temperature Contours (a) Without and (b) With Second Tunnel	46
4.11	Part of a Three-Dimensional Mesh of FS#2	47
4.12	Displacement Obtained from an Axisymmetric Model at Extensometer Room Wall (FS#2).	48
4.13	Deviation from Correct Solution Due to Large Incremental Loads for (a) Hardening Material and (b) Locking Material.	50
4.14	Solution Convergence of the Initial Stress Method for a Locking Material for (a) a Thick-Walled Cylinder and (b) a Plane Strain Compression Block	52
4.15	Distribution of Parameters Along Radius of Cylinder for (a) Temperature, (b) Stress at 1.0 Second, and (c) Stress at 2.0 Seconds	55
4.16	Three-Dimensional Mesh for FS#2.	56
4.17	Comparison of Temperatures for FS#2 Predicted by FINEL and SATAN.	57
4.18	Temperatures Predicted in FS#2 Using Two-Dimensional Axisymmetric Model DAMSWEL	58
4.19	Comparison of Two-Dimensional Results for FS#2 Predicted by DAMSWEL and FINEL.	60
4.20	Comparison of Displacements for FS#2 Using (a) FINEL and (b) DAMSWEL.	61
5.1	Mesh and Thermal Boundary Conditions for FS#1	62
5.2	Initial In Situ Stress and Stress Boundary Conditions for FS#1	63

5.3	Temperature Contours for FS#1 (a) 100 Days, (b) 300 Days, and (c) 500 Days	65
5.4	Temperature History for Thermocouple 1T01T01	66
5.5	Stress History at IRAD Gage 1C02C01	67
5.6	(a) Deformed Mesh and (b) Principal Stress at 100 Days for FS#1	68
5.7	(a) Deformed Mesh and (b) Principal Stress at 300 Days for FS#1	69
5.8	(a) Deformed Mesh and (b) Principal Stress at 500 Days for FS#1	70
5.9	(a) Radial and (b) Hoop Stresses at 100 Days at Heater Mid-Height for FS#1	71
5.10	(a) Radial and (b) Hoop Stresses at 300 Days at Heater Mid-Height for FS#1	72
5.11	(a) Radial and (b) Hoop Stresses at 500 Days at Heater Mid-Height for FS#1	73
5.12	Mesh and Thermal Boundary Conditions for FS#2.	74
5.13	Temperature Contours for FS#2 (a) 100 Days, (b) 300 Days, and (c) 500 Days	75
5.14	Temperature Prediction at Instrument 2T03T04	76
5.15	Stress Prediction at Instrument 2U09C01	77
5.16	(a) Deformed Mesh and (b) Principal Stress at 100 Days for FS#2	79
5.17	(a) Deformed Mesh and (b) Principal Stress at 300 Days for FS#2	80
5.18	(a) Deformed Mesh and (b) Principal Stress at 500 Days for FS#2	81
5.19	(a) Radial and (b) Hoop Stresses at 100 Days at Heater Mid-Height for FS#2	82
5.20	(a) Radial and (b) Hoop Stresses at 300 Days at Heater Mid-Height for FS#2	83
5.21	(a) Radial and (b) Hoop Stresses at 500 Days at Heater Mid-Height for FS#2	84
5.22	Radial Displacement at Heater Mid-Height at 500 Days for FS#2.	85
5.23	Temperature Prediction at Instrument 2E05T05 for FS#2.	86
5.24	Displacement at Instrument 1U01U02 for FS#1.	87
5.25	Displacement at Extensometer Collar 2E21E00 for FS#2	88
5.26	Stress Prediction at Instrument 1C02C01 for FS#1	89
5.27	IRAD Gage Showing Transient Tangential Stress Behavior when Heater Power Levels are Changed, FS#2	90

TABLES:

3.1	Power Levels for FS#1 and FS#2	8
3.2	Summary of Rock-Mass Classifications for Blocks in FS#1 and FS#2.	17
3.3	Rock-Mass Classification Values	19
3.4	Average Modulus of Deformation Values.	20
4.1	Summary of Rock-Mass Classifications for Blocks in FS#1 and FS#2.	28
4.2	Statistical Summary of FS#1 and FS#2 Rock-Mass Parameters	29
4.3	Cross-Anisotropic Deformational Properties	38

PREFACE

A test program has been established by the Basalt Waste Isolation Project (BWIP) at the Near-Surface Test Facility (NSTF), located at Gable Mountain on the Hanford Site near Richland, Washington. The objective of the NSTF test program is to provide information on the thermomechanical response of basalt to thermal loading. This information will be used to assess the feasibility of using basalt as a geologic media for the permanent disposal of nuclear waste. Should a decision be made to proceed with the development of a repository for such disposal, the information collected at the NSTF would provide part of the data base needed to support repository design, construction, and licensing.

The existing NSTF test program concerns basalt thermomechanical response on the canister scale or in the very-near field (within several meters of a canister). The general logic for this canister-scale test program at the NSTF is illustrated in Figure 1. Three stages of tests are planned. The initial stage, presently in operation, involves full-scale electric heater tests. The second and third stages are comprised of a jointed block test and nuclear waste tests, respectively, and are scheduled to be in operation by 1982. The logic illustrated in Figure 1 indicates that a comparison will be made between actual test results and the predicted response obtained by numerical modeling of the test. The model shall be considered validated for repository design at the canister scale when agreement between actual and predicted responses is obtained. It can then be used to support the licensing process.

The process shown in Figure 1 is an iterative process. If satisfactory agreement between actual and predicted response is not reached, several options will be considered. These options include revising the model, revising the input data to the model, and/or revising the in situ tests. A detailed description of the logic for the first stage of testing, Full-Scale Tests #1 and #2, is provided in Figures 2 and 3. The numerical modeling and predictive data which will be used in the comparison with actual test responses are discussed in this report.

A primary objective of the NSTF test program is to provide the data base needed to validate numerical models at the canister scale for repository design and licensing. The logic of this validation process is shown in Figures 1, 2, and 3. The process consists of constructing a pre-test data base from laboratory testing, in situ testing, and geologic characterization of the rock mass during construction. This pre-test data base is then integrated with an evaluation of the fundamental thermal and thermomechanical relationships in order to construct the numerical model. The numerical model consists of a set of conservation or equilibrium equations, a set of initial conditions, and a set of boundary conditions which describe the problem to be modeled. A supplementary set of equations (consisting of the constitutive relations) are then developed to provide for the solution and prediction of the specific test response. For Full-Scale Tests #1 and #2, the responses are temperature, stress, and displacements on the canister scale (very-near field).

Model validation as illustrated in Figures 1, 2, and 3 is accomplished by comparing predictions with actual field measurements. If there is satisfactory agreement, then the model and the process of developing the model is considered to be valid. If agreement is not achieved, then the iterative process previously discussed is initiated and appropriate revisions in the model, input data, and/or test program are made.

The validation of numerical codes is not, however, as straight forward as the above explanation indicates. What constitutes agreement between actual and predicted data is not easily defined in precise terms. Model validation can be said to be complete when discrepancies between actual and predictive data are characterized as random. This requires that large systematic errors in instrumentation, laboratory measurements, and geologic characterization of the rock mass be identified and accounted for in analysis of the NSTF Full-Scale Heater Tests. This, in turn, requires that sufficient sampling of input parameters to the model be undertaken to reduce random errors to acceptable levels.

The amount of time and effort spent in identifying and accounting for systematic errors and reducing random errors is a function of how the model is to be used. In actuality, a model is validated for a specific use or application. The same model may be valid in some applications and not in others. This ultimately means that satisfactory agreement between actual and predicted data requires an understanding and definition of the particular use of the model.

The flow chart in Figure 4 illustrates the process for the design of a repository. Repository design requires that a wide range of geometric-scale problems be solved. For example, for the very-near field, it may be the selection of canister spacing. For the near field, it may be selection of pillar sizes. For the far-field scale, it may be the effects of thermal loading on ground uplift/subsidence, and the ground-water regime. The full-scale heater tests relate specifically to selecting design-allowable waste and canister temperatures, and design-allowable rock temperatures and stresses within several meters of the canister borehole. The full-scale heater tests will, therefore, provide the geotechnical data base and experience in analyzing thermomechanical phenomena needed by the designer in selecting a rational design criteria for canister design in the very-near field.

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FIGURE 1. Very-Near-Field (Canister-Scale) Repository.

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RCP8102-136

FIGURE 2. Full-Scale Heater Test #1.

RCP8102-137

FIGURE 3. Full-Scale Heater Test #2.

RCP8110-142

FIGURE 4. The Repository Design Process.

1.0 INTRODUCTION

The Near-Surface Test Facility (NSTF) at Gable Mountain has been selected for an initial in situ testing program in basalt. The tests have been scheduled to progress in two phases, Phase I and Phase II. The Phase I tests include Full-Scale Heater Tests #1 and #2 (FS#1 and FS#2), which are the subject of the present modeling effort. The layout of the NSTF tests is detailed in BWIP Staff (1979).

Specific objectives of the NSTF tests are listed below:

1. Measurement of thermomechanical rock-mass properties to:
 - Determine average rock-mass properties and, if possible, properties as functions of stress, direction, temperature, rock quality, and size.
 - Verify magnitude of properties and phenomena under complex geometries and boundary conditions (e.g., heater experiments).
2. Characterization of rock-mass properties for generalization of NSTF conditions to other sites.
3. Determination of thermomechanical rock-mass response under operating and overload conditions:
 - Determine upper-thermal power load which the basalt can withstand before failure.
 - Investigate rock/liner interactions.
 - Demonstrate the thermomechanical behavior of basalt under the range of conditions possible with alternate waste forms, various repository configurations, and waste loading densities.
4. Provision of response data for verification and validation of numerical analyses of tests.

This report contains a description of numerical analyses used in generating predicted thermomechanical response of the basalt in NSTF FS#1 and FS#2.

The methodology used in assessing the thermal and mechanical parameters characterizing the basalt is described and the values of all input parameters detailed. The selection of these input parameters represents an engineering assessment based on the results available from laboratory tests and in situ measurements. The influence of the geological conditions at the site and the state of the rock mass on these input parameters is considered. An analysis is made of the effects of assumptions inherent in the model and accuracy of available input data.

The predictive modeling analysis used, as input, the best information available as of April, 1980 and was completed prior to test startup. Since that time additional information has been added to the data base which supports the predictive modeling activity. Some of the data available in April was preliminary in nature (i.e., rock-mass deformation properties determined by the borehole jacking method); other data has been supplemented by additional testing (i.e., laboratory, thermal, and mechanical properties). As additions to the NSTF data base are made, an assessment will be made as to the effect of such additions on the predictive modeling and test analysis.

2.0 CONCLUSIONS

A transient nonlinear thermal stress, finite element model using the code DAMSWEL (Boonlualohr et al., 1980b), has been used to predict temperature, displacement, and stress changes in the rock mass surrounding FS#1 and FS#2. From the results of this model, predicted time histories have been generated for each instrument (thermocouples, IRADs, USBMs and extensometers) monitoring these tests.

The geological characterization study, the in situ measurements, and the laboratory tests made on samples of the basalt at the test location, indicate the following:

- The fractured nature of the basalt requires that it be modeled mechanically as a transversely homogeneous isotropic medium with five independent constants. Deformational properties in directions parallel to the test room floor are equal in value but are different to values in directions normal to the plane of the floor.
- The deformational behavior of the fractures necessitates that the constitutive stress/strain relationship of the rock be modeled as a bilinear locking material.
- The mechanical properties of the rock, including Young's modulus, and the coefficient of thermal expansion can be considered independent of temperature, for the temperature range to be encountered during the tests.
- Care must be taken in choosing deformational moduli and Poisson's ratios for a transversely isotropic material so that physical constraints are not violated.
- The thermal properties of the basalt are homogeneous but are dependent on temperature.
- The heat transfer coefficient between rock and air, though not accurately known, will not impact significantly the instrument predictions.

The finite element model idealization for both heater tests is axisymmetric about the main heater. In this idealization, the thermal properties are temperature dependent and the mechanical properties are transversely isotropic, with a bilinear stress/strain relationship. Numerical tests were conducted to determine the effect of assumptions implicit in the model idealization. It was found that:

- The assumption of axisymmetry is valid except in a localized region close to a peripheral heater and close to the extensometer room.
- Neglecting the presence of the extensometer room does not affect the temperatures, stresses, or absolute displacements at instruments located within the rock area being monitored. It does, however, affect displacements at the extensometer heads located in the wall of the extensometer room. The absolute movement of these heads should be determined for accurate predictions of extensometer readings.
- Heat transfer boundary conditions can be used to model the effect of heat transfer to and from the air in the tunnels.

The results of the numerical studies show the major influence of rock-mass modulus on induced stresses. Difficulties in trying to predict the modulus characteristics of fractured rock indicate the need for a more fundamental understanding of the constitutive behavior of the rock mass.

Additional deformability studies are considered essential if meaningful predictions and interpretations are to be made. In addition to those already proposed for the jointed-block test, plate-bearing tests are recommended.

Further evaluation of the in situ stress field at the NSTF should be considered, particularly in terms of establishing a proven test procedure in the fractured basalt. The use of the CSIRO (Australia) triaxial strain cell is recommended.

3.0 FULL-SCALE HEATER TESTS #1 AND #2

3.1 INTRODUCTION

The Phase I Heater Tests #1 and #2 are described in detail in BWIP Staff (1979), and only a brief description is included here.

3.1.1 Full-Scale Electric Heater Test #1

A full-scale electric heater 2.5 m long and 0.36 m in diameter, surrounded by eight peripheral heaters (4.6 m heated length) is proposed to simulate expected repository operating conditions on a canister scale for approximately one year and then at double the operating thermal loading for a further period. The test has been designed to determine the following information:

- The temperatures, strains, and displacements on a canister scale
- Validation of in situ thermomechanical properties and how they vary with temperature and stress
- Verification of numerical models
- The mechanical and thermal stability of the heater and the basalt rock mass at equivalent thermal loadings of twice those proposed for conceptual repository design.

Full-Scale Heater Test #1 is illustrated diagrammatically in Figure 3.1. The central and peripheral electric heaters are shown in Figures 3.2a and 3.2b, respectively. A plan view of the test layout showing heater and instrument hole locations is illustrated in Figure 3.3. All of the heaters in this test will have power levels varying with time (Table 3.1).

3.1.2 Full-Scale Electric Heater Test #2

A single full-scale electric heater test is proposed to induce borehole decrepitation to determine the upper limit of acceptable thermal power of a single canister. The test consists of an overload thermal gradient test and is designed to provide the following information:

- The determination of the maximum acceptable power input from a single canister by determining the thermal conditions under which decrepitation on rock failure renders the borehole mechanically unstable
- The modes and subsequent thermal and mechanical effects of rock decrepitation in the borehole canister field
- Data for validation of in situ thermomechanical properties of basalt and the effects of temperature and fracturing on those properties
- The correlation of results with the Full-Scale Heater Test #1
- The verification of numerical models, including those with the ability to model nonlinear rock failure.

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FIGURE 3.1. Diagrammatic Illustration of FS#1 with Eight Peripheral Heaters.

(a) FULL-SCALE HEATER

(b) PERIPHERAL HEATER

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FIGURE 3.2. Diagram of Electric Heaters (a) Central and (b) Peripheral.

FIGURE 3.3. A Plan View of the Test Layout Showing Heater and Instrument Hole Locations (Moak and Wintczak, 1980).

TABLE 3.1. Power Levels for FS#1 and FS#2 (BWIP Staff, 1979).

Power Levels for Heater Test #1				Power Levels for Heater Test #2		
Time (Days)	Step	Central Power (kW)	Peripheral Power (kW)	Time (Days)	Step	Central Power (kW)
0 - 30	1	0	0.25	0 - 9	1	1.0
30 - 90	2	1.00	0.25	90 - 210	2	3.0
90 - 210	3	1.00	0.50	210 - 300	3	5.0
210 - 300	4	2.00	1.00	300 - 420	3	5.0
300 - 420	4	2.00	1.00	420 - 510	3	5.0
420 - 510	5	2.00	1.00*	510 - 540	4**	0
510 - 540	5	2.00	1.00*	540 - 630	5	5.0
540 - 630	5	2.00	1.00*	630 - 720	5	5.0
630 - 720	5	2.00	1.00*	720 - 900	6†	0
720 - 900	6†	0	0			

*Four peripheral heaters only, other four reduced to zero (see diagram below).

**Step 4 heater removal for borehole inspection.

†Cool Down.

Heater Switching Diagram for
FS#1 Step 5

The heater and instrumentation layout is illustrated in plan in Figure 3.4. The heater is the same as the central heater in FS#1 (see Figure 3.2a) and has stepped power levels varying with time (see Table 3.1).

3.2 GEOLOGICAL CHARACTERIZATION

The geological characterization of the NSTF FS#1 and FS#2 has been reported in full by Moak and Wintczak (1980). This characterization study consisted of mapping and quantifying rock-mass quality using the present underground openings and a large number of vertical and horizontal boreholes. The underground openings and borehole cores were logged in detail, photographed, and all of the borehole and line survey log data were computerized. The full-scale heater boreholes were completely photographed with downhole equipment. Some of the larger monitoring holes were characterized using impression packers. The

FIGURE 3.4. A Plan View of a Main Heater and Instrumentation Layout for FS#2 (Moak and Wintczak, 1980).

geological data were summarized in terms of fracture frequency and rock-mass quality for two rock-mass classification schemes (Bieniawski, 1973; Barton et al., 1974). The volume of rock around both heaters was subdivided and characterized to determine the homogeneity of the rock mass around the heaters. The results of the geological characterization study of Moak and Wintczak (1980) are briefly discussed in the next section. In situ Goodman Jack tests were undertaken in a number of boreholes (Shuri et al., 1980). The results from these tests are summarized in Section 3.2.2. Limited in situ stress measurements have been conducted at the NSTF by the hydrofracturing technique. The results from these tests were not conclusive due to the necessary assumptions concerning the principal stress orientations and accuracy of the technique itself.

3.2.1 Rock-Mass Characterization

The NSTF, located in the entablature of the Pomona Member of the Saddle Mountains Basalt, is approximately 45 m beneath the surface of Gable Mountain, an anticlinal outcrop on the Hanford Site. Tunnel excavations penetrate the Elephant Mountain Member, the Rattlesnake Ridge interbed, and the entablature of the Pomona Member. On the basis of the available data from the Pomona entablature, three joint sets have been identified by their angle of dip. The high-angle set, from 66 to 90 degrees, comprises 47% of the over 7,000 joints measured. These represent the faces of the polygonal cooling columns and some extend to 2 m in length. The 38- to 65-degree set (23%) and the 0- to 37-degree set (30%) are generally discontinuous joints limited to the confines of an individual column. None of the joints were capable of being identified between boreholes or from the subsurface to the tunnel invert.

No tectonically induced joints were recognized in the joint study. Joint infilling was generally less than 0.3 mm and the filling was primarily a smectite clay with occasional calcite, pyrite, or silica.

Because of the preferential direction of the boreholes and tunnels, there was a bias in the strike direction of the joints sampled. Efforts to eliminate this bias were generally successful, but no preferred strike direction was seen in any of the three joint sets. It is presumed that the polygonal cooling pattern of the basalt results in a random orientation of the joints.

The rock, although highly jointed, may be classified as "good rock" using various engineering rock-mass classifications developed for an overall assessment of its constructibility.

3.2.1.1 Determination of Joint Sets. Characterization of the joint sets in the arrays for FS#1 and FS#2 was a process of joint analysis of the geomechanical logs, detail line maps, and core logs. Using this information, joint sets were defined. From the boreholes, both horizontal and

vertical, volumes of the rock mass surrounding FS#1 and FS#2 were delineated. Twelve blocks, Figures 3.3 to 3.7, were outlined for each heater test, and joint orientations, dip groups, and rock-mass classifications were determined. The entablature of the Pomona is highly jointed due to the polygonal cooling pattern and numerous low-angle joints. The polygonal columns are continuous, some exceeding 2.4 m in length, but observations were limited by the dimensions of the tunnel and the shotcrete support system. Over their length, the columns are sinuous or undulating, varying in overall dip by 15 to 20 degrees. The number and size of column faces often varied every 0.61 to 1.5 m along the length of the column. This, in conjunction with the low-angle joints which were usually continuous only within a single column 15 to 31 cm in diameter, prohibited any type of correlation from the tunnel invert to depth or between boreholes.

The sinusoidal nature of the columns made the waviness measured in the geomechanical logs less meaningful, since the waviness occurs over approximately a 1-m length of the joint and the core provides only several centimeters of the joint for analysis. The joint apertures averaged approximately 0.2 mm and varied by ± 0.1 mm. With such small average apertures and minor variations, this parameter was not useful in defining joint sets.

The undulating nature of the discontinuities in polygonal columns caused statistical variation in the strike and dip measurements, making joint sets difficult to determine and poorly defined. Histograms of dip frequency were plotted for each of the boreholes. These plots demonstrate the biased sampling of joints by horizontal and vertical boreholes. As expected, the horizontal holes intersected a higher number of high-angle joints, while vertical holes intersected a majority of low-angle joints. By plotting equal numbers of joint dips from horizontal and vertical boreholes, an unbiased dip frequency can be obtained. With these histograms, three groups or sets of joints were determined: the 66- to 90-degree dip comprising 47% of the measured joints, the 0- to 37-degree dip comprising 30%, and the 38- to 65-degree dip, the remaining 23%.

Using these groupings, histograms of frequency versus strike were plotted. The 0- to 37-degree and 38- to 65-degree plots showed no definable pattern, but the 66- to 90-degree plot has two broad groupings centered around 90 and 270 degrees. These two groups are apparent whenever the 66- to 90-degree-dip group from any of the horizontal holes is plotted. However, if the 66- to 90-degree-dip group from only the vertical holes is used, there is no definable pattern. This is attributed to the layout of the horizontal holes, which were drilled due south, creating a bias in the orientation of the joints intersected.

FIGURE 3.5. Cross Section of Extensometer Room, Looking South (Moak and Wintczak, 1980).

FIGURE 3.6. Full-Scale Heater Test #2, Looking East (Moak and Wintczak, 1980).

FIGURE 3.7. Full-Scale Heater Test #1, Looking East (Moak and Wintczak, 1980).

When the 66- to 90-degree-dip group from the detailed line maps in the heater test room are plotted, the 90- and 270-degree groups are not apparent, but a group from 40 to 160 degrees is dominant. This dominance is also due to a directional bias, the east-west direction of the heater test room. The convention for measuring the joint strike, 90 degrees counterclockwise from the joint dip, contributes to the strike location of this biased grouping. These two factors, coupled with the predominantly southerly dip of the high-angle joints caused by the anticlinal structure, all contribute to the biased dominance of the 40- to 160-degree grouping.

In summary, the Pomona entablature is composed of three joint groups or sets defined by the frequency of dip:

- 0 to 37 degrees
- 38 to 65 degrees
- 66 to 90 degrees.

The first two groups represent 53% of the total number of joints measured. The remaining 47% are in the 66- to 90-degree group. The 66- to 90-degree group are undulating and represent faces of the polygonal cooling columns. These joints may be continuous up to 2.5 m in length. The remaining two groups are discontinuous, seldom crossing the joint boundaries of the 66- to 90-degree group. Analysis of strike orientation data did not reveal preferred orientations of these joint sets. The cooling patterns of the basalt joints are not oriented to a preferred direction, but only in sets delineated by dip frequency.

3.2.1.2 Rock-Mass Classification. Engineering classifications of the rock mass surrounding FS#1 and FS#2 were developed. Three engineering classifications were used: the rock-mass rating (RMR) (Bieniawski, 1979); the Q System (Barton et al., 1974); and the rock-quality designation (RQD) (Deere et al., 1969). The RQD system is the summation of the lengths of core pieces greater than 10 cm in length divided by the percentage of core recovered per run. The Geomechanics Classification (Bieniawski, 1979) considers five rock-mass parameters:

- Uniaxial compressive strength
- Rock-quality designation
- Discontinuity spacing
- Condition of discontinuities
- Groundwater conditions
- Joint orientation.

Each of these is assigned a numerical value and the rock-mass rating is calculated from the formula

$$\text{RMR} = \text{RQD}/5 + \text{UCS} + \text{Ds} + \text{Dc} + \text{G} + \text{Jo}$$

where

	<u>VALUE</u>	<u>DESIGNATION</u>
RQD is the rock-quality designation	Table 3.2	
UCS is uniaxial compressive strength	15	313 MPa
Ds is discontinuity spacing	6	70 to 90 mm
Dc is discontinuity condition	27	Slightly rough aperture <1 mm, no weathering
G is groundwater	13	Inflow <10 L/min
Jo is joint orientation	-5	Fair.

The Q System (Barton et al., 1974) considers six categories in determining the rock-mass classifications:

- Rock-quality designation
- Joint set number
- Joint roughness number
- Joint alteration number
- Joint water reduction factor
- Stress reduction factor.

TABLE 3.2. Summary of Rock-Mass Classifications for Blocks in FS#1 and FS#2 (Moak and Wintczak, 1980).

Block	Joint Density Per Meter	RQD	RMR	Q
Blocks in FS#1				
1-1-A	15.85	35	63.0	11.67
1-1-B	15.32	42	64.4	14.00
1-1-C	12.92	45	65.0	15.00
1-2-A	15.19	45	65.0	15.00
1-2-B	12.99	46	65.2	15.33
1-2-C	14.20	31	62.2	10.33
1-3-A	14.92	42	64.4	14.00
1-3-B	19.38	39	63.8	13.00
1-3-C	16.33	28	61.6	9.33
1-4-A	15.45	35	63.0	11.66
1-4-B	15.71	36	63.2	12.00
1-4-C	13.74	49	65.8	16.33
Average	15.15	39(poor)	63.9(good)	13.14(good)
Blocks in FS#2				
2-1-A	17.19	44	64.8	14.67
2-1-B	12.89	62	68.4	20.67
2-1-C	11.31	69	69.8	23.00
2-2-A	15.12	40	64.0	13.33
2-2-B	16.69	54	66.8	18.00
2-2-C	11.22	65	69.0	21.67
2-3-A	16.17	39	63.8	13.00
2-3-B	13.91	52	66.4	17.33
2-3-C	11.38	58	67.6	19.33
2-4-A	15.84	39	63.8	13.00
2-4-B	14.14	51	66.2	17.00
2-4-C	11.41	57	67.4	19.00
Average	13.94	53(fair)	66.5(good)	17.50(good)

RQD = Rock quality designation.

RMR = Rock-mass rating.

Q = Q system.

These parameters were assigned values based on Barton et al. (1974) classifications and the formula:

$$Q = (RQD/J_n) (J_r/J_a) (J_w/SRF)$$

where

	<u>VALUE</u>	<u>DESIGNATION</u>
RQD is the rock-quality designation	see Table 3.2	
J _n is the joint set number	7.5	Three joint sets
J _r is the joint roughness number	2.5	Rough-smooth undulating
J _a is the joint alteration number	1.0	Unaltered joint walls
J _w is the joint water reduction factor	1.0	Minor inflow, <5 liter/min
SRF is the stress reduction factor	1.0	$\sigma_e/\sigma_1 = \sigma_t/\sigma_1 = 1.6$

was used to derive Q (see Table 3.2).

To aid in evaluating these formulas and as a comparative tool, histograms of the frequency of joints per borehole were plotted. In many of the boreholes, zones of rubble caused by the drilling technique and/or the intersection of several joints were recorded. These zones were included in the histograms using a factor of six joints per foot. A line was drawn on these histograms to denote the number of joints in the rubble zones. Using these histograms joint densities for each of the 12 blocks in FS#1 and FS#2 were calculated (see Table 3.2).

The RQD and joint densities are variable with each hole, but limited cross-hole similarities can be seen. Generalities can be made by comparing the RQD of the vertical holes in the blocks to determine possible zones of damage due to the excavation method. The initial 0.8 to 1.0 m below the collars of the holes have average RQD values of less than 20. The values increase to an average of approximately 40 to 50 at depths of 5.5 to 6.0 m below the collar. Rock-quality designation values vary from 0 to 100 below this point, but give an overall indication of reasonably competent rock.

Values for joint densities, RQD, RMR, and Q were calculated for each of the twelve blocks in FS#1 and FS#2 and are summarized in Table 3.2. These calculated values indicate that the highly jointed Pomona unit in these blocks can be classified as "good rock" in the Q and rock-mass rating systems (Table 3.3). The RQD rating (Table 3.3) is lower, because it only considers one of the many parameters used in the other classifications. As previously stated, the rock was easily broken along healed

joints, due to weakening by the excavation and the borehole drilling; hence, few pieces are greater than 10 cm in length.

TABLE 3.3. Rock-Mass Classification Values.

Rock-Mass Quality (Q) (Barton et al., 1974)	
Exceptionally poor	0.001 - 0.01
Extremely poor	0.01 - 0.1
Very poor	0.1 - 1.0
Poor	1.0 - 4.0
Fair	4.0 - 10.0
Good	10.0 - 40.0
Very good	40.0 - 100.0
Extremely good	100.0 - 400.0
Exceptionally good	400.0 - 1000.0
Rock-Quality Designation (RQD) (Deere et al., 1969)	
Very poor	0 - 25
Poor	25 - 50
Fair	50 - 75
Good	75 - 90
Excellent	90 - 100
Rock-Mass Rating (RMR) (Bieniawski, 1979)	
Very poor	0 - 20
Poor	21 - 40
Fair	41 - 60
Good	61 - 80
Very Good	81 - 100

The classifications indicate better overall ratings for the blocks of FS#2. This most likely is attributable to the overall lower RQD values in FS#1 caused by inconsistent excavation and drilling procedures. It is believed that the overall competency is not significantly different in FS#1 from that in FS#2.

3.2.2 Goodman Jack Results

The in situ rock-mass modulus was determined from Goodman Jack tests undertaken in vertical and horizontal boreholes around the full-scale heater tests (Shuri et al., 1980). The results exhibited a strongly non-linear dependence of moduli on applied load, resulting in the rock-mass modulus being expressed as a bilinear relationship of stress. The results from horizontal and vertical boreholes indicated that the modulus determined from the Goodman Jack results was virtually independent of direction. The average moduli for these tests are tabulated in Table 3.4.

TABLE 3.4. Average Modulus of Deformation Values (Shuri et al., 1980).

Statistical Group	Pressure Level, psia	Depth, ft ^b	Average Tangent Modulus x 10 ⁶ psi	Range
Modulus Values for Pressures Greater than 3,000 psi				
Horizontal Boreholes	>3,000	all	2.72 $\pm$ 0.27	0.87 - 8.79
Vertical Boreholes	>3,000	20, 25	1.90 $\pm$ 0.28	0.83 - 5.83
		5, 10	1.00 $\pm$ 0.10	0.53 - 2.02
Modulus Values for Pressures Less than 3,000 psi				
Horizontal Boreholes	<1,000	all	0.37 $\pm$ 0.06	0.14 - 0.99
	1,000 - 3,000	all	1.24 $\pm$ 0.18	0.50 - 2.87
FS#1, Vertical Boreholes	<3,000	all	0.63 $\pm$ 0.11	0.20 - 2.02
FS#2, Vertical Boreholes	<1,000	all	0.32 $\pm$ 0.04	0.19 - 0.48
	1,000 - 3,000	all	0.90 $\pm$ 0.21	0.41 - 2.14

^aMultiply by 6.894757 to convert to Pascals.

^bMultiply by 0.3048 to convert to meters.

3.3 LABORATORY MATERIAL PROPERTIES

The laboratory material properties of rock core taken from the basalt at the NSTF have been: density (ρ), thermal conductivity (k), specific heat (C_p), coefficient of linear thermal expansion (α), Young's modulus (E), Poisson's ratio (ν), and strength (S_c). A number of these properties have been determined for a temperature and confining pressure range of 25^o to 300^oC and 0 to 14 MPa, respectively. The thermal properties k , C_p , and α have been determined by Duvall et al., (1978), Erickson and Krupka (1980), Martinez-Baez and Amick (1978), and FSI (1980). The mechanical properties have been determined by Duvall et al., (1978) and FSI (1980). There are some inconsistencies in the results from these tests due to variations in equipment and test procedures.

3.3.1 Thermal Properties

The thermal conductivity calculated from the tests undertaken by Erickson and Krupka (1980), Duvall et al., (1978) and Martinez et al., (1978) are illustrated in Figure 3.8. These results are in good agreement. There are, however, slight differences in the trend of thermal conductivity with temperature measured by Duvall et al., (1978), Martinez-Baez and Amick (1980), and Erickson and Krupka (1980). Over the temperature range of 25^o to 300^oC, the differences between the respective data are slight.

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FIGURE 3.8. Thermal Conductivity of Pomona Basalt as a Function of Temperature (Erickson and Krupka, 1980).

The relative enthalpy for Pomona basalt at the NSTF as a function of temperature is illustrated in Figure 3.9 for Duvall et al., (1978) and Erickson and Krupka (1980). There exists a significant difference between these data at elevated temperatures. Since the data of Erickson and Krupka (1980) do match very closely the relative enthalpies reported in the literature for other basalts, greater credence should be placed on these results. The specific heat determined from these relative enthalpy values is illustrated in Figure 3.10 as a function of temperature.

The coefficient of linear thermal expansion as a function of temperature, determined by FSI (1980), is shown in Figure 3.11. These results are in close agreement with those obtained by Duvall et al., (1978).

3.3.2 Mechanical Properties

The Young's modulus (E) and Poisson's ratio (ν) of intact cores of Pomona basalt have been determined by FSI (1980), as independent of confining pressure, Figures 3.12 a and b, respectively. The Young's modulus was also found to be independent of temperature (Figure 3.13), while Poisson's ratio did indicate some variation. This variation of Poisson's ratio was not completely determined and further studies are planned to quantify any temperature dependence of ν .

4.0 NUMERICAL MODEL

4.1 ROCK-MASS PROPERTIES

From the geological characterization studies of Moak and Wintczak (1980), it is apparent that the rock mass is moderately fractured throughout the entire region of both in situ heater tests. The fracture orientations are dominantly near-vertical and near-horizontal. The vertical fractures are associated with the columnar structure and there appears to be no preferential strike direction of these features. Also, no major pervasive joints were detected in either test location. Therefore, it can be concluded that the basalt rock mass around the heaters has a transversely anisotropic structure of joints. The homogeneity of the rock mass can be ascertained from the geological classification indices tabulated in Table 4.1 for each rock-mass block. The fracture frequency (f) (number per meter), rock-mass rating (RMR), and rock quality (Q) are similar for each block in both test locations, as summarized in Table 4.2.

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FIGURE 3.9. Relative Enthalpy ($H_T - H_{298 K}$) for Basalt as a Function of Temperature (Erickson and Krupka, 1980).

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FIGURE 3.10. Specific Heat (C_p) for Basalt as a Function of Temperature (Erickson and Krupka, 1980).

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FIGURE 3.11. Coefficient of Thermal Expansion Versus Temperature (FSI, 1980).

(a)

(b)

RCP8102-156

FIGURE 3.12. Elastic Properties of Intact Cores of Pomona Basalt (a) Young's Modulus and (b) Poisson's Ratio (FSI, 1980).

FIGURE 3.13. Young's Modulus (E) Versus Temperature (FSI, 1980).

These standard deviations are very small for a rock-quality index parameter, thus giving additional support to the assumption of homogeneity. It is important to highlight the fact that $\log Q$ should be used in the statistical analysis of Q . These ranges of RMR and Q virtually all lie in a designated rock quality of "good" for both classification schemes (Bieniawski, 1973; Barton et al., 1974). Since little to no variation (joint filling, etc.) of the rock mass was detected, except the fracture frequency from one block to another, it can be concluded that the basalt rock mass around the heaters is reasonably homogeneous. The Goodman Jack results (Shuri et al., 1980) confirm the conclusions stated above. In these results, no definite variations of the in situ modulus were detected for the borehole depth or rock-mass classification index.

The required rock-mass properties for the predictive analyses are the thermal conductivity (k), specific heat (C_p), density (ρ), coefficient of thermal expansion (α), surface heat-transfer coefficient (ω), deformational properties, and rock-mass strength. The significance of these properties to the response of the basalt rock mass subjected to elevated temperatures has been previously discussed in detail (Hardy and Hocking, 1978; Hocking and St. John, 1979). The conclusions from these previous studies were that:

- The surface heat-transfer coefficient of basalt to air has a minimal impact on the resulting temperatures at the instrument locations.
- The deformational moduli have a minimal impact on the thermally induced rock displacements, but a significant impact on the stresses.

TABLE 4.1. Summary of Rock-Mass Classifications for Blocks in FS#1 and FS#2 (Moak and Wintczak, 1980).

Block	Joint Density Per Meter	RQD	RMR	Q
Blocks in FS#1				
1-1-A	15.85	35	63.0	11.67
1-1-B	15.32	42	64.4	14.00
1-1-C	12.92	45	65.0	15.00
1-2-A	15.19	45	65.0	15.00
1-2-B	12.99	46	65.2	15.33
1-2-C	14.20	31	62.2	10.33
1-3-A	14.92	42	64.4	14.00
1-3-B	19.38	39	63.8	13.00
1-3-C	16.33	28	61.6	9.33
1-4-A	15.45	35	63.0	11.66
1-4-B	15.71	36	63.2	12.00
1-4-C	13.74	49	65.8	16.33
Average	15.15	39(poor)	63.9(good)	13.14(good)
Blocks in FS#2				
2-1-A	17.19	44	64.8	14.67
2-1-B	12.89	62	68.4	20.67
2-1-C	11.31	69	69.8	23.00
2-2-A	15.12	40	64.0	13.33
2-2-B	16.69	54	66.8	18.00
2-2-C	11.22	65	69.0	21.67
2-3-A	16.17	39	63.8	13.00
2-3-B	13.91	52	66.4	17.33
2-3-C	11.38	58	67.6	19.33
2-4-A	15.84	39	63.8	13.00
2-4-B	14.14	51	66.2	17.00
2-4-C	11.41	57	67.4	19.00
Average	13.94	53(fair)	66.5(good)	17.50(good)

RQD = Rock quality designation.

RMR = Rock-mass rating.

Q = Q system.

TABLE 4.2. Statistical Summary of FS#1 and FS#2 Rock-Mass Parameters.

	f	RMR	Q
Maximum Range of Parameters			
FS#1	13.0 - 19.4	63 - 65	9.3 - 16.3
FS#2	11.3 - 17.2	64 - 70	13.0 - 21.7
Standard Deviations for Normal Distribution			
FS#1	--	+1.2	+2.4, -2.0
FS#2	--	+2.1	+3.7, -3.1

- The coefficient of thermal expansion has a dominant impact on the thermally induced displacements.
- The temperature field in the rock is dependent on the rock-mass thermal conductivity and specific heat.

Therefore, for temperatures, k and C_p are most significant; for displacements, k , C_p , α , and Poisson's ratio are most important; and for stresses, k , C_p , α , and deformational properties are most important. The rock-mass properties selected for the predictive analyses are discussed in the following sections under thermal properties (k , C_p , and α) and mechanical properties (deformational and strength).

4.1.1 Thermal Properties

Granite rock mass thermal properties k and C_p were observed to be almost unaffected by the jointing, provided water was absent (Cook and Hood, 1978). Since there are no significant clay infillings in the joints nor any significant heterogeneities present, it is highly likely that k and C_p of the basaltic rock mass will be similar to its intact value. The evidence from Stripa (Cook and Hood, 1978), strongly supports such a conclusion. From the present laboratory data (see Section 3.3) the Pacific Northwest Laboratories' (PNL) results have the most credence as being accurate because of the techniques used. The other laboratory data were generated with equipment and procedures which gave rise to modest errors in determining both k and C_p . The PNL data yielded temperature-dependent thermal conductivity and specific heat as given by Equations 4.1a and 4.1b:

$$k = 1 \times 10^{-3} (T-273) + 1.53 \text{ W/m } ^\circ\text{K} \quad (4.1a)$$

$$C_p = 1.28 \times 10^3 - 0.108T - 4.8 \times 10^7 T^{-2} \text{ J/kg } ^\circ\text{K}. \quad (4.1b)$$

The coefficient of linear thermal expansion (α) has only been determined from intact basalt samples. The data from Duvall et al., (1978) and FSI (1980) are in general agreement and these studies indicated that α was mildly dependent on temperature. The FSI data gave rise to α being temperature dependent as given by Equation 4.2, which is in close agreement with the Duvall et al., (1978) results:

$$\alpha = (0.61 \times 10^{-2} (T-273) + 4.9) \times 10^{-6}/K. \quad (4.2)$$

It was assumed that the rock-mass coefficient of thermal expansion was the same as the rock's intact value. This assumption was based on observations of rock response in the Stripa tests. The thermal expansion of the rock is most likely less in situ compared to the intact value. This would be due to the expansion of the rock into free open voids. The joints will not directly affect the rock deformational response by modifying the coefficient of thermal expansion. However, their low initial stiffness at low stress levels will impact the resulting strains and stresses. From the limited data set, the dependence of α on applied stress is not apparent; therefore, it was assumed that α was independent of applied stress.

As mentioned earlier, the surface heat-transfer coefficient (ω) has only a minimal influence on the resulting temperatures at the instrument locations. Quantitative values of ω for basalt were not available; therefore, it was necessary to estimate it. From ventilation studies in mines, ω was estimated to range from 1 to 45 W/m²·K (Hocking and St. John, 1979). The high value of 45 W/m²·K was due to the fact that the thermal transfer from groundwater inflow was included. For the case of negligible groundwater inflow, it is estimated for basalt that ω varies from 1 to 10 W/m²·K. A value of 1 W/m²·K was selected for the predictive analyses.

4.1.2 Mechanical Properties

From the geological characterization studies it was concluded that the basalt rock mass around both in situ heater tests was transversely isotropic in the vertical direction. Due to the low initial stress levels present, it is imperative that any locking behavior as shown in Figure 4.1 be adequately modeled. This would require either a bilinear or continuous function of deformational properties with stress. It was concluded from the results of loading basalt-intact rock and joints that a bilinear relationship would suffice. In fact, the Goodman Jack results (Shuri et al., (1980) confirm that the basalt rock mass for FS#1 and FS#2 exhibits bilinear deformational behavior. The quantification of these cross-anisotropic terms is important in calculating the thermally induced stresses and displacements. There are a number of methods which are available to determine the modulus of the rock mass: (1) rock-mass classification schemes (RMR and Q), (2) fracture-frequency logs and joint stiffnesses, and (3) Goodman Jack results.

FIGURE 4.1. Normal Loading of a Fracture in Basalt (Iwai, 1976).

4.1.2.1 Rock-Mass Classification Schemes. Engineering classification schemes have been used for numerous applications in geotechnical engineering. The two most popular ones at present are the geomechanics classification by Bieniawski (1973) and the Q system by Barton et al., (1974). Both schemes were used in the geological characterization of FS#1 and FS#2 (Moak and Wintczak, 1980). The geological classification determined the rock-mass quality, RMR, and Q for each volume of rock (see Table 3.2).

The geomechanics classification, RMR, has been correlated with the in situ modulus of deformation as illustrated in Figure 4.2. The relationship determined between RMR and the in situ modulus is given by Equation 4.3 (Bieniawski, 1979). For the average value of RMR (=65) for all the blocks in the tests, the predicted in situ modulus for the basalt rock mass is 3×10^4 MPa as given by Equation 4.3.

$$E_m = 2 \times \text{RMR} - 100 \text{ (GPa)} \quad (4.3)$$

where E_m is the rock mass in situ modulus.

FIGURE 4.2. Correlation Between In Situ Modulus of Deformation and Geomechanics Classification Rock-Mass Rating (RMR) (Bieniawski, 1979).

A relationship with the Q classification scheme and the in situ modulus has been proposed (BWIP Staff, 1980). This relationship for an isotropic material is given by Equation 4.4 and shown diagrammatically in Figure 4.3.

$$\frac{E_m}{E_R} = \left(\frac{Q}{1,000} \right)^{\frac{K}{3}} \quad (4.4)$$

where Q is the rock-mass quality, E_R is the intact rock Young's modulus, K is a stress relief coefficient equal to 1 when $\sigma_3 > 5$ MPa, to 1.4 for $1 < \sigma_3 < 5$ MPa, and to 2 when $\sigma_3 < 1$ MPa, and σ_3 is the in situ minimum principal confining stress. The in situ rock-mass modulus as given by Equations 4.3 and 4.4 are 3×10^4 and 2.2×10^4 MPa, respectively, for the higher stress levels. The agreement between the two relationships for RMR and Q, Equations 4.3 and 4.4, at the higher confining stress level is relatively good.

4.1.2.2 Fracture Frequency. Provided the fracture frequencies and orientation of the joint sets and the joint normal and shear stiffnesses are known, the in situ equivalent continuum rock-mass modulus can be determined.

FIGURE 4.3. Rock-Mass Deformation Modulus Reduction Factor as a Function of Rock Quality and Minimum Principal Stress.

Iwai (1976) has studied normal cyclic loading of an induced fracture in basalt. The results of his experiments show that the specimen did not exhibit significant cyclic effects; in fact, the loading and unloading paths were virtually identical (see Figure 4.1). Iwai concluded that Goodman's (1976) dimensionless expression for the stress-deformation curve of a fracture was applicable to his results as given by Equation 4.5:

$$\frac{\Delta V}{V_{mc} - \Delta V} = A \left(\frac{\sigma'_n}{\xi'} \right)^t \quad (4.5)$$

where ΔV , V_{mc} , and ξ' are joint closure, limited joint closure, and normal joint seating effective stress respectively; σ'_n is the normal effective stress acting on the joint; and A and t are joint deformational constants. Equation 4.5 can be reformulated in terms of the joint normal stiffness as given by Equation 4.6:

$$k_n = \frac{\sigma_n}{\Delta V} = \frac{\sigma_n}{V_{mc}} \left(1 + \frac{1}{A} \left(\frac{\xi'}{\sigma'_n} \right)^t \right). \quad (4.6)$$

Assuming that the joint sets are regularly spaced, the rock-mass moduli can be determined from Equation 4.7 for an equivalent cross-anisotropic continuum, provided that the joint normal and shear stiffnesses and joint set spacings are known.

$$E_1 = \frac{E}{1 + \frac{E}{k_n S_v}} \quad E_2 = \frac{E}{1 + \frac{E}{k_n S_h}} \quad G_2 = \frac{G}{1 + \frac{G}{k_s S_h}} \quad (4.7)$$

where $E_1 = E_{xx} = E_{yy}$, k_n and k_s are the normal and shear stiffnesses of the joints, S_v and S_h are the spacing of the vertical and horizontal joint sets, $E_2 = E_{zz}$, $G_2 = G_{zx} = G_{yz}$, and E and G are the moduli of the intact material. Thus, from Equation 4.6, the equivalent rock-mass modulus can be determined as a function of normal applied stress and joint set spacing using Equation 4.7. The rock-mass modulus E_{zz} is a nonlinear function of stress, which for practical purposes can be approximated by a bilinear relationship as shown in Figure 4.1. Data on the variation of shear stiffness with normal stress has been collected by Barton (1971) for both in situ and laboratory specimens. From this data the shear stiffness may be assumed to be given by Equation 4.8:

$$k_s = \frac{\tau}{\Delta S} = \frac{\sigma_n'}{V_{sc}} \left[1 + \frac{1}{A} \left(\frac{\xi'}{\sigma_n'} \right)^t \right] \quad (4.8)$$

where τ is the applied shear stress, ΔS is the shear displacement, and V_{sc} is the shear counterpart of V_{mc} . This implies that the shear stiffness is independent of shear stress and dependent only on the applied normal stress. In the model presented, it was assumed that the shear modulus obeys a bilinear relationship similar to that for the normal deformation modulus.

The average fracture frequency observed in the two heater tests is approximately 14 joints/m. Using Equations 4.6 and 4.7 with $A = 0.45$, $t = 0.5$, and $\xi' = 0.022$ (Hardy et al., 1979) then E_1 and E_2 are approximately 0.5×10^4 and 3.0×10^4 MPa for normal stress levels less than 5 MPa and greater than 5 MPa respectively. The shear modulus G_2 can be determined in a similar fashion.

4.1.2.3 Goodman Jack. Goodman Jack tests have been conducted in boreholes around the full-scale heater tests (Shuri et al., 1980). These results showed a nonlinear deformational behavior of the rock mass over a range of jacking pressures. The interpretation of the Goodman Jack results is subject to a number of errors because of the variable pressure distribution at the platen/rock interface. The diametric deformation across the platens is highly dependent on the pressure distribution at the platen/rock interface. A number of corrections (Heuzé and Salem, 1976), (Hustrulid, 1976, 1979) have been proposed for the Goodman Jack;

however, their validity has not been established. The corrections for Hustrulid require an estimate of the controlling parameter, the contact area between the platens and the rock. The results are useful, nevertheless, since they qualitatively yield behavioral trends and provide relative quantitative comparisons between results. It has been observed that for stiff hard rocks the Goodman Jack method consistently underestimates the in situ rock-mass modulus. The results from the Goodman Jack tests yielded an in situ tangent modulus ranging from 0.21 to 0.827×10^4 MPa and 0.7 to 1.86×10^4 MPa at applied stresses less and greater than 21 MPa, respectively.

The close agreement between the estimated in situ deformational modulus for the two rock-mass classification schemes and from the fracture-frequency logs, supports adopting these techniques to determine the value of rock-mass modulus. The disparity in in situ modulus between the Goodman Jack method and the other techniques is most probably due to the fact that the Goodman Jack consistently underestimates the stiffnesses of stiff hard rocks. Therefore, the values of moduli used in the predictive analyses were determined from the rock-mass quality (RMR and Q) and the fracture-frequency logs.

In generalizing the deformational relationships to three dimensions, it was assumed that the rock mass behaves as a cross-anisotropic medium. Since the applied stress field may not be homogeneous, it was necessary to substitute the mean applied effective stress for σ'_n when determining E_1 and ν_1 ; thus avoiding the generation of further anisotropic terms. It is imperative to strictly adhere to the physical bounds on these constitutive parameters when manipulating equivalent continuum parameters that represent a discontinuous medium. The necessary and sufficient conditions that the quadratic form of the strain-energy matrix be positive is given by Pickering (1970). The allowable ranges of the parameters are:

$$\begin{aligned} E_1, E_2 \text{ and } G_2 &> 0 \\ 1 > \nu_1 &> -1 \\ n(1 - \nu_1) - 2\nu_2^2 &> 0. \end{aligned} \quad (4.9)$$

The rapid change of the ratio $n = E_1/E_2$ due to the bilinear behavior could violate these conditions if not checked. The physical allowable ranges of these moduli and Poisson's ratios, as given by Equation 4.9, are illustrated in Figure 4.4.

For isotropic materials it has been observed that the effective Poisson's ratio of a body containing very flat cracks is less than its intact value when the cracks are open and greater than its intact value when closed (Jaeger and Cook, 1977). Thus, flat open cracks have a similar effect on Poisson's ratio of a body and its deformational modulus; while in the case of a body containing closed cracks, the effective bulk

modulus must be equal to its intact value. The Poisson's ratios, ν_{xy} and $\nu_{zx} = \nu_{yz}$, for the anisotropic rock mass were determined from the above observed behavior and the limiting conditions given in Equation 4.9.

RCP8102-164

FIGURE 4.4. Physical Limits on Anisotropic Material Parameters (Pickering, 1970).

From the relationships presented, a bilinear stress/strain model was developed for the fractured rock. The stress/strain relationship is given by $\underline{\sigma} = \underline{D} \underline{\epsilon}$ (Zienkiewicz, 1977) where $\underline{\sigma}$ and $\underline{\epsilon}$ are the stress and strain vectors and $\underline{D}$ is the elasticity matrix as defined in Equation 4.10.

$$\underline{D} = \frac{E_2}{(1 + \nu_1)(1 - \nu_1 - 2n\nu_2^2)} \begin{bmatrix} 1 - \nu_1^2 & n\nu_2(1 + \nu_1) & n\nu_2(1 + \nu_1) & 0 \\ & n(1 - n\nu_2^2) & (\nu_1 + n\nu_2^2) & 0 \\ \text{Symmetric} & & n(1 - n\nu_2^2) & 0 \\ & & & m(1 + \nu_1)(1 - \nu_1 - 2n\nu_2^2) \end{bmatrix} \quad (4.10)$$

where $m = \frac{G_2}{E_2}$ and E_1 , ν_1 , E_2 , ν_2 , G_2 , and n are as defined previously.

The vertical Young's modulus and Poisson's ratio E_2 and ν_2 , were assumed to be a function of σ_z as given by Equation 4.11.

$$\begin{aligned} E_2 &= E'_2, \nu_2 = \nu'_2 \text{ for } \sigma_z \leq \sigma_{0z} \\ E_2 &= E_2, \nu_2 = \nu_2 \text{ for } \sigma_z > \sigma_{0z} \end{aligned} \quad (4.11)$$

where E'_2 , E_2 and ν'_2 , ν_2 are the initial and final values of Young's modulus and Poisson's ratio, and σ_{0z} is the stress limit where they change their values.

The horizontal Young's modulus and Poisson's ratio, E_1 and ν_1 , were assumed to be a function of σ_r and σ_θ as given by Equation 4.12. For the present analysis this function was assumed to be the mean value $1/2(\sigma_r + \sigma_\theta)$ to avoid generating further anisotropic deformational terms due to the inhomogeneous nature of the applied stresses.

$$\begin{aligned} E_1 &= E'_1, \nu_1 = \nu'_1 \text{ for } \frac{1}{2} (\sigma_r + \sigma_\theta) \leq \sigma_{or\theta} \\ E_1 &= E_1, \nu_1 = \nu_1 \text{ for } \frac{1}{2} (\sigma_r + \sigma_\theta) > \sigma_{or\theta} \end{aligned} \quad (4.12)$$

where E'_1 , E_1 and ν'_1 , ν_1 are the initial and final values of Young's modulus and Poisson's ratio, and $\sigma_{or\theta}$ is the stress limit.

Finally G_2 is assumed to be a function of σ_z similar to that given in Equation 4.11 for E_2 :

$$\begin{aligned} G_2 &= G'_2 \text{ for } \sigma_z \leq \sigma_{0G}, \\ G_2 &= G_2 \text{ for } \sigma_z > \sigma_{0G} \end{aligned} \quad (4.13)$$

where G'_2 and G_2 are the initial and final values of the shear modulus, and σ_{0G} is the stress limit.

The actual values of the cross-anisotropic terms for the deformational properties of the basalt rock mass are listed in Table 4.3. The various combinations of these parameters satisfy the criteria of a positive definite strain-energy matrix as given by Equation 4.9. The stress limit on changing from the low to high values and vice versa was set at 3.0 MPa. This stress level was determined from the rock-mass stiffness versus stress relationship given by Equations 4.6, 4.7, and 4.8. Fitting a bilinear curve for this relationship resulted in the stress level for moduli changes to be 3.0 MPa. The vertical Young's modulus, E_2 , was selected slightly higher than horizontal value, E_1 , because the fracture frequency of the horizontal joint sets was in

general slightly less than the vertical ones. For FS#1 a histogram of number of fractures versus dip for 178 fractures taken from one horizontal and one vertical borehole (holes 1M3 and 1E20) results in 78 fractures with dip between 0 and 45 degrees and 120 fractures between 45 and 90 degrees. For FS#2 (holes 2M12 and 2E23) the corresponding numbers were 74 and 124 (Moak and Wintczak, 1980).

TABLE 4.3. Cross-Anisotropic Deformational Properties.

Property	Value	
	Low	High
E_1 ($\times 10^4$ MPa)	0.6	3.0
E_2 ($\times 10^4$ MPa)	0.8	4.0
G_2 ($\times 10^4$ MPa)	0.2	1.0
ν_1	0.02	0.3
ν_2	0.03	0.25

4.2 GEOMETRIC IDEALIZATION

In the nonlinear analysis used for the predictive data, it is assumed that the rock response is axisymmetric about the central heater. The validity of this assumption depends on the following:

- The response of the rock to symmetric loads
- The degree of symmetry of the loads, and the magnitude of the effect of any nonsymmetric loads
- The degree of symmetry of the boundary conditions and the magnitude of the effect of any nonsymmetric boundaries.

The characteristic response of the basalt at the NSTF heater test location was discussed in the previous section where it was deduced that the rock may be modeled as a homogeneous but cross-anisotropic medium. The plane of anisotropy is such that the central heater can be considered as an axis of symmetry for loads symmetric about this axis.

The degree of symmetry of the thermal loads and boundary conditions and the magnitude of the effect of the nonsymmetric conditions on the results of the tests are detailed here.

4.2.1 Thermal Loads

The thermal loads on the rock are predominantly provided by the heaters and, to a lesser degree, by the air in the heater and extensometer drifts. In FS#1, the eight peripheral heaters destroy this symmetry, especially when they are not symmetrically energized.

Full-Scale Heater Test #1 was analyzed three-dimensionally by the finite-element method to determine the errors involved in modeling the problem axisymmetrically. The finite-element mesh used to model this test consisted of a wedge slice as illustrated in Figure 4.5. This model takes into account the lines of symmetry posed by the problem for a near-field analysis. The location of the portions of the central and peripheral heaters in this mesh are also shown in Figure 4.5. The problem was analyzed for an elapsed time up to one year.

The temperature contours on a vertical section passing through a peripheral heater and a vertical section passing between two neighboring peripheral heaters are illustrated in Figure 4.6a and 4.6b, respectively, for an elapsed time of six months. The temperature contours plotted on a vertical circumferential section are shown in Figure 4.6c for an elapsed time of nine months. This circumferential section is between the centers of two neighboring peripheral heaters. This figure illustrates that, except for a small localized region in the vicinity of the 200°C contour, the problem at this time possesses approximate axisymmetry.

At 420 days four of the peripheral heaters are deactivated, which leads to an asymmetric loading condition. The effect of this loading on the temperature field is shown in Figure 4.7. It is seen that maximum temperature variation between orthogonal directions is approximately 25°C at 630 days. At other times and with other radii, the variation is not significant.

The heat flux into the tunnels will depend on the air velocity, the surface heat-transfer coefficient, the water vapor present, and any groundwater inflow. At the room boundaries it is assumed that heat is transferred away according to the formula given in Equation 4.14:

$$f_L = \omega(T - T_0) \quad (4.14)$$

where ω is the heat-transfer coefficient, T_0 and T are the air and rock temperatures, respectively, and f_L is the thermal-flux density.

RCP8102-158

FIGURE 4.5. Three-Dimensional Mesh Used for Analysis of FS#1.

(a)

(b)

(c)

RCP8102-159

FIGURE 4.6. Vertical Sections Passing (a) Through a Peripheral Heater and (b) Between Two Neighboring Heaters; (c) Temperature Contours on a Vertical Circumferential Section ($^{\circ}\text{C}$).

FIGURE 4.7. Temperatures at FS#1 Mid-Height for Asymmetric Thermal Loading.

As stated in Section 4.1 and reported in Hocking and St. John (1979), the temperature field in the vicinity of the instrumentation is insensitive to the heat-transfer coefficient. Further evidence was provided by running a three-dimensional linear model using the program FINEL (Hitchings, 1978). These results indicated that the difference in temperature at 500 days at a point 6 m from the heater along the axis of the tunnel and at a point 6 m from the heater at floor height in the wall of the tunnel, was approximately 1°C (Figure 4.8). This small difference indicates that the error due to the heat-transfer boundary in an axisymmetric analysis will be small.

4.2.2 Physical Boundaries

To determine the effect of the extensometer room and far-field boundaries, finite element calculations were performed for both geometries; a single heater tunnel, and a heater tunnel with adjacent extensometer tunnel. The meshes are illustrated in Figures 4.9a and 4.9b, respectively. The thermal loading consisted of a line of heaters in the floor of the test room. The thermal power was selected at 1 kW/m along the tunnel length. The tunnels were ventilated with a constant air temperature of 25°C and a rock surface heat-transfer coefficient of $5 \text{ W/m}^2\cdot\text{K}$. The temperature contours in the rock for an elapsed time of 3.0 yr for the two cases: (a) a single tunnel and (b) two tunnels, are illustrated in Figure 4.10. In this figure, it is impossible to distinguish between the two cases of modeling a single and a double-tunnel geometry. In the case of the thermomechanical interaction, it was also impossible to differentiate between the two cases in respect to the stresses induced in the rock around the test room (Hocking and St. John, 1979). However, for assessing the deformations at extensometer collars of the horizontal boreholes, it was concluded that the extensometer tunnel would need to be modeled. A three-dimensional thermomechanical analysis was performed on FS#2 as illustrated in Figure 4.11. The radial displacements at the room wall reached a maximum of 2.09 mm after approximately 730 days. This is significantly larger than that illustrated in Figure 4.12 for a similar axisymmetric model. The difference in these displacements indicates that the assumption of axisymmetry may not be good near the extensometer room and that a fully three-dimensional model may be necessary to give good predictions in this region.

4.3 NUMERICAL FORMULATION

The thermal stress analysis was conducted using a finite-element program called DAMSWEL, which consists of a transient nonlinear heat-conduction module (Williams et al., 1979) (Boonlualohr, et al., 1980b) and a nonlinear stress module (Boonlualohr, et al., 1980a). The numerical formulation of the transient thermal analysis has been described in detail in the cited references and will not be described here. The thermal analysis modulus can analyze highly nonlinear material properties dependent on temperature. Both the rock thermal conductivity and enthalpy were temperature dependent in these analyses. The thermal

RCP8102-145

FIGURE 4.8. Part of the Three-Dimensional Mesh Used by FINEL.

(a)

(b)

RCP8102-161

FIGURE 4.9. (a) Two-Dimensional Mesh with Insert Element and
(b) Alternate Insert for Analysis of Extensometer Room Effects.

(a)

(b)

RP8102-162

FIGURE 4.10. Temperature Contours (a) Without and (b) With Second Tunnel.

RCP8102-151

FIGURE 4.11. Part of a Three-Dimensional Mesh of FS#2.

FIGURE 4.12. Displacement Obtained from an Axisymmetric Model at Extensometer Room Wall (FS#2).

stress formulation, which involves modeling the rock mass as a cross-anisotropic medium with a bilinear locking stress/strain relationship, is discussed in depth in this section. The nonlinear stress analysis employs the incremental tangent stiffness method to solve the nonlinear problem by piecewise linearization. To minimize the drift from the true solution and to utilize larger load increments, a residual force correction technique is used.

The governing equation for a general nonlinear analysis may be written in terms of linear and nonlinear contributions (Boonlualohr, 1977) as given in Equation 4.15:

$$(\underline{K} + \underline{K}^*) \Delta u = \Delta P \quad (4.15)$$

where Δu are the unknown incremental displacements, and ΔP are the incremental loads and where $\underline{K}$ and $\underline{K}^*$ are the linear and nonlinear contribution stiffness matrices, respectively. While $\underline{K}$ remains unaltered for all load increments, $\underline{K}^*$ varies as a function of current displacement $u + \Delta u$. Thus, Δu cannot be directly solved since $\underline{K}^*$ also contains the unknown Δu . There are basically two approaches to solve for Δu , the incremental and the iterative.

4.3.1 Incremental Tangent Stiffness Approach

In order to solve for Δu , $\underline{K}^*$ is calculated from the displacement u of the previous load increment, rather than the current displacement $u + \Delta u$, in which Δu is unknown. This iteration is known as the incremental tangent stiffness method. It is obvious that the increment has to be kept small to minimize the error due to the linear approximation of $\underline{K}^*$. This error will result in a cumulative drift away from the true load-displacement curve as shown diagrammatically in Figure 4.13.

4.3.2 Iterative Approach

For a larger load increment, this cumulative error cannot be ignored. The nonlinear contribution $\underline{K}^*$ in this case can be determined by the process of iteration. There are two basic iterative techniques which are commonly used, the Newton-Raphson and the Modified Newton-Raphson.

4.3.2.1 Newton-Raphson Iteration. In this method, $\underline{K}^*$ is first determined using the known value of u from the previous load increment. The displacement increment Δu is then calculated. The unbalanced or residual force (Zienkiewicz, 1977; Hofmeister et al., 1971) due to the error in the linear approximation of $\underline{K}^*$ can now be calculated from the governing constitutive relationships. The corrective cycle is then made by applying the residual force together with the updated $\underline{K}^*$ calculated from the current value of $u + \Delta u$. The process is repeated until convergence is achieved.

RCP8102-148

FIGURE 4.13. Deviation from Correct Solution Due to Large Incremental Loads for (a) Hardening Material and (b) Locking Material.

4.3.2.2 Modified Newton-Raphson Iteration. Although the convergence rate of the Newton-Raphson technique is generally fast, the computational effort required to solve the problem at each iteration becomes a significant drawback. To save computational time, K^* can be held unchanged for several iterations and be updated only when the rate of convergence begins to deteriorate. This procedure is known as the modified Newton-Raphson iteration. The well-known initial stress method (Zienkiewicz et al., 1969) in fact used a modified Newton-Raphson iteration technique with K^* held at zero for all iterations. Figure 4.14 illustrates two examples of the convergence process of the initial stress method for a locking material. The initial stress method works well with perfectly plastic and hardening materials (Zienkiewicz et al., 1969; Valliappan et al., 1976); however, the method is subject to possible nonconvergence for locking materials as illustrated in Figure 4.14.

4.3.2.3 Solution Technique for Locking Material. In view of the possible unsuitability of the initial stress method for locking material, the incremental tangent stiffness method was preferred in this analysis. To minimize the drift of the load deflection curve inherent in this method, a residual force correction is applied along with the new load increment and solved using an updated K . For any element undergoing a change in properties during a load increment, the stresses are calculated using the previous and modified elasticity matrix $\underline{D}$. The modified elasticity matrix $\underline{D}$ is determined at the exact stress level when any material properties are modified. This process is repeated until no further modification of $\underline{D}$ is required.

The algorithm for the present analysis can be summarized for each load increment as follows:

1. Upon solution of a load increment, the stress difference due to material nonlinearity can be calculated from:

$$\Delta \underline{\sigma} = \underline{D} \Delta \underline{\epsilon} - \underline{D}^* \Delta \underline{\epsilon}$$

where $\underline{D}^*$ is the modified elasticity matrix. The total stress is then updated:

$$\underline{\sigma}_n = \underline{\sigma}_0 - \Delta \underline{\sigma}$$

where $\underline{\sigma}_0$ is the current stress at the end of load increment and $\underline{\sigma}_n$ is the updated stress.

RCP8102-149

FIGURE 4.14. Solution Convergence of the Initial Stress Method for a Locking Material for (a) A Thick-Walled Cylinder and (b) A Plane Strain Compression Block.

2. The stress difference $\Delta\sigma$ must be transferred by the residual force $\underline{R}$

$$\underline{R} = \int \underline{B}^T \Delta\sigma \, dV$$

where $\underline{B}$ is the usual strain displacement matrix. The residual force $\underline{R}$ is now applied together with the next load increment ΔP :

$$\left(\int \underline{B}^T \underline{D}^* \underline{B} \, dV \right) \Delta \underline{u} = \Delta \underline{P} + \underline{R}.$$

3. The equilibrium condition between the internal and external forces must be ensured before proceeding to the next load increment. This can be done by calculating the out-of-balance force given by:

$$\underline{\psi} = \underline{P} - \int \underline{B}^T \sigma \, dV$$

where $\underline{\psi}$ is the out-of-balance force vector and $\underline{P}$ is the total applied loads together with the reactions at the prescribed boundaries. σ is the total stress vector at the end of current load increment. This corrective amount of unbalanced force is then applied to the structure without modifying the stiffness matrix. This step is necessary to bring the stress solution to equilibrium before a further load increment can be applied.

4.3.3 Test Examples

The method of analysis as described above has been applied to a number of test problems. The problem of a thick-walled cylinder with pressures p_i , p_z , and p_o applied at the inner, top, and outer surfaces is illustrated in Figure 4.14a. The ratio of $P_i:P_z:P_o$ was set at 1:1:1 for all loading increments. The ratio of initial Young's modulus and its subsequent value was assumed to be 1 to 100 with a Poisson's ratio of 0.25. The mesh consists of three eight-node isoparametric elements. The ratio of P/P_C , where P is the applied force in the vertical direction and P_C is the force at which Young's modulus changes its initial value, is plotted against the displacement at the inner radius (see Figure 4.14a). The computed results with and without residual corrections are illustrated in Figure 4.14a along with the exact solution. It can be seen that improved accuracy is achieved with the residual correction. Similar plots are also compared in Figure 4.14b for the plane strain compression block with a Young's modulus ratio of 1 to 100 and a Poisson's ratio of 0. These results are also in good agreement with the exact calculation.

The problem of an axisymmetric solid cylinder with an applied temperature of 100°C along the centerline is shown in Figure 4.15. Figure 4.15a illustrates the transient temperature distribution along the radial distance for various times. The material deformational properties assumed for this problem are given in Table 4.3. The thermal conductivity, specific heat, and thermal expansion were 1.0 W/m·K, 1×10^{-2} J/m³·K, and 6.5×10^{-6} /K, respectively. The distributions of σ_z along the radial distance are plotted in Figures 4.15b and 4.15c for elapsed times of 1.0 and 2.0 seconds, respectively. It can be observed that distributions of σ_z for the locking material analyses lie between the two bounds of the linear stress analyses using the higher and lower values of E and ν . The residual correction method results in a smoother distribution of σ_z than those determined without the residual correction.

4.4 CHECKS ON ACCURACY OF MODEL

The computer codes DAMSWEL (Boonlualohr et al., 1980b) and FINEL (Hitchings, 1978) have been verified against many known analytical solutions and a high degree of confidence can be placed in the code logic. However, even with such checks, errors still might occur in the idealization or during the input of parameters. To guard against such errors, the heater tests were modeled axisymmetrically using two different programs (DAMSWEL and FINEL) and in three dimensions using FINEL and SATAN (an in-house Dames and Moore program). These check runs used linear material properties and the same thermal loading as for FS#2. The three-dimensional mesh used to model FS#2 in both FINEL and SATAN is shown in Figure 4.16 and consisted of 92 twenty-node brick elements with 658 nodes. The heater mid-height is at -4.21 m. The thermal program was run for 1,000 days with a ten-day time step. Displacements and stresses were calculated at 200-day intervals. Figure 4.17 shows a comparison of the temperatures at a depth of -3.60 m predicted by FINEL and SATAN and excellent agreement is seen.

A single heater similar to FS#2 was modeled axisymmetrically using DAMSWEL with a similar mesh as is used in generating the predictive data set, but with the heater mid-plane at a depth of -5.00 m. Figure 4.18 shows the temperatures at the same distance radially from the heater but at a depth of -4.16 m. As expected, the temperatures are a little higher than in the three-dimensional models because the point is nearer the heater mid-plane, but the correspondence between the two-dimensional and the three-dimensional models is good.

(a)

(b)

(c)

RCP8102-150

FIGURE 4.15. Distribution of Parameters Along Radius of Cylinder for (a) Temperature, (b) Stress at 1.0 Second, and (c) Stress at 2.0 Seconds.

RCP8102-146

FIGURE 4.16. Three-Dimensional Mesh for FS#2.

RCP8102-152

FIGURE 4.17. Comparison of Temperatures for FS#2 Predicted by FINAL and SATAN.

RCP8102-153

FIGURE 4.18. Temperatures Predicted in FS#2 Using Two-Dimensional Axisymmetric Model DAMSWEL.

Full-Scale Heater Test #2 was also modeled axisymmetrically using both FINEL and DAMSWEL with linear material properties. The FINEL mesh was much coarser than that used for DAMSWEL and consisted of only 39 elements and 140 nodes, compared with 111 elements and 374 nodes for DAMSWEL. The heater mid-plane in both models was -4.21 m. Figure 4.19 shows a comparison of the temperatures at thermocouple 2T02T02 and good agreement is found between the models. This indicates that the difference in the meshes makes little difference to the results and that no refinement of the DAMSWEL mesh is necessary. Comparison of the displacements produced by the two models is shown at 200-day intervals in Figure 4.20 and again good agreement is seen. The close correspondence between the results of FINEL and DAMSWEL gives confidence in the validity of the codes, in the input data, and in the idealization, especially since the results were produced by two independent methods and by personnel working independently.

5.0 PREDICTIVE RESULTS

5.1 FULL-SCALE HEATER TEST #1

Full-Scale Heater Test #1 was modeled using the nonlinear transient thermal stress program DAMSWEL. The mesh and initial boundary conditions for the thermal calculations are illustrated in Figure 5.1. For the calculation of stress the same finite element mesh was used, but the elements representing the main heater and air gap were removed to give a free surface at the borehole wall. This resulted in a mesh of 96 elements and 329 nodes.

The in situ stress state was assumed to be geostatic. This stress state is modified by the presence of the tunnel and the first step of the solution process is to find the equilibrium stress distribution when the boundaries of the model are subjected to the geostatic stress. The resulting stress field is also shown in Figure 5.2. A more complicated analysis of the effect of the tunnel on the stress state is unwarranted because of the assumption of axisymmetry and the uncertainty of the real in situ stresses.

The transient thermal calculation was run for 1,000 days with a time step of two days, and stress calculations were executed every 10 days. In analyzing the results of these models the following should be considered:

- The borehole wall is a free surface, and normal stress components must be zero there.
- Peripheral heaters are situated at a radius of 0.9 m in FS#1 but the boreholes are not modeled.

RCP8102-154

FIGURE 4.19. Comparison of Two-Dimensional Results for FS#2 Predicted by DAMSWEL and FINEL.

RCP8102-155

FIGURE 4.20. Comparison of Displacements for FS#2 Using (a) FINEL and (b) DAMSWEL.

FIGURE 5.1. Mesh and Thermal Boundary Conditions for FS#1 (x/y Scale Changed for Clarity).

FIGURE 5.2. (a) Initial In Situ Stress and (b) Stress Boundary Conditions for FS#1.

- The heat-loading path of each heater will be reflected in the response of the rock. Sudden increases or decreases in stress will occur when heater power levels are changed.
- A temperature change at the heater results in a thermal wave which travels at a finite velocity through the basalt. Points distant from the heater respond at later times and with less amplitude than points close to the heater.

Temperature contours around the borehole are shown in Figure 5.3 at 100, 300, and 500 days. The effect of the peripheral heaters at a radius of 0.9 m is clearly visible as a distortion of the smooth contours. Four of the peripheral heaters are deactivated at 420 days and the temperature near the borehole decreases as shown in Figure 5.4 for thermocouple 1T01T01. The heater power-level changes are seen as abrupt changes in slope of the curve. The changes in power levels are also reflected in the stress throughout the test area as shown in Figure 5.5 for IRAD gage 1C02C01. This shows that the maximum stress occurs at approximately the same time as the maximum temperature, i.e., 420 days. The stress and deformation patterns around the main borehole are shown in Figures 5.6 to 5.8 at 100, 300, and 500 days, respectively. The radial and hoop stresses at heater mid-height are plotted against radial distance for 100, 300, and 500 days in Figures 5.9 to 5.11. The hoop stress is largest at the borehole wall (approximately 46 MPa at 300 days). It rapidly decreases almost to the geostatic stress within 2 m of the borehole. The change in slope of the stress curves which occurs at 3.0 MPa is due to the bilinear stress/strain relationship which changes slope at this stress value. The radial stress is zero at the borehole wall but rises rapidly to its maximum value at a radius of approximately 0.5 m. The effect of the peripheral heater at 0.9 m and the bilinear stress/strain relationship is clearly visible.

5.2 FULL-SCALE HEATER TEST #2

The mesh used to model FS#2 is shown in Figure 5.12. The same in situ stress field was assumed as for FS#1. The time step for the thermal calculation was 2 days. Stress and displacements were calculated every 10 days. Figure 5.13 shows the temperature contours about the main heater at 100, 300, and 500 days. Figure 5.14 shows the time history of thermocouple 2T03T04 and the changes in slope can be directly related to the thermal loading of the main heater (see Table 3.1). Two maxima of temperature occur, one at approximately 510 days and the other at approximately 720 days. Figure 5.15 shows the stress at IRAD gage 2U09C01 and the maximum radial stress is reached at 510 days with a peak of lesser magnitude at 720 days.

FIGURE 5.3. Temperature Contours for FS#1 (a) 100 Days, (b) 300 Days, and (c) 500 Days.

NSTF TEST NO. 1

FIGURE 5.4. Temperature History for Thermocouple 1T01T01.

NSTF TEST NO. 1

FIGURE 5.5. Stress History at IRAD Gage 1C02C01.

DEFORMATION PLOT

X DEFORMATION ENLARGED BY 1500
Y DEFORMATION ENLARGED BY 350

0 1 cm

(a)

PRINCIPAL STRESS PLOT

STRESS SCALE 0 20.0 MPa

(b)

FIGURE 5.6. (a) Deformed Mesh and (b) Principal Stress at 100 Days for FS#1.

(a)

(b)

FIGURE 5.7. (a) Deformed Mesh and (b) Principal Stress at 300 Days for FS#1.

DEFORMATION PLOT

X DEFORMATION ENLARGED BY 300
Y DEFORMATION ENLARGED BY 100

0 1 cm

(a)

PRINCIPAL STRESS PLOT

STRESS SCALE 0 75.0 MPa

(b)

FIGURE 5.8. (a) Deformed Mesh and (b) Principal Stress at 500 Days for FS#1.

FIGURE 5.9. (a) Radial and (b) Hoop Stresses at 100 Days at Heater Mid-Height for FS#1.

FIGURE 5.10. (a) Radial and (b) Hoop Stresses at 300 Days at Heater Mid-Height for FS#1.

(a)

(b)

FIGURE 5.11. (a) Radial and (b) Hoop Stresses at 500 Days at Heater Mid-Height for FS#1.

FIGURE 5.12. Mesh and Thermal Boundary Conditions for FS#2 (x/y Scale Changed for Clarity).

FIGURE 5.13. Temperature Contours for FS#2 (a) 100 Days, (b) 300 Days, and (c) 500 Days.

NSTF TEST NO. 2

FIGURE 5.14. Temperature Prediction at Instrument 2T03T04.

NSTF TEST NO. 2

FIGURE 5.15. Stress Prediction at Instrument 2U09C01.

The displacements of the mesh and the principal stresses in a vertical plane through the heater are illustrated in Figures 5.16 to 5.18. Figures 5.19 to 5.21 show the radial and hoop stresses at heater mid-height (-4.21 m) at 100, 300, and 500 days respectively. The maximum hoop stress occurs at 500 days and has a value of 50.7 MPa at the borehole wall. The radial displacement with distance from the borehole is shown at 500 days at the heater mid-height in Figure 5.22. The maximum displacement of 1.13 mm occurs at approximately 2.0 m from the borehole. The radial displacement at the borehole is 0.15 mm at the heater mid-height.

5.3 PREDICTIONS OF INSTRUMENT RESPONSE

The output of the numerical model gives results for temperatures and displacements at the nodal points and for strains and stresses at the Gaussian points. The response of each instrument is obtained by interpolating the results from nodal or Gaussian points to the appropriate instrument location. The same shape functions as used in the finite element programs were used for this interpolation. Figures 5.23 to 5.27 illustrate the time histories produced for a thermocouple, a USBM gage, an extensometer, and two IRAD gage components. The abrupt changes in slope of the instrument response can be directly related to changes in the heater power levels (see Table 3.1). It is interesting to note the transient response predicted in the tangential stress component, as shown in Figure 5.27 at 510 days and 720 days, when the central heater is switched off. The sequence of events which causes this is as follows. The material near the heater cools first and contracts causing the radial stress to decrease. Elements of the rock farther from the heater, which were in equilibrium, must now move radially inwards to maintain equilibrium. Such a decrease in radius of the element causes the tangential stress to rise. Later the element itself cools causing the tangential stress to fall.

DEFORMATION PLOT

X DEFORMATION ENLARGED BY 1500
Y DEFORMATION ENLARGED BY 700

0 1 cm

(a)

PRINCIPAL STRESS PLOT

STRESS SCALE 0 75 0 MPa

(b)

FIGURE 5.16. (a) Deformed Mesh and (b) Principal Stress at 100 Days for FS#2.

FIGURE 5.17. (a) Deformed Mesh and (b) Principal Stress at 300 Days for FS#2.

DEFORMATION PLOT

X DEFORMATION ENLARGED BY 450
Y DEFORMATION ENLARGED BY 150

0 1 cm

(a)

PRINCIPAL STRESS PLOT

STRESS SCALE 0 12.5 MPa

(b)

FIGURE 5.18. (a) Deformed Mesh and (b) Principal Stress at 500 Days for FS#2.

FIGURE 5.19. (a) Radial and (b) Hoop Stresses at 100 Days at Heater Mid-Height for FS#2.

FIGURE 5.20. (a) Radial and (b) Hoop Stresses at 300 Days at Heater Mid-Height for FS#2.

FIGURE 5.21. (a) Radial and (b) Hoop Stresses at 500 Days at Heater Mid-Height for FS#2.

NSTF TEST NO. 2

FIGURE 5.22. Radial Displacement at Heater Mid-Height at 500 Days for FS#2.

NSTF TEST NO. 2

THERMOCOUPLE COMPONENT 5 I.D. E5 T5 AT CO-ORDS 1.75 150.00 -2.27

FIGURE 5.23. Temperature Prediction at Instrument 2E05T05 for FS#2.

NSTF TEST NO. 1

FIGURE 5.24. Displacement at Instrument 1U01U02 for FS#1.

NSTF TEST NO. 2

EXTENSOMETER COMPONENT . I.D. E21E. AT CO-ORDS 9.95 93.50 -4.41

FIGURE 5.25. Displacement at Extensometer Collar 2E21E00 for FS#2.

NSTF TEST NO. 1

FIGURE 5.26. Stress Prediction at Instrument 1C02C01 for FS#1.

NSTF TEST NO. 2

FIGURE 5.27. IRAD Gage Showing Transient Tangential Stress Behavior When Heater Power Levels are Changed, FS#2.

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APPENDIX

CONVENTIONS ADOPTED

Within the numerical computation for the prediction of the instrument measurements in NSTF FS#1 and FS#2, the following conventions and assumptions are used:

1. The predicted stress measurements for IRAD's are defined so that a compressive stress component is positive.
- 2. The predicted strain measurements for USBM's are defined so that a strain component giving an increase in the diameter of the borehole is positive.
3. The predicted displacement measurements of an extensometer anchor are given relative to its corresponding collar point. The sign convention adopted here is that a reduction in the length of the extensometer is defined to be positive (Figure A-1). The actual formula used for this calculation is:

$$U_{\text{ext}} = L_f - L_i$$

where L_i is the initial length of the extensometer,
 L_f is the final length of the extensometer,
 and U_{ext} is the change in length of the extensometer.

Note that the initial positions of the anchor and collar points are defined respectively as $p^a = (x^a, y^a, z^a)$ and $p^c = (x^c, y^c, z^c)$. The displacements of the anchor and collar points are defined respectively as $\underline{u}^a = (u_x^a, u_y^a, u_z^a)$ and $\underline{u}^c = (u_x^c, u_y^c, u_z^c)$. Using the above, L_i and L_f can be written explicitly as:

$$L_i = [(x^p - x^a)^2 + (y^p - y^a)^2 + (z^p - z^a)^2]^{1/2}$$

$$L_f = [(x^p + u_x^p - x^a - u_x^a)^2 + (y^p + u_y^p - y^a - u_y^a)^2 + (z^p + u_z^p - z^a - u_z^a)^2]^{1/2}.$$

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FIGURE A-1. Typical Positions of the Anchor and Collar Points of an Extensometer.

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