AN ABSTRACT OF THE THESIS OF

DONALD J. DODDS for the MASTER OF SCIENCE (Name of student) (Degree)
in CIVIL ENGINEERING presented on May 24 1973

Title: A MANUAL FOR PLATE LOADING TESTS PERFORMED ON ROCK

Abstract approved: Redacted for privacy Dr. W. L. Schroeder

Plate loading tests in rock are deceptively simple tests to perform. Consequently, many varieties have been developed in the past 60 years. From each of the varieties various investigators have added their own developments and modifications, some good, and some not so good. The result has been a confusion of claims and counterclaims which has produced a situation concerning the plate loading test similar to the Aesop's fable of the blind man and the elephant.

The purpose of this paper is to provide a single reference which presents a suitable method for performing each of the various types of tests and reducing the data therefrom. The advantages and disadvantages of each test are discussed along with practical suggestions to assist the reader in selecting the type of test best suited for application to his problem.

All forms of the test dutifully generate numbers; and because rock
is not homogeneous, isotropic, elastic or continuous as assumed in
theory, the results produced from these numbers can add to the confusion.
Most of the confusion can be alleviated by understanding how each of the
various boundary conditions affects the results. While it is recognized
that actual experience with the tests is the best means of eliminating
this confusion, this paper discusses the major boundary conditions, how
each tends to influence the test and how the unwanted results of the
boundary conditions are best separated from the useful data.

The useful product of a plate loading test is generally the modulus
of deformation of the tested material, but other parameters may be obtained.
The paper closes with a discussion of the other material properties
and in situ characteristics which can be produced by proper utilization of
test data.
A Manual for Plate Loading Tests
Performed on Rock

by

Donald J. Dodds

A THESIS
submitted to
Oregon State University

in partial fulfillment of
the requirements for the
degree of

Master of Science

June 1974
APPROVED:

Redacted for privacy

Associate Professor of Civil Engineering
in charge of major

Redacted for privacy

Head of Department of Civil Engineering

Redacted for privacy

Dean of Graduate School

Date thesis is presented: 24 May 1973

Typed by Judith Brecht for DONALD J. DODDS
ACKNOWLEDGMENTS

The generous aid and encouragement from instructors, family and friends have inspired the accomplishment of this thesis. The author is specifically indebted to Dr. W. L. Schroeder, Dr. J. Richard Bell and Dr. Keith F. Olds for their time, labor and counsel. Thanks is also extended to Mike Kelly and Ken Faught for their drafting and photography work and to Judi Gard for her help in typing and proofreading.
# TABLE OF CONTENTS

**INTRODUCTION** ........................................................................................................... 1  
History ......................................................................................................................... 1  
General ......................................................................................................................... 2

**PLATE LOAD METHODS FOR ROCK PROPERTIES** ....................................................... 4  
Rigid Plate ..................................................................................................................... 4  
  Equipment  
  Procedure  
  Testing  
  Data Reduction  
  Advantages  
  Disadvantages  
Flexible Plate Test ......................................................................................................... 14  
  Equipment  
  Procedure  
  Testing  
  Data Reduction  
  Advantages  
  Disadvantages  
Pressure Chamber ......................................................................................................... 26  
  Chamber Construction  
  Instrumentation  
  Testing  
  Data Reduction  
  Advantages  
  Disadvantages  
Radial Jacking ............................................................................................................... 34  
  Equipment  
  Procedure  
  Testing  
  Data Analysis  
  Advantages  
  Disadvantages  
Cable Jacking ............................................................................................................... 38  
  Equipment  
  Procedure  
  Testing  
  Data Reduction  
  Advantages  
  Disadvantages  
Borehole Jacking .......................................................................................................... 45  
  Goodman Jacking
TABLE OF CONTENTS (cont.)

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equipment</td>
<td>54</td>
</tr>
<tr>
<td>Procedure</td>
<td></td>
</tr>
<tr>
<td>Testing</td>
<td></td>
</tr>
<tr>
<td>Data Reduction</td>
<td></td>
</tr>
<tr>
<td>Advantages</td>
<td></td>
</tr>
<tr>
<td>Disadvantages</td>
<td></td>
</tr>
<tr>
<td>Summation</td>
<td>54</td>
</tr>
<tr>
<td>FACTORS BEARING ON INTERPRETATION OF RESULTS</td>
<td>56</td>
</tr>
<tr>
<td>General</td>
<td>56</td>
</tr>
<tr>
<td>Rock Fabric</td>
<td>56</td>
</tr>
<tr>
<td>Anisotropy</td>
<td></td>
</tr>
<tr>
<td>Heterogeneity</td>
<td></td>
</tr>
<tr>
<td>Continuity</td>
<td></td>
</tr>
<tr>
<td>Summation</td>
<td>60</td>
</tr>
<tr>
<td>Interface Conditions</td>
<td></td>
</tr>
<tr>
<td>Size of the Test Chamber</td>
<td>61</td>
</tr>
<tr>
<td>Pressure Effects</td>
<td>63</td>
</tr>
<tr>
<td>In Situ Stress</td>
<td>64</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>66</td>
</tr>
<tr>
<td>Strain Measurements</td>
<td>67</td>
</tr>
<tr>
<td>DATA UTILIZATION</td>
<td>71</td>
</tr>
<tr>
<td>Modulus of Deformation</td>
<td>71</td>
</tr>
<tr>
<td>Creep Coefficients</td>
<td>73</td>
</tr>
<tr>
<td>In Situ Strength Determinations</td>
<td>78</td>
</tr>
<tr>
<td>In Situ Stress Determinations</td>
<td>79</td>
</tr>
<tr>
<td>Stress Relief and Blast Damage</td>
<td>81</td>
</tr>
<tr>
<td>Elastic-Plastic Responses</td>
<td>87</td>
</tr>
<tr>
<td>Summation</td>
<td>91</td>
</tr>
<tr>
<td>CONCLUSIONS</td>
<td>94</td>
</tr>
</tbody>
</table>
LIST OF FIGURES

2.1 Typical vertical rigid plate loading test in progress.............. 5
2.2 Typical horizontal rigid plate loading test in progress.......... 5
2.3 Typical jack leg used in tunnels.................................. 8
2.4 Relationship between theoretical and hypothetical stress and strain under a rigid die.............................................. 8
2.5 Equipment layout for first generation jacking tests................ 15
2.6 Section through second generation jacking test setup.............. 17
2.7 Relationship between theoretical and hypothetical stress and strain under a flexible pad uniformly loaded..................... 20
2.8 Hydrostatic pressure chamber sections.............................. 28
2.9 View of hydrostatic pressure chamber during construction........ 28
2.10 Sketch of the radial jacking test.................................. 35
2.11 Photograph of the test setup....................................... 35
2.12 Effect of short length of loading of radial jack analyzed by superposition of radial deformation measured.................. 39
2.13 Cable jacking test.................................................. 41
2.14 Plan view of test layout............................................. 41
2.15 Goodman jack testing................................................ 47
3.1 Theoretical and typical interface conditions........................................... 62
3.2 Areas to be avoided during testing................................... 62
3.3 Stress at depth beneath square plate loaded with 400,000 pounds. 65
LIST OF FIGURES (cont.)

3.4 Comparison of moduli for different rock types. .................. 65
3.5 Deviator stress vs. Poisson's ratio. ................................. 69
3.6 Typical multiple borehole extensometer used for deflection measurement. .................................................. 69
4.1 Relationship between tangent, secant and recovery moduli .................. 74
4.2 Constant stress, creep curve. ......................................... 74
4.3 Typical curve showing stress history ................................ 80
4.4 Typical curve showing blast damage. .................................. 82
4.5 Typical curve showing deep layer of stress relieved rock. ........ 84
4.6 Typical curve showing blast damage. .................................. 85
4.7 Theoretical tangential stress distribution away from an opening in elastic material. .............................................. 86
4.8 Tangential stress away from opening showing stress relief process. ......................................................... 86
4.9 Inelastic deformation and in situ stress curve showing percent stress relief as function of inelastic deformation. .. 88
4.10 Typical curve showing increase in strain with decrease in load. ................................................................. 93
<table>
<thead>
<tr>
<th></th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Values of constant &quot;m&quot; in Equation 2.7, rectangular side ratio</td>
<td>20</td>
</tr>
<tr>
<td>2.2</td>
<td>Values of constants in Equation 2.18.</td>
<td>49</td>
</tr>
<tr>
<td>2.3</td>
<td>Comparison of test methods</td>
<td>49</td>
</tr>
<tr>
<td>4.1</td>
<td>Field moduli</td>
<td>75</td>
</tr>
<tr>
<td>4.2</td>
<td>Rock qualities exhibited in curves</td>
<td>90</td>
</tr>
</tbody>
</table>
NOTATIONS

a = constant
A = plate area
b = radius of plate rotation
C = constant
C_1 = constant
C_2 = constant
C_3 = constant
C_4 = constant
C_5 = constant
d = distance from measuring point to center of plate
D = distance from measuring point to center of chamber
E = modulus of deformation
Ef = modulus of deformation from flexible plate test
Er = modulus of deformation from rigid plate test
h = diameter of borehole
k = constant
K = constant
L = width of square plate
Ln = natural log
m = constant from Table 2.1
n = elliptical integral
p = internal pressure
P = total applied load
q = applied unit load
Q_h = hydraulic line pressure
r = radius of bearing plate
r_1 = internal radius of annular ring
r_2 = external radius of annular ring
R = pressure chamber radius
t = time
T = temperature
w = deflection
w' = corrected deflection
w_ave = average plate deflection
w_cor = deflection of plate corner
w_d = deflection at point "d" distance from center of plate
w_D = radial deformation of point "D"
w_e = deflection off plate edge
w_h = borehole diametrical deflection
w_max = maximum deflection
w_z = deflection at depth "z" under center of plate
z = depth to measuring point under center of plate
a = constant
\( \mu \) = Poisson's ratio
$\sigma$ = stress
$\sigma_{ave}$ = average stress
$\sigma_1$ = major principal stress
$\sigma_2$ = intermediate principal stress
$\sigma_3$ = minor principal stress
$\sigma'$ = deviator stress
$\sigma'_{ult}$ = ultimate deviator stress
INTRODUCTION

1.1 History

The origin of the plate loading test appears to be lost in the emergence of soil mechanics sometime around the turn of the Nineteenth Century. In the 1870's the concept of allowable soil pressure was developed and plate loading tests certainly followed closely afterward. The test was undoubtedly used as back-up for some of the allowable bearing value tables generated at that time. In the early 1900's, the test fell into disuse because:

1. the test is rather awkward and time consuming;
2. the results were difficult to extrapolate into actual design;
3. simpler and more accurate methods to obtain bearing values were developed.

A modification of the test survived in highway departments as a method to determine subgrade reaction (CBR, etc.) which, along with some large-scale pressure chamber tests, seemed to be its most useful application until the 1930's when the test was used to obtain the modulus of deformation on weak rock foundations for dams (Droughin, 1936 and Niederhoff, 1939). The Niederhoff paper is an excellent paper as it gives the test procedure and results obtained in great detail. The
test as described in the Niederhoff paper was essentially the same as
the soil test, ASTM D1194, except that the loads were larger and,
therefore, more awkward to apply. The next significant advance came
in 1943 when the test was performed in an exploratory adit in an arch
dam in France (Habib, 1943). It was here that the cumbersome reac-
tion load was eliminated by using a simple beam to transfer the load
to the opposite side of the adit. Further refinement waited until double-
curve, high arch dams forced the emergence of rock mechanics in the
early 1950's, and throughout the last 20 years the use of the test has
steadily increased.

1.2 General

Plate loading tests on rock are commonly performed for informa-
tion to aid the design of underground power houses, large underground
cavities or major tunnel projects as well as foundation design for large
concrete dams, for dams where weak rock is encountered in the founda-
tion, or in the foundation and abutments of the arch dams. The inform-
ination most commonly used is the modulus of deformation; however,
much additional useful data can be obtained and the test is rapidly be-
coming one of the primary descriptive tests in practical rock mechanics.
In general, the test is performed on an exposed rock surface by applying
a normal force to the surface and then measuring the resultant displace-
ments of the rock. In most instances, the applied stress and the rela-
ted displacements are analyzed by elastic theory to produce the desired
material properties of the rock. This load is generally applied by hydraulic jacks in tunnels, galleries, or boreholes where the reaction of the jack can be conveniently taken by the opposing wall.

The interpretation of the data obtained from plate loading tests is far from a simple matter. There are many reasons for this difficulty, some of which are covered in a later section of this paper. This apparent lack of consistency in test results arising from interpretation methods has led to many modifications of the test as various investigators sought the "perfect test" form. As a result there exists a myriad of test methods and variations. From these variations six basic types have evolved. They are:

1. Rigid plate
2. Flexible plate
3. Pressure chamber
4. Radial jacking
5. Cable jacking
6. Borehole jacking

In order to familiarize the reader with each of the various types and allow him to select the test best suited to his application, each of these forms will be discussed in detail in the succeeding section.
Fig. 2.1 Typical vertical rigid plate loading test in progress.

Fig. 2.2 Typical horizontal rigid plate loading test in progress.
2. Supporting cradle to hold beams during assembly for horizontal test.

3. Hydraulic ram, generally 100- to 400-ton capacity.

4. Pressure gauge and fittings.

5. Hydraulic pump.

6. Dial gauges sensitive to 0.0001 inch and necessary holding rods; the number of gauges varies, but six are most commonly used (three on each end).

7. Dial gauge holding brackets.

2.1.2 Procedure

When the location for the test has been chosen, all loose rock is cleaned from the test areas, and relatively smooth bearing surfaces are prepared using a chipping hammer, stopper, or a jack leg (Figure 2.3). If the chuck tender on the jack leg holds the drill rod just above the bit and forces the bit back and forth across the proposed bearing area while the jack leg is being operated at a low setting, a very fine bearing surface is produced in short order. After the two bearing surfaces are prepared, approximately parallel to each other, the test proceeds as follows:

1. The beam support cradle is set up and leveled;

2. Sections of beams, the ram and bearing plates are placed on the cradle and bolted together. The total
length of the assembled beam should allow for minimum clearance between bearing plates and rock bearing surfaces;

3. The assembly is again leveled;

4. The opening between the bearing plate and rock is then packed with a high strength grout, epoxy resin or sulfur. The sulfur works well on horizontal tests, but care should be taken that the sulfur does not ignite underground because of the SO$_2$ fumes generated and the generally poor ventilation;

5. The gauge holding post and brackets are installed, being sure they are founded well away from test influences;

6. The dial gauges are installed so that they are in contact only with the bearing plates.

After a sufficient time — normally 24 hours, but dependent on the particular grout used in the bearing pads — has passed to allow the bearing cap to attain a compressive strength in excess of the maximum test load, the rock test is begun.

2.1.3 Testing

Each test consists of several cycles of loading and unloading and recording the resulting rock or plate deflections. It is recommended that at least three cycles be performed with ten readings made during
FIG. 2.3 Typical jack leg used in tunnels.

FIG. 2.4 Relationship between theoretical and hypothetical stress and strain under a rigid die.
each cycle and that the load be held constant for 15 minutes at the maximum and minimum load to allow time-dependent deformations in the rock to develop. If deformation continues, the wait may be extended to see if the deformation stabilizes. In any case, the instability should be noted before proceeding. One complete test generally requires six to eight hours to set up and four hours to perform.

2.1.4 Data Reduction

The theoretical basis for the rigid plate test is the well known Boussinesq solution (Timoshenko, 1951) for the stresses caused by the indentation of a rigid die into a semi-infinite solid. The equations given are all for circular plates; if rectangular plates are used an additional correction factor must be added to the proper equation. Figure 2.4 shows the relationship between the theoretical and actual stress and deformation under a rigid die pressed into a homogeneous semi-infinite elastic solid and a rigid die pressed into a typical rock material.

Theoretically, as shown in Figure 2.4a, the stress becomes infinite at the edges of the plate. This would occur in only a truly elastic material. Most rock does not fall in that theoretical category, and in almost all instances local plastic failure takes place which distributes the load in a highly indeterminate manner, shown hypothetically in Figure 2.4b. This distribution is caused by cracks, joints, failure of highly stressed rock and the non-homogeneity of the rock and has been the
cause of much concern over how well the Boussinesq equation really fits the actual rigid plate test conditions. It will be shown later that this concern is at the present time insignificant when compared with other factors. The unequal distribution of the actual strain shown in Figure 2.4d is caused by non-axial loading of the plate and non-homogeneity of the rock under the plate. The stronger material tends to take more than its fair share of the load, but in many instances it still deforms less than the weaker material. Rather than use the actual complicated stress condition, or the theoretical stress and strain conditions, the average values, which are easily obtained in the field, are substituted into the Boussinesq equation and the following formula for the deformation modulus is obtained.

\[ E = \pi r \sigma_{ave} (1 - \mu^2)/2w_{ave} \]  

(Eq. 2.1)

where:

- \( r \) = radius of bearing plate
- \( \sigma_{ave} \) = average stress applied to plate
- \( \mu \) = Poisson's ratio
- \( w_{ave} \) = average plate deflection taken from three measurements, 120 degrees apart around the periphery of the plate.

An attempt to correct for the rotation, mentioned above and implied by Figure 2.4b was proposed by Kruse (1963) and is as shown in the following equation.
\[ E = \pi r \frac{\sigma_{\text{ave}} (1 - \mu)}{2w_{\text{ave}}} \times \frac{(2b - r)}{2(b - r)} \]  

(Eq. 2.2)

where:

\( b \) = the radius of plate rotation

The modification is not recommended for general use because the assumptions used for the derivations are contradictive; the correction itself is generally away from the side of safety, and presently the accuracy of the test does not warrant this type of precision. An immediate inconsistency is noted as the radius of plate rotation \( b \) approaches the radius of the plate causing the modulus of deformation as defined in Equation 2.2 to approach infinity. A reason for this may be the contradiction between the assumption that the material is homogeneous and the actual rotation caused by non-homogeneity.

In some instances measurements of surface deflection have been taken outside the loaded area. These measurements have the advantage of providing back up data at very little additional cost. If deflection is measured outside the loaded area of the plate, the modulus is obtained using the following relationship.

\[ E = \frac{2}{3} \frac{\sigma_{\text{ave}} (1 - \mu^2)}{\pi d w_d} \left( \frac{3r^2 \pi + 6r^2 - 6d^2 - 2}{w_d} \right) \]  

(Eq. 2.3) (Coates, 1965)

where:

\( d \) = the distance from the measuring point to the center of the plate

\( w_d \) = deflection at point "d"
It should be pointed out that the moduli obtained with Equation 2.3 are moduli which represent the response of the point under the measurement instrument and because the rock is not the ideal material assumed in theory, the overall material reaction may be quite different.

2.1.5 Advantages

The advantages of the rigid test are as follows:

2.1.5.1 Speed

The test can normally be set up in six to eight hours and the actual testing time is in the order of four hours.

2.1.5.2 Low Expense

The cost of the rigid plate test is minimal, approximately $600 per test, not including the test adit. The borehole variety of the loading test may be considered less expensive but only if the cost of the borehole is neglected as part of the test cost.

2.1.5.3 Simplicity of Operation

No complex electronic equipment is needed to perform testing. Average plate stress and average plate deflection are quantities easily obtained in the field because only static force and deflection need be measured.
2.1.5.4 Average Modulus

The rigid plate tends to average the effect of stress on all the material under the plate. The resulting average deformation produces a modulus representative of the total rock volume affected by the plate. This type of averaging process is essential if a mass modulus of the rock material is required. It is an implied assumption that the rock volume affected by the test is representative of the larger mass to be affected by actual works proposed.

2.1.6 Disadvantages

2.1.6.1 Small Loaded Area

The area of the rock tested is generally in the range of one to four square feet. In most instances when the modulus of a large area is needed, or the effect of widely spaced joints is required, this test is inadequate. Non-homogeneity can often be handled by performing enough tests to submit the results to a quasi-statistical analysis.

2.1.6.2 Low Stress Level

The handling of a ram large enough to stress the rock under the plate to greater than 4,000 psi is difficult because of the great weight and bulk required in a hydraulic ram of this size. This limitation on the total load results in modulus values for a stress range
of generally 25 percent or less of the rock's compressive strength.

2.1.6.3 Theoretical Boundary Conditions

The difference between the theoretical boundary conditions and the actual boundary conditions caused by the actual plate not being perfectly rigid, and the stress near the plate edges not being infinite are disadvantages only in homogeneous, isotropic, continuous, elastic rock. In actual rock, the error caused by these factors is small, as shown later, and is lost in the more serious error contributed by uncontrolled variables other than the rigidity of the plate.

2.2 Flexible Plate Test

The flexible plate test was developed in an attempt to avoid the theoretical difficulties mentioned above due to the rigidity of the plate. This refinement attempts to supply a uniform load to the rock surface by the insertion of a pad of flexible material such as rubber or a hydraulic cushion like a flat jack between the bearing pad and the rock, and thereby meet the boundary conditions of the Boussinesq equation. This test has evolved through two generations; the first generation test is shown in Figure 2.5, and the second generation equipment is shown in Figure 2.6. The second generation test will be discussed below.

2.2.1 Equipment

1. Test beam, consisting of a cluster of four separate
FIG. 2.5 Equipment layout for first generation jacking tests.
beams (see Figure 2.6), each beam has an adjustable screw leg.

2. Two circular flat jacks with center holes.

3. Two multiple position borehole extensometers.

4. One-tunnel-diameter extensometers.

5. Pressure gauge, hydraulic fittings, and pump.

6. Readout equipment for borehole extensometers.

2.2.2 **Procedure**

1. The bearing surfaces are prepared as previously described under the rigid plate test.

2. A hole is drilled in the center of each bearing surface.

3. The borehole extensometers are positioned at various depths depending on the diameter of the bearing plate. Anchors are generally placed as close as possible to the surface and spaced farther apart until the deepest anchor is set beyond the practical influence of the load, approximately ten plate diameters.

4. The grout pads are poured in a manner similar to the method previously described under the rigid plate test.

5. The flat jacks and the test beams are assembled on the pads.
FIG. 2.6 Section thru second generation jack test setup.
6. Tunnel-diameter extensometers are positioned between the two bearing pads.

2.2.3 Testing

1. The load is applied in four cycles at pressure increments of 250 psi. A cycle consists of loading and unloading in ten equal increments.

2. The maximum and zero load are held for 15 minutes to allow the rock time-dependent deformation to stabilize. If stability is not achieved in 15 minutes, wait an additional 15 minutes and if deformation is still occurring the test is continued. In all cases the instability should be noted.

3. Deformation measurements are taken at each 250 psi increment or decrement.

The test takes approximately three weeks to prepare and perform (Benson, 1970). The actual performing of the test can take as little as 4 hours with the remainder of the time used for preparation of the test.

2.2.4 Data Reduction

The data is reduced by using the Boussinesq solution for a uniformly distributed load on a semi-infinite elastic medium. The abundance of published formulae for reduction of the data obtained from
flexible plate tests is an indication of the overwhelming favoritism of this test with theorists. The relationship between the theoretical and the actual stress distribution and deflection is shown on Figure 2.7.

The non-homogeneous nature of the rock and weak areas like joints, cracks and faults cause variations from the theoretical. Neglecting the actual deflection curve for the moment, it is apparent from examining the theoretical deformation distribution that the deflection depends on where the measurement is taken on the plate. If the maximum plate deflection is used — that is, the deflection at the center of the plate — and if the plate is circular, the equation for the modulus of deformation is:

\[ E = 2(1 - \mu^2) q r / w_{\text{max}} \]  
(Eq. 2.4)  
(Timoshenko, 1951)

If the deflection is measured at the edges of the circular plate, the modulus is given by the following equation.

\[ E = 4(1 - \mu^2) q r / \pi w_e \]  
(Eq. 2.5)  
(Timoshenko, 1951)

The third useful relationship for plate deflection is the average plate deflection and if that is measured the equation becomes:

\[ E = 0.54 \pi q r (1 - \mu^2) / w_{\text{ave}} \]  
(Eq. 2.6)  
(Timoshenko, 1951)

It may be worthwhile to diverge here a moment and compare the equation given for the rigid plate test (Eq. 2.1) using average deflection, 

\[ E = \pi r \sigma_{\text{ave}} (1 - \mu^2) / 2w_{\text{ave}} \]  
(Eq. 2.1)  
(Timoshenko, 1951)
FIG. 2.7 Relationship between theoretical and hypothetical stress and strain under a flexible pad uniformly loaded.

Table 2.1

Values of Constant "m" in Equation 2.7

<table>
<thead>
<tr>
<th>Rectangular Side Ratio</th>
<th>1:1</th>
<th>1:15</th>
<th>1:2</th>
<th>1:3</th>
<th>1:5</th>
<th>1:10</th>
<th>1:100</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.95</td>
<td>0.94</td>
<td>0.92</td>
<td>0.88</td>
<td>0.82</td>
<td>0.71</td>
<td>0.37</td>
</tr>
</tbody>
</table>
with Equation 2.6 for the flexible plate test using average deflection:

\[ E = \frac{0.54\pi r q (1 - \mu^2)}{w_{ave}} \quad \text{(Eq. 2.6)} \]

(Timoshenko, 1951)

Dividing \( E_f \) by \( E_r \) and noting that \( q \) and \( q_{ave} \) are the average applied stresses and are, therefore, numerically equal, an eight percent difference in answers is obtained with the flexible plate producing higher results.

\[ \frac{E_f}{E_r} = \frac{0.54\pi r q (1 - \mu^2) 2w_{ave}}{\pi r q_{ave} (1 - \mu^2) w_{ave}} = 1.08 \]

This indicates that the upper boundary for error caused by the rigid plate, in fact not being rigid, is only eight percent and is on the side of safety.

For rectangular plates and average deflection, Equation 2.3 may be modified as follows (Timoshenko, 1951).

\[ E = m P \frac{(1 - \mu^2)}{w_{ave}\sqrt{A}} \quad \text{(Eq. 2.7)} \]

where:

- \( A \) = plate area
- \( P \) = total load, and
- \( m \) is taken from Table 2.1

If a square plate is used and the maximum deflection or the deflection at the center of the plate utilized, then the modulus equation is as follows.

\[ E = \frac{1.12 q L (1 - \mu^2)}{w_{max}} \quad \text{(Eq. 2.8)} \]

(Timoshenko, 1951)

If corner deflections are used, then the modulus is found by the follow-
ing relationship.

\[ E = 0.56 \frac{q L (1 - \mu^2)}{w_{\text{cor}}} \]  

(Eq. 2.9)  
(Timoshenko, 1951)

Many investigators have attempted to measure surface deflection of the rock outside of the loaded area because, theoretically, displacements are less sensitive to pressure distributions under the plate and are influenced to a greater extent by the rock at depth. In order to obtain the modulus of deformation using this deflection, the following equation is used,

\[ E = 4 (1 - \mu^2) \frac{q d n}{\pi w_d} \]  

(Eq. 2.10)  
(Timoshenko, 1951)

where:

\[ d \] = the distance from plate center line to the measuring point,

\[ w_d \] = the deflection measured at point "d", and

\[ n \] = an elliptical integral given by the following equation

\[ n = \int_{0}^{\pi/2} \frac{\sqrt{1 - r^2 \sin^2 e}}{d^2} \, de - \int_{0}^{\pi/2} \frac{d e}{\sqrt{1 - (r^2) \sin^2 e}} \]  

(Eq. 2.11)

However, in actual practice mixed results are obtained when individual point deflections are measured due to non-homogeneity, discontinuity and anisotropic characteristics of most rock. This tends to invalidate any advantages gained by this type of refinement.
One of the prime objections to plate loading tests in general is that the surface rock is damaged due to the tunneling process and that the modulus obtained from the tests is not the true modulus of the undisturbed rock. Theoretically, the modulus of the undisturbed rock can be obtained simply by measuring deflections at depth. Using these deflections at depth along the axis of the bearing plate, the modulus is obtained from Equation 2.12.

\[
E = \frac{P}{2\pi w_z} \left[ z^2 (1 + \mu) / (r^2 + z^2)^{3/2} + 2(1 - \mu^2) / (r^2 + z^2)^{1/2} \right] \quad \text{(Eq. 2.12)}
\]

(Timoshenko, 1951)

where:

\[
z = \text{the depth from the bearing plate to the measuring point}
\]

The disadvantages to these types of measurement are all basically practical and are as follows:

1. The stress level at depth is considerably lower than the already minimum acceptable stress level presently applied to the surface rock;

2. If the modulus is going to be used for set or lining design where the surface rock provides an important percentage of the overall reaction, possibly the modulus needed is the modulus of the surface rock;

3. The installation of the gauges at depth disturbs the
in situ conditions and may invalidate the results.

In an attempt to avoid the difficulties encountered in objection 3, listed above, the bearing plate has been changed in some instances to an annular ring. The equations used for this type of plate are as follows (Shannon and Wilson, Inc., 1964).

Surface deflection - center of annular ring:
\[
E = 2q(1 - \mu^2) (r_2 - r_1)/w_{\text{max}}
\]  
(Eq. 2.13)

Surface deflection - edge of the plate:
\[
E = 4q(1 - \mu^2) (r_2 - r_1)/\pi w_e
\]  
(Eq. 2.14)

where:
\[
\begin{align*}
r_1 &= \text{hole radius} \\
r_2 &= \text{plate radius}
\end{align*}
\]

Surface deflection outside of the loaded area is little affected by the presence of the hole in the plate and Equation 2.10 is used with errors of less than one percent (Shannon and Wilson, Inc., 1964).

The deflection at depth along the center line produces modulus values described by:
\[
E = \frac{q}{w_z} \left[ \frac{z^2 (1 + \mu) + 2(1 - \mu^2)}{(r_2^2 + z^2)^{\frac{3}{2}}} + \frac{z^2 (1 + \mu) - 2(1 - \mu^2)}{(r_1^2 + z^2)^{\frac{3}{2}}} \right]
\]  
(Eq. 2.15)

2.2.5 Advantages

The condition of uniform load under the plate is theoretically easy to obtain and most investigators consider that boundary conditions of the test fit Boussinesq's assumptions for a uniformly dis-
tributed load more closely than theoretical boundary conditions are met by the rigid plate test.

2.2.5.1 Increase in Plate Size

With the use of flat jacks to apply the load, the bearing plate diameter can be increased to two feet and more without as much increase in bulk of the test equipment as encountered when hydraulic rams are used to apply the load.

2.2.6 Disadvantages

2.2.6.1 Cost

The cost of the test is considerably more than rigid plate tests, in some instances as much as 15 times more expensive.

2.2.6.2 Complexity

The increase in complexity of the flexible plate test, especially in the method of deflection measurement, is all out of proportion with any increase in accuracy of the results obtained. For example, in one series of tests (Bureau of Reclamation, 1965) it was considered important to eliminate thermal effect on the measuring systems. The test area was partitioned off, kept dark except for flashlights and all personnel were kept outside of the test area except for a few minutes when reading the gauges was necessary.
2.2.6.3 Sensitivity

Point measurement of deflection produces a point modulus. Note, on Figure 2.7b illustrating a hypothetical deflection curve, that if the strain is measured at points "a" and "b" a much different modulus is obtained than if points "c" and "d" are selected for strain measurement. Because the selection of the measuring points is essentially arbitrary and because they are point measurements, this can lead to serious errors in data reduction, considerable scatter in the results, and may completely negate the effect of the larger plate diameter.

2.2.6.4 Safety

The flexibility and inherent low strength of the flat jacks require that they be confined. This confinement in many instances is not subjected to rigid quality control, and can fail. The result of such a failure causes debris, pieces of rock, bearing pad or flat jack, along with hot hydraulic oil under high pressure to be projected around the test cavity. All of these projectiles can cause serious injury or even death.

2.3 Pressure Chamber

Hydrostatic pressure chambers were developed very early in the history of plate loading tests and are used to measure the reaction of a rock to stress over large areas. These tests cover a much larger area
than any other test method used to determine material constants, and as such they provide results that measure the mass behavior of the material. In this type of test a portion of the test adit itself is sealed off and lined to provide the hydraulic chamber. This chamber is then pressurized and the deflection of the chamber and the surrounding rock is measured to obtain the necessary rock properties. Figures 2.8 and 2.9 show a typical layout and photograph of an actual test chamber (Dodds, 1965).

2.3.1 Chamber Construction

The test chamber is excavated and scaled very thoroughly to remove all loose and drummy (rock that does not ring when struck by a hammer) rock from the interior of the chamber cavity. The ribs, back and invert should be made as free of irregularities and loose material as physically possible. The chamber is then lined with reinforced concrete. In some instances unlined chambers have been used, but this should be avoided because the strain measurements in this type of a chamber provide only point deformation and not the desired mass deformation. Measuring the volume change in the pressurizing fluid could produce satisfactory bulk chamber deformation, but at the present time the loss of fluid through leaks, the relatively low applied pressures, and the accuracy of available instruments to measure volume all combine to preclude the use of this method.

The reinforced concrete should be placed to allow the chamber
FIG. 2.8 Hydrostatic pressure chamber sections

FIG. 2.9 View of hydrostatic pressure chamber during construction. (Fig.s from "Measurement & Analysis of Rock Physical Properties on the Dez Project, Iran").
to expand under pressure. After the concrete is placed, low pressure grouting is performed to fill any voids between the concrete lining and the rock.

The chamber is sealed on either end with concrete bulkheads through which a manhole, water pipes, air vents and various equipment conduits must pass. In some instances the chamber is constructed in the end of an adit which necessitates the use of only one bulkhead, but requires an additional length of chamber to reduce end effects.

The chamber is then lined with a watertight membrane to prevent fluid from entering the surrounding rock during the test and affecting the pore water pressure in the rock.

2.3.2 Instrumentation

The instrumentation of the chamber is such a small percentage of the overall cost of the test, and it is the portion which supplies the final data, that redundancy should be built in to reduce the possibility of invalid conclusions based on data from malfunctioning instruments. The instruments are of the type that can be read remotely such as extensometers utilizing LVDTs (linear variable displacement transducers), strain gauges and acoustic or vibration wire elements. Instruments should be placed in a minimum of three arrays of four instruments each. Each instrument array consists of a vertical, horizontal and two diagonal extensometers set perpendicular to the
longitudinal axis of the tunnel to measure diametrical changes of the chamber. In addition, borehole extensometers can be installed in the rock surrounding the chamber to measure response at depth. Care should be taken to see that the instrumentation does not puncture the waterproof membrane, or that the waterproofing does not interfere with the action of the extensometer. It is advisable to use invar steel wherever necessary and to control the temperature of the fluid to match the rock temperature, thus minimizing the thermal effects on the test results.

2.3.3 Testing

Each test consists of several cycles of loading and unloading and recording the resulting rock deflections. It is recommended that at least three cycles be performed with ten readings made during each cycle and the load held constant for 30 minutes at the maximum and minimum load to monitor any time-dependent properties of the rock. If deformation continues, the wait may be extended to see if deformation stabilizes; in any case, the instability should be noted before proceeding. One complete test generally requires six to eight hours to perform and two to three months to set up. Because of the long set-up and short testing time, the chamber is often used for sustained loading tests where the load on the rock and the resulting deformations are recorded for several weeks.
2.3.4 Data Reduction

Data from pressure chamber tests can be analyzed by using classical thick cylinder theory. In order to meet the boundary conditions imposed by the thick wall cylinder theory the loaded length of the test chamber must be large when compared with the diameter of the test chamber. This condition allows the end effects, caused by the finite length of the test chamber to be neglected in the calculations. Normally, the depth of the test chamber exceeds ten times its diameter which allows the assumption that the external radius of the thick walled cylinder is infinite. Under these conditions the equation for the modulus is as follows.

\[ E = \frac{(1 + \mu) p R^2}{D w_D} \]  

(Eq. 2.15)  
(Clark, 1965)

where:

- \( p \) = internal pressure
- \( R \) = radius of chamber
- \( D \) = distance from measuring point to center of chamber
- \( w_D \) = radial deformation

2.3.5 Advantages

2.3.5.1 Mass Modulus

The pressure chamber tests have the advantage
of affecting a larger rock mass than any other form of loading test. This large loaded area reduces the effect of localized weak spots like cracks, joints and soft material as well as the isolated areas of resistive material, and the tests produce a reliable overall modulus.

2.3.5.2 State of Stress

The state of stress in the walls of the tunnel is easier to model mathematically. Also, the depth to which the rock is affected by loading is related to the size of the bearing area, therefore, the results of the pressure chamber tests and the large bearing areas are less affected by surface disturbance.

2.3.5.3 Direction of Measurement

The modulus of deformation can be computed with deflections measured in a number of directions perpendicular to the cavity axis simultaneously during any one test. This measurement enables the anisotropy of the rock to be estimated under fixed conditions.

2.3.5.4 Hoop Stresses

The pressure chamber method introduces tensile hoop stresses in the rock and these stresses may exceed any residual compression and result in the opening of radial cracks in the rock. These radial cracks in turn reduce the observed stiffness of the rock and
in some instances this reduced modulus may be important. For example, a tunnel used as a penstock could encounter this type of loading system and a pressure chamber test may be the only way to adequately analyze the actual field conditions.

2.3.6 Disadvantages

2.3.6.1 Low Stress Levels

The maximum stress level attained in a pressure chamber test to date was on the order of 320 psi, with the average test peak pressure on all other tests being 300 psi (Dodds, 1965). In most rock deformation moduli are a function of the applied pressure, and these low values of applied test pressure generally do not subject the rocks to the range of pressures expected under design load. Whether or not this is a significant factor depends on the extent to which the modulus is stress dependent. If necessary, laboratory tests can be performed on rock cores at different confining pressures and loads to determine the extent of this sensitivity. From these same laboratory tests a coefficient can be obtained to adjust the field modulus to a value more in line with the expected or actual stresses.

2.3.6.2 Cost

The pressure chamber tests are the most expensive form of loading test to perform. The average cost is approximately
$200,000 per test, and the test, including construction of the chamber, takes from two to three months to perform. If the test adit excavation is ignored, then these tests are almost 400 times as expensive as rigid plate tests.

2.4 Radial Jacking

The radial jacking test was developed to avoid some of the major difficulties of the pressure chamber test; i.e., time and cost, while still retaining some of its advantages. This method has been pioneered by the Austrians (Kujundzic, 1963 and Lanter, 1962) and modified by the U. S. Bureau of Reclamation (Wallace, 1968). Figure 2.10 is a sketch of the test and Figure 2.11 shows an actual photograph of the test setup.

2.4.1 Equipment

The following equipment is needed for the radial jacking tests:

1. sixteen flat jacks, 96 inches long by 6 inches wide;
2. eight sets of aluminum circular sets;
3. wooden cushions between circular sets and flat jacks;
4. four tunnel-diameter extensometers;
5. eight borehole extensometers;
FIG. 2.10 Sketch of the radial jacking test.

6. hydraulic manifold, pump, oil reservoir and pressure gauge.

2.4.2 Procedure

1. Site preparation - Site preparation consists of the removal of rock damaged in the excavation required to give access to the site, the obtaining of the proper shape and dimensions for installation of the test equipment, and the provision of a reasonably smooth rock surface for application of the test loads.

2. Installation of subsurface measuring devices - Eight boreholes are drilled radially, perpendicular to the axis of the tunnel. The extensometers are installed with the deepest anchor beyond the zone of influence of the proposed load. These extensometers should be monitored to determine whether the test site is essentially stabilized from the excavation prior to the start of the jacking test.

3. Installation of the circular sets, wooden blocking and flat jacks - This equipment is all suspended from anchor rods drilled into the rock.

4. The voids between the rock and flat jacks are pumped full of concrete.
5. The diametrical extensometers are installed across the opening.

6. The hydraulic pump and manifold are connected and the system is purged of air.

2.4.3 Testing

After the concrete has cured sufficiently to provide bearing strength in excess of the maximum test load, the testing is performed in cycles similar to the previously described tests.

2.4.4 Data Analysis

Using the equation developed by Love (1944) for the radial displacement of a point in a thick wall circular cylinder with infinite outer radius, the following equation is obtained.

\[ E = \frac{p (1 + \mu) R}{w'} \]  

(Eq. 2.17)

where:

\[ p \] = internal pressure

\[ w' \] = the corrected deflection at depth "R"

Jaeger and Cook (1963) obtained this same solution using radial pressure over diametrically opposed sections of a borehole wall where the sections are semi-circular sectors. Both solutions consider uniform stress; however, neither of these solutions accounts for the finite length of the loaded area. Tranter (1946) solved this problem for finite length
2C, but the solution is rather complex. The Bureau of Reclamation developed a computer program based on Tranter's work to solve for the modulus (Misterek, 1969). Lauffer and Seeber (1961) measured radial displacement both inside and outside the loaded length and utilized the principles of superposition to graphically obtain the surface deflections that would have occurred if the loaded length had been infinite. The use of this method is demonstrated in Figure 2.12 and graphically accounts for the finite length and seems to be of the most practical use.

2.4.5 Advantages

The advantages of the radial test are similar to the pressure chamber advantages except that the radial test is more economical, more easily performed and the test subjects the rock to high stress levels.

2.4.6 Disadvantages

The radial test has disadvantages which are also similar to the disadvantages of the pressure chamber test except that the loaded area is less for the radial test.

2.5 Cable Jacking

The cable jacking test was developed in an attempt to eliminate the expense of driving an exploration tunnel to provide an area to perform the type of jacking tests previously mentioned in this paper. This method was proposed by Zienkiewitz and Stagg (1967), a drawing of the test
FIG. 2.12 Effect of short length of loading of radial jack analyzed by superposition of radial deformation measured.
layout is shown in Figure 2.13. It was developed as a simple test using one cable and pad arrangement. From this original idea grew a more complex system composed of a four-pad array, as shown in Figure 2.14. The assumptions necessary to utilize the data obtained from this test reduce the results to the same order of accuracy obtained on the simpler test, and therefore, negate many of the advantages of the more complex test.

The simple, single-pad test will be discussed. The more complex test is essentially the same except that the load is also applied in the plane parallel to the loaded surface.

2.5.1 Equipment

The equipment necessary for this test is as follows:

1. three 100-ton hydraulic jacks;
2. one 45-foot multi-strand steel cable and one 2 1/2-foot diameter, 1-foot deep steel cable head;
3. an aluminum bridge for mounting of the dial gauges;
4. dial gauges sensitive to 1/10,000 of an inch;
5. hydraulic lines, manifold, pump and pressure gauge.

2.5.2 Procedure

1. One 40-foot deep NX borehole is drilled in the center of the proposed bearing area. The borehole should be sufficiently deep to provide the necessary anchor length plus a clearance between the pad and the
FIG. 2.13 Cable jacking test

FIG. 2.14 Plan view of test layout. (O.C. Zienkiewicz and K.G. Stagg, "Cable Method of In Situ Rock Testing", 1966.)
anchor of 10 pad diameters.

2. The bearing surface under the pad is cleaned and prepared by leveling and removing the loose and drummy rock.

3. The multi-strand cable is anchored in the borehole by grouting the necessary length to provide anchorage sufficient to withstand the maximum, intended applied load.

4. The 3-foot-diameter concrete bearing pads are formed centered on the hole with the cable passing freely through a metal sleeve.

5. The jacks are positioned on the bearing plates, the cable head is set in position on the jacks, and the multistrand cable is anchored in the head by filling the head with concrete.

6. All concrete is allowed to obtain sufficient set to provide adequate strength for testing.

7. The hydraulic lines and manifold are positioned and connected to the pumps and jacks.

8. The gauge-holding bridge and the gauges are positioned so that three dial gauges, set 120 degrees apart, measure the deflection of the bearing pad. Additional gauges are placed to measure deflection of the rock surface adjacent to the pad.
2.5.3 Testing

The testing is performed in a similar manner to the testing described in the rigid plate test section. The testing, using four pads, is considerably slower because there are more dial gauges to read and, also, the maximum load is held on the primary jacks while the load is applied in the direction parallel to the surface. The test of the four-pad system requires two and one-half days to complete, not including the set-up time. In comparison, the single-pad system takes about 4 to 6 hours to test.

2.5.4 Data Reduction

The data reduction is the same as that described for the rigid plate tests. The hole can be neglected if the diameter of the concrete pad is large in comparison to the diameter of the center hole. If the concrete bearing pad is thick, care should be taken to embed the dial gauge measuring arms near the surface of the rock so as to eliminate excessive deflection of the pad from affecting the readings.

A primary advantage of the four-pad test is that it is possible to obtain data relative to the anisotropic properties of the rock mass. Zienkiewitz and Stagg (1967) present a solution for the five elastic constants, but the solution is so complex that computer data reduction is essential. In order to avoid this complication, Zienkiewitz and Stagg simplified the equation by assuming Poisson's ratio to be zero in both
directions. This assumption provides a solution no more realistic than the measuring of isotropic moduli in different directions by using separate tests. At the risk of being labeled "head-in-the-sandish", it may well be pointed out that, presently, design engineers are well pressed just utilizing the isotropic modulus. In conclusion, it is felt that the simple single-pad test is best suited to present needs, and that the complex four-pad program should await further sophistication in the field of rock mechanics.

2.5.5 Advantages

The major advantage is that the test can be performed on the surface which eliminates the need of a test adit to provide adequate reaction for the test load. Also, in many instances, arch dams for one, the moduli of the surface rock is necessary for design and the cable test can be performed at the exact location and in the direction where the design load will be applied.

2.5.6 Disadvantages

The major disadvantage is the effect of the presence of deep weathered surface rocks on the results of the tests, and the flexibility of the concrete bearing plate. The nonrigidity of the plate was discussed earlier under rigid plate tests. The additional flexibility of the concrete bearing pad does not materially affect the conclusions drawn in that discussion.
2.6 Borehole Jacking

There are essentially two borehole jack techniques available: the Goodman jack, which applies unidirectional pressure over diametrically opposed sections of the drill hole wall; and the pressure meter, which applies radial pressure to the entire circumference of the drill hole. The pressure meter test in rock is essentially in the development stage and is not considered operational by the author and is therefore only mentioned in passing. The borehole jacking method, discussed herein, will be the Goodman jack method.

2.6.1 Goodman Jack

2.6.1.1 Equipment

The following equipment, shown in Figure 2.15, is used to perform the Goodman jack tests:

1. Goodman jack;
2. Schaevitz TR100, LVDT (linear variable displacement transducers), readout box;
3. low pressure and high pressure hydraulic lines;
4. electric readout cable;
5. BX casing to install and manipulate the jack;
6. hydraulic pump and pressure gauge.
2.6.1.2 Procedure

1. Thread low pressure and high pressure hydraulic lines and electric readout cable through first section of BX casing, Figure 2.15a.

2. Attach hydraulic lines and cable to Goodman jack and thread jack onto casing by twisting casing to prevent fouling of lines and cable, Figure 2.15c.

3. Place jack and casing in borehole, Figure 2.15c, add additional casing and length of cable and lines as necessary to position jack at desired depth.

4. Orient test surfaces and record compass bearing of the axis of the applied load.

5. Attach hydraulic lines to pump and the electric cable to the readout box.

6. Apply pressure until jack expands against sides of the drill hole. Record the LVDT readings as zero readings.

2.6.1.3 Testing

The load is applied in ten equally spaced increments, and is held 15 minutes at the peak of each cycle to determine if the material exhibits time-dependent strain characteristics. The load is then removed in decrements to obtain an unloading curve and a minimum
A. Threading the lines into casing.

B. Jack and casing.

C. Connecting jack to casing.

D. Goodman jack, readout box and hand pump.

E. Installing jack in the hole.

Fig. 2.15
load is held for 15 minutes to allow the rock to rebound. Two additional and similar cycles of loading are performed to successively higher loads. The jack is then collapsed and rotated 90 degrees, and the above procedure repeated. Upon completion of the second test, the jack is collapsed and removed from the boreholes.

2.6.1.4 Data Reduction

The equation used to obtain the modulus values was derived by assuming the actual pressure to be a constant radial boundary pressure plus shear and radial pressures distributed sinusoidally over the width of the plate. If the angle subtended by the width of the plate is about 45 degrees, little effect on the results occurs from the finite plate width. The results of a finite element program (Hall, 1972) showed that a reduction of 14 percent in the value of the modulus was necessary to adjust for the actual length of the jack. The modulus is obtained by the use of the following equation (Hall, 1972) and Table 2.2.

\[ E = 0.86 K \mu \frac{P h}{w_h} \]  

(Eq. 2.18)

where:

- \( P \) = applied load
- \( h \) = diameter of borehole
- \( w_h \) = diametrical deformation

In the Goodman device the hole diameter (h) equaled three inches, and plate pressure (Q) equaled 93 percent of the hydraulic line pressure.
<table>
<thead>
<tr>
<th></th>
<th>0</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
<th>0.25</th>
<th>0.30</th>
</tr>
</thead>
<tbody>
<tr>
<td>( ) ( K() )</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( K () )</td>
<td>1.38</td>
<td>1.29</td>
<td>1.29</td>
<td>1.28</td>
<td>1.27</td>
<td>1.25</td>
<td>1.23</td>
</tr>
<tr>
<td>( 2.40 ( K () ) )</td>
<td>3.07</td>
<td>3.10</td>
<td>3.10</td>
<td>3.07</td>
<td>3.05</td>
<td>3.00</td>
<td>2.95</td>
</tr>
<tr>
<td>( ) ( ) )</td>
<td>0.35</td>
<td>0.40</td>
<td>0.45</td>
<td>0.50</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( K () )</td>
<td>1.20</td>
<td>1.17</td>
<td>1.13</td>
<td>1.09</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( 2.40 ( K () ) )</td>
<td>2.88</td>
<td>2.81</td>
<td>2.71</td>
<td>2.62</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Rigid Plate</th>
<th>Flexible Plate</th>
<th>Chamber</th>
<th>Cable Jacking</th>
<th>Radial</th>
<th>Borehole Jacking</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cost</td>
<td>5</td>
<td>2</td>
<td>0</td>
<td>3</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>Ease of Execution</td>
<td>5</td>
<td>2</td>
<td>1</td>
<td>3</td>
<td>0</td>
<td>4</td>
</tr>
<tr>
<td>Modulus Area</td>
<td>2</td>
<td>2</td>
<td>5</td>
<td>3</td>
<td>4</td>
<td>0</td>
</tr>
<tr>
<td>Amount of Applied Stress</td>
<td>3</td>
<td>2</td>
<td>0</td>
<td>4</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>Total</td>
<td>15</td>
<td>7</td>
<td>6</td>
<td>13</td>
<td>6</td>
<td>13</td>
</tr>
</tbody>
</table>
Putting these values into Equation 2.18 we obtain:
\[ E = 2.4 K \mu \frac{Q_h}{wh} \]  
(Eq. 2.19)

The complete derivation of these equations can be found in Goodman, Van and Heuze (1968).

The U. S. Corps of Engineers, Missouri River Division Laboratory (Ohnishi, 1971) and the U. S. Bureau of Mines, Spokane Mining Research Center (1969) have performed independent studies on the Goodman jack and both report low modulus results compared with other methods.

2.6.1.5 Advantages

2.6.1.5.1 Low Cost

The actual cost of the Goodman jack test is relatively low, approximately $1000 to $2000 per test depending on hole depth. The deeper the hole the more expensive the drilling and the more difficult it is to place the instrument at the desired test depth. For instance, it may take four men 2 to 3 hours to set and retrieve the instrument at 200 feet. However, the most significant savings comes from the deletion of a test adit. If the only purpose of the test adit is to obtain rock modulus information, and the rock is within 300 feet of the surface, the test adit can be deleted and the test performed from the surface with a savings of several hundred thousand dollars.
2.6.1.5.2 Properties at Depth

In test cavities which are excavated by blasting, the surface rock on the walls of the cavities is affected by the de-stressing of the rock in the cavity walls. Most loading tests are performed against these walls. The effect these changes make on the undisturbed value of the deformation modulus is hard to determine and is the subject of concern and controversy. The borehole jack is a way of getting behind this damaged rock to test the so-called undisturbed rock. Admittedly, the drill hole used in the borehole jacking test does actually disturb the rock; however, even if the test adit were bored by a tunneling machine the borehole jack would still supply a modulus of deformation closer to the undisturbed values. The reason for this is that both the relative size of the test to the drill hole size and the high loading stress applied by the borehole jack reduce the effect of the drill hole's disturbance of the in situ conditions considerably below the effect caused by a regulation loading test performed in a bored test adit.

Modulus values at depth may also be required to supply closer control on a finite element grid. One of the significant advantages to finite element analysis is that the material properties can be changed at each nodule point. The borehole jack can get information on the material properties of the actual location of the nodule point and thereby strengthen this already powerful analytic tool.
2.6.1.5.3 High Applied Stress Levels

The borehole jack can apply surface loadings of 10,000 psi, several times larger than any of the other tests discussed in this paper. These high stresses have the advantage of extending the range of applied stress, over which the modulus is known, to approximately 50 percent of the rock's compressive strength; in weaker rock actual failure may take place. This failure is dependent upon the strength of the rock, the in situ stress field and the rock fabric. The necessary information on the rock fabric can be obtained by examining the cores from the drill hole and by the use of a borehole camera before and after the test. Now, if one of the other variables are known such as strength (laboratory tests) or in situ stress field (overcoring or flat jack tests), then information on either the in situ stress field or in situ strength can be found.

2.6.1.6 Disadvantages

2.6.1.6.1 Small Bearing Area

The approximate size of the bearing surface is 3 by 8 inches. This small bearing area limits the depth and sphere of influence the test has on the surrounding rock. The rock affected by the Goodman jack is a cylinder some 12 inches long with a diameter of approximately 3 feet. The amount of joints, cracks and weak areas
contained in this volume is minimal and so the results of the test generally show values for in-place, intact rock. If an abnormality does pass under the plate, the relatively large area allotted to the abnormality compared with that allotted to the normal rock tends to overwhelm the results and produce a very low value of the modulus; the end result then being that the upper and lower bounds are obtained rather than the deformation modulus of the material mass.

2.6.1.6.2 Test Site Selection

Because the test is performed in a borehole, care must be taken to prevent broken rock from falling out of the sides of the hole and into the drill hole during the test. These small pieces of rock can lodge between the borehole and the instrument and wedge it in the hole. Thus, careless placement of the instrument may result in loss of the instrument, loss of the drill hole and delay of the testing program. This enforced care in test site selection leads the engineer to select test areas in sound rock in the borehole rather than chance the loss of several tens of thousands of dollars. The end result is that the deformation modulus obtained from the study is from the stronger basically intact material. This stronger material can be studied more cheaply and with better control in the laboratory, and it tends to decrease the real value of the Goodman jack test.
2.6.1.6.3 Limitation of Depth

The practical limit in which the borehole jack tests can be performed is in the vicinity of 300 feet from the surface. Holes of greater depth become increasingly difficult and time consuming to drill and test. Most major projects are generally much deeper and/or have larger budgets for rock mechanics studies which include a test adit where large-scale loading tests can be performed. These two factors tend to limit the Goodman jack to use in minor, shallow projects or preliminary studies.

2.7 Summation

In summation of this section, a comparison of the major advantages and disadvantages of each individual type of test is shown on Table 2.3. Each of the six tests is ranked, according to the opinion of the author, on this table from zero to five in ascending order of desirability with five being the most desirable. Cost, as used here, does not include the cost of the test adit itself as in many instances the adit is used for other equally important reasons. The ease of execution and the amount of applied stress are self-explanatory. The modulus area is the area of rock to which the modulus pertains. From this table it can be seen that the rigid plate is the most desirable, the borehole jack and cable jacking tests closely following. Regardless of the type of test selected, care still must be taken in performing and interpreting the data. Many
factors involved in these two operations can render meaningless data and useless results. While both performance and interpretation require experience and judgment which best can be obtained in actual practice, the remainder of this paper is devoted to a discussion of these two operations with the purpose of serving as an adjunct rather than a substitute for practical experience.
FACTORS BEARING ON INTERPRETATION OF RESULTS

3.1 General

In order to perform and analyze a plate loading test properly, it is essential that the interrelationship between the test and its environment and various rock properties be thoroughly understood. Consideration of the effects on the test caused by such things as: rock fabric, interface conditions, chamber size, plate pressure effects, in situ stress level, Poisson's ratio and strain measurement methods can change apparently spurious readings into meaningful results.

3.2 Rock Fabric

3.2.1 Anisotropy

Most rocks are composed of an aggregate of crystals or amorphous particles joined in some instances by varying amounts of cementing materials. The chemical composition and/or the size of the crystals may be relatively homogeneous as in limestone or very heterogeneous as in granite. Large grained igneous rocks are composed of randomly oriented mineral crystals of various sizes, shapes and strengths. Clastic, sedimentary rocks consist of an assemblage of detrital particles derived from other rocks in a matrix commonly of calcium carbonate or silica. Metamorphic rocks are produced by the action of heat, stress or heated fluids on other rocks. All these rocks contain
minerals which exhibit anisotropic characteristics. If there is any preferred orientation of these crystals in the rock this will lead to anisotropism of the rock itself. For example, on a series of tests performed in two types of metamorphic rock, horizontal tests in a gneiss produced an average of $E = 5.44 \times 10^6$, vertical tests in the same rock averaged $E = 2.29 \times 10^6$, a difference of 2.4 times. In a schist, horizontal tests on a specific job averaged $1.0 \times 10^6$, while vertical tests averaged $4.5 \times 10^5$, a difference of 2.2 times (Foundation Sciences, Inc., 1967).

3.2.2 Heterogeneity

Rock is rarely homogeneous. However, since engineering is not an exact science, the heterogeneity of most rocks is seldom of much concern when bearing areas of two to ten square feet are used. Generally, very little diligence is needed to insure that the bearing plate is placed wholly within one type of material. If this care is taken the homogeneity of most rocks is as good as the homogeneity of concrete and produces little effect on the test results. On occasion, tests are performed where non-homogeneity can be a problem. For instance, tests performed on a dense vesicular basalt, where the vesicles were large compared to the plate size, produced results which varied by 30 percent (Foundation Sciences, Inc., 1973); but this is rare and in most instances the scatter of the test results, blamed so conveniently on heterogeneity of the material, is caused by one of the other not so easily recognized culprits discussed in this section.
3.2.3 Continuity

On a scale with dimensions ranging from feet to hundreds of feet, the structure of some rocks is continuous, but it is more often interrupted by cracks and joints. The cracks and joints usually occur in sets which are more or less parallel and regularly spaced; also, there are usually several of these sets in different directions so that the rock mass is broken up into a blocky, discontinuous structure.

It is the presence of this structure and our inability to model it successfully in the laboratory which has given the greatest impetus to field plate loading tests. The effect of the joints is to reduce the value of the effective modulus for large areas. That is to say, before any effect of jointing is noted, joint spacing has to be small, two to six inches, or the loaded areas have to be large or heavily loaded. This is not to say that an occasional low spurious reading cannot be obtained by careless placement of a bearing plate too close to a joint or crack.

3.2.4 Summation

The only methods available presently to combat these difficulties arising from rock fabric are to:

1. map the geology of the test site very precisely and apply judgment to the results;
2. use average plate deflection rather than point de-
flections;
3. perform enough tests to apply statistical judgment to the data obtained;
4. modify the theory to accept anisotropic values of the modulus of deformation and Poisson's ratio.

Item one is, of course, an invaluable aid for any type of exploration and rock testing program. The general mapping should be performed on a scale suitable to show sufficient detail. For example, one inch to five feet is a good size. Further detailed mapping should be performed on each of the individual test sites and immediate surroundings at a scale of perhaps one inch to one-half foot.

Item two, the most commonly used way to solve the problem caused by fabric, has led to larger and larger plate sizes to average the extremes produced by individual tests. It must be realized that this is not the only method to correct for fabric or even the best method. For instance, if the modulus of deformation is to be used to design the flange width of a steel tunnel rib or size of the foot block for the rib, a smaller test plate size would provide more meaningful data. Even if it is for a massive rock unit, judgment still must be used on the geographical extent and the sharpness of the change in modulus across the boundary as well as the numerical difference between adjacent moduli.

For example, if there are one or two individual rock elements that exist homogeneously throughout the separate sections of a tunnel, with sharp changes in modulus between the sections and very little var-
iance throughout an individual material, or if large joint spacing is an important factor, pressure chamber or radial jacking tests may be called for. However, if the modulus varies continually along the chamber, then item three may be a better solution. Numerous small tests allow for statistical manipulation to obtain average values over large areas as well as the gradation of the modulus values along the rock unit and the rock anisotropic properties.

Item four, the modification of an existing theory to allow for anisotropic conditions, is presently cumbersome, unduly precise and beyond the needs of the industry.

3.3 Interface Conditions

The condition between the steel bearing plate and the test material is a subject often ignored by investigators. The assumption is that the two bearing surfaces are smooth and parallel. In actual fact these surfaces are neither. Generally, these are prepared by chipping to remove loose and damaged rock and the surface is then made as flat and parallel as possible with an air tool. The final preparation is made by pouring a grout or sulfur bearing pad under the plate. While this all sounds reasonable enough, observations of the field test often show a divergence from parallel of as much as four inches across a ten-inch plate. Rock is sometimes a difficult material to shape as it tends to break along cleavage and fracture planes and the test engineer must
often settle for what the rock gives him, which may well invalidate any theoretical arguments as to the nature of the stress field under the plate. The ideal interface condition would be as shown in Figure 3.1a. A typical interface condition may be as shown in Figure 3.1b.

In Figure 3.1b, not only is the level of stress imparted to the rock in question, but how much does the flexibility of the pad change the rigid die assumption and conversly how much rigidity does the pad produce in a flexible plate test? Depending on the relative moduli of the pad and rock, the deflection of a 4-inch-thick pad can become significant in the determination of the true rock modulus, and should be removed from the calculations.

3.4 Size of the Test Chamber

The size of the test chamber should not be overlooked in relation to the plate size. In particular, care should be taken in smaller chambers to insure that the assumptions of a half space or a quarter space are met. The boundary conditions imparted by the roof and floor produce serious errors if the method of analysis is based upon Boussinesq's theory. It is common to see photographs of test sites showing 2-foot-diameter, or larger, plates in an 8-foot, or smaller, test adit. In an 8-foot adit the plate size should be limited to about twelve inches in width or the method of data reduction should be altered to allow for the actual boundary conditions rather than use the
FIG. 3.1 Theoretical & typical interface conditions.

FIG. 3.2 Areas to be avoided during testing.
Boussinesq equation which assumes a semi-infinite body.

As well as the size of the opening, care should be taken to see that test sites are selected so they are away from areas of shape change within the chamber, as shown in Figure 3.2. All the irregularities shown in this figure can cause stress concentrations that may affect the test results.

3.5 Pressure Effects

Currently, plate load and size vary greatly from test to test and in turn affect the applied pressure. Understanding the effect plate pressure has on test results is essential to proper test design and the interpretation of the test results. Figure 3.3 is a plot showing stress at depth according to Boussinesq's theory for two different plate sizes both having identical total loads. From studying this curve, it can be seen that for the smaller plate approximately one half of the area under the stress-depth curve is contained between the surface and two feet of depth; whereas, the area under the curve for the larger plate in the same depth range is only about one quarter of the total area. Therefore, the smaller plate results will be affected more by surface rock which is in turn more susceptible to blast damage and stress relief effects. Also, note that from depths greater than two feet, the stress in the rock is essentially uniform regardless of the width of the plate; therefore, stress in the rock at depths greater than two feet is proportional to the total plate load and
is essentially independent of plate size. It is apparent from this curve that larger and larger plate sizes are not the best method to obtain information about the material at depth, and that somewhere in the range of 18 to 24 inches is a plate width which produces a good combination of high stress level at depth and a minimum of near surface effect for use as the ideal plate width.

To increase the stresses at greater depths, it is necessary to increase the applied load. Currently the typical load is generally in the range of 200 tons whether using the standard hydraulic jacks or the hydraulic cell, flat jack system. To increase the load using hydraulic jacks would require the use of excessively heavy and unwieldly equipment which is very difficult to use in the confines of the test adit. Increasing the load on the flat jack system can produce larger deformation in the rock than can be tolerated by the flat jack which, in turn, could produce violent destruction of the jack and extreme danger to the testing personnel. Better tests will be performed when either of these two problems are solved.

3.6 In Situ Stress

The in situ stress acts as a confining load during plate loading tests. Confinement causes an increase in modulus. Figure 3.4 shows the results of laboratory study on the effect of confinement on three types of deformation moduli for four types of rock (Foundation Sciences, Inc.,
Stress at depth beneath square plate loaded with 400,000 lbs.

**FIG. 6.**

- **Unconfined**
- **Confined, \( \sigma_3 = 1000 \) psi

---

**FIG. 3.4** Comparison of moduli for different rock types.

- **TANGENT MODULUS**
- **SECANT MODULUS**
- **RECOVERY MODULUS**
1972). The increase in modulus varied from 20 to 100 percent with the addition of only 1000 psi confinement. The stresses in the plane of the tunnel wall, which act as confining loads (see Figure 4.8) can be two to four times greater than the undisturbed stress level which easily ranges from 500 to 2000 psi. The resulting confining loads can produce significant change in modulus values. This is important when making measurements of moduli in the same material on either side of a fault plane where values of in situ stress may vary greatly. Also, confinement should call for a reduction of moduli obtained from underground surface testing when applied to elements at depth in finite element models.

3.7 Poisson's Ratio

The modulus of deformation obtained from the results of any of the plate loading tests is a function of Poisson's ratio. The term \((1 - \mu^2)\) appears in most equations used for reduction of plate loading data; this would produce a maximum error of 12.5 percent if a median Poisson's ratio of .25 were assumed. The pressure chambers or radial jacking test formulas contain a \((1 + \mu)\) term which can lead to errors as large as 25 percent with the same assumptions. Further, Poisson's ratio, commonly assumed to be a constant for any given material, is stress dependent in rock especially under triaxial loading conditions. Figure 3.5 shows the relationship between deviator stress and Poisson's ratio for two sandstones and one schist sample (Dodds, 1972). The
negative portion of the curve is produced because of non-isotropy and large axial deflections due to recovery of stress damage in the early portions of the loading cycle. Poisson's ratios greater than 0.5 result after the sample has failed. It can easily be seen that Poisson's ratio is anything but constant. A good portion of the curve can be considered linear, but the complete curve can be made to fit:

\[
\mu = 0.25 - a \frac{\text{ctn}(\pi \sigma')}{\sigma'_{\text{ult}}} \tag{Eq. 3.1}
\]

where:

\[
\begin{align*}
\mu &= \text{Poisson's ratio} \\
a &= \text{constant} \\
\sigma'_{\text{ult}} &= \text{ultimate deviator stress} \\
\sigma' &= \text{deviator stress}
\end{align*}
\]

The complete implications of these findings is not yet known; however, this characteristic renders any attempt of refining the accuracy of the deformation modulus below 10 percent to be useless unless Poisson's ratio is treated as a stress-dependent variable in the modulus equation.

3.8 Strain Measurements

The measurements of deformation have been made with ordinary dial gauges, joint meters, and borehole extensometers. Measurements of deformation are taken from plate movement, surface rock movements and rock movement at depth.
Dial gauges are mounted independent of the jacking equipment on supports which are outside the area to be affected by the test. The gauges should have one-quarter to one-half inch of travel and be sensitive to one-ten thousandth of an inch. The gauges are used to obtain the movement of the bearing plates and surface rock near the plates. Tunnel diameter extensometers employ dial gauges to measure the relative movement between the opposite sides of the tunnel. Dial gauges perform quite well and are the most accurate method of obtaining measurements of rock movement. Reading loss can come from sticky gauges, movement of the plate beyond the range of the gauge and jarring the test equipment. All these difficulties are caused by carelessness on the part of the test engineer and are easily corrected with a little attention to detail.

Joint meters have been installed under the bearing plates. A flange at the top of a 10-inch meter is cast into the concrete pad and a 14-foot steel rod is attached to the bottom of the meter. The lower end of this rod is grouted at the bottom of a 15-foot hole. A waterproof cable from the gauge extends through the concrete pad and the change in resistance of a pair of resistance coils is read on a whetstone bridge. The precision of the instrument is quite high; however, the use of electronic equipment in the wet, dirty underground environment causes the accuracy to suffer and the use of joint meters to detect surface movement is considered inferior to dial gauges.
FIG. 3.5 Deviator stress vs Poisson's ratio.

FIG. 3.6 Typical multiple borehole extensometer used for deflection measurement.
To measure the deflection at depth in the rock, a multiple-position borehole extensometer, similar to the one shown in Figure 3.6, is installed. The multiple-position borehole extensometer in the figure is the rod type of extensometer where the rock movement is sensed by LVDTs. Extensometers using wire instead of rods and SR4 strain gauges instead of LVDTs are also used. These instruments suffer from the same precision-accuracy problems as do the joint meters; however, they do have the advantage of measuring deformation at multiple points away from this surface.

It is important to remember that regardless of the type of device used to measure deflection, sufficient measurements must be taken to allow the average plate deformation to be calculated, and regardless of the size of the bearing plate, if only one measurement point is used then the modulus obtained is the modulus of the rock represented by calculation from deflection at one point.
DATA UTILIZATION

The primary product developed from plate loading tests is the modulus of deformation. However, considerable additional information on the material properties can be obtained such as: primary creep coefficients, strength parameters, in situ stress levels, amount of stress damage and judgment factors on elastic-plastic response. Not all this information is available on every test, but it occurs often enough to be of significant value to be discussed.

4.1 Modulus of Deformation

It must be remembered that the equations used in computing the moduli are generally based on elastic theory. These equations assume that the rock mass is an isotropic, homogeneous body with ideal elastic properties. The validity of the computed deformation modulus will depend on the extent to which the in situ rock conforms to the above assumptions and how well the actual test conditions conform to the conditions imposed upon the completed structure. An error of ten percent would be considered excellent and twenty percent acceptable at the present time.

Because rock is not truly an elastic material, there is considerable latitude available in selecting the type of modulus to be calculated, and in some instances this can result in large numerical differences in the moduli. All moduli are calculated using the same equation; the only difference is the portion of the curve that is used to obtain the
deformation and stress changes. The three moduli most commonly used are: the tangent modulus, the secant modulus and the recovery modulus, all of which are shown on Figure 4.1.

1. The tangent modulus of deformation is the slope of the stress-strain curve obtained between two adjacent sets of data points or over the segment of the loading curve judged as the most representative of elastic response by the investigator. It neglects the end effects of the curve and is better suited to small stress changes.

2. The secant modulus of deformation is the slope of the stress-strain curve between zero stress and the stress in question. This modulus should be used for complete load steps from zero to the desired load. The initial, concave upwards section of the stress-strain curve is often attributed to closing of micro-cracks and other stress-damage-type phenomena; as such, the ratio between the secant modulus and the tangent modulus can be used as a means of measuring the stress damage of the material. A ratio of one indicates no stress damage.

3. The recovery modulus of deformation is a tangent modulus taken from the portion of the stress-strain
curve where the stress is being removed. This modulus is generally higher than the other two moduli and is used in calculations where unloading conditions exist. The difference between the tangent and recovery moduli indicate the material's capacity for hysteresis or energy storing capabilities. In a linearly elastic material all three moduli would be identical. Table 4.1 is a tabulation compiled by the author of typical values of the above mentioned deformation moduli for various types of materials taken from past reports on field testing.

4.2 Creep Coefficients

The study of time-dependent effects, or creep, is important in many rock mechanics applications. There are many reviews on creep of rocks, Robertson (1964) and Murrel and Cruden (1971).

In general, most rock has a deformation cycle which can be broken down into four parts:

1. instantaneous elastic strain,
2. primary creep,
3. secondary creep,
4. tertiary creep to failure.

The elastic strain takes place when the load is applied, A-B, Figure 4.2. This is followed by a deformation which gradually decreases in time, pri-
FIG. 4.1 Relationship between tangent, secant and recovery moduli.

FIG. 4.2 Constant stress, creep curve.
<table>
<thead>
<tr>
<th></th>
<th>Tan Modulus $\times 10^6$ psi</th>
<th>Secant Modulus $\times 10^6$ psi</th>
<th>Recov Modulus $\times 10^6$ psi</th>
<th>Number of Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ave</td>
<td>Max</td>
<td>Min</td>
<td>Ave</td>
</tr>
<tr>
<td>Quartzite</td>
<td>2.1</td>
<td>5.4</td>
<td>0.4</td>
<td>1.6</td>
</tr>
<tr>
<td>Granodiorite</td>
<td>2.6</td>
<td>8.1</td>
<td>0.6</td>
<td>2.4</td>
</tr>
<tr>
<td>Biotite Schist</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.9</td>
</tr>
<tr>
<td>Chlorite Schist</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.3</td>
</tr>
<tr>
<td>Graphite Schist</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.3</td>
</tr>
<tr>
<td>Augen Gneiss</td>
<td>1.3</td>
<td>1.6</td>
<td>0.8</td>
<td>0.9</td>
</tr>
<tr>
<td>Granite Gneiss</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>4.2</td>
</tr>
<tr>
<td>Siltstone</td>
<td>2.0</td>
<td>2.7</td>
<td>1.3</td>
<td>1.1</td>
</tr>
<tr>
<td>Limestone</td>
<td>0.3</td>
<td>0.4</td>
<td>0.1</td>
<td>0.2</td>
</tr>
<tr>
<td>Argillite</td>
<td>1.8</td>
<td>3.6</td>
<td>0.5</td>
<td>1.8</td>
</tr>
<tr>
<td>Amphibolite</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.3</td>
</tr>
<tr>
<td>Scoriaceous Basalt</td>
<td>1.0</td>
<td>2.7</td>
<td>0.1</td>
<td>0.6</td>
</tr>
<tr>
<td>Vesicular Basalt</td>
<td>4.4</td>
<td>8.2</td>
<td>2.2</td>
<td>1.2</td>
</tr>
</tbody>
</table>
mary creep, also called elastic flow, B-C on Figure 4.2.

If the load is released at $t_1$ there is an immediate elastic recovery, C-D, and a viscoelastic recovery, D-E over $t_1$, $t_2$. If the load is maintained past time $t_1$, a constant strain rate — secondary creep — follows which includes some permanent deformation of the rock. After a given limit of secondary creep has occurred the strain begins to accelerate — tertiary creep — until failure. Information on tertiary creep generally requires stress levels higher than most plate loading tests can produce and, therefore, information from plate loading tests concerning this phase of creep is nonexistent.

The tests performed to date are of insufficient time span to be able to accurately or realistically predict secondary creep. The two factors which control the accuracy at which time-dependent strain can be predicted are the accuracy at which strain can be measured during the tests and the length of time the test is performed. The accuracy to which deformation is generally measured during the testing is on the order of 0.0002 of an inch. The maximum length of time the tests are performed is approximately 600 hours. This provides a minimum increment of measurement of $10^{-10}$ inches/second which is not sufficient to measure secondary creep rate in most materials under low temperature and pressure conditions.

The only practical information from plate loading tests on the creep cycle of the material is that information obtained on primary creep
characteristics. Primary creep is dependent mainly on time \((t)\), imposed stress \((\sigma)\), confining stress \((\sigma_2, \sigma_3)\) and temperature \((T)\). That is:

\[
\epsilon = F(t), F(\sigma_1), F(\sigma_3, \sigma_2), F(T)
\]  

(Eq. 4.1)  

(Swanson, 1971)

Equation 4.2 represents a method that has been used in the past to predict time-dependent strain.

\[
\epsilon = c_1 \sigma_1^2 \sigma_3^3 t^4 e^{-c_5/T}
\]  

(Eq. 4.2)

It can be seen from examining Equation 4.2, experimental determination of the five constants in a laboratory would be a difficult and expensive task. Further complications are inherent since the individual form of each function can change; for example, according to the creep model assumed for analysis, various investigators (Jaeger and Cook, 1969) have used the following forms for \(F(t)\):

\[
\epsilon = C t \alpha 
\]  

(Eq. 4.3)

\[
\epsilon = C \ln(t)
\]  

(Eq. 4.4)

\[
\epsilon = C \ln(1 + \alpha t)^a
\]  

(Eq. 4.5)

\[
\epsilon = C(1 + kt)\alpha - 1
\]  

(Eq. 4.6)

where:

\[\alpha, C, k\] = material constants

\[\ln\] = natural log

As a result, the number of tests needed to completely define the time-dependent strain characteristics of a material becomes large, and to the author's knowledge it has never been done. Laboratory, as opposed to field, studies offer the best possibility of controlling the many variables
involved. However, the sustained plate loading test offers significant advantages over the laboratory test if the tests are performed so that temperature and confinement are nearly constant for the test and the prototype. Then only two variables must be explored: time and the applied load, and if the applied load is known, or at least if its limits are known, obtaining the creep equation is almost automatic by comparison. To obtain this type of information, the load on the plate is held constant for a period of from 3 to 30 days and the strain is recorded at various time intervals. The only difficulties encountered are the usual problems of experimental error and the homogeneous and isotropic characteristics of the material. The primary creep coefficients are generally obtained by a root-mean-square curve fit to the strain-time data.

4.3 In Situ Strength Determinations

Coates (1965) utilized plate loading test data to obtain bearing capacities of rock. However, this method has some current limitations in that it requires the use of such a small plate or large load to produce failure stress in normal rock that its practical value is confined to very weak material. But when failure does occur in a loading test, as indicated by the slope of the stress-strain curve, significant strength information can and should be obtained by the use of this theory.

It has been claimed by Talobre (1957) that in situ strength can be obtained from plate loading data and the judicious use of combined stress theory. Both Kruse (1963) and this author have, on separate occa-
sions, attempted to obtain Mohr's envelope by the method outlined and find serious errors in the procedure which render meaningless results. It was essentially concluded by both parties that the procedure was not adaptable in our studies.

4.4 In Situ Stress Determinations

On some loading tests the material behaves as shown on Figure 4.3. Not all tests exhibit this behavior but a sufficient number of tests, in various parts of the world, have followed this pattern which would preclude it as being a chance result. It can be seen from examining Figure 4.3 that there exists a sharp break in the load deformation curve and, further, this break occurs near the previous cycle's high load. This change in moduli is probably due to strain hardening, an effect similar to preconsolidation in clays. At the beginning of the first cycle, this same steepened curve can also be noted; it follows logically that in this case the material has been strain hardened by this amount previous to the first cycle. It is reasonable, also, to conclude that this hardening occurred because of the maximum, historical in situ stress level.

The in situ stress indicated on this curve is approximately 350 psi, which agreed well with independent in situ stress measurements in the area of this test. When this type of curve occurs, the four cycles generally produce sufficient information to allow a graphical estimate of
FIG. 4.3 Typical curve showing stress history.
the break point on the curve from which the in situ stress level is taken.

A significant advantage to this stress measurement is that it measures stress perpendicular to the plane of the wall and is relatively unaffected by stress concentrations. The results shown in Figure 4.3 were obtained from a plate loading test in a chlorite schist.

4.5 Stress Relief and Blast Damage

A subject of much controversy in any discussion of plate loading tests is the effect of surface disturbance of the rock, and its effect on the test results along with the completed structure. Engineering judgment must be used to temper the test results. This judgment can be based upon examination of the test site to determine if the plate bearing areas are fresh, durable rock and in some instances upon examination of the load-deformation curve. The curve, shown in Figure 4.4, exhibits a reaction common to rock that is extensively damaged. The concave upward portion of this curve is caused by closing of small cracks and fractures in the rock produced by blast damage or stress relief or both. Notice that the concave portion of the curve is not present in the later cycles. Also there is an increase in the secant modulus for each cycle. As the material at depth is stressed to higher levels, its influence on the modulus increases. In this particular test the modulus change between cycle 3 and cycle 4 is not as great as between cycles 2 and 3 which would indicate that the modulus obtained from the fourth cycle is
FIG. 4.4 Typical curve showing blast damage.
close to the undamaged modulus because of stress damage during cavity excavation. Not in all cases is this phenomenon transient. Figure 4.5 shows this reaction on all four cycles indicating a deep layer of stress-relieved or blast-damaged rock. The amount of elasticity shown by the material tested in Figure 4.5 is quite high. In contrast, the curve shown in Figure 4.6 exhibits no elasticity and even though its modulus is quite high, it retains any deformation imposed by the test. Here again it is seen that the concave portion is only on the beginning curve. Comparing these two figures we can get some information on the type and cause of the cracking that has occurred. The type of behavior shown in Figure 4.6 is diagnostic of blast damage where a few, large, widely spaced cracks are opened during the blast and are then closed during the first cycle of the test. Figure 4.5 is more typical of stress relief damage where numerous small cracks are opened in the material by stressing the material beyond its strength.

This stress relief of the near-surface rock may be a factor that must be considered in the evaluation of test results when the in situ stress level approaches 40 to 50 percent of a rock's strength (either tensile or compressive, depending on the type of loading). Figure 4.7 and 4.8 show a general stress gradient beyond an adit surface when no stress relief is taking place at various points in time.

The high stress levels near the surface of the opening, in many instances, surpasses the strength of the surrounding rock thus produc-
FIG. 4.5 Typical curve showing deep layer of stress relieved rock.
FIG. 4.6 Typical curve showing blast damage.
FIG. 4.7 Theoretical tangential stress distribution away from an opening in elastic material.

FIG. 4.8 Tangential stress away from opening showing stress relief process.
ing numerous small fractures which weaken the rock and distribute the load of the material further from the opening. These fractures, in addition to producing the concave, upward portion of the curve mentioned above, also tend to make the rock inelastic in some instances. Figure 4.9 uses data from tests on a high arch dam project and shows this possible relationship between inelastic deformation and stress relief (Dodds, 1965). The percent divisions were obtained statistically from the information obtained from the field tests.

It can also be seen from Figure 4.8 that the effects of stress relief decrease with depth. An indication of this depth can be obtained by performing two plate loading tests on the same axis with different size plates and then comparing the obtained moduli. Because the depth to which the rock is affected is proportionate to the diameter of the plate, the larger plate should show less effects of the surface conditions. For example, one test would use a six-inch-wide plate and the second test would use a twenty-inch-wide plate. The first test would measure the modulus of the material essentially within one foot of the surface. The larger test would be representative of the material within a depth of four feet. This same type of comparison could be made with the use of a borehole jack test by successively taking measurements in a borehole at different distances away from the surface.

4.6 Elastic-Plastic Responses

Information of a general nature can be obtained from an analy-
FIG. 4.9 Inelastic deformation and in situ stress curve showing percent stress relief as function of inelastic deformation.
sis of the shape of the curve. This information, when made available, can serve as a basis for numerous decisions requiring engineering judgment which are necessary during design and construction of underground facilities. A table similar to Table 4.2 can be produced by examining the plots of the load-deformation data. The terms used in this table are defined as follows.

Elastic Material - Elastic material is a material which deforms under load and the deformation is recovered upon removal of the load. The loading and unloading curves are almost identical and the energy used during loading is recovered.

Plastic Material - Plastic material is a material which deforms under load and essentially none of the deformation is recovered upon removal of the load. The loading energy is lost by the rearrangement or crushing of the material particles.

Viscoelastic Material - A viscoelastic material is a material which exhibits time-dependent strain or creep under a constant load. The stress tends to drain from these materials as they rearrange themselves to seek a lower energy level.

Crack Closing - Crack closing is essentially self-explanatory. The test energy is used in moving the rock blocks to physically close cracks rather than compressing the rock material. This occurrence is characteristic of stress or blast damage in the rock. If desired, this column can be subdivided into two columns; i.e., stress
### Table 4.2

Rock Qualities Exhibited in Curves

<table>
<thead>
<tr>
<th>Test #</th>
<th>Material</th>
<th>Elastic</th>
<th>Plastic</th>
<th>Visco-elastic</th>
<th>Crack Closing</th>
<th>Load Memory</th>
<th>Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>DVB*</td>
<td>XXX</td>
<td></td>
<td></td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>DVB</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>LSB°</td>
<td>XXX</td>
<td>XXX</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>LSB</td>
<td>X</td>
<td>XX</td>
<td>XXX</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5**</td>
<td>P-P#</td>
<td>X</td>
<td>XX</td>
<td>XX</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>P-P</td>
<td>XXX</td>
<td></td>
<td></td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>DVB</td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>DVB</td>
<td>XX</td>
<td>XX</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>DVB</td>
<td>XXX</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10**</td>
<td>DVB</td>
<td>XX</td>
<td>X</td>
<td>XXX</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>LSB</td>
<td>X</td>
<td>XXX</td>
<td></td>
<td>X</td>
<td>XXX</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>LSB</td>
<td></td>
<td>XXX</td>
<td>XXX</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13**</td>
<td>P-P</td>
<td>XXX</td>
<td>XX</td>
<td>XXX</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>P-P</td>
<td>XX</td>
<td></td>
<td>XX</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>P-P</td>
<td>XXX</td>
<td>XX</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* DVB = Dense Vesicular Basalt  
° LSB = Light Scoriaceous Basalt  
P-P# = Puka Puka  
** = Invalid Test  

XXX = Exhibits strong qualities  
XX = Exhibits moderate qualities  
X = Exhibits slight qualities
damage and blast damage, by using the criteria outlined earlier in paragraph 4.5.

**Load Memory** - Certain rock material exhibits an ability to retain a record of the peak load imposed upon the material. This is demonstrated by a steep initial curve and then a definite break to a flatter curve on higher loading.

**Failure** - Failure under the loading test is considered to have occurred when the curve reaches essentially a horizontal slope. This occurs only in very weak material or during tests with very small bearing surfaces; i.e., 16 to 24 square inches.

4.7 **Summation**

In summation, it can be said that much usable information besides the deformation modulus can be obtained from plate loading tests. This information, when correlated with the results of the plate loading tests and other in situ testing, can provide much additional detail. While rock mechanics is not yet an exact science, the use of this additional detail, as color, adds a dimension to the results commonly referred to as the art of rock mechanics. Additional work is urged in this area on other in situ test results and even on plate loading test results. For example, there is a bothersome occurrence on some tests that has occurred often enough around the world to rule out chance error, but so far it has defied rational explanation: on the return portion of
the last cycle of the test plot, shown in Figure 4.10, note the continued deformation after the load has been reduced. It may have something to do with yield strength or confining load, but at this writing sufficient data have not been gathered for rational explanation.
FIG. 4.10 Typical curve showing increase in strain with decrease in load.
CONCLUSIONS

In conclusion, the plate loading test is a useful tool of rock mechanics which is well on its way to becoming one of the major diagnostic tests. The most universally useful loading test would be a rigid plate test with a 16-inch circular bearing plate loaded with a 400-ton ram. The following points are additional significant conclusions found in this paper.

1. The accuracy of the test is limited to ±10 percent of the modulus obtained. Any attempts to increase the precision beyond this limit during practical testing are presently a waste of time and money.

2. The rigid plate, borehole jack and cable jacking have the highest general benefit coefficient, and the pressure chamber and radial tests have the least general benefit coefficient with the flexible plate falling in between.

3. Point measurements are extremely sensitive to the assumptions of ideal material. A single point measurement of deformation from the loading of a large plate may produce a modulus significantly different from the average modulus of all the material under the plate and be useful in predicting the response of only the material under the measuring point.
4. Much usable information besides deformation moduli is obtained from plate loading tests.
BIBLIOGRAPHY


20. __________, 1969, Tocks Island Project, spillway—rock mechanics studies, p. 112.


