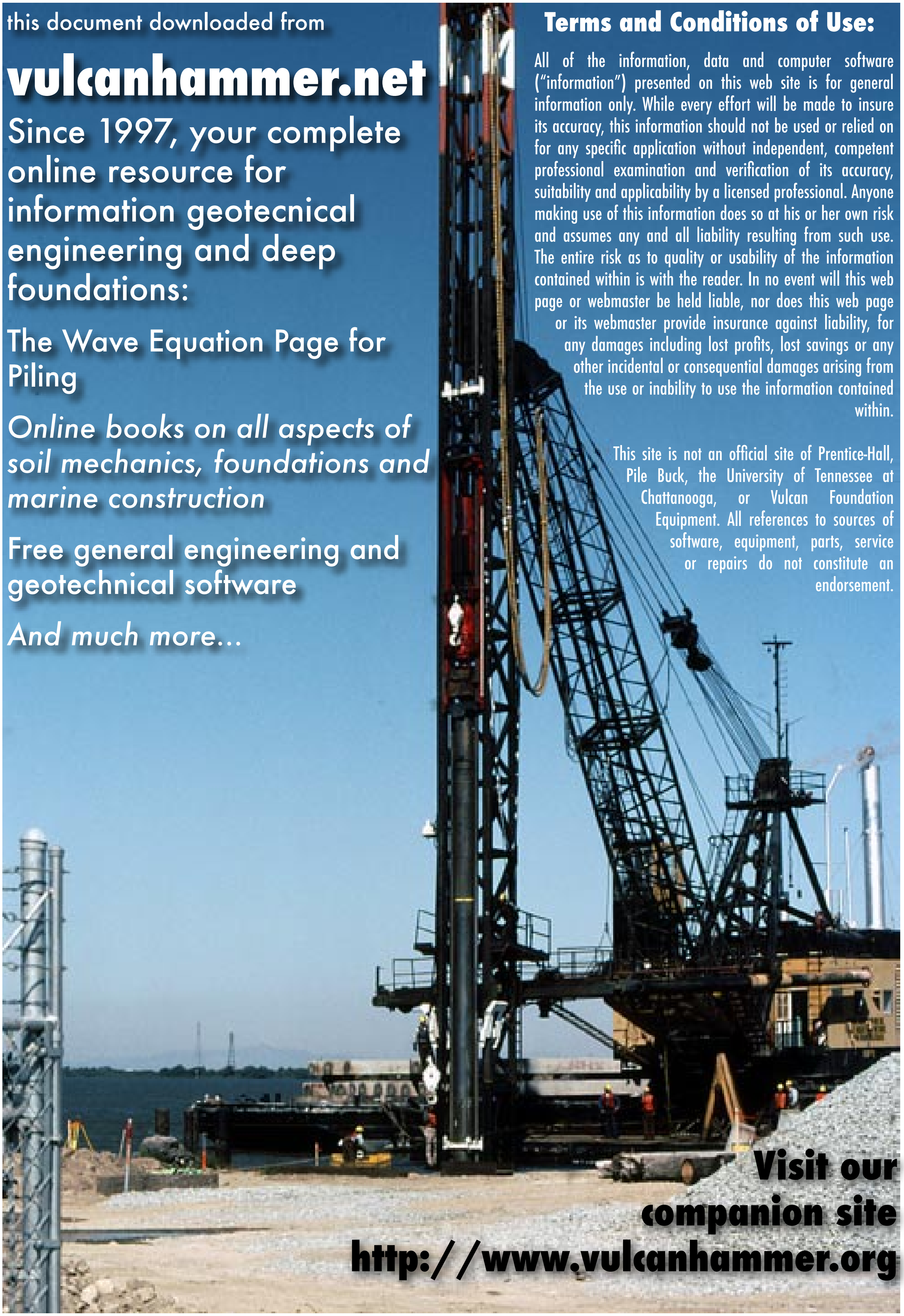

LITERATURE REVIEW OF ROCK PROPERTIES FOR ANALYSIS OF NAVIGATION STRUCTURES FOUNDED ON ROCK

by

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(ABSTRACT)

A review of behavioral rock properties used for input to the finite element method are summarized. Rock properties presented in the literature were primarily obtained from laboratory specimens. Methods to determine applicable field properties via testing, calculations and empirical correlations are included.

Suggested behavioral properties of the structural concrete-to-rock interface are proposed.

Specific property values, resulting from the literature review, are presented as input for a finite element parametric evaluation of navigation structures.

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I owe my parents and family much gratitude. Their moral support was instrumental to my success.

My fiancee, Missy, deserves special thanks. Without her love, understanding and friendship this work could not have been completed. To her I dedicate this theises.

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1.0 INTRODUCTION

Many U.S. Corps of Engineers navigation structures, constructed on rock, have been in existence for a considerable number of years. Throughout the lifetime of these structures, normal wear, tear and material degradation has occurred. Additionally, design standards have been modified as a result of advances in Civil Engineering.

Recognizing the need to reevaluate the stability of existing navigation structures, the Corps of Engineers established the program "Repair Evaluation Maintenance & Rehabilitation" (REMR). However, when conventional design standards were imposed on existing structures,in a number of cases only marginal safety factors were realized. Since the existing structures have performed well over time, the applicability of conventional design standards was questioned. To investigate possible discrepancies in conventional design, the finite element method was suggested for a comparative evaluation.

The conventional method of stability analysis considers the foundation as rigid and uses moment and force equilibrium to evaluate the potential for overturning and shear failure. Alternatively, the finite element method of stability analysis is an application of the direct stiffness method where the interaction between load and displacement is evaluated. In order to successfully apply the finite element method, the behavior of the rock foundation and structure - rock interface must be properly modeled.

The purpose of this report is to review the literature on rock foundation behavior as to general behavior trends and parameters which can define the behavior. In this work, it was assumed that two possible approaches would be used to model the foundation. First, the rock would be a linear elastic medium with single valued parameters. Second, it would be a nonlinear medium with possibly multi-valued parameters. The most suitable approach is a function of the degree of detail needed to accurately simulate the rock foundation. Additionally, the literature review addresses the question of modeling of, and selecting parameters for, the rock-structure foundation interface.

2.0 ROCK PROPERTIES - General

Modeling a rock foundation for a finite element evaluation includes consideration of the physical property, unit weight (γ), and its stiffness and strength.

Stiffness refers to the response of the rock to normal loading. Typically, this response is given by a stress-versus-strain relationship, where the ratio between stress and strain, at any point, is termed modulus. The stress-versus-strain response of rock can be characterized in one of three ways: elastic, strain-softening or strain-hardening (Figure 1). Many geologic materials display aspects of one or all of these types of behavior (Figure 2).

Elastic stress-versus-strain response indicates a constant modulus value. General elastic behavior is exhibited by unfractured basalt, diabase, gabbro, quartzite, very strong sandstone.

Strain-softening behavior is represented by an increase in incremental strain with each increment of load. Consequently, the modulus value decreases with increasing load. General strain softening behavior is exhibited by unfractured shales, siltstones, tuff, soft limestones, and by bedded coal loaded parallel to bedding.

In contrast to strain-softening, strain-hardening behavior is represented by a decrease in incremental strain with each increment of load, thus increasing the modulus value with increasing load. General strain-hardening is exhibited by unfractured sandstone, coal and other bedded rocks loaded normal to bedding, and rock salt.

The stiffness of a material is often typified by an averaged slope (modulus) of the stress-versus-strain curve corresponding to a certain range of stress. Stress dependency may, however, warrant the use of multiple modulus values.

The strength of rock is usually given by the Mohr-Coulomb strength parameters and/or the unconfined compressive strength. The Mohr-Coulomb strength parameters result directly from triaxial lab testing. As is true for stiffness, the angle of internal friction may either be averaged as a single value or represented as stress dependent.

2.1 EFFECT OF JOINTS ON ROCK PROPERTIES

In order to evaluate the properties of a foundation rock mass, the influences of joints and discontinuities must be considered. Of the rock properties used in the finite element method, only the parameter unit weight is considered independent of discontinuities. However, factors such as the spacing and attitude of joints with respect to loading, the condition of observed joints and the presence of groundwater can strongly influence deformation properties.

2.2 METHODS AVAILABLE FOR THE DETERMINATION OF ROCK MASS PROPERTIES

Available methods for evaluating the behavior of a specific rock mass include the use of field test results and/or lab test results corrected for the presence of discontinuities. Direct loading field tests make use of loading jacks and displacement gauges to record the response of the rock mass under field loading conditions. Field tests may also include the direct use of seismic wave velocities. Such tests, however, are often overly influenced by the presence of discontinuities, which are not representative of the entire rock mass (e.g., those created by blasting).

Alternative approaches do exist for the evaluation of rock mass behavior using lab test data and/or local geologic information. One such method uses an empirical scheme to relate the geologic classification and the intact rock properties (lab data) to rock mass behavior. Another uses intact rock properties and specific information on rock jointing, along with the theory of discontinua, to compute rock mass behavior.

3.0 INTACT ROCK-BEHAVIOR AND PROPERTIES

Results of laboratory tests to determine intact rock properties are readily available in the literature. Compilations of physical and behavioral rock properties are given by Deere (1966), Kulhawy (1975), and Lama and Vutukuri (1978) among others. Typically, the compilations present values of unit weight (γ) or density (ρ), Poisson's Ratio (ν), unconfined strength (σ_c), Mohr-coulomb strength parameters (c, ϕ) and modulus (E). Reported modulus values may be applicable to the initial tangent, the tangent value at 50% uniaxial compressive strength, or to a secant value at given percent of compressive strength. Compilations (Deere, 1966) also present tangent modulus values for various stress levels. Figure 3 presents an example of secant modulus, tangent modulus and initial tangent modulus.

Rock property values are generally presented as averaged parameters. For example, a single value of Poisson's Ratio is often assigned to a given rock, or rock mass, without regard to dependence on the state of stress. Additionally, even when stress levels are reported for the modulus value, the assigned value will be approximated as applicable for a broad range of stress.

3.1 TYPICAL INTACT ROCK PARAMETERS

From the compilations, typical values of unit weight (or density) range from 90 lbs/ft^3 (1.44 g/cm^3) for a pumice tuff (Kulhawy, 1975) to 256 lbs/ft^3 (4.10 g/cm^3) for quartzite (Lama and Vutukuri, 1978). More typical values are in the range of 120 lbs/ft^3 to 200 lbs/ft^3.

Poisson's Ratio values range from 0 to over 20 (Lama and Vutukuri, 1978). Values in excess of 0.5 indicate strong dilation - unusual for most rock masses. An averaged value of Poisson's Ratio, excluding dilatent values, is cited by Kulhawy (1975) as 0.20.

Extreme values in unconfined compressive strength can range from as low as 21.5 lbs/in^2 (0.148 MPa), for siltstone, to as high as 79,750 lbs/in^2 (550 MPa), for nephrite (Lama and Vutukuri, 1978). Since nephrite is an uncommon foundation rock type, an upper bound of 51,500 lbs/in^2 (360 MPa), for basalt (Kulhawy, 1975), is a more realistic value for a parametric study.

Mohr - Coulomb strength parameters, as determined by triaxial lab testing, range from c=0 for porous andesite, tuff, chalk and shale loaded parallel to bedding, to 22,520 lb/in^2 (155 MPa) for fine grained quartz diorite (Kulhawy, 1975). The angle of internal friction is reported to range from $\phi=0^{\circ}$, for very porous pyroxine andesite, to $\phi=64^{\circ}$, for basalt (Kulhawy, 1975).

Intact modulus values, without regard to definition (i.e., initial, tangent, secant), range from 3.7×10^{4} lbs/in^{2} (0.255 GPa), for claystone, to 3.0×10^{7} lbs/in^{2} (210 GPa), for eclogite (Lama and Vutukuri, 1978). Eclogite is, however, an uncommon rock. The more common granite gives values as high as 1.5×10^{7} lbs/in^{2} (100 GPa).

3.2 REPRESENTATION OF STRESS DEPENDENT BEHAVIOR

Because the stress-strain response of rock is dependent on the state of stress, Kulhawy (1975) suggests the use of intact rock parameters compatible with the Duncan and Chang (1970) stress dependent soil representation. In this approach, the tangent modulus of an elastic to strain-softening stress-strain curve ($Et=d\sigma/d\varepsilon$ at any point on a curve) is represented as a hyperbola of the form:

$$E_t = E_i \left[1 - \frac{(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_f} R_f\right]^2$$

in which E_t = tangent modulus, E_i = initial tangent modulus, $(\sigma_1 - \sigma_3)$ = mobilized deviator stress, $(\sigma_1 - \sigma_3)_f$ = failure deviator stress and R_f = failure ratio. R_f will always be less than or equal to unity and is considered independent of confining pressure (σ_3). High values of R_f indicate a high degree of nonlinearity. The failure stress is represented by the Mohr-Coulomb strength envelope as:

$$(\sigma_1 - \sigma_3)_f = \frac{(2c\ \cos\phi + 2\sigma_3 \sin\phi)}{(1-\sin\phi)}$$

The stress dependency of the initial tangent modulus is represented by the following relationship:

$$E_i = K\ P_a \left[\frac{\sigma_3}{P_a}\right]^n$$

in which K = modulus number, n = modulus exponent, σ_3 = confining pressure, and P_a = atmospheric pressure in consistent units. K and n are determined by plotting E_i versus σ_3 on log-log scales and fitting a straight line to the data. For additional information see Duncan and Chang (1970).

Representative intact non-linear parameters (K, n, R_f) are given by Kulhawy (1975) for various types of igneous, metamorphic and sedimentary rocks. The reported range of values for the hyperbolic parameters are: K = 0.1 to 1100, n = 0.1 to 1.2 and R_f = 0.2 to 1.0.

Information on the stress dependent representation of Poisson's Ratio is given by Kulhawy and Duncan (1975) and representative hyperbolic parameters are given by Kulhawy (1975).

4.0 ROCK MASS BEHAVIOR - DIRECT TESTING

Direct testing of a foundation rock mass is used to evaluate design parameters applicable to a discontinuous rock mass for major structures. Methods of direct testing include propagation of seismic waves, and in situ application of loading directly to the rock mass.

4.1 SEISMIC

Correlations between seismic velocity and rock properties are based on low strain elastic theory. Poisson's Ratio is computed as a function of the ratio of the primary wave to the shear wave velocity. The rock mass modulus is computed as a function of mass density, primary wave velocity and Poisson's Ratio. Deere (1969) presents the general solution for calculation of dynamic modulus and Poisson's Ratio. Deere (1969) also points out that the dynamic modulus is calculated from low stress states applied over a short time span, and that the strains used to compute dynamic modulus are very small. Consequently, the dynamic modulus is higher than the static field modulus.

4.2 IN SITU LOADING

A direct method of determining the deformation modulus of a discontinuous rock mass is the use of field load tests. In situ load versus displacement tests are advantageous because they test the rock mass under conditions similar to a full scale project. The cost for these tests, however, is high and the results can only be considered an estimation (Bieniawski, 1978). Difficulties in the interpretation of the results of in situ deformation tests may result from such factors as inconsistent site preparation, blasting effects, and/or the presence of anomalous discontinuities.

Many field loading test methods exist for the determination of static modulus. These include the plate bearing test, the flat jack test, the radial press test, the pressure chamber test and the borehole jacking test. From the results of these tests, the deformation modulus of the rock mass is determined by applying general elastic theory to the load-versus-displacement response.

As is the case for the behavior of intact rock, the load-versus-deformation response of a rock mass may indicate elastic, strain-softening, or strain-hardening behavior.

Figure 4 presents the results of two plate bearing tests on gneiss, which show both strain softening and strain hardening behavior. Figure 4 also presents modulus rosettes, which show graphically the solution of the elastic modulus equation (Shannon and Wilson, 1964, from Deere, 1969).

Table 1 presents a comparison between the in-situ test modulus and the laboratory modulus for a variety of projects. Note the degree of scatter in the data. A similar comparison, between the intact and field modulus values, is presented by Deere (1969). Based on data gathered from four sites, the ratio between the field and lab modulus was observed to range from 0.1 to 1.0. For a detailed discussion of the intrinsic errors and the associated problems of the various in situ methods, see Bieniawski (1978).

4.3 IN-SITU SHEAR STRENGTH

Similar to a laboratory direct shear test, large in-situ blocks are subjected to shearing forces. By using a normal load jack and a shear load jack, it is possible to evaluate the Mohr-Coulomb strength parameters. Test results are presented by Ruiz and Camargo (1966), Ishii et. al. (1966), Krsmanovic and Popovic (1966), and Dodds (1970), to

Table 1 Field and Laboratory Moduli: From Major Projects

Name of project, reference, rock type and date	Type of *in-situ* test	No. of tests	*In-situ* modulus, GPa Range	*In-situ* modulus, GPa Mean	Laboratory modulus, GPa Range	Laboratory modulus, GPa Mean	Remarks
Oroville Dam [23]	Plate bearing	5	8.3–12.4	10.4	74.5–105.0	89.0	Wide scatter in flat
Massive amphibolite	Tunnel relaxation	22	4.1–51.7	17.9			jack results yet rock
1961	Flat jacks	30	9.7–113.5	51.8			uniform
Tumut 2 [11]	Plate bearing	6	1.8–52.0	6.9	41.5–86.1	59.1	Large scatter in plate
Gneiss/granite	Tunnel relaxation	3	—	11.0			bearing results 30:1
1962	Flat jacks	6	34.5–83.0	57.5			
	Pressure chamber	2	13.8–20.6	17.7			
Poatina [43]	Flat jacks	?	16.6–22.1	20.6	31.0–45.0	34.5	
Mudstone							
1965							
Dworshak Dam [17, 18]	Plate bearing	24	3.5–34.5	23.5	—	51.7	G = original
Massive granite gneiss	Goodman jack G	14	11.6–18.6	14.5			results by
1966	S		16.5–36.4	23.6			Goodman [17]
	H		42.8–74.5	53.5			
Tehachapi Tunnel [17, 18]	Plate bearing	4	3.5–5.5	4.8	—	77.9	S = corrected
Fractured diorite gneiss	Goodman jack G	4	4.1–7.1	5.8			by Heuze and
1967	S		3.5–7.9	5.8			Salem [18]
	H		15.9–26.9	22.5			
Crestmore Mine [17, 18]	Plate bearing	2	12.0–18.7	15.0	—	47.5	H = same data
Massive marble	Flat jacks		12.4–20.5	12.4			recalculated by
1968	Goodman jack G		9.3–11.7	10.4			Hustrulid [24]
	S		11.7–17.0	14.0			
	H		36.5–46.2	40.9			
Turlough Hill [42]	Large flat jacks	4	9.6–40.2	29.2	8.0–20.2	15.0	
Granite							
1969							
Lake Delio [44]	Plate bearing	12	7.5–20.4	9.5	15.0–32.4	28.5	From 1966 to 1973,
Gneiss	Pressure chamber	20	9.7–26.2	18.2			at the Witbank-
1970							Breyten coalfields,
Gordon Scheme [45]	Plate bearing	8	—	19.0	38.0–91.0	67.0	S.A., 44 large scale
Quartzite	Dilatometer	2	—	25.0			*in-situ* tests were
1971	Tunnel relaxation	10	—	25.0			conducted on coal
	Flat jacks	16	28.0–96.0	58.0			pillars in compres-
							sion [7]. These tests
Churchill Falls [46]	Plate bearing	10	34.5–48.2	41.5	45.0–75.0	55.0	gave the *in-situ*
Massive gneiss							modulus $E_M = 4.0$
1972							GPa (2.9–5.0 GPa)
							while the laboratory
Waldeck II [47]	Plate bearing	?	3.0–7.0	5.0	—	20.0	modulus of the coal
Greywacke	Radial press		4.5–10.0	—			was $E_L = 5.2$ GPa
1973	Tunnel relaxation		—	15.0			(4.6–6.1 GPa)
Mica Project [48]	Plate bearing	12	8.3–48.3	27.6	24.5–32.0	27.0	
Quartzite gneiss	Flat jacks	19	—	28.8			
1974	Goodman jack	132	—	16.6			
Channel Tunnel [41]	Plate bearing	?	2.03–3.41	2.4	0.44–0.91	0.7	
Chalk							
1975							
LG-2 Project [49]	Plate bearing	?	38.0–60.9	50.0	—	80.0	
Massive granite							
1976							
Dinorwic [40]	Flat jacks	8	—	—	75.0–140.0	105.0	Flat jack tests
Slate	RQD index		—	50.0			unsuccessful
1977							

1 GPa = 1.45×10^5 psi

(From Bieniawski, 1978)

name just a few. A typical test device and the results from an in situ direct shear test (Dodds, 1970) are given in Figure 5. In Figure 6, Mohr-Coulomb strength parameters are given from a suite of in situ tests on discontinuous limestone (Krsmanovic and Popovic, 1966). In Figure 6, the friction angle (ϕ) is stress dependent.

5.0 ROCK MASS BEHAVIOR FROM EMPIRICAL METHODS

Due to the high costs, difficulties in interpretation, and concern about the reliability of in situ testing, empirical methods for the evaluation of rock mass behavior are more often used.

5.1 CORRELATION WITH ROCK QUALITY DESIGNATION

One of the earliest correlations made between rock condition and modulus was based on the Rock Quality Designation (RQD)* (Deere, 1969). The empirical correlation between RQD and rock mass modulus is based on a best fit line through data points of known RQD and field

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* The RQD is the ratio of cumulative length of NX core greater than 4 inches to attempted length of core drilled.

determined modulus values for four job sites. Results of this correlation are scattered (Figure 7). However, by evaluating the RQD, the modulus can be estimated without lab or field testing. An approximation of the rock mass modulus can be obtained at a small cost since standard coring is used for most rock foundation studies.

Another correlation between RQD and rock properties involves the use of a modulus reduction factor, which is defined as the ratio of the field modulus to the lab modulus. When case studies are correlated, less scatter is observed than with the use of RQD alone. Bieniawski (1978) and Deere (1969) present relationships between the modulus reduction factor and the RQD. Figure 8 presents the correlation by Bieniawski (1978). He warns, however, that in a number of case projects, the RQD approach was either impossible to apply or resulted in misleading field modulus values.

5.2 GEOMECHANICAL CLASSIFICATION PROCEDURES

Originally developed for estimating stand-up time in tunnel design the Geomechanics Classification (RMR) System also gives an approximation of Mohr-Coulomb strength parameters (Bieniawski, 1974). The Rock Mass Rating (RMR) has also been correlated empirically to the deformation modulus of a rock mass.

The Geomechanics Classification is based on summing numerical values relating to the quality or condition of six rock mass parameters. These parameters are: 1) Point load or uniaxial compressive strength, 2) Drill core quality (RQD), 3) Discontinuity spacing, 4) Condition of discontinuities, 5) Presence of groundwater, and 6) Attitude of joints with respect to loading. A classification rating is obtained from the summation of the numeric designations for each of these six categories. Bieniawski (1974) presents five categories ranging from "very poor" to "very good" which result from the RMR rating. These categories estimate Mohr-Coulomb strength parameters ranging from $\phi < 15^{o}$, $c < 14{,}500$ psi (100 kPa) to $\phi > 45^{o}$, $c > 58{,}000$ psi (400 kPa). Figure 9 presents the classification.

Recognizing the problem of comparing RQD to in situ modulus or modulus ratio, Bieniawski (1978) compared the Rock Mass Rating to in situ modulus values determined from field load tests. The resulting correlation is presented on Figure 10. It yields the equation EM = 2RMR - 100, where Em is the rock mass modulus expressed in GPa (1GPa = 1.45×10^{5} lb/in^{2}). The prediction error of this relationship is approximately 18%. The proposed relationship, however, is only suitable for values of RMR greater than 50.

5.3 CORRELATION WITH SEISMIC WAVE VELOCITY

In contrast to the application of small strain theory to the evaluation of the rock mass modulus, empirical correlations between seismic velocity and field load tests are available. One such method is the "Petite Sismique" technique, which specifies the amount of input energy in terms of a sledgehammer falling one meter. Using the amplifier gain controls to generate a wave form of constant amplitude, the frequency, based on the first one-half wave length, is evaluated as:

$$f = [2\ (t_b - t_a)]^{-1}$$

where f is in hz, and t_a and t_b are the times to the first positive and negative peaks, respectively. Using a linear correlation with field load test data yields the relationship:

$$E_m = 0.056\ f - 9.2.$$

where f is in hz, and E_m is in GPa. A presentation of the "Petite Sismique" technique is given by Bieniawski (1978). Although the empirical correlation fits the existing data base well, additional field application will further verify the suggested relationship.

6.0 ROCK MASS BEHAVIOR - GEOMECHANICAL MODELS

There are two geomechanical models which can be used for the calculation of rock mass modulus using intact rock modulus, discontinuity spacing, and stiffness. The discontinuity normal stiffness (K_n) is the slope of a compression load versus deformation curve for a discontinuity. The value of K_n (in units of F/L^3) can be determined from compression tests on discontinuities. Table 2 presents a summary by Kulhawy (1978) of discontinuity normal stiffness values from reports by Goodman (1968, 1972) and Mahtab (1970).

6.1 DUNCAN AND GOODMAN GEOMECHANICAL MODEL

The Duncan and Goodman (1968) model is based on the rock mass system and geometry, as shown in Figure 11. The rock mass, characterized by the modulus E_r, is transected by joints with equal spacing of S_i. Each joint has a normal

TABLE 2 Discontinuity Normal Stiffnes Values

Rock Type	K_n * (GPa/m)	E_r ** (GPa)	E_r/K_n (m)
- Boise Sandstone With Dry Rough Sawed Joint	35.1	7.46	0.21
- Marly Sand Filled Joint	1.96	___	___
- Closely Jointed Shale Zone in Limestone	0.24	___	___
- Wet Shale Interbed	0.26	___	___
- Lyon's Sandstone w/Clay Joint	5.59	2.48	0.44
"	5.40	"	0.46
"	5.40	"	0.46
"	5.43	"	0.46
- Sierra White Granite With	5.21	22.06	4.23
Clay Joint	16.91	"	1.30
"	67.59	"	0.33
"	7.22	"	3.06

* Normal Stiffness
** Intact Rock Modulus

1 GPa/m = 3684 lbs/in^3

1 meter = 39.37 in

(from Kulhawy, 1978)

stiffness of k_{ni}. The resulting rock mass modulus is given by the equation:

$$E_{mi} = \left[\frac{1}{E_r} + \frac{1}{S_i K_{ni}}\right]^{-1}$$

Changing the subscript i to follow axes x, y, or z allows calculation of the rock mass modulus in three directions.

The Poisson's Ratio of the rock mass is given by:

$$\nu_{ij} = \nu_{ik} = \nu_r \frac{E_{mi}}{E_r}$$

where:

ν_r = Poisson's Ratio of intact rock

E_i = Rock mass modulus in the i direction

E_r = Intact rock modulus

i = x, y, and z respectively

j = y, z, and x respectively

k = z, x, and y respectively

6.2 KULHAWY MODIFIED METHOD

While the Duncan and Goodman (1968) model is accurate, the use of the parameter S_i makes it somewhat impractical. Recognizing the need to incorporate more accessible parameters into the assessment of rock mass modulus, Kulhawy (1978) modified the Duncan - Goodman model.

In order to evaluate the rock mass modulus (E_m), Kulhawy (1978) proposed the use of RQD, coupled with a laboratory derived assessment of E_r/K_n (intact modulus to discontinuity stiffness ratio, in meters). Table 2 presents values of E_r/k_n (Kulhawy, 1978). Figure 12 presents the modulus reduction factor (E_m/E_r) versus RQD for various ratios of E_r/K_n.

To use an example, given a granite with E_r = 22 GPa, RQD = 50% and E_r/k_n = 0.5m, the resulting modulus reduction factor is 0.12 (see Figure 12). The field modulus is therefore taken as 2.6 GPa (3.7 x 10^5 psi).

7.0 RANGES OF KEY PARAMETERS FOR ROCK MASSES SUBJECTED TO FOUNDATION LOAD.

One of the objectives of this investigation is to establish bounds for key rock mass parameters, in order to guide selection of reasonable values for analytical studies of navigation structures founded on rock. The information presented previously in this report is used in the following paragraphs for this purpose.

7.1 UNIT WEIGHT

As noted earlier, all evidence suggests that the unit weight of intact and jointed rock are, for practical purposes, the same. Typical values determined from the review fall between 120 pcf and 200 pcf.

7.2 MOHR COULOMB STRENGTH PARAMETERS (PHI, C)

One of the simplest ways to obtain the range of Mohr-Coulomb strength parameters is to use the information provided with the RMR classification system proposed by Bieniawski (1974). This approach suggests that the cohesion and the friction bounds are related. Thus, if the cohesion is low, the friction angle will also be low.

There are five classification categories in the RMR system. On the low end, cohesion is 14,500 psi or less, and friction angle is 15 degrees or less. At the high end, cohesion is 58,000 psi or more, while the friction angle is 45 degrees or more. Based on the results of a rock core exploration, the RMR method will yield a rating which relates to the strength parameters of "very poor" to "very good" rock with three intermediate categories.

7.3 MODULUS AND POISSON'S RATIO

There are a wider number of means for determining ranges of modulus for rock masses than for strength parameters. This is true partly because the data base is larger, and in part because more attention has been directed towards the parameter. In Table 3, ranges are provided using four different sources:

1.) Field load tests (Bieniawski, 1978).
2.) The RMR method (Bieniawski, 1978).
3.) Petite Sismique correlations (Bieniawski, 1978).
4.) Geomechanical model calculations (Duncan and Goodman, 1968; Kulhawy, 1978).

While values for the first three of these methods are essentially just abstracted from tables, those for the fourth are calculated, and deserve some brief comment. The upper bound for the modulus in the geomechanical model approach is achieved when the rock is free of joints and is of high quality. Under such circumstances, the modulus of the rock mass is simply that of the intact rock. As noted earlier in this text, a reasonable upper bound for intact rock is 1.5×10^7 psi. This upper bound is somewhat larger

TABLE 3 Range of Field Modulus From Various Sources

Source	Range of E_{field} lbs/in^2 (GPa)
Field Load Tests	260,000 (1.8) - 16,500,000 (113.5)
RMR-Method	1,450,000 (10) - 10,200,000 (70)
"Petite Sismique"	300,000 (2) - 6,500,000 (15)
Geomechanical Model	10,000 (0.07) - 15,000,000 (103)

$$1 \text{ GPa} = 1.45 \times 10^6 \text{ lbs/in}^2$$

than that indicated by other information sources, although there is general agreement that an upper bound modulus in the range of 1.0×10^7 psi is appropriate.

The value of the lower bound of the modulus range for the geomechanical model can approach zero in the extreme, since it can be assumed that jointing is pervasive, is closely spaced, and that the joints have very low stiffness values. This is not, however, a realistic assumption, particularly because sites with such conditions would not be used "as is" for the foundation of a navigation structure. In order to arrive at a more reasonable lower bound, the following approach is used:

1.) The rock is assumed to have an RQD value of 25%. This would be very poor to poor rock, according to Deere (1969).

2.) The intact rock modulus is taken as 1.7×10^5 psi, which is a lower bound, given by Kulhawy (1975).

3.) The value of the parameter E_r/k_n, is taken as 13.7 ft (4.2 m), which leads to a modulus reduction factor of 0.05 (Kulhawy, 1978).

4.) Multiplying the intact rock modulus from Step 3, by the modulus reduction factor of Step 4, a lower bound value of rock mass modulus of 8,700 psi is obtained. For practical purposes, this can be assumed to be 10,000 psi.

The lower bound obtained by the geomechanical model calculation is one order of magnitude lower than that indicated by any of the other three information sources (see Table 3). As a frame of reference, the geomechanical model lower bound modulus is only about one order of magnitude larger than that obtained for a competent soil. This bound would thus appear to be as low as should be gone in representing possible poor rock conditions. Additional support for the geomechanical lower-bound is suggested by the RQD correlation (Deere, 1969). Although the rock mass modulus values are shown only for RQD values greater than 30%, interpolating the relationship for RQD = 25% indicates a modulus value of approximately 10,000 psi (0.07 GPa). Using the value of 10,000 psi as a representative modulus for "very poor" rock, a reinterpretation of Bieniawski's E_m vs RMR relationship is extended for a range in RMR from 10 to 50, and is shown in Figure 13.

Techniques for field determination of Poisson's Ratio values for rock masses are virtually nonexistent. Fortunately, the range of values for this parameter is relatively well defined. It is common to assume values for rock between 0.1 and 0.5. If Poisson's Ratio is less than 0.5, it is a compressible material, and will contract in volume upon application of the load. If Poisson's Ratio is 0.5, the material is incompressible, and its volume will not change under load. Kulhawy (1975) presents a summary of the values for intact rock that have been published in the literature, and, excluding extreme cases, the average Poisson's Ratio is 0.2. Values of intact Poisson's Ratio are also cited by the Corps of Engineers (Appendix A), the average value being 0.19. Using the Duncan-Goodman geomechanical model concept, the intact rock Poisson's Ratio should be multiplied by a modulus reduction ratio in order to obtain a value applicable for a rock mass. Using a modulus reduction ratio of 0.75 - applicable for average rock conditions - we obtain a rock mass Poisson's Ratio of 0.15.

8.0 STRESS DEPENDENT REPRESENTATION OF ROCK MASS

It is not uncommon to see results showing nonlinear behavior for intact rock or rock masses. Many alternatives exist in an analysis that would allow for this response. One might simply perform an analysis using a single value for modulus, where the modulus is selected to average the nonlinear response over the load range of interest. An alternative, nonlinear elastic approach could be used where the modulus of the rock mass is adjusted under the action of successive load increments. Finally, some form of elasto-plastic model could be used with a plastic formulation designed to induce a nonlinear response.

One of the first two options would be likely to be acceptable as a solution since, for the navigation structure foundation, the stresses in the rock will be in a working range well below failure. The latter alternative, however, presents difficulties in obtaining parameters for the rock.

Should a nonlinear analysis be required, Kulhawy (1975) has suggested that, for intact rock, the nonlinear elastic model developed by Duncan and Chang (1970) for soil can be used. Kulhawy also provides parameters for the model for various types of rocks. This approach, however, is limited to cases where the stress strain curve can be reasonably approximated by a hyperbola, and where the rock is intact.

In most instances, of course, the foundation of a navigation structure will not be intact rock, but rather a jointed medium. In such cases, there are two methods that may be used if a nonlinear response is to be included. Some field load tests show rock masses to have a load response which can still be modeled using a hyperbolic approach. It may therefore be possible to use the Duncan-Chang model for this case, and to calibrate it by a trial and error fitting between predicted and observed behavior.

Alternatively, a nonlinear model could be developed using the Duncan-Goodman (1968) geomechanics model concept as a framework. In the original version for this approach the individual components, i.e., the intact rock and the joints, were assumed to have linear responses. It is possible however, in the context of a finite element analysis, to allow each component to have a behavior which is stress dependent, each following its own set of controls. At each stage of loading, the finite element analysis would check the governing equations for the intact rock and for the joints, to allow for an updating of the modulus or stiffness. Depending on the relationships in the modeling, the rock mass could generate either a hardening a or softening of the response as the load is applied.

The importance of including a nonlinear response in analytical studies for a navigation structure foundation has not been established. If there is a large effect of the modulus in parametric studies using a variety of rock foundation modulus values, it can provide an index for this kind of information. If the modulus is important, this suggests that the incorporation of variations of modulus during the loading could influence the predicted response. On the other hand, if the value of the rock foundation modulus is not important, modeling of variations of the modulus during loading should not be important.

9.0 EVALUATION OF THE CONCRETE-TO-ROCK INTERFACE OF THE NAVIGATION STRUCTURE FOUNDATION

The foundation interface at the bottom of the navigation structure is an important element when considering possible sliding of the structure on the rock. The interface behavior is a function of the interaction of the concrete of the structure with the top of the rock foundation. Conventional analyses focus on the strength of this interface. In finite element analyses, the interface strength is still important, but this approach also allows consideration of the effects of the stress-deformation response of the interface. This can be characterized in terms of stiffness, both normal to the interface, k_n, and tangential to it, k_s. Both stiffnesses have the same meaning and definition as those used for describing the behavior of rock joints.

9.1 STRENGTH OF THE CONCRETE-TO ROCK INTERFACE

Surprisingly, little hard information exists on the concrete-to-rock interface extant in the navigation structure environment. Most of the data for this review were found in reports prepared by Corps of Engineers personnel for specific project evaluation of concrete-to-rock interfaces prepared in the laboratory, or in some cases, actual interfaces cored from navigation structure foundations (Thorton, et al., 1981; Stowe and Pavlov, 1961; Stowe, 1978; Stowe and Warringer, 1975; and Pace, et al., 1981).

Laboratory tests performed by the Corps of Engineers include the strength evaluation of pre-cut concrete-to-rock, and bonded concrete-to-rock. Pre-cut samples were prepared in the laboratory and sheared to obtain Mohr-Coulomb strength parameters (See Stowe, 1978; Stowe, et al., 1980; Stowe and Pavlov, 1981; Pace, et al., 1981; and Appendix A). Bonded concrete-to-rock samples were obtained from a rock core exploration and sheared to obtain Mohr-Coulomb strength parameters (Stowe, et al., 1980; Stowe, 1978; Stowe and Warringer, 1975; and Stowe and Pavlov, 1981; and Appendix A). Shear stress versus deflection response is provided by Thorton, et a.l (1981), and Stowe and Warringer (1975) for three bonded concrete-to-rock samples, as shown on Figures 14-16.

Mohr-Coulomb strength parameters for the bonded concrete-to-rock interface reported by the Corps of Engineers range from c = 1.6 tsf, for concrete-to-dolomite, to c = 19 tsf, for concrete-to-limestone. Peak friction angles are reported to vary from 49°, for concrete-to-shale to 78°, for concrete-to-dolomite. Reinterpreted raw data from Corps of Engineers direct shear tests on bonded concrete-to-rock yielded residual friction angles of 38° to 57° (Figures 14, 15, and 16). Residual friction angles obtained from tests on precut samples are reported to range from 20°, for concrete-to-shale to 30°, for concrete-to-limestone. As the precut interface does not correctly model the actual structural interface, this data is considered very conservative. Appendix A includes concrete-to-rock interface strength parameters used in the design of various referenced Corps of Engineers projects.

Additional information on the shape of the Mohr-Coulomb strength envelope for a concrete-to-limestone interface is given by Krsmanovic and Popovic (1966). Figure 17 shows the Mohr-Coulomb strength envelope for five tests conducted on in-situ limestone capped with concrete (Krsmanovic and Popovic, 1966). Note, in Figure 17, the stress dependency on the frictional angle, ϕ.

Some studies of the concrete-to-rock interface have been made in work on drilled pier foundations where the drilled piers were pushed or pulled from the ground (Horvath, 1978). However, these data cannot be put into the same context of the direct shear tests since the lateral stress on the interface is not known, and the rock mass around the pier takes part in the deformation of the system. Thus these results are not considered applicable to this investigation.

9.2 CONCRETE-TO-ROCK INTERFACE STIFFNESS

As noted previously, there is relatively little information on the testing of the concrete-to-rock interface. This becomes a problem when considering deformation response, since most of the available tests are performed with the sole intent of obtaining the strength of the interface. Thorton et al. (1981), and Stowe and Warringer (1975), however, provide test results from three direct shear tests on the bonded concrete-to-rock interface. Based on these test results, the shear stiffness (k_s) of the bonded concrete-to-rock interface is evaluated. Figures 14, 15, and 16 present the shear stress versus deflection response for the projects referenced. The test results generally indicate a linear k_s relationship and brittle failure beyond peak strength. Additionally, the mobilization of peak strength is at a very low displacement.

As with rock joints, it is possible to characterize the stress-deformation response of the interface in terms of the normal and shear stiffness. Only the shear stiffness can be obtained from the tests results in Figures 14-16 since no information is given about the response when normal stress is applied to the interface.

The shear stiffness, k_s, is determined as the slope of the shear stress-shear deformation plots. For all of the tests on bonded interfaces, the shear stiffnesses range from 10,000 to 30,000 lbs/in^3. Shear stiffness values applicable to precut interfaces are not provided by the Corps of Engineers' Reports. By way of comparison, Kulhawy (1978) gave an average k_s for interface tests on rock joints of 10,400 lbs/in^3 (2.82 GPa/m) (see Table 4 of this report).

The lack of information on the response of the concrete-to-rock interface under normal loading necessitates the use of judgment in selecting values for normal stiffness. It is possible to draw some useful information from tests on rock joints. In earlier discussion of this subject, it was pointed out that the preponderance of results suggested a value of 30,000 lbs/in^3 (8.14 Gpa/m) for a variety of rock joints (see Table 2 of this report). It would, however, seem reasonable to expect the normal stiffness for a concrete-to-rock interface to be higher than this for a number of reasons. First, a rock interface often contains foreign matter, which can be compressible, e.g., clay or gouge. The concrete-to-rock interface should, however, be cleaned during foundation preparation activities. Second, the interface surfaces in the rock interface may only be in contact over small areas, due to asperities or saw-tooth effects. The concrete is poured directly onto the rock while in a fluid condition, and placed carefully so as to insure the best possible contact.

Table 4 Discontinuity Shear Stiffness Values

Test (1)	Test designation (2)	Specimen description (3)	Discontinuity: Thickness, in centimeters (4)	Discontinuity: Area, in square centimeters (5)	Shear Stiffness, K_s, in giganewton per cubic meter: Range (6)	Shear Stiffness, K_s, in giganewton per cubic meter: Average (7)
DLL-1[a]	10-14.6 run 3—University of Illinois	Berea sandstone—dry, sawed joint	—	82	—	29.80
DLL-2	6-14.7 run 7—University of Illinois	Limestone—dry, sawed joint	—	82	—	8.73
DLL-3	QP2—University of California, Berkeley	Boise sandstone—dry, rough saw cut	—	5	—	1.29
DLL-4	Blackingstone—Imperial College	Granite—dry, rough joint from breaking beam	—	144-205	0.99-1.57	1.32
DLL-5	Delgado—Imperial College	Slate—dry, natural cleavage surface	—	500	—	0.79
DLL-6	Vouglans Dam	Limestone—oolitic, compact to stylolitic	—	575	1.68-4.61	2.78
DLL-7	Larouviere—1965	Marl layers in limestone—saturated	0.1-0.3	605-730	0.51-3.73	2.13
DL-1[b]	Larouviere—1964	Marly partings in limestone—saturated	0.1-5.0	51-63	0.85-5.69	2.89
DL-2	Vouglans Dam	Limestone with marly joints—dry	0.025-2.0	28-47	2.26-23.55	9.75
DL-3	L'Aquila Meridional Apennine	Sandstone—marl contact	—	—	0.11-0.17	0.15
DL-4	Bresica Meridional Prealps	Phyllitic schist fractures	—	—	0.14-0.40	0.22
DL-5	B-2 Sarajevo	Limestone—slightly rough bedding	—	1,500	0.20-1.37	0.84
DL-6	B-3 Sarajevo	Limestone—rough bedding surfaces	—	1,600	0.25-7.35	3.06
DL-7	D-2 Sarajevo	Limestone—rough unfilled fractures	—	1,600	1.26-2.61	1.98
DL-8	Malpasset MII6	Foliated gneiss and mylonite	4.0-5.0	10.6-24.5	0.69-3.69	2.36
DL-9	Delgado—Imperial College	Porphyry—dry, natural joint surface	—	500	0.26-1.94	1.02
DL-10	B-1 Sarajevo	Limestone—mylonite along bedding	—	1,500	0.28-5.69	1.25
DL-11	Vouglans Dam	Moist marly joint in limestone	1.5-3.0	980-1,243	0.41-4.71	1.70
DL-12	B-1 Sarajevo	Limestone—thin shale seams along bedding	—	1,500	0.32-8.33	3.08
DL-13	Vouglans Dam	Marly joint—saturated	1.3-3.2	1,030-1,240	0.02-1.86	0.78
DL-14	D-1 Sarajevo	Limestone—smooth unfilled fractures	—	1,600	0.20-1.28	0.51
DL-15	Torino Gran Paradiso Massiccio	Granitic gneiss fractures	—	—	0.09-0.12	0.11
DL-16	Vouglans Dam	Limestone with marly joints—saturated	0.025-0.2	24-40	0.14-31.60	7.41
DIS-1[c]	1P Krivoklat	Bedding plane in greywacke	0.5-0.8	2,265	—	0.23
DIS-2	2P Krivoklat	Bedding plane in greywacke	>0.1	3,510	—	1.21
DIS-3	3P Krivoklat	Bedding plane in greywacke	Closed, clean	3,660	—	2.26
DIS-4	Vouglans Dam	Marly sand filled joint	0.1-0.2	44,000	—	2.34
DIS-5	K1 Kurobe IV Dam	Vertical fault	Thick, uneven	—	0.17-0.23	0.20
DIS-6	SLL Khajuri Dam	Closely jointed shale zone in limestone	0.2-0.5	50,000	0.01-0.02	0.02
DIS-7	SLS Khajuri Dam	Shale interbed—wet	0.2-0.5	50,000	0.01-0.02	0.02
DIS-8	C-2 Dolesice Dam	Schistosity plane in amphibolite	—	5,000	—	0.59
DIS-9	Jupia Dam	Unbonded basalt—sandstone contact	—	307,900	—	0.11
	Overall summary or average		Closed—5.0	5-307,900	0.01-31.60	2.82

[a]DLL series—laboratory direct shear tests over limited stress range.
[b]DL series—laboratory direct shear tests.
[c]DIS series—in-situ block direct shear tests.
Note: 1 GN/m³ = 3,684 lb/cu in.

(From Kulhawy, 1978)

The normal stiffness for the concrete-to-rock interface, thus, can reasonably be expected to easily exceed 30,000 lbs/in^3 (8.14 GPa/m).

It is suggested that the significance of this parameter be studied in the finite element analyses, using the figure of 30,000 lbs/in^3 as a lower bound.

10.0 PARAMETER VALUES RECOMMENDED FOR STUDIES OF NAVIGATION STRUCTURE FOUNDED ON ROCK

This review attempts to document the likely ranges of key parameters for a rock foundation serving to support a navigation structure. A summary of the findings is presented in this section with emphasis on values that should be considered in finite element studies of the navigation structure problem.

10.1 ROCK FOUNDATION MODULUS

The review showed that rock mass modulus values, from a lower bound of 10,000 psi to an upper bound of 10,000,000 psi, could reasonably be anticipated in the field. The lower bound represents a relatively soft rock with closely spaced fractures. The upper bound represents unfractured granite. the upper bound modulus, as described by the

Bieniawski (1978) RMR method, would apply to a "good" or "very good" rock mass.

While analyses representing the rock by a single modulus value should consider the two extremes of this parameter, neither is appropriate for average conditions. It is logical to represent the average case with an intermediate modulus. For this purpose, a value of 3,000,000 psi is recommended; a value rated as a "fair" rock by the RMR classification procedure.

The literature review also showed that the response of a rock mass is, in some cases, nonlinear. While it is not apparent that it is necessary to provide a model of this behavior to ensure a good prediction in a finite element analysis, several procedures are available to simulate the nonlinear response, if necessary. It is suggested, however, that the sensitivity of the solution to variations in modulus be examined. If the results are not influenced by modulus in the linear analyses, it is unlikely that precise modeling of a nonlinear rock stiffness is worthwhile.

10.2 POISSON'S RATIO

Neglecting extreme cases, the Poisson's Ratio value applicable to rock does not vary over a wide range. Further, the exact choice of a value of this parameter will not significantly affect the outcome of the analyses. It is therefore suggested that a value of 0.15 be used, since as this is representative of an average rock mass response.

10.3 CONCRETE-TO-ROCK INTERFACE

The behavior of the concrete-to-rock interface is characterized in terms of the Mohr-Coulomb strength parameters, and the shear and normal stiffnesses. The literature review showed that there are very few test results for this type of interface. Based on the available data for bonded surfaces, the average peak friction angle for the interface is 60° to 65° (see Figures 14-16 and Appendix A). Residual friction angles taken from tests on bonded surfaces are shown in Figures 14-16, and range from 38° to 57°.

While cohesion will exist where the concrete is bonded to the rock, this is usually neglected in design since it is uncertain whether the bond will remain in force over the design life of the structure. Where philosophy is adopted in analyses for this work, it should be remembered that it introduces an element of conservatism.

Test results that allow direct determination of the shear stiffness of the concrete-to-rock interface are very limited, and no results exist for loading normal to the interface. From the available shear tests with deformation data, a shear stiffness of 10,000 lbs/in^3 represents a reasonable value. A lower bound on the normal stiffness is set by tests on rock interfaces. These suggest a value of 30,000 lbs/in^3. Some effort should be made to check the degree of influence of this parameter, since it is clear that the normal stiffness of the concrete-to-rock interface is higher than this value. It is not obvious however how much larger it should be.

11.0 SUMMARY

The evaluation of navigation structures founded on rock can be accomplished by either conventional stability analysis or finite element analysis. Although conventional stability analysis may yield overly conservative results, it is not dependent on foundation behavior. The finite element analysis, however, must include the behavioral properties of the foundation. Therefore, the foundation model for finite element analysis must consider the rock type, the integrity of the rock mass, and the strength and behavioral properties of the structure-foundation interface.

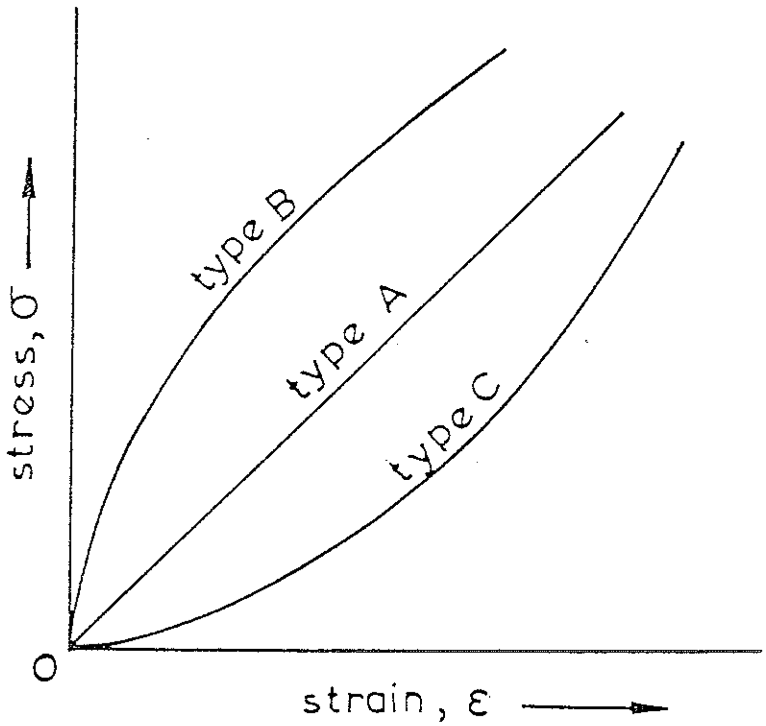


Figure 1 General Stress Versus Strain Response

Type A: Elastic Behavior
Type B: Strain Softening Behavior
Type C: Strain Hardening Behavior
(After Lama and Vutukuri, 1978)

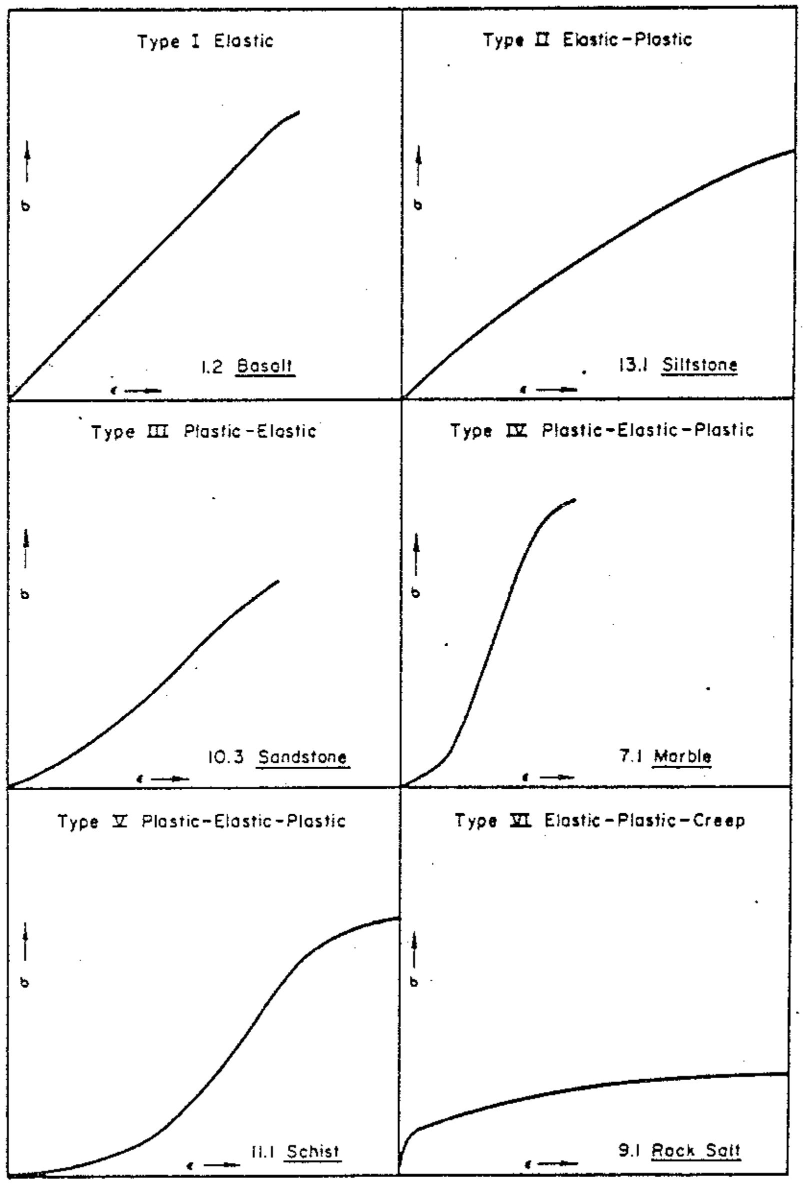


Figure 2 Typical Stress Versus Strain Response for Intact Rock (From Deere, 1966)

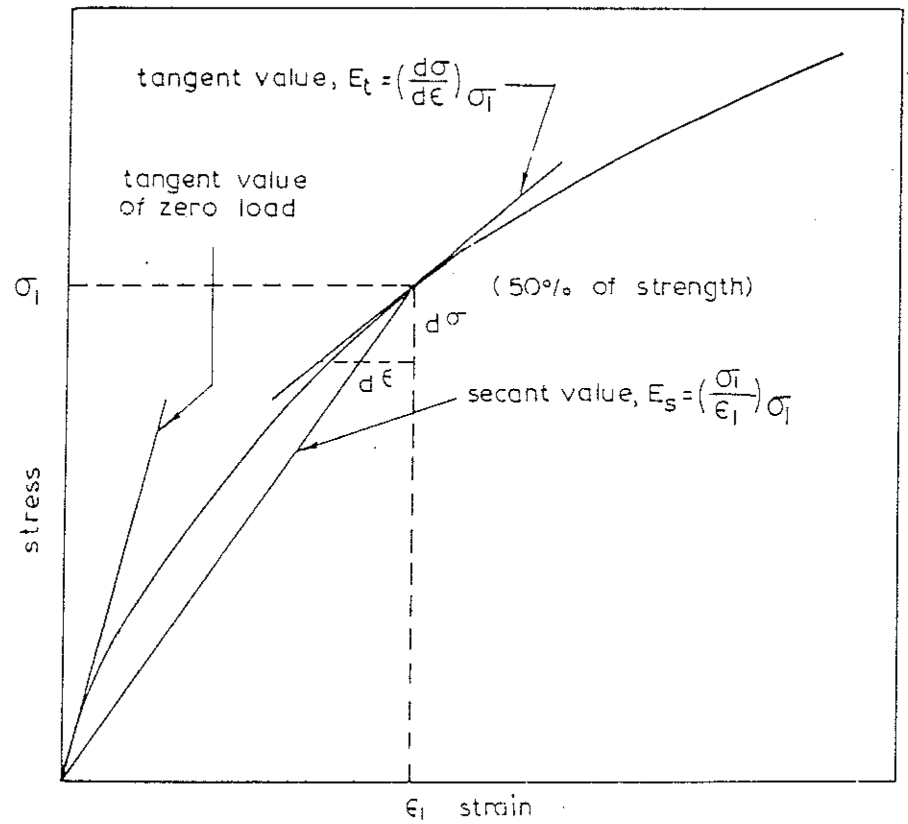
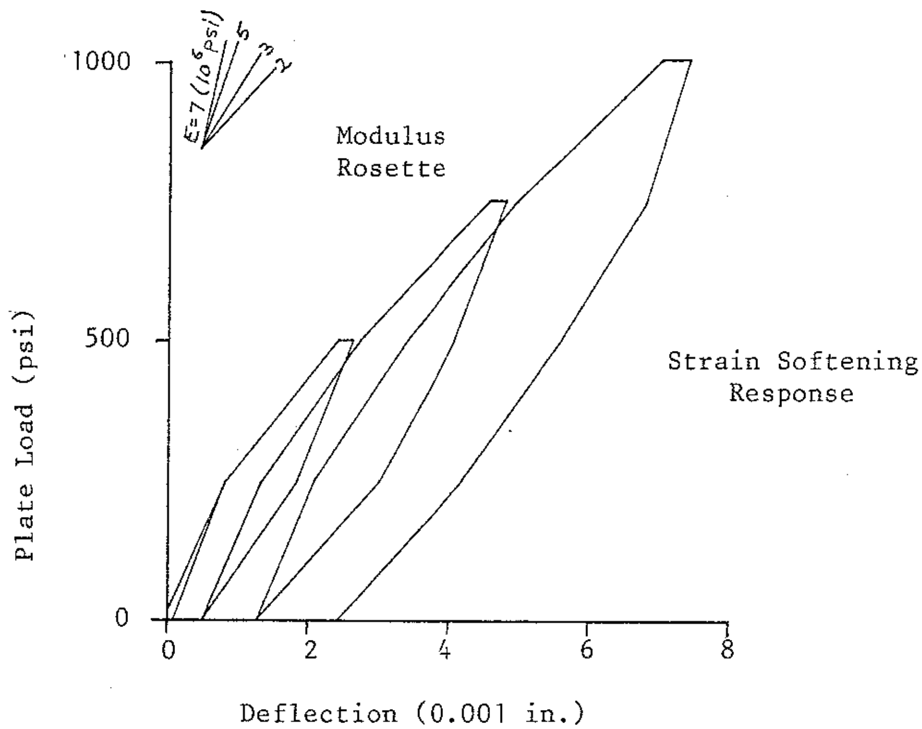


Figure 3 Methods of Representing Modulus
(From Lama and Vutukuri, 1978)



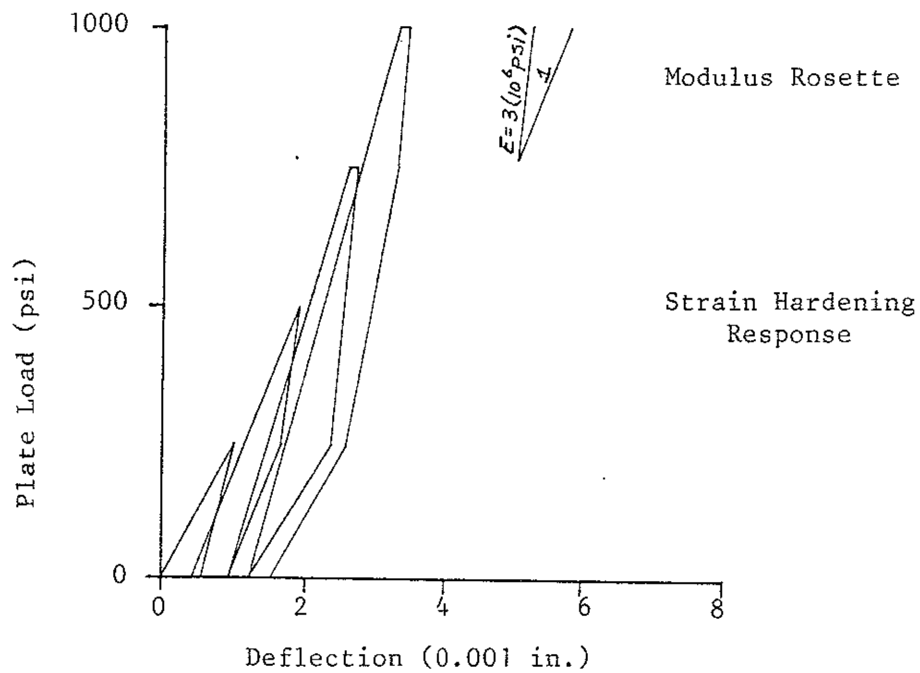
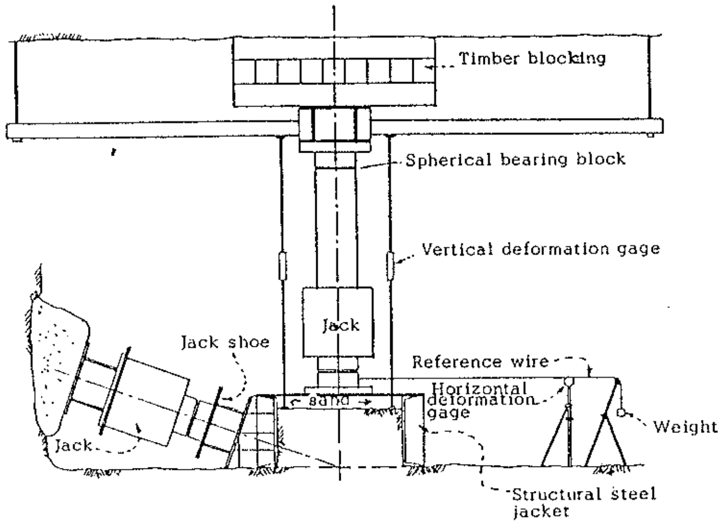


Figure 4 Stress Versus Deflection from Field Load Tests. Dworshak Gneiss (Shannon and Wilson, 1964, from Deere, 1969)



Test Device

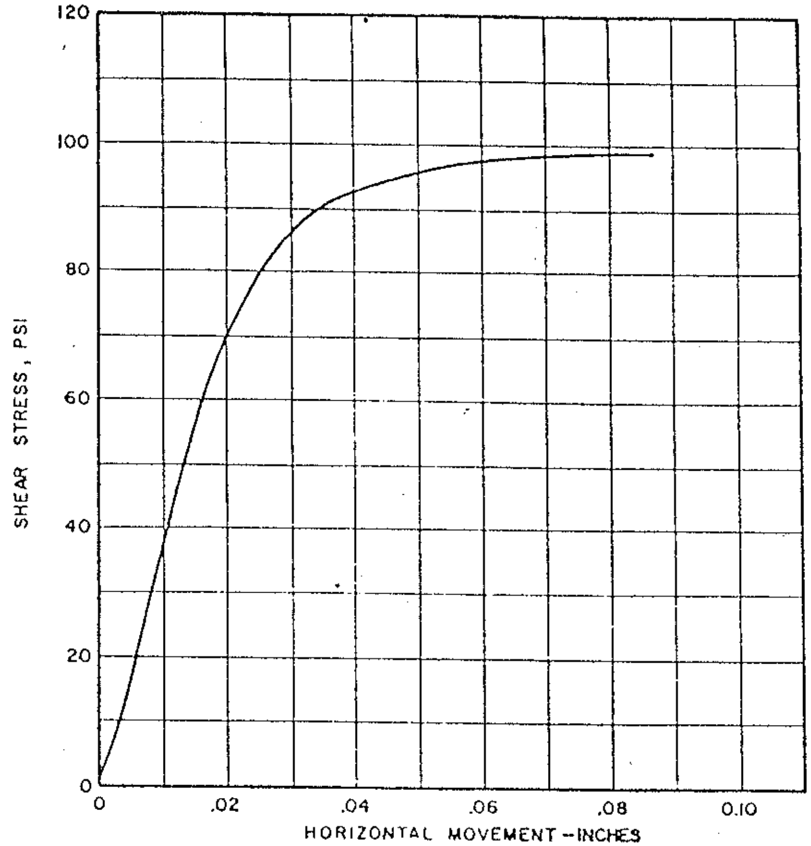


Figure 5 Stress Versus Deflection from Field Shear Test. (Millinger, 1966, from Dodds, 1970)

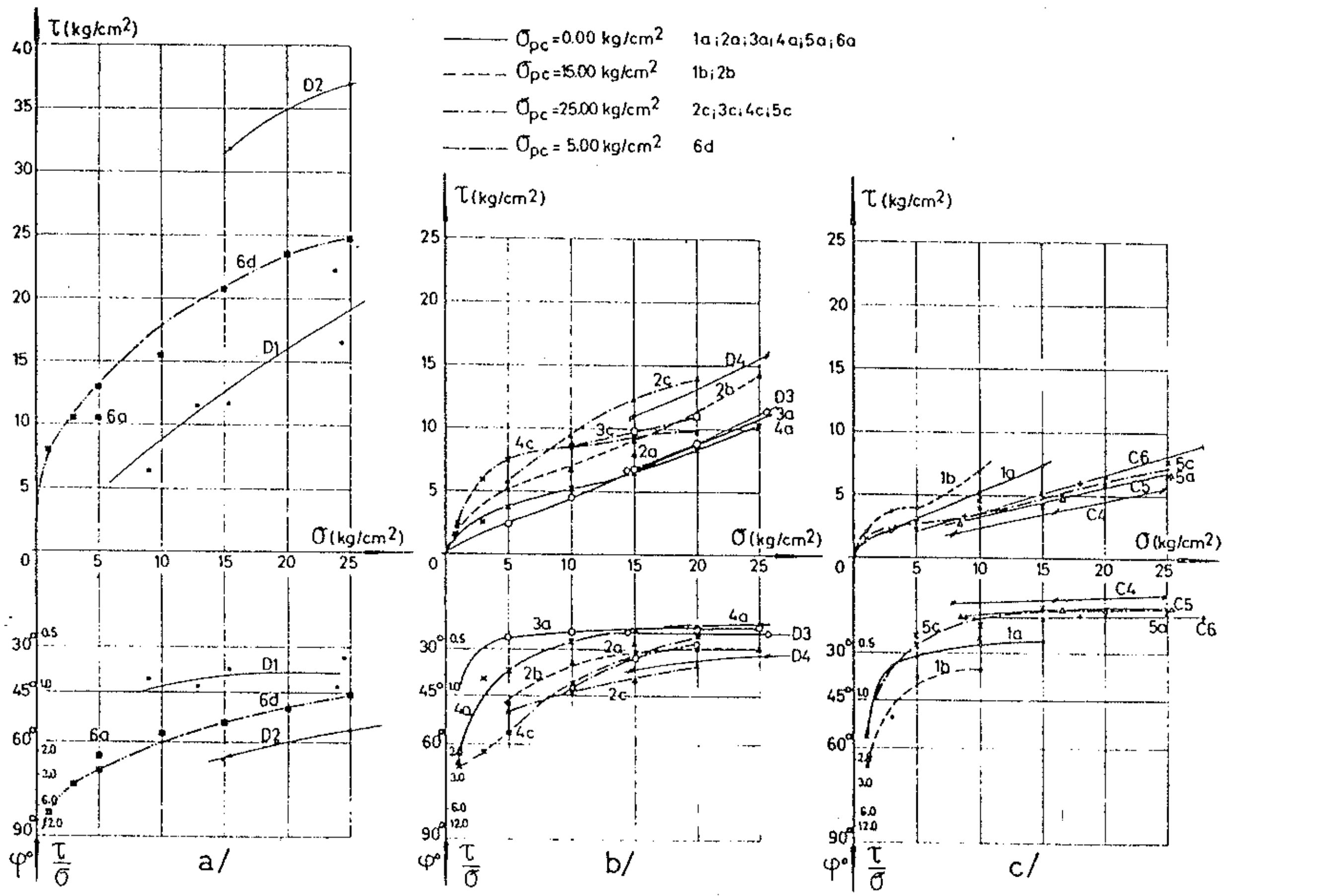


Figure 6 Strength Properties from Field Shear Testing. Case A: Limestone with clean discontinuities, Case B: Limestone with thin clayey discontinuities, Case C: Limestone with clayey layers. (After Krsmanovic & Popovic, 1966)

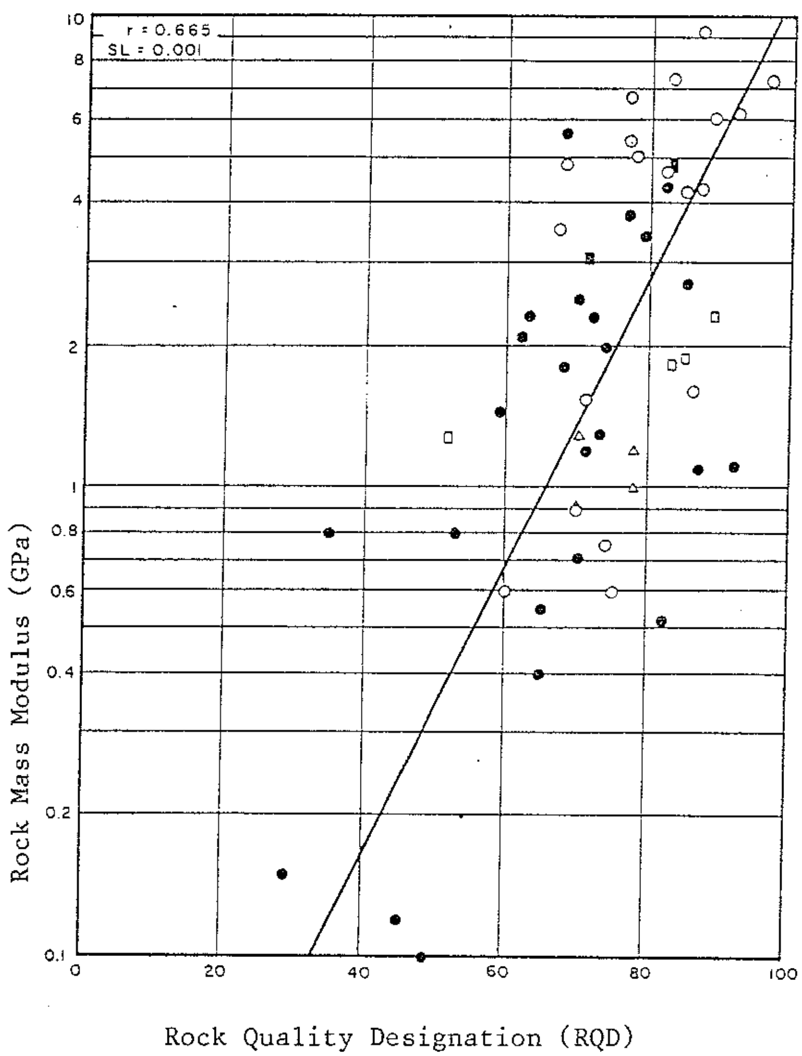


Figure 7 RQD Versus Rock Mass Modulus
(From Deere, 1969) 1 GPa = 1.45×10^5 psi.

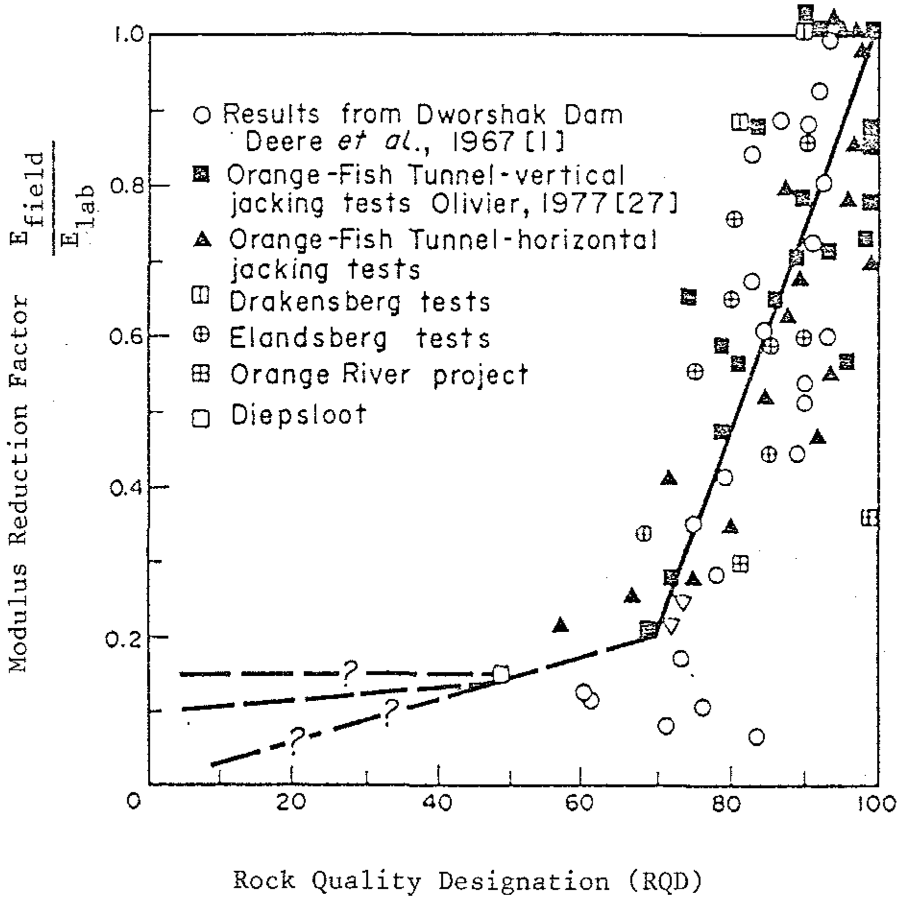


Figure 8 RQD Versus Modulus Reduction Factor (From Bieniawski, 1978)

A. Classification parameters and their ratings

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point-load strength index	> 10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial compressive strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5-25 MPa	1-5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2 m	0,6 - 2 m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surfaces. Not continuous No seperation Unweathered wall rock.	Slightly rough surfaces. Separation < 1 mm Slightly weathered walls	Slightly rough surfaces. Separation < 1 mm Highly weathered walls	Slickensided surfaces OR Gouge < 5 mm thick OR Separation 1-5 mm. Continuous	Soft gouge > 5 mm thick OR Separation > 5 mm. Continous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length	None	< 10 litres/min	10-25 litres/min	25 - 125 litres/min	> 125		
			OR	OR	OR	OR	OR		
		Ratio $\frac{\text{joint water pressure}}{\text{major principal stress}}$	0	0,0-0,1	0,1-0,2	0,2-0,5	> 0,5		
			OR	OR	OR	OR	OR		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. Rating adjustment for discontinuity orientations

Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. Rock mass classes determined from total ratings

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. Meaning of rock mass classes

Class No	I	II	III	IV	V
Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2,5 m span	30 minutes for 1 m span
Cohesion of the rock mass	> 400 kPa	300 - 400 kPa	200 - 300 kPa	100 - 200 kPa	< 100 kPa
Friction angle of the rock mass	< 45°	35° - 45°	25° - 35°	15° - 25°	< 15°

Figure 9 Geomechanics Classification
(After Bieniawski, 1974)

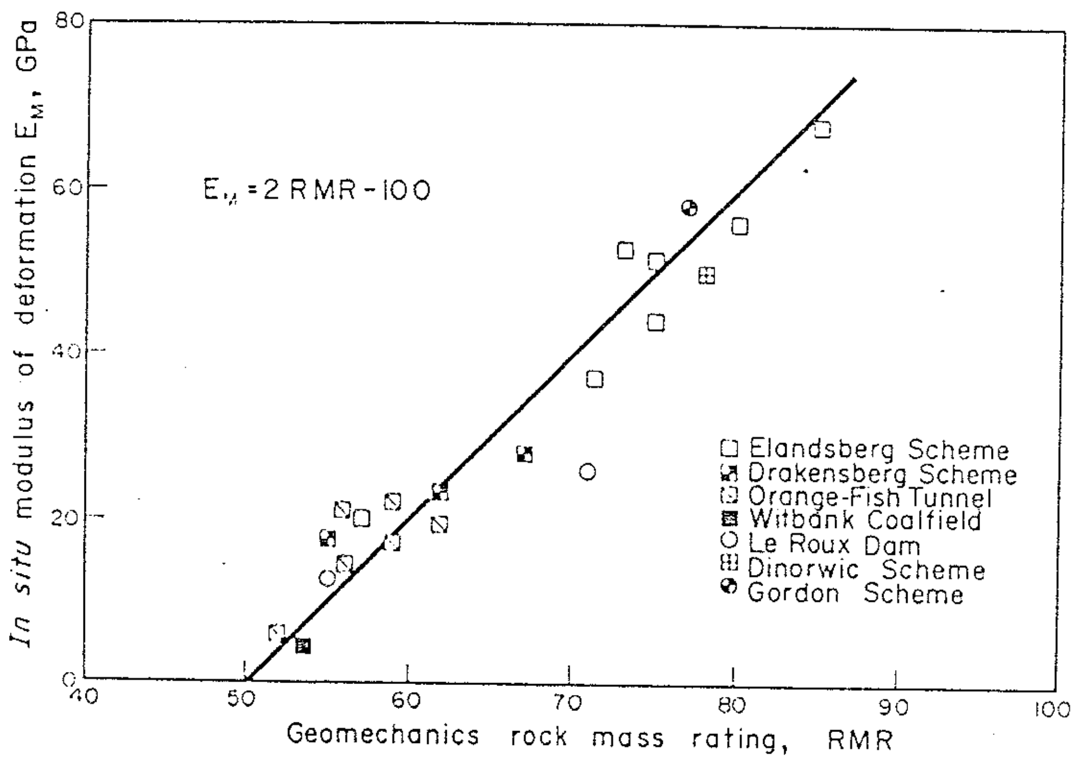


Figure 10 RMR Versus Field Modulus
(From Bieniawski,[5] 1978)
1 GPa = 1.45×10^5 psi

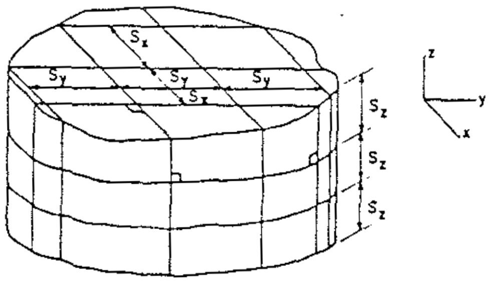


Figure 11 Geomechanical Rock Mass Model
(From Kulhawy, 1978)

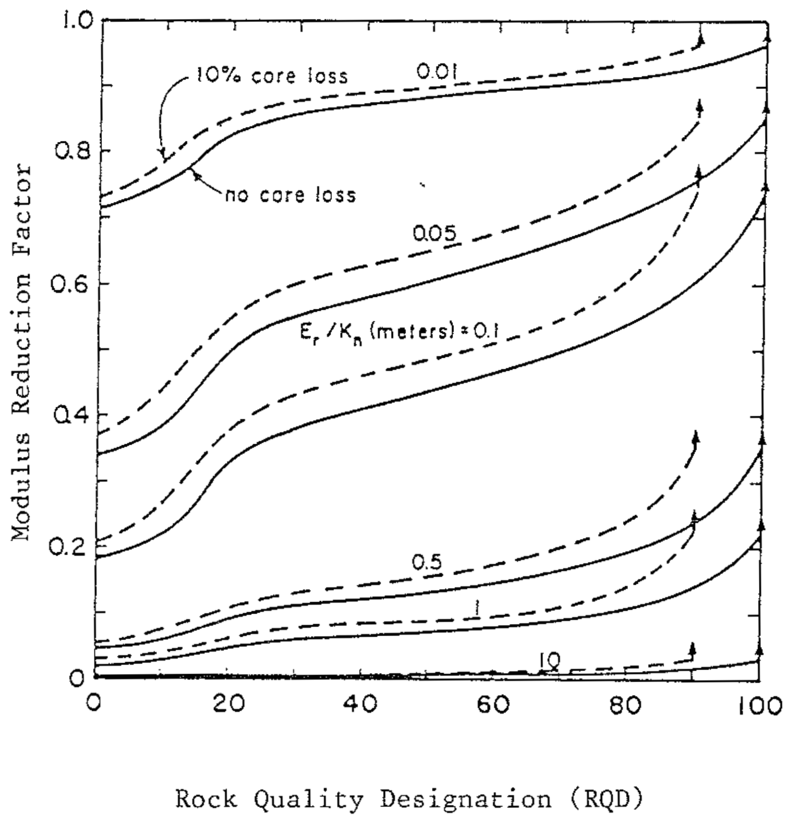


Figure 12 RQD, E_r/K_n Versus Modulus Reduction Factor (From Kulhawy, 1978)

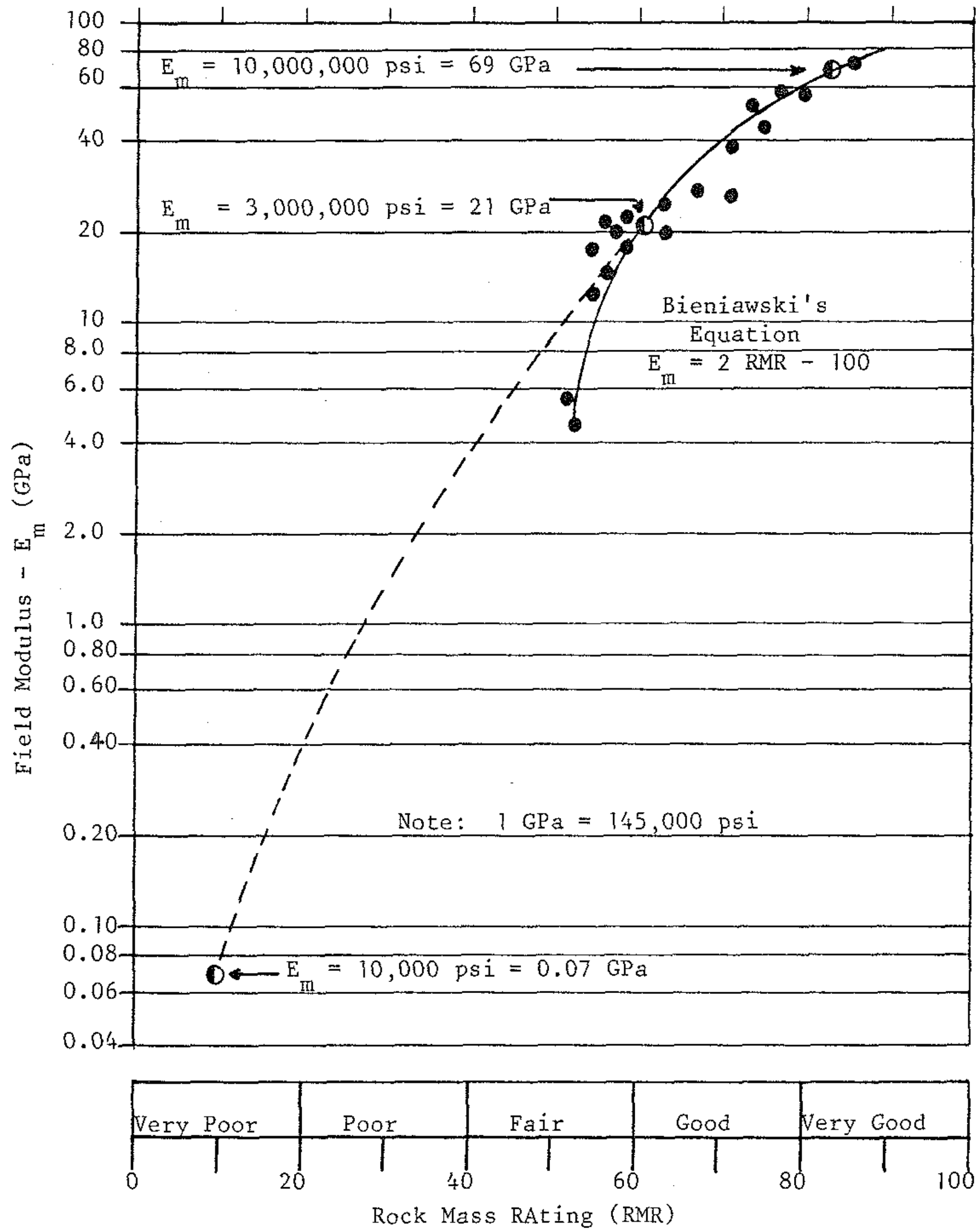


Figure 13 - Relationship Between Rock Modulus and Rock Mass Rating
(After Bieniawski, 1978)

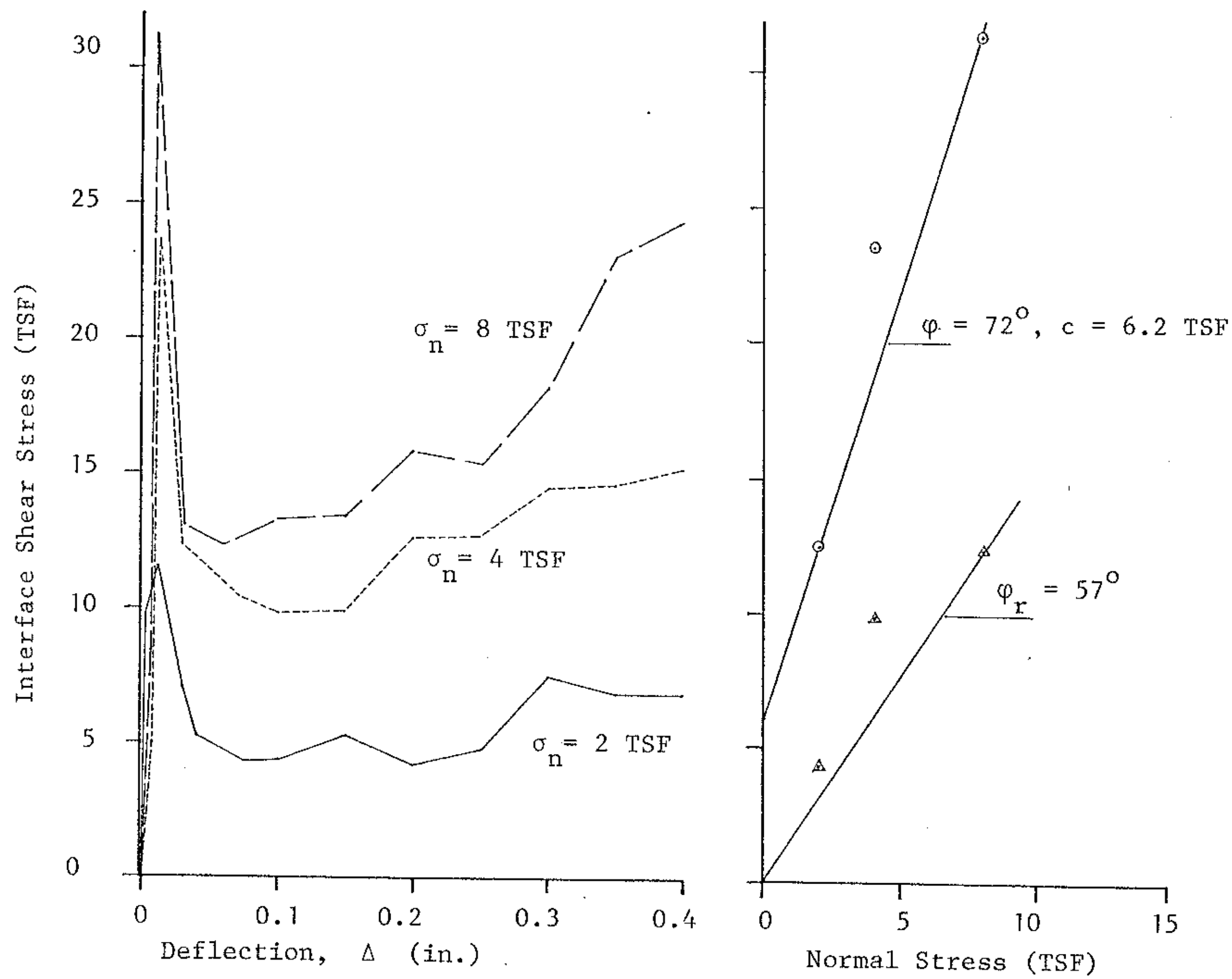


Figure 14 Interface Shear Stress Versus Deflection. Concrete-to-Very Hard Sandstone (After Thorton et. al., 1981)

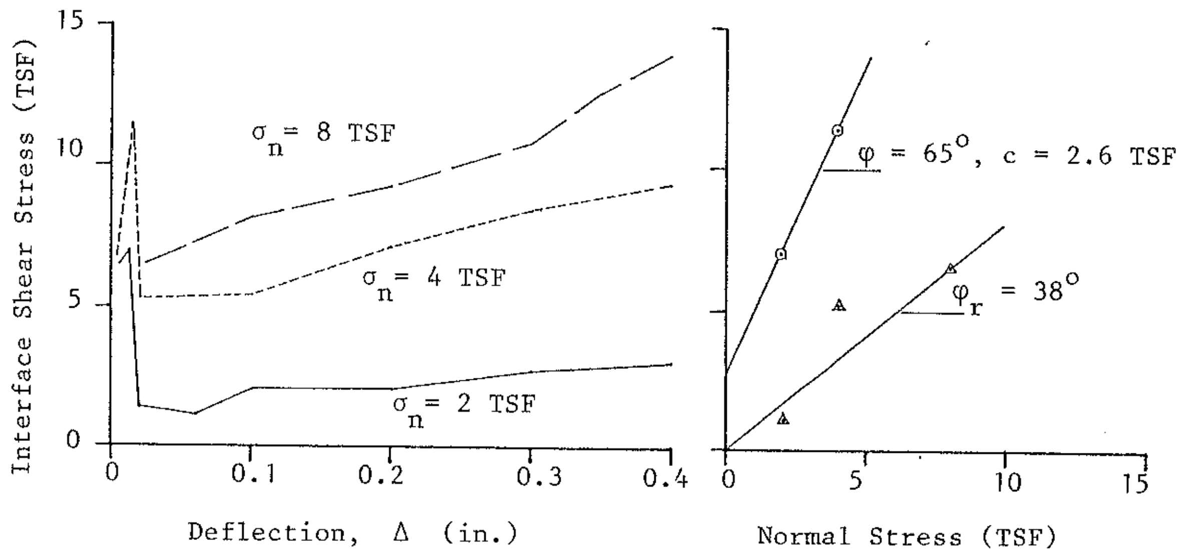


Figure 15 Interface Shear Stress Versus Deflection. Concrete-to-Shale (After Stowe and Pavlov, 1981)

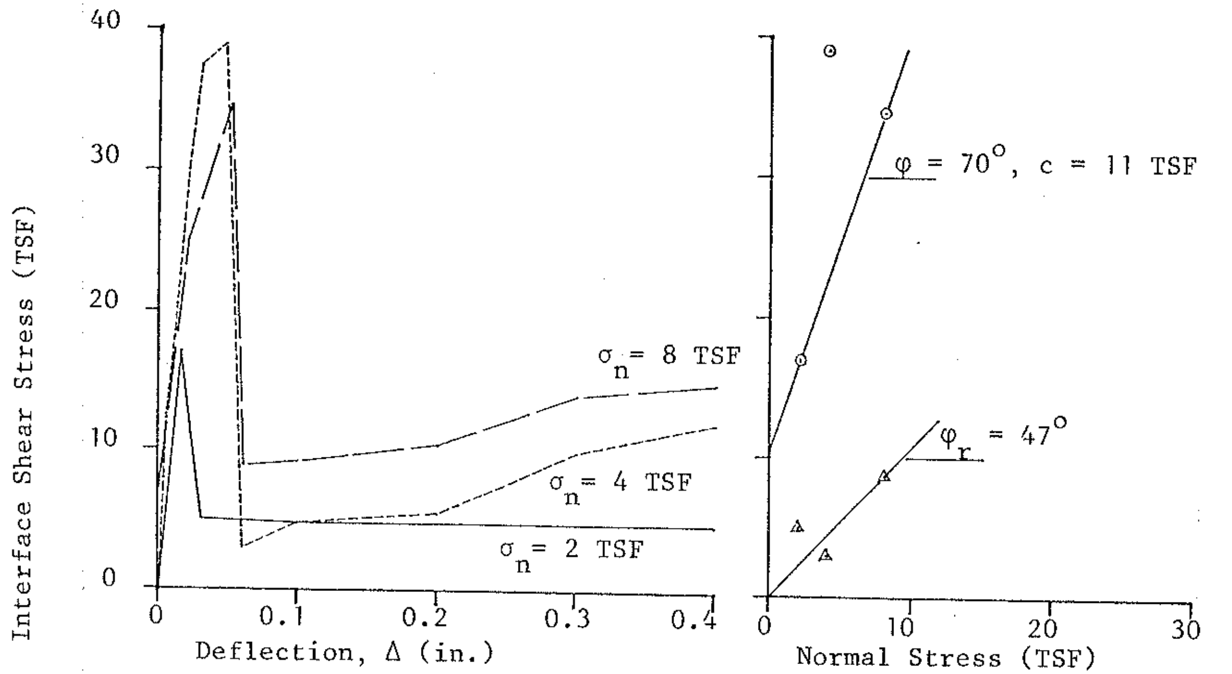


Figure 16 Interface Shear Stress Versus Deflection. Concrete-to-Limestone (After Stowe and Pavlov, 1981)

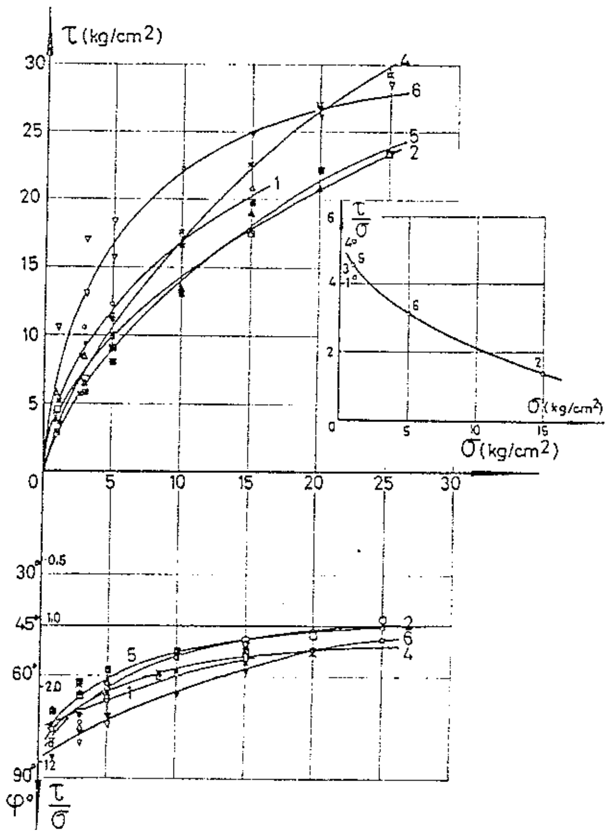


Figure 17 Strength Envelopes of Concrete-to-Limestone
(After Krsmanovic and Popovic, 1966)

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APPENDIX A

TABLE A-1

DESIGN VALUES (Stowe, et al, 1980)

	Dolomite
Characterization Properties	
Effective Unit Weight, lb/ft^3	168.0
Dry Unit Weight, lb/ft^3	164.3
Compressive Strength, psi	13,820
Direct Tensile Strength, psi	95.0
Shear Strength	
Concrete-on-rock	c=18 tsf, $\phi=67^o$
Cross bed, 45^o	c=29 tsf, $\phi=68^o$
Clay-filled parting (along stylolitic bedding)	c=9.2 tsf, $\phi=48^o$
Bedding plane (along stylolitic bedding)	c=49 tsf, $\phi=61.5^o$
Precut, rock-on-rock	c=0, $\phi_r=22^o$
Precut, concrete-on-rock	c=0, $\phi_r=26^o$
Natural joint, smooth	c=12.4 tsf, $\phi=23.2^o$
Bond Strength, tsf	117.4
Modulus of Elasticity x 10^6 psi	8.94
Poisson's Ratio	0.28
Shear Modulus x 10^6 psi	3.49

TABLE A-2

DESIGN VALUES (Stowe and Pavlov, 1981)

	Limestone	Gry to Brn Shale	Grn Shale	Grn Clay
Characterization Properties				
Dry Unit Weight, lb/ft^3	175.0	144.8	155.8*	124.2
Effective Unit Weight, lb/ft^3	176.5	155.4	163.7*	138.6
Compressive Strength, psi	6280*	1830*	--	--
Tensile Strength, psi	--	195	--	--
Engineering Design Properties				
Shear Strength				
Intact	--	c=3.6 tsf* $\phi=47.2^o$	c=6.0 tsf* ϕ=38.3 tsf c=0 $\phi_r=34.3^o$	
Filled parting				
green clay seam	c=2.3 tsf* $\phi=43^o$ c=0 $\phi_r=23.3^o$	--	--	c=0* $\phi=12.5^o$ --
shale/clay seam	c=1.6 tsf* $\phi=34.3^o$ c=0 $\phi_r=21^o$	--		
Precut, rock-on-rock	--	c=0 $\phi_r=21.4^o$	--	--
Concrete-on-rock	c=19 tsf $\phi=67.2^o$ c=0 $\phi_r=37^o$	c=5.6 tsf $\phi=49^o$	--	--
Precut, concrete-on-rock	c=0 $\phi_r=30^o$**	c=0* $\phi_r=20^o$	--	--
Cross-bed	c=49.9 tsf* $\phi=54.5^o$	c=5.7 tsf $\phi=46.3^o$	--	--
Modulus of Elasticity, 10^6 psi	5.51	0.25	--	--
Poisson's Ratio	0.26	0.36	--	--
Shear Modulus, 10^6 psi	2.19	0.10	--	--

* New data obtained during compliance and scour detection studies.
** Generally excepted values.

TABLE A-3

DESIGN VALUES (Pace, et al, 1981)

Property	Location	Numerical Value Local	Numerical Value Overall
Coefficient of sliding friction between concrete and foundation (ϕ), deg	Land wall River wall Dam Headgate	30° 24'	30° 24'
Cohesion between concrete and foundation (C), ksf	Land wall River wall Dam Headgate	0.04	0.04
Effective unit weight of concrete (γ_m), pcf	Land wall River wall Dam Headgate	149.80 149.19 149.30	149.59 149.93
Dry unit weight of concrete (γ_d), pcf	Land wall River wall Dam Headgate	139.06 137.69 138.89	138.66 139.16
Effective unit weight of foundation material (γ_m), pcf	Land wall River wall Dam Headgate	169.71 169.50 169.80	169.60 169.20
Dry unit weight of foundation material (γ_d), pcf	Land wall River wall Dam Headgate	167.70 167.07 -	167.38 -
Saturated unit weight of backfill material (γ_s), pcf	Land wall	144	144
Dry unit weight of backfill material (γ_d), pcf	Land wall	115	115
Modulus of Elasticity of concrete (E_c), psi	Land wall River wall Dam Headgate	4.23×10^6 4.50×10^6 4.59×10^6 4.32×10^6	4.41×10^6
Poisson's ratio of concrete (μ)	Land wall River wall Dam Headgate	0.24 0.22 0.24 0.21	0.23

(Continued)

TABLE A-3 (Cont'd)

DESIGN VALUES (Pace, et al, 1981)

Property	Location	Numerical Value Local	Numerical Value Overall
Modulus of elasticity of foundation material (E_f), psi	Land wall	0.97×10^6	0.97×10^6
	River wall		-
	Dam	0.97 × 106	
	Headgate	-	
Poisson's ratio of foundation material (μ)	Land wall	0.12	0.12
	River wall		-
	Dam	0.12	
	Headgate	-	
Foundation material pullout resistance based on area between grout and rock, kips/foot of depth	Land wall	9	9
	River wall		
	Dam		
	Headgate		
Compressive strength from unconfined compressive test of nondeteriorated concrete, psi	Land wall		4400
	River wall		
	Dam		
	Headgate		
Tensile strength from tensile splitting tests of nondeteriorated concrete, psi			295
Tensile strength from direct tension tests of nondeteriorated concrete, psi			198
Average tensile strength from tensile splitting and direct tension tests of nondeteriorated concrete, psi			247
Compressive strength from unconfined compressive test of slaty-shale foundation, psi			900
Tensile strenght from direct tension test of slaty-shale foundation, psi			43
Angle of internal friction, ϕ, from triaxial test of slaty-shale foundation (deg)			42-49
Cohesive strength, c, from triaxial test of slaty-shale foundation, psi			10-230

TABLE A-4

DESIGN VALUES (Stowe, 1978)

	Concrete <4.3 ft Depth	Concrete >4.3 ft Depth	Dolomite	Silt	Gravel
Characterization Properties					
Effective (wet) unit weight, lb/ft^3	147.1	151.5	164.4	82.8	140.0
Dry unit weight, lb/ft^3	135.8	141.8	159.0	--	--
Bearing capacity, tsf	310	368	432.0	--	--
Tensile strength, tsf	32	39	8.8	--	--
Shear strength:					
Concrete-on-rock	--	--	c=8.0 tsf $\phi=78^o$	--	--
Intact	--	--	c=33.5 tsf $\phi=77^o$	c=0.0 $\phi=30^o$*	--
Cross bed, 45^o	--	--	c=29 tsf $\phi=68^o$	--	--
Clay-filled parting	--	--	c=0.43 tsf $\phi=45^o$	--	--
Precut, rock-on-rock	--	--	c=0.4 tsf $\phi=22^o$	--	--
Precut, concrete-on-rock	--	--	c=0.0 $\phi=26^o$	--	--
Natural joint	--	--	c=5.93 tsf $\phi=46^o$	--	--
Modulus of elasticity psi x 10^6	2.39	4.86	2.30	--	--
Poisson's ratio	0.19	0.21	0.17	--	--
Shear modulus, psi x 10^6	1.00	1.98	0.98	--	--
Coefficient earth	--	--	--	0.50	0.45

* $\phi=30$ suggested by CDO.

TABLE A-5

DESIGN VALUES (Stowe and Warringer, 1975)

	Dolomite	Friable Sandstone	Competent Sandstone	Shale
Index Properties:				
Dry Unit Weight, lb/ft^3	157.0 (1)	127.7	138.3	110.4
Wet Unit Weight, lb/ft^3	162.0 (1)	140.2	147.7	129.9
Porosity, pct	9.6	20.3	16.8	34.8
Bearing Capacity, tsf	350	39	320	--
Tensile Strength, psi	110	75	175	--
Shear Strength:				
Intact	c = 90.0 tsf (2) Ø = 56^o	-- (3)	c = 40.3 tsf Ø = 65.5^o	c = 0.12 tsf Ø = 14.5^o
Natural Joint	--	--	c = 1.45 tsf Ø = 30.5^o	c = 0.0 Ø = 38^o
Shale-Filled Parting	c = 0.25 tsf Ø = 26^o	--	--	--
Precut, Rock-on-Rock	c = 0.0 Ø = 31^o	c = 0.0 Ø = 33.5^o	c = 0.0 Ø = 31^o	c = 0.45 tsf Ø = 19^o
Concrete on Rock (4)	c = 1.60 tsf Ø = 63^o	--	--	--
Modulus of Elasticity x 10^6 psi	1.82	-- (4)	2.00	-- (6)
Poisson's Ratio	0.13	-- (5)	0.12	-- (7)
Shear Modulus x 10^6	0.65	--	0.69	--

(1) Lower value previously reported by NCC; density γ_d = 142 lb/ft^3; γ_s = 146 lb/ft^3.
(2) Lower value previously reported by NCC; c = 72.0 tsf; Ø = 54.5^o.
(3) Lower value previously reported by NCC; c = 1.08 tsf; Ø = 38^o (moderately weak).
(4) Value previously reported by NCC; 1.40 x 10^6 psi.
(5) Value previously reported by NCC; 0.30.
(6) Value previously reported by NCC; 0.36 x 10^6 psi.
(7) Value previously reported by NCC; 0.225.

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Carl Philip Benson was born April 27, 1955 in Gary, Indiana, attended public schooling in Silver Spring, Maryland and graduated from Montgomery Blair High School in 1973. Mr. Benson received the degree of Bachelor of Science in Geology from Colorado State University in 1977. Prior to relocation to Seattle, Washington he worked as a geologist in Colorado for the U.S. Geological Survey and a mining and engineering consulting firm. In Seattle, Mr. Benson worked as a project level engineering geologist for two geotechnical consulting firms. Specific assignments were in many western states, including Alaska.

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