FOUNDATION DESIGN OF CAISSONS ON GRANITIC AND VOLCANIC ROCKS

GEO REPORT No. 8

T.Y. Irfan & G.E. Powell

GEOTECHNICAL ENGINEERING OFFICE
CIVIL ENGINEERING DEPARTMENT
HONG KONG

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PREFACE

In keeping with our policy of releasing information of general technical interest, we make available some of our internal reports in a series of publications termed the GEO Report series. The reports in this series, of which this is one, are selected from a wide range of reports produced by the staff of the Office and our consultants.

Copies of GEO Reports have previously been made available free of charge in limited numbers. The demand for the reports in this series has increased greatly, necessitating new arrangements for supply. In future a charge will be made to cover the cost of printing.

The Geotechnical Engineering Office also publishes guidance documents and presents the results of research work of general interest in GEO Publications. These publications and the GEO Reports are disseminated through the Government's Information Services Department. Information on how to purchase them is given on the last page of this report.

A. W. Malone
Principal Government Geotechnical Engineer
April 1995

EXPLANATORY NOTE

This GEO Report consists of the following two technical review documents prepared by the Advisory Section of the now defunct New Works Division on the subject of caisson design in igneous rocks :

1 FOUNDATION DESIGN OF CAISSONS ON GRANITIC ROCKS : A TECHNICAL REVIEW

T.Y. Irfan & G.E. Powell

This report was originally produced as GCO Report No. 16/82

CONTENTS

1. INTRODUCTION

Foundation design is based on two basic criteria; bearing capacity and settlement. The basic problem in foundation design is the choice of an allowable bearing stress which will not exceed the safe bearing capacity of rock and the settlement of the structure caused by deformation of the rock will be less than the allowable settlement for the structure.

The methods commonly in use for the selection of a design bearing stress for deep foundations on rock fall into four main categories :

(a) building codes,

(b) empirical rules,

(c) rational methods based on bearing capacity and settlement analyses, and

(d) field load tests.

Regardless of the method used for bearing stress analysis, the allowable bearing stresses specified by the various existing Building Codes, based on local experience, tend to control the design. Many building codes recommend a presumptive value for the allowable bearing stress and very few relate these values to simple quantitative rock parameters. In the case of structures imposing large loadings, the code values are normally conservative and the settlements are unknown. Field load tests are expensive and very few load tests have been carried out to determine the bearing stresses of deep foundations, such as bored piles and hand-dug caissons, in rock.

Parameters considered in the design of pile and caisson foundations on rock, based on bearing capacity and settlement analyses, are :

(a) the strength of the intact rock material and the rock mass,

(b) the rock mass modulus,

(c) the structure of the rock mass,

(d) the construction practice,

(e) the embedment ratio for base resistance, and

(f) the socket roughness, if side resistance is taken into account.

In the general case, the load on a deep foundation is carried partly by skin friction and partly by end bearing. In the design of rock caissons in Hong Kong, skin friction is usually neglected. Evans et al (1982) stated that "structural designs (in Hong Kong) are based on practically no foundation yield and standard building designs have very small settlement constraints, much smaller than would be the case in European practice; no allowance is made for skin friction". However, it must be realized that, for the section socketed within rock, full skin friction is mobilized after much smaller movements than is for end-bearing, and hence a substantially reduced portion of the total applied load is transferred to the base (Gill, 1980).

In Hong Kong, granitic rocks are of common occurrence, and an increasing number of heavy structures are being built on granitic rock foundations. Presumptive allowable bearing stresses of about 5 MPa have typically been used as design values for caissons (end-bearing) in slightly weathered granite. Minimum RQD values of 75 percent have been specified to reach this value in most works contracts.

In this report, various well-established and internationally used design methods for pile foundations on rock have been reviewed. Allowable bearing stresses and likely settlements for granitic rock foundations have been calculated by various methods, with particular reference to the fresh to moderately weathered granitic rocks including granodiorites.

2. ALLOWABLE BEARING STRESS

The mechanism of bearing capacity failure of an intact rock is much more complex than either assumed or observed in the failure of a soil mass. Ultimate bearing capacity of rock has very little engineering significance and the calculation of the ultimate bearing capacity is complex. Failure may be defined by :

$$q_{fi} = 2.7\ q_u \text{ (Ladanyi, 1966)} \quad \ldots\ldots\ldots\ldots \quad (1)$$

where q_{fi} is the stress at incipient failure and q_u is the unconfined compressive strength of the rock. Safe bearing capacity in rocks approximates to the unconfined compressive strength after allowing a factor of safety of 3. In a case where the rock is stronger than the concrete, the bearing capacity of the rock imposes no practical limitation on the foundations, and the allowable bearing capacity is then governed by the bearing stress at which allowable settlement occurs.

2.1 Building Codes

Many building codes specify presumptive allowable bearing stresses. However, they differ considerably in their recommendations for the same kind of rock (Table 1). A number of building codes relate the allowable bearing stress to the uniaxial compressive strength of the rock material (e.g. Uniform Building Code of America, 1964, Dallas Building Code, 1968) by the following formula :

$$q_a = K\ q_u \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

where q_a = allowable bearing stress,

q_u = uniaxial compressive strength of the intact rock material, and

K = a factor, usually 0.2 to take account of the nature of jointing of the rock.

The values calculated by this method do not include increases allowed for embedment into rock. The British Code (British Standards Institution, 1972) and the Chicago Code (1972) allow an increase in bearing stress of 20% for every 300 mm embedment up to a maximum equal to twice the allowable bearing stress.

2.2 Empirical Correlation between RQD and Allowable Bearing Stress

Peck et al (1974) suggested an empirical correlation between the allowable contact stress and the Rock Quality Designation, RQD (Figure 1). The RQD value is an approximate measure of the joint intensity of the rock mass, which in turn is an indicator of the rock mass compressibility. The authors stated that these values are based on settlement criteria and should not be increased for a foundation embedded into rock.

2.3 Canadian Foundation Engineering Method

Allowable bearing stress is calculated by the following formula (Canadian Geotechnical Society, 1978) :

$$q_a = K_{sp}\, q_u\, d \quad \ldots\ldots\ldots\ldots \quad (3)$$

where K_{sp} is an empirical factor which depends upon spacing of discontinuities and includes a factor of safety of 3 (Table 2), q_u is the average unconfined compressive strength of rock cores and d is the depth factor given by the formula :

$$d = 0.8 + 0.2\,\frac{H_s}{D} \leqslant 2 \quad \ldots\ldots\ldots\ldots \quad (4)$$

where H_s is the depth of socket in rock having a strength equal to q_u and D is the diameter of socket.

K_{sp} is also calculated by the following formula which takes into account spacing of discontinuities, c, thickness of discontinuities δ , and the footing width B :

$$K_{sp} = \frac{3 + c/B}{10\sqrt{1 + 300\,\delta/c}} \quad \ldots\ldots\ldots\ldots \quad (5)$$

3. SETTLEMENT OF FOUNDATIONS ON ROCK

For foundations on fresh rock, settlement is so small that it is hardly worth considering except for special structures where total and differential settlement must be extremely small. For large diameter caissons and piles on fresh granitic rocks, the bearing capacity of the rock is no limitation since the rock is always stronger than the concrete. Foundation design in fresh rock is influenced by the nature and intensity of discontinuities as explained in the previous section. In weathered rock, in addition to these parameters, the degree and type of weathering control the design.

In order to determine an allowable bearing stress it is necessary to estimate the settlement of the foundation. Settlement in rock foundations can be calculated by elastic analysis. For a rigid pile foundation, settlement is given by the following formula :

$$s = \frac{\pi}{2} \cdot \frac{q(1 - \nu^2) \cdot r}{E_m} \cdot I_s \quad \ldots\ldots\ldots\ldots \quad (6)$$

where s = settlement,

q = uniform load per unit area,

E_m = deformation modulus of the rock mass under the pile,

I_s = depth reduction factor (Burland, 1970),

r = radius of pile, and

ν = Poisson's ratio.

3.1 Rock Mass Modulus

Deformation modulus of the rock mass, E_m, is the primary factor in settlement calculations. It is difficult to define and measure the mass deformation moduli when the rock mass is jointed and weathered. Hobbs (1975) defined a rock mass factor, j, as the ratio of the deformation modulus of a rock mass, E_m, in any identifiable lithological and structural component to that of the deformation modulus of the intact rock, E_i, comprising the component :

$$j = \frac{E_m}{E_i} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

Most rocks exhibit a linear relationship between the modulus of elasticity of intact rock and the uniaxial compressive strength. Granites are good examples of this, and it is possible within limits to estimate the intact modulus from the compressive strength. Figure 2 shows a plot of modulus of elasticity against compressive strength for fresh and weathered granitic rocks.

3.1.1 Effect of Weathering on Rock Mass Modulus

The strength and elastic modulus of intact rock specimens are drastically reduced with increased weathering.

The modulus ratio, $M_r = E_i/q_u$, is only marginally affected as the rock material weathers (Figure 2). The discontinuities have an increasingly smaller influence on the mass modulus as the modulus of the intact rock decreases, i.e. the j-value increases with increase in weathering (Table 3).

RQD values and fracture frequency can be related quite closely to mass/intact rock modulus ratio. Table 4, based on Deere et al (1966) and Coon and Meritt (1970), gives typical j-values associated with RQD ranges and fracture intensity.

Woodward et al (1972) in their textbook 'Drilled Pier Foundations' suggested, the use of reduction factor, α_E ($E_{mass}/E_{seismic}$) vs RQD as an approximate way to obtain the rock mass modulus for settlement calculations. Peck et al (1974) based their design stress values on RQD (Figure 1). When this chart is used, limiting settlement 'should not exceed 12.7 mm, even for large loaded areas'. Although it is difficult to compute settlements using this approach, it does provide a convenient starting point for evaluating foundations on rock masses.

Bieniawski (1975a, 1978) correlated the reduction factor α_E with Geomechanics Classification Rating of the rock mass (Figure 3). The Rock Mass Rating (RMR) is based on six rock mass parameters including weathered state, RQD, strength of intact rock, and spacing and condition of discontinuities. Hoek and Brown (1980) constructed the information presented in Bieniawski's paper to establish a relationship between in situ deformation modulus of rock mass and rock mass class (Figure 4). Kulhawy (1978) and, Kulhawy and Goodman (1980) suggested the use of a geomechanical model to establish equivalent rock mass properties from the individual properties of the rock material and the discontinuities. The rock material is characterized by the elastic properties E_r and ν_r, while the discontinuity properties are defined by a normal stiffness, K_n, and a shear stiffness, K_s. A plot of reduction factor α_E, against discontinuity spacing is shown in Figure 5. RQD values are correlated with mean discontinuity spacing in Figure 6. These two figures are combined in Figure 7 which gives the modulus reduction factor as a function of RQD for no core loss and 10% core loss, i.e. representing weak zones which cannot be recovered by conventional coring. To use Figure 7, RQD, E_r and K_n values must be known. The RQD is simply determined by logging the rock cores, while E_r can be determined from uniaxial compression tests on cores in the laboratory. The normal stiffness K_n can only determined from compression tests on discontinuities. For preliminary design purposes, where testing may not be warranted, E_r and K_n can be calculated from the published data.

3.1.2 Plate Loading Tests

Rock mass deformation modulus or rock mass factor can be determined more accurately by plate load test, and settlement calculated by the formula :

$$\frac{S_f}{S_p} = \left(\frac{r_f}{r_p}\right) a \quad \ldots\ldots\ldots\ldots\ldots \quad (8)$$

where S_f = settlement of foundation,

S_p = settlement calculated from the plate load test,

r_f = radius of the foundation,

r_p = radius of the plate, and

a depends on the plate dimensions but is approximately unity.

In situ field testing can be time-consuming and expensive, and therefore it is usually only warranted for special structures, relatively heavy loading, and unusual or particularly poor rock mass conditions including highly to completely weathered rocks. In the absence of plate load tests, one of the methods based on borehole RQD or fracture intensity and laboratory deformation modulus given above can be used to determine the modulus reduction factor and the settlement under specified load conditions.

4. ROCK SOCKET

In the case of rock caissons, load transfer through skin friction on the portion of the shaft within the overburden, i.e. residual soil and completely weathered rock, is normally small and usually neglected. However, the portion of the caisson socketed in rock contributes significantly to load transfer. Studies indicate that shaft resistance values can be large and can account for a significant portion of the load support capacity of drilled pier and caisson foundations (Ladanyi, 1977; Pells & Turner, 1979; Horvath et al, 1980; Williams et al, 1980; Donald et al, 1980). The results show that the transfer of load into the embedding medium depends on :

(a) the ratio of moduli of concrete and the medium,

(b) the depth of embedment,

(c) the presence of seams or layers of weaker rock, and

(d) the Poisson's ratio.

Some building codes suggest increase of presumptive bearing stresses with depth of embedment to a maximum of twice the basic value. For example, the Chicago Building Code allows the end-bearing stress to be increased by 20% for every 300 mm embedment into the rock up to a maximum value of 20 MPa with a socket of 1.5 m deep in massive, crystalline rock. The Canadian Foundation Engineering Method includes the effect of the rock socket in the calculation of bearing stress by the following formula :

$$q_a = K_{sp}\ q_u\ d \qquad (3)$$

where d is the depth factor and equal to

$$d = 0.8 + 0.2\ \frac{H_s}{D} \leqslant 2 \qquad (4)$$

where H_s = depth of the socket in rock having a strength q_u, and

D = diameter of the socket.

Coates (1967) suggested the use of the following formula to calculate the average allowable bond strength (adhesion) for piers socketed into rock :

$$\text{Bond strength} = 0.05\ f'_c \qquad (9)$$

where f'c = 28 day compressive strength of concrete.

The Draft Australian Piling Code recommended the use of following formulae (Williams et al, 1980) :

$$f_s = 0.05\ q_u \qquad (10)$$

$$f_b = 0.5\ q_u \qquad (11)$$

where q_u = unconfined compressive strength of rock,

f_s = mobilized side resistance, and

f_b = mobilized base resistance.

The choice of these values was based on both on acceptable deformations and on safe load capacity. If so, their use in design can be assumed to satisfy both settlement and load capacity criteria.

Rosenberg and Journeaux (1976) proposed a load capacity criterion based on an elastic solution for load distribution in a pile, using laboratory and field strength tests. The peak side resistance was found to depend primarily on rock strength. Figure 8 shows a correlation of measured bond strength and unconfined compressive strength for six different rock types.

Load test data on socketed piers has been collected for over 50 sites, located mainly in Australia, Canada, Britain and USA, and shaft resistance plotted on a log-log scale against uniaxial compressive strength of rock by Horvath et al (1980) in Figure 9. Diameter of the piles ranged from 400 to 1 220 mm and the embedment ratio, L_s/D_s, from 1 to 20. The relationship can be closely approximated using the following equation :

$$S_r = b\sqrt{f'_c} \quad . . . . (12)$$

where b = 0.2 to 0.25,

S_r = shaft resistance, and

f'_c = controlling compressive strength, i.e. the unconfined compressive strength of the weaker material, either concrete or rock.

As a final comment on the subject of rock roughness, it should be noted that the roughness of the pier-socket wall interface has a significant influence on the magnitude of mobilized shaft resistance. However, there is very little quantitative data available on this aspect.

Table 5 is taken from Thorne (1980) and shows the attained socket adhesions from a number of cases reported in the literature compared with unconfined compression test data from rock cores. The results indicate that the governing factor in strong rocks is the concrete strength. End-bearing stresses are also shown in the same table. The only failures recorded are in rocks with open or clay-filled joints at 10 mm spacing or closer. Table 6 also summarizes various test data on shaft resistance.

5. DESCRIPTION OF GRANITIC ROCKS

5.1 Classification of Granites

Granites contain quartz and feldspars as essential constituents. Classification is based on texture and grain size, and the geological subdivision into three classes (alkali granites, adamellites and granodiorites) is on the basis of type and relative proportions of feldspars (Hatch et al, 1972).

5.2 Rock Mass Classification of Granites

Rock mass characterization for engineering purposes involves the description of the geological nature of the rock mass, including the petrographic properties and details of discontinuities and structures, coupled with an assessment of the engineering properties of the rock material. Weathering grade classification is an extremely useful tool for rock description. Assessment of weathering grade has become an important aspect of rock mass description for engineering purposes (Dearman, 1976). The distribution and grades in a weathering profile plays an important part in the recognition and assessment of uniformity of foundation condition (Knill & Jones, 1965).

Engineering geological schemes for grading rock masses have not yet been developed. The best that can be achieved is a detailed semi-quantitative or quantitative description of rock material properties plus description of discontinuity and weathering state. The weathering grades, backed up by discontinuity and strength grades, provide the best indication of rock mass grade. Rock and soil material may be described in qualitative terms. Field index tests, such as Schmidt hammer and slakeability, have been used successively for the description of slope materials in Hong Kong (Hencher & Martin, 1982). Such a material description scheme may be used to define the weathered material in each foundation layer.

In a civil engineering context, assessment of foundation conditions depends mainly on the logging of drill cores. Percentage core recovery, recognition of weathered rock, and estimation of proportion of discoloured to fresh rock permit determination of distinctive mass weathering zones I to V. Calculation of RQD (Rock Quality Designation) has to be relied upon to complete the rock mass characterization.

5.2.1 Description of Weathering State of Rock Mass

Recognition of distinctive weathering grades in the rock mass may be based on the degree of discolouration, the rock/soil ratio, and the presence or absence of the original rock fabric. The descriptive schemes for weathering grades of rock material and rock mass recommended by BS 5930 (BSI, 1981) have been used in this report.

The weathering profile in the rock mass may therefore be described in terms of the distribution of the various types of weathered rock materials within it, and the effects that weathering has had on discontinuities. Experience with a variety of rock types, including granite, has shown that the classification is of general application, but in some cases subdivisions of grades are necessary.

Variations in intensity of weathering are generally gradational, but sometimes very sharp boundaries exist between the material components in one grade. In the former, engineering properties for each mass will be represented by a spectrum of values; in the latter, a bimodal range of properties is given relating to the two distinct components of the mass grade.

5.2.2 Description of Weathering Grades of Rock Material

A descriptive scheme for weathering grades of rock material may be established on the basis of the changes associated with mechanical and chemical weathering (Table 8). Individual stages listed in Table 8 may be subdivided using qualifying terms, for example 'partially discoloured', 'wholly discoloured' and 'slightly discoloured' to aid in description of the material being examined. If desired such terms may be quantified.

6. ENGINEERING PROPERTIES OF WEATHERED GRANITIC ROCKS

6.1 General

Quantitative test data used in the calculation of allowable bearing stresses and settlements of foundations on granite (Tables 9 and 10) have largely been based on the work carried out by Irfan (1977) on a wide spectrum of weathered granites including granodiorites (Irfan & Dearman, 1978; Dearman & Irfan, 1978 a,b). Published laboratory test data on the strength and deformation properties of granitic rocks have also been included for comparison (Table 11). There is very little published systematic data on the engineering properties of weathered Hong Kong rocks.

6.2 Uniaxial Compressive Strength and Elastic Properties

Table 9 sets out the classification of weathered granite in terms of uniaxial compressive strength, elastic modulus and point load strength based on the laboratory determination of engineering properties. Uniaxial compressive strength testing was carried out in accordance with the methods given in Hawkes and Mellor (1970). The tangent Young's modulus and Poisson's ratio were calculated from the third loading cycle at 50% of the ultimate stress (Deere & Miller 1966; ISRM, 1978). Lower-bound values and mean values have been selected as design parameters for each rock mass class (Table 10) and used in the calculation of bearing stress and settlement for pile and caisson foundations in granites. Published engineering parameters for granites based on the data compiled by Kulhawy (1975) and, Lama and Vutukuri (1978) are given in Table 11. The values quoted in the literature are generally for 'fresh' granites. Rock mass deformation modulus and the effect of weathering on mass engineering properties are discussed in Section 3.1.1.

6.3. Point Load Strength

Point load strength has been widely used to estimate uniaxial compressive strength of rocks in the field and laboratory (D'Andrea et al, 1965; Broch & Franklin, 1972; Bieniawski, 1975b). Figure 10 shows the correlation of compressive strength with point load strength for weathered granitic rocks (Dearman et al, 1978). Reliability of the point load test has been improved with recognition of the size dependancy of the results (Bieniawski, 1975b) and standardization of the method (ISRM, 1973). The test is applicable over a wider range of strengths than the Schmidt hammer (Irfan & Dearman, 1978).

7. ALLOWABLE BEARING STRESSES FOR GRANITIC ROCKS

7.1 Allowable Bearing Stress

7.1.1 Building Codes

Granites, when fresh or slightly weathered, are massive crystalline rocks (excluding microfractured granites). Presumptive allowable bearing stress specified by many national building codes is 10 MPa (Table 1), without taking into account the rock socket effect. Moderately weathered granite may be assumed as 'sound foliated rock' with presumptive bearing stresses around 4 MPa.

Table 12 gives allowable bearing stresses calculated by using lower-bound and average uniaxial compressive strength values for each weathering grade of rock mass and K = 0.2 as specified by some building codes. Minimum allowable bearing stress for the fresh to slightly weathered granite using this approach is over 20 MPa.

7.1.2 RQD and Allowable Bearing Stress

By using the empirical correlation between the allowable contact stress and RQD suggested by Peck et al (1974), maximum, average and minimum allowable bearing stresses for various weathering grades of granite have been calculated (Table 13). The minimum allowable bearing stress for the slightly to fresh granites is over 6.5 MPa and average values are over 10 MPa. If design is based on these values, the settlement of the caissons should not exceed 12.5 mm as stated by Peck et al (1974).

7.1.3 Canadian Foundation Engineering Method

Allowable bearing stresses for the fresh to moderately weathered granites calculated by the Canadian Foundation Engineering Method, using $K_{sp} = 0.1$ for a joint spacing of 300 mm, are tabulated in Table 14. Lower-bound values of allowable bearing stress, using minimum uniaxial compressive strength of intact rock material, is over 10 MPa for the slightly weathered to fresh granite. The rock mass is assumed to have favourable characteristics with the rock surface perpendicular to the foundation and with no open discontinuities.

K_{sp} values have also been determined for various weathering grades and caisson diameters of 1 to 4 m (Tables 15 and 16). A joint spacing of 300 mm and a joint thickness of 1 mm for the fresh to slightly weathered granite and 5 mm for the moderately weathered granite have been adopted as the lower-bound values. The above relationship is valid for thickness of discontinuities less than 25 mm if filled with soil or rock debris. This may be the case for the moderately weathered granite where the material around the joints may either be extremely weak or completely weathered to soil. The allowable bearing stress calculated by this method is much higher than the bearing stress determined assuming $K_{sp} = 0.1$ for the whole rock mass.

7.2 Settlement

Settlements have been calculated for a range of bearing stresses of 5, 7.5, 10 and 15 MPa for pier and caisson foundations of 1 to 6 m diameter, using the following formula (6) given in Section 3.

Depth reduction factor of $I_s = 0.85$ for $\nu = 0.22$ has been adopted to allow for an average embedment of the foundation into sound rock of approximately one diameter (Burland, 1970, Figure 12). In computing the settlement, it is assumed that all the load is transferred to the base (i.e. end-bearing) and reduction of the load due to rock socket effect has been neglected for the overlying rock. This is an extremely conservative assumption.

Mass deformation modulus for each type of foundation layer has been computed from lower-bound elastic modulus of intact rock in each weathering grade for various rock mass factors or reduction coefficients, corresponding to various RQD classes, to take into account the jointing in the rock mass (Table 10). In the final analysis of settlement for each class of rock mass, a rock mass factor of 0.8 for the fresh granite, 0.5 for the slightly weathered granite and 0.2 for the moderately weathered granite have been selected as design factors based on average RQD of each class of rock mass (Deere et al, 1966).

The values calculated by this method (Table 17) are the upper-bound values for settlement, as conservative values for rock mass factors and elastic modulus have been adopted in calculating the rock mass modulus. Effect of rock socket in reducing the end-bearing stress and settlement has been ignored.

The maximum settlement computed for caisson diameters of 1 to 6 m is less than 9.5 mm for the slightly weathered granite and less than 19.1 mm for the moderately weathered granite for a high bearing stress of 15 MPa.

In Figures 11 to 14, total settlements computed for each class of weathered granite have been plotted against the pile diameter for various RQD classes.

7.3 Rock Socket

Shaft resistance or bond strength of piles and caissons socketed into granitic rock have been calculated using the following formula recommended by the Draft Australian Piling Code, and also suggested by Pells et al (1978) and Coates (1967) :

$$\text{Shaft resistance (bond strength)} = 0.05\sqrt{f_c'} \quad . \ . \ . \ . \ . \quad (9)$$

The uniaxial compressive strength of the moderately weathered to fresh granite is very much higher than the uniaxial compressive strength of the concrete used for piles. Therefore uniaxial compressive strength of the concrete, is taken as the controlling strength. The shaft resistance values for different grades of concrete are tabulated in Table 18. The shaft resistance has also been calculated by using the following formula (Table 19) :

$$\text{Shaft resistance} = b \ \sqrt{f_c'} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad (12)$$

where b = 0.20 to 0.25

A conservative approach has been adopted in the calculation of shaft resistance. Higher values of shaft resistance have been reported in the literature for weaker rocks such as sandstones and shales (see Tables 5 and 6). The roughness of the caisson-socket wall interface has not been taken into account. The presence of narrow seams and open joints down the socket wall is advantageous as these can produce the required roughness by being washed out during excavation (Pells et al, 1980).

8. CONCLUSIONS

Allowable loadings for end-bearing piles on rock are traditionally conservative. For many structures no great saving would be achieved by adopting higher allowable bearing stresses. However, for large structures imposing heavy loadings founded on numerous deep caissons and piles, considerable saving could be achieved by increasing the allowable bearing stresses and hence reducing the number of caissons or piles.

Safe bearing capacities of 5 MPa for massive crystalline igneous rock in hard sound condition (core recovery greater than 85%) and 3 MPa for medium hard rock (core recovery greater than 50%) have been specified by the Hong Kong Building (Construction) Regulations. However, allowable bearing stresses quoted in most other building codes for the same rock are much higher, usually 10 MPa or more.

It is recommended that the allowable bearing stress for Hong Kong granites could be increased significantly.

Minimum allowable bearing stresses for granites have been determined by using the following methods :

(a) Building Codes - presumptive bearing stress

(b) Building Codes - empirical factors

(c) Canadian Foundation Engineering Methods

(d) RQD method

(e) Settlement

In the calculation of allowable bearing stress for granitic rock for the above methods (c), (d) and (e), the strength of the intact rock material, the deformation modulus of the intact rock and rock mass, the structure of the rock mass (jointing, joint spacing, RQD), the weathered state of the discontinuities and the rock mass have been taken into consideration. Minimum allowable bearing stresses for granitic rock determined by various conventional design methods, using conservative geotechnical parameters, are set out in Table 20.

The design parameters proposed are the upper-bound values for settlement, as conservative parameters for rock mass factor, j, and elastic modulus have been adopted. The effect of rock socket in reducing the end-bearing stress and settlement has also been ignored.

Rock-socketed caissons could be designed to carry their design load in side resistance plus base resistance, provided that no doubt exists regarding the mobilization of either component. Studies indicate that shaft resistance values for strong rocks can be large (Horvath et al, 1980). Shaft resistance of caissons socketed into slightly to moderately weathered granite calculated by conservative methods using the compressive strength of concrete as the weaker material is 1 to 2 MPa depending on the type of concrete used. The roughness of caisson-socket wall interface has not been taken into account. To date, no completely satisfactory design method for such rock sockets has been published. Only recently, Williams et al (1988) suggested a design method based on a combination of settlement and strength criteria.

No laboratory or field testing has been carried out to determine the engineering properties of Hong Kong foundation rocks. Engineering parameters of granite used in this report are based on the work carried out by Irfan (1977) on a wide spectrum of weathered granitic rocks. Lower-bound values have been selected for each grade of rock mass. These values should be verified during construction of caissons by a field and laboratory testing programme.

Highly weathered granite is composed of various proportions of discoloured, weakened rock material and friable soil; the distribution and depth of this zone is variable. If caissons are to be founded in such materials and in extremely fractured areas, field tests such as pressuremeter, plate loading and caisson loading tests should be carried out.

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LIST OF TABLES

Table 1 - Presumptive Allowable Bearing Stress for Rock Specified by Various Building Codes

Code		Massive Crystalline Bedrock (MPa)	Sound Foliated Rock (MPa)	
BOCA	1970	10	4	
Boston	1970	10	5	
Chicago	1970	10	10	20% increase for every 300 mm embedment into rock
Los Angeles	1970	10	4	
Uniform Building Code	1970	0.2 q*	0.2 q*	
San Fransisco	1969	3-5	3-5	
National Building Code of USA	1967	10	4	
Dallas	1968	0.2 q*	0.2 q*	
CP 2004 (BSI, 1972)	1972	10	4	Can be increased for embedment into rock
New York City	1970	6	6	
Hong Kong	1976	5	3	

Legend :

q* Uniaxial compressive strength of intact rock.

Table 2 - Classification of Rock Mass in Terms of Discontinuity Spacing and Empirical Coefficient, K_{sp}

Term	Joint Spacing (m)	K_{sp}
Very Widely Jointed	3	0.4
Widely Jointed	1 - 3	0.25
Moderately Closely Jointed	0.3 - 1	0.1

Table 3 - Effect of Weathering on Rock Mass Properties (Hobbs, 1975)

Term	Grade	Rock Mass Properties
Fresh and Slightly Weathered	I II	In faintly and slightly weathered rock it is possible that the j-value, owing to the reduction in stiffness of the joints as a result of penetrative weathering alone, will show a fairly sharp decrease compared with that of the same rock in fresh state. The intact modulus, by definition, is unaffected by penetrative weathering. The safe bearing capacity is not therefore affected by faint weathering and may be only slightly affected by slight weathering.
Moderately Weathered	III	In moderately weathered rock the intact modulus, and strength, can be very much lower than in the fresh rock and thus j-value will be higher than in the fresh state, unless the joints and fractures have been opened by erosion or softened by the accumulation of weathering products. The intact modulus and strength can be measured in the laboratory, and the bearing capacity assessed, in the same way as for fresh rock. Triaxial tests may be more appropriate than uniaxial tests, and it would be advisable to adopt conservative values for the factor of safety.
Highly Weathered	IV	In highly weathered rock difficulties will generally be encountered in obtaining undisturbed samples for testing. If samples are obtained the strength and modulus will generally be underestimated, frequently by large margins, even with apparently undisturbed samples. In such rock in-situ tests with either the Menard pressuremeter or the plate should be carried out to determine the bearing capacity and settlement characteristics. The greatest difficulties in assessing bearing capacity and settlement are likely to be encountered in highly weathered rocks, in which the rock fabric becomes increasingly disintegrated or increasingly more plastic.
Completely Weathered Residual Soil	V VI	In completely weathered rock and residual soil it may be possible to obtain fair quality samples depending upon the parent rock type and the consistency of the product. Generally the samples will tend to be less disturbed than when taken in the same rock in the highly weathered state. The bearing capacity and settlement characteristics of rock in these extreme states can be assessed using the usual method for testing soils.

Note : In residual granitic soils the usual methods of laboratory testing may be difficult to perform satisfactorily, or may lead to severe disturbance of the sample. As a result there are difficulties in assessing bearing capacity and settlement characteristics by these procedures.

Table 4 - RQD Classes and Rock Mass Factors (Based on Deere, et al, 1966 and Coon & Merritt, 1970)

Quality Classification	R.Q.D (%)	Fracture Frequency per m	Velocity Index, V_F^2/V_L^2	Mass Factor, j
Very poor	0-25	15	0-0.2	0.2
Poor	25-50	15-8	0.2-0.4	0.2
Fair	50-75	8-5	0.4-0.6	0.2-0.5
Good	75-90	5-1	0.6-0.8	0.5-0.8
Excellent	90-100	1	0.8-1.0	0.8-1.0

Legend :

V_F Seismic velocity in the field
V_L Seismic velocity in the laboratory on cores.

Table 5 - Socket Adhesion, Unconfined Compressive Strengths and Achieved End-bearing Stresses of Various Rock Foundations (Thorne, 1980)

REF. NO.	LOCATION	ROCK TYPE	UNCONFINED COMPRESSION (Qu) MPa	ROCK SOCKET MAX. ATTAINED STRESSES				LAB. SECANT MODULUS	CALC. FIELD MODULUS	
				ADHESION		END BEARING				
				MPa	÷Qu	MPa	÷Qu	MPa	MPa	÷Qu
14	WESTMEAD AUSTRALIA	Wianamatta Shale (Shear zones to 100 mm in otherwise fresh rock)	34	2.5	0.07 (1)	28	0.83	3700	3000	88
14	NEWCASTLE AUSTRALIA	Tighes Hill Sandstone	10 to 15	2.5 (2)	0.2	14 (2)	1	-	2500	700
14	BRISBANE AUSTRALIA	Tuff (75mm clay seam just below base)	15 to 20	-	-	1.6	-	-	485	28
14	PERTH AUST.	King Park Shale		1.3	-	-	-	-	1250	-
3 -	SYDNEY AUST.	Sandstone, fresh defect free	27.5	3.0	0.11 (1)	50	1.8	-	1920	70
2	AUCKLAND NEW ZEALAND	Sandstone & Siltstone Variably cemented	0.7 to 1.0	-	-	18.8	22	-	121	140
4	BRISBANE AUST (PLATE TEST)	Argillite	18.2	-	-	12.7 & 7.5	0.7 (1) 0.4 (1)	-	450	25 PLATE TEST
6	NANTICOKE CANADA	Fresh Limestone contains 3mm thick bituminous shale seams at 0.6 to 1.0 m spacing	55 to 125	-		22	>0.18	-	15000	170
6	NANTICOKE CANADA	Slightly Weathered Limestone shale seam as above	55 to 125	-		22	>0.18		4000	45
6	OTTOWA CANADA	Shale, occasional recemented moisture fractures and "thin mud" seams. Intact core lengths 75 to 250 mm	55	3.1	0.06	27.8	0.5	-	Deflection to small to measure	
5	CALIFORNIA U.S.A.	Highly fractured unevenly weathered Sandstone and Shale	-	1.0	-	-	-		124	-
5	U.K.	Hard Shale Fractures 0.3 to 1.0 m spacing	8	1.2 (2)	0.15	1.4 (2)	.17	-	400	50
5	U.K.	Shale (Joints @ 10 to 20 mm)	-	0.25 (1)	-	4.2	-	-	100	-
8	BROOKFIELD NOVA SEOFIA	Shale contains "frequent' weathered zones. Disintegrates on exposure	0.5	0.25 (4)	0.50 (0.007 f'c)	Pullout Test		Rock strength probably conservative		
9	CONVENTRY U.K.	Sandstones & Siltstones with Mudstone bands 0.6 MPa W about 1 m spacing. Fractures @ 100mm or closer spacing.	8.0	0.22 (2)	0.03	3.65 (2)	0.45	-	130	17
10	MELBOURNE AUSTRALIA	Moderately hard, Mudstone and sandstone. Joint spacing average 4 to 10mm many clay filled.	20 (confined at 700 kPa)	0.88 (1)	0.044 (1)	8.05 (1)	0.40 (1)	-	70 to 80	
11	SYDNEY AUSTRALIA	Fissile shale, joints 20 to 100mm spacing. Clay seams 5 to 10mm thick, 20 to 40mm spacing in 150mm shear zone just below base of pile	6 to 40 estimated 30 average	2.48 (3)	0.08	21.8 (3)	0.6	-	2300 to 3200	100
12	HALIFAX CANADA (PLATE TEST)	Steeply dipping weathered slates core recovery BX 65%	-	-	-	3.4	-	-	-	-
12	NORTH QUEBEC CANADA	"Sound Granite" core recovery AXT 95%	-	-	-	24.7	-	-	-	-
12	QUEBEC CANADA (PLATE)	Sandstone, RQD 10%	-	-	-	26.8 (1)	-	-	-	-
12	LABRADOR (PLATE)	Friable Iron formations similar to weathered & Friable sandstone.	-	-	-	5.4 (1)	-	-	-	-
12	CANADA	Fractured & sheared andesite shear zones with soft green chlorite on surfaces BXL core recovery 33 to 75%	10.3	1.1 (1)	0.11 (1) (.06f'c)	-	-	-	390	38
12	CANADA	Horizontally bedded shale core length 75 to 125 mm	20.7	1.7 (1)	0.08 (1) (0.05f'c)	Pullout Test			1130	55
12		Weathered fractured inter-bedded sandstone & shales	-	1.0	(0.03f'c)					
13	ERARING N.S.W.	CLAYSTONE	2.6 to 10.8 5.5 average	Socketted load equivalent to 13.6 MPa end bearing alone or 1.3 MPa in adhesion alone.				Field test showed high creep (5)		

(1) Failure Attained (2) Proportion of end bearing/adhension estimated (3) Concrete shaft failed

(4) Shaft grooved, stress given to outside of grouve. (5) High water inflows thought to have softened claystone.

Table 6 - Shaft Resistance Values for Various Rock Types (from Horvath et al, 1980)

Rock Type	No. of Tests	Unconfined Compressive Strength MPa (psi)	Shaft Resistance MPa (psi)
Shale and Sandstone	28 (50 to 16 000)	0.35 to 110 (17 to 440+)	0.12 to 3+
Limestone and Chalk	10 (150 to 1000+)	1 to 7+ (17 to 300)	0.12 to 2.1
Others	2	0.35 to 10.5+ (50 to 1500+)	0.12 to 1.1 (17 to 160)

Table 7 - Scale of Weathering Grades of Rock Mass (BSI, 1981)

Term	Description	Grade
Fresh	No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.	I
Slightly Weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering.	II
Moderately Weathered	Less than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.	III
Highly Weathered	More than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	IV
Completely Weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	V
Residual Soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI

Table 8 - Description of Weathering Grades of Rock Material (BSI, 1981)

Term	Description
Fresh	No visible sign of weathering of the rock material.
Discoloured	The colour of the original fresh rock material is changed and is evidence of weathering. The degree of change from the original colour should be indicated. If the colour change is confined to particular mineral constituents this should be mentioned.
Decomposed	The rock is weathered to the condition of a soil in which the original material fabric is still intact, but some or all of the mineral grains are decomposed.
Disintegrated	The rock is weathered to the condition of a soil in which the original material fabric is still intact. The rock is friable, but the mineral grains are not decomposed.
Note : The stages of weathering described above may be sub-divided using qualifying terms, for example, 'partially discoloured', 'wholly discoloured', and 'slightly discoloured', as will aid the description of the material being examined. These descriptive qualifying terms may be quantified if necessary.	

Table 9 - Engineering Properties of Granitic Rocks (Based on Irfan, 1977; Dearman & Irfan, 1978 a, b; Irfan & Dearman, 1978)

Mass Weathering Grade	Material Weathering Grade	Uniaxial Compressive Strength, UCS_s * (MPa)	Point Load Strength, PLS_s * (MPa)	Tangent Young's Modulus, E_t (GPa)	Poisson's Ratio
I Fresh	Fresh	> 150	> 6	60 - 80	0.18 to 0.38
II Slightly Weathered	Partially Discoloured	120 - 150	5 - 6	40 - 60	
	Completely Discoloured	100 - 150	4 - 6	30 - 50	
III Moderately Weathered	Weakened	50 - 100	2 - 4	15 - 30	

s * Saturated condition

Table 10 - Summary of Engineering Properties of Weathered Granitic Rocks Used in the Calculation of Allowable Bearing Stresses

Mass Weathering Grade	Uniaxial Compressive Strength		Tanget Young's Modulus	Rock Mass Factor	Rock Mass Modulus	Poisson's Ratio
	Mean UCS (MPa)	Lower Bound, UCS (MPa)	Lower Bound, E_t (GPa)	Lower Bound, j	(GPa)	
Fresh	175	150	60	0.8	48	0.22
Slightly Weathered	125	100	30	0.5	15	0.22
Moderately Weathered	75	50	15	0.2	3	0.22

Table 11 - Engineering Properties of Granites (Based on the data compiled by Kulhawy, 1975, and Lama & Vutukuri, 1978)

Rock Type	Uniaxial Compressive Strength (MPa)	Modulus of Elasticity (GPa)	Poisson's Ratio	Reference
Granite (mainly fresh)	120 to 300 very few less than 120	50 to 75 very few less than 50	0.18 to 0.27	Lama & Vutukuri, 1978
Granite (mainly fresh)	110 to 325 very few less than 100	55 to 75 very few less than 55	0.14 to 0.39	Kulhawy, 1975
Granite (slightly altered-weathered)	55 to 90	8 to 45	0.10 to	Kulhawy, 1975

Table 12 - Allowable Bearing Stresses for Weathered Granitic Rocks, for K = 0.2

Mass Weathering Grade	Uniaxial Compressive Strength		Allowable Bearing Stress	
	UCS-min (MPa)	UCS-av (MPa)	q_a-min (MPa)	q_a-av (MPa)
I Fresh	150	175	30	35
II Slightly Weathered	100	125	20	25
III Moderately Weathered	50	75	10	15

Table 13 - Allowable Bearing Stresses for Weathered Granitic Rocks Based on the RQD Method

Mass Weathering Grade	R.Q.D.[1]	Allowable Bearing Stress, q_a (MPa)		
		min	av.	max
Fresh	75-100 usually 90	12	20	30
Slightly Weathered	50-90	6.5	12	20
Moderately Weathered	50-75	6.5	10	12

Note : (1) RQD values are from Dearman et al, 1978.

Table 14 - Allowable Bearing Stresses for Weathered Granitic Rocks Calculated by the Canadian Foundation Engineering Method, $K_{sp} = 0.1$

Mass Weathering Grade	Uniaxial Compressive Strength		Allowable Bearing Stress	
	UCS-min (MPa)	UCS-av (MPa)	q_a-min (MPa)	q_a-av (MPa)
I Fresh	150	175	15	17.5
II Slightly Weathered	100	125	10	12.5
III Moderately Weathered	50	75	5	7.5

Table 15 - Empirical Coefficient, K_{sp}, for Weathered Granitic Rocks

Mass Weathering Grade	K_{sp} Pile Diameter (m) 1	2	3	4
I Fresh	0.23	0.22	0,22	0.22
II Slightly Weathered	0.23	0.22	0.22	0.22
III Moderately Weathered	0.13	0.13	0.13	0.12

Table 16 - Allowable Bearing Stresses Determined by the Canadian Foundation Engineering Method, Using Calculated K_{sp} Values

Mass Weathering Grade	Uniaxial Compressive Strength		K_{sp} (1)	Allowable Bearing Stress	
	UCS-min (MPa)	UCS-av (MPa)		q_a-min (MPa)	q_a-av (MPa)
I Fresh	150	175	0.23	34.5	40.3
II Slightly Weathered	100	125	0.23	23.0	28.8
III Moderately Weathered	50	75	0.13	6.5	9.8

Note : (1) Assuming embedment of the pile into sound rock for approximately one diameter

Table 17 - Settlement of Granite Foundations Under Various Loads (Sheet 1 of 3)

Fresh Granite

Ei = 60 GPa

Pile Radius (m)	0.5				1				2				3			
Bearing Stress (MPa)	5	7.5	10	15	5	7.5	10	15	5	7.5	10	15	5	7.5	10	15
Reduction Coeff. I_j	0.8				0.8				0.8				0.8			
Settlement (mm)	0.07	0.10	0.13	0.20	0.13	0.20	0.26	0.40	0.26	0.40	0.53	0.79	0.40	0.60	0.79	1.19
Reduction Coeff., I_j	0.5				0.5				0.5				0.5			
Settlement (mm)	0.11	0.16	0.21	0.32	0.21	0.32	0.42	0.63	0.42	0.63	0.85	1.27	0.63	0.95	1.27	1.91
Reduction Coeff., I_j	0.2				0.2				0.2				0.2			
Settlement (mm)	0.26	0.40	0.53	0.79	0.53	0.79	1.06	1.59	1.06	1.59	2.12	3.18	1.59	2.38	3.18	4.76

Table 17 - Settlement of Granite Foundations Under Various Loads (Sheet 2 of 3)

Slightly Weathered Granite

Ei = 30 GPa

Pile Radius (m)	0.5				1				2				3			
Bearing Stress (MPa)	5	7.5	10	5	5	7.5	10	15	5	7.5	10	15	5	7.5	10	15
Reduction Coeff., I_j	0.8				0.8				0.8				0.8			
Settlement (mm)	0.13	0.20	0.26	0.40	0.26	0.40	0.52	0.80	0.52	0.80	1.04	1.60	0.78	1.20	1.56	2.40
Reduction Coeff., I_j	0.5				0.5				0.5				0.5			
Settlement (mm)	0.22	0.32	0.42	0.64	0.44	0.64	0.84	1.28	0.88	1.28	1.68	2.56	1.32	1.92	2.52	3.84
Reduction Coeff., I_j	0.2				0.2				0.2				0.2			
Settlement (mm)	0.52	0.80	1.06	1.58	1.04	1.60	2.12	3.16	2.08	3.20	4.24	6.32	3.12	4.80	6.36	9.48

Table 17 - Settlement of Granite Foundations Under Various Loads (Sheet 3 of 3)

Moderately Weathered Granite

Ei = 15 GPa

Pile Radius (m)	0.5				1				2				3			
Bearing Stress (MPa)	5	7.5	10	15	5	7.5	10	15	5	7.5	10	15	5	7.5	10	15
Reduction Coeff., I_j	0.8				0.8				0.8				0.8			
Settlement (mm)	0.26	0.40	0.53	0.79	0.53	0.79	1.06	1.59	1.06	1.59	2.12	3.18	1.59	2.38	3.18	4.76
Reduction Coeff., I_j	0.5				0.5				0.5				0.5			
Settlement (mm)	0.42	0.64	0.85	1.27	0.85	1.27	1.69	2.54	1.69	2.54	3.39	5.08	2.54	3.81	5.08	7.62
Reduction Coeff., I_j	0.2				0.2				0.2				0.2			
Settlement (mm)	1.06	1.59	2.12	3.18	2.12	3.18	4.23	6.35	4.23	6.35	8.47	12.7	6.35	9.53	12.7	19.05

Table 18 - Shaft Resistance Values for Piles Socketed into Sound Rock by the Australian Piling Method ($S_r = 0.05\ f_c'$)*

Concrete Grade	Uniaxial Compressive Strength f_c' (MPa)	Shaft Resistance, S_r (MPa)
20	20	1.0
30	30	1.5
40	40	2.0

Legend :

f_c' Uniaxial compressive strength of the weaker material, concrete in this case

Table 19 - Shaft Resistance Values for Piles Socketed into Sound Rock by Horvath et al (1980) Formula ($S_r = 0.22 \sqrt{f_c'}$)

Concrete Grade	Uniaxial Compressive Strength, f_c' (MPa)	Shaft Resistance , S_r (MPa)
20	20	1.0
30	30	1.2
40	40	1.4

Table 20 - Summary of Allowable Bearing Stresses Calculated by Various Design Methods and Proposed Design Values for Granitic Rocks

Rock Type	Allowable Bearing Stress (MPa)							
	Building Codes		Canadian Foundation Engineering Method				RQD Method	Proposed Design Value
	K = 0.2		K_{sp} = 0.1		K_{sp}			
	q-min	q-av	q-min	q-av	q-min	q-av	q-av	q-min
Fresh Granite	30	35	15	17.5	34.3	40.3	20	15
Slightly Weathered Granite	20	25	10	12.5	23.0	28.8	12	10
Moderately Weathered Granite	10	15	5	7.5	6.5	9.8	6.5	5(1)

Note : Up to 10 per cent by volume of completely decomposed or disintegrated soil is permitted.

LIST OF FIGURES

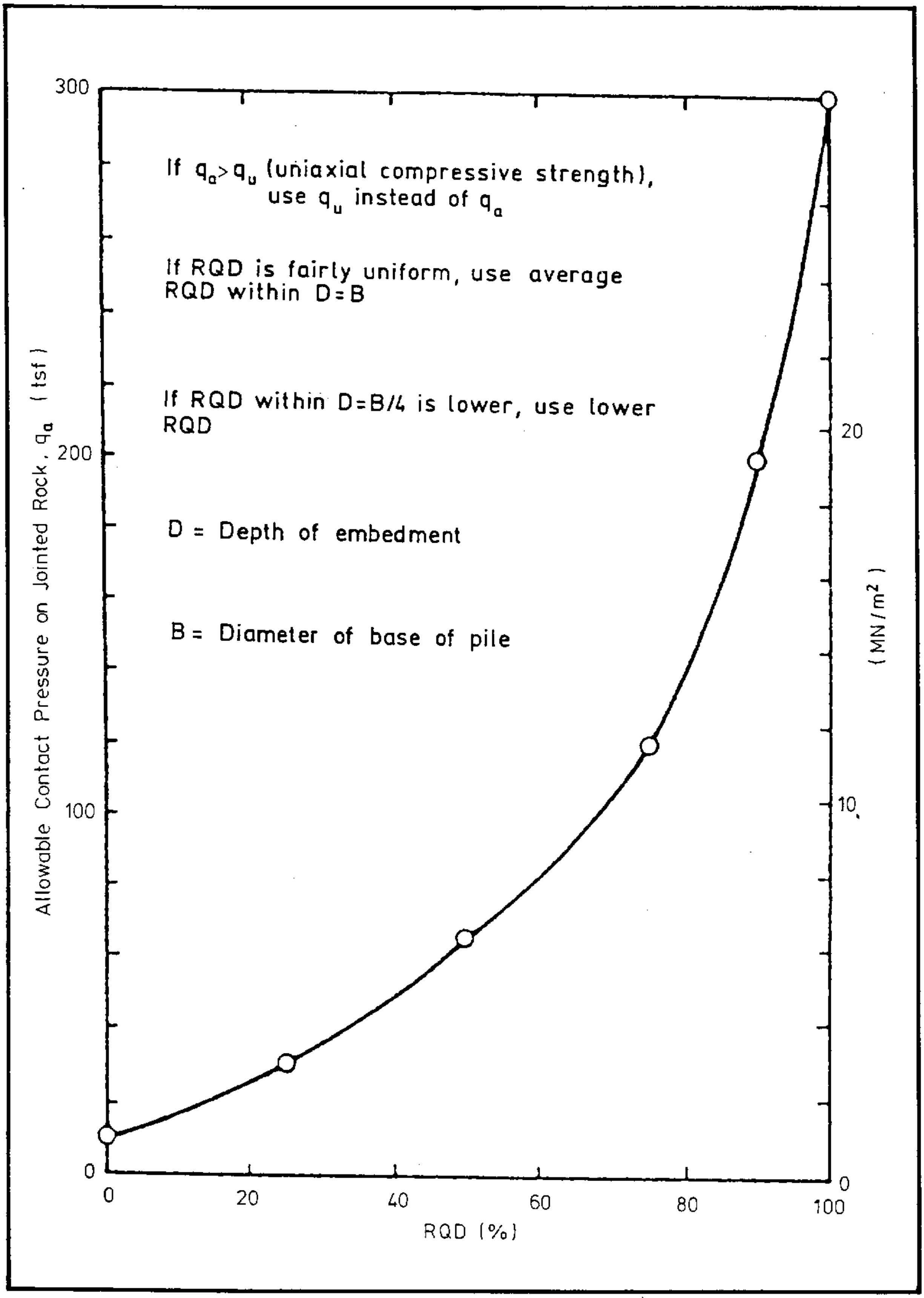
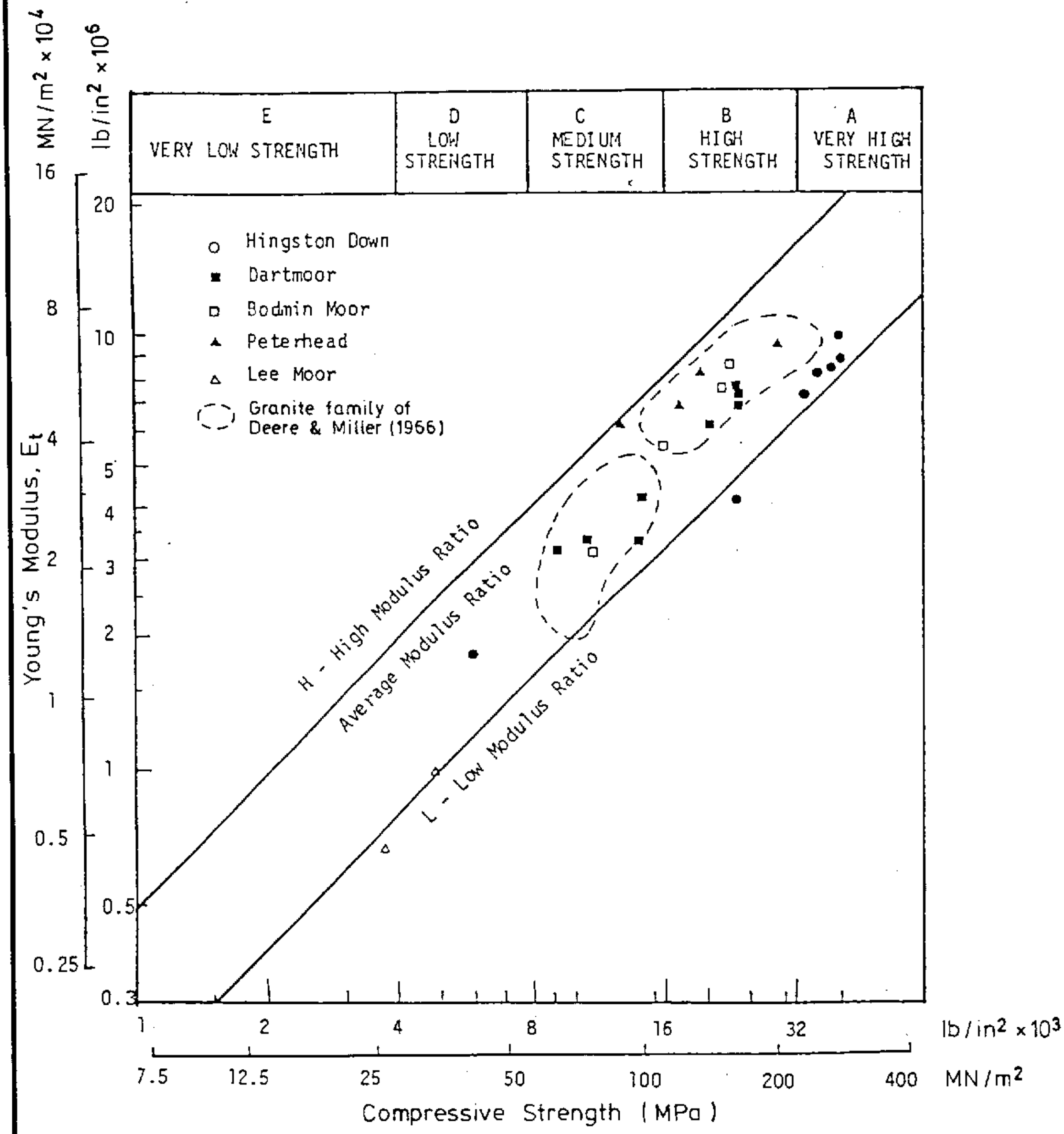


Figure 1 - Allowable Contact Pressure on Jointed Rock
(After Peck, Hanson & Thornburn, 1974)



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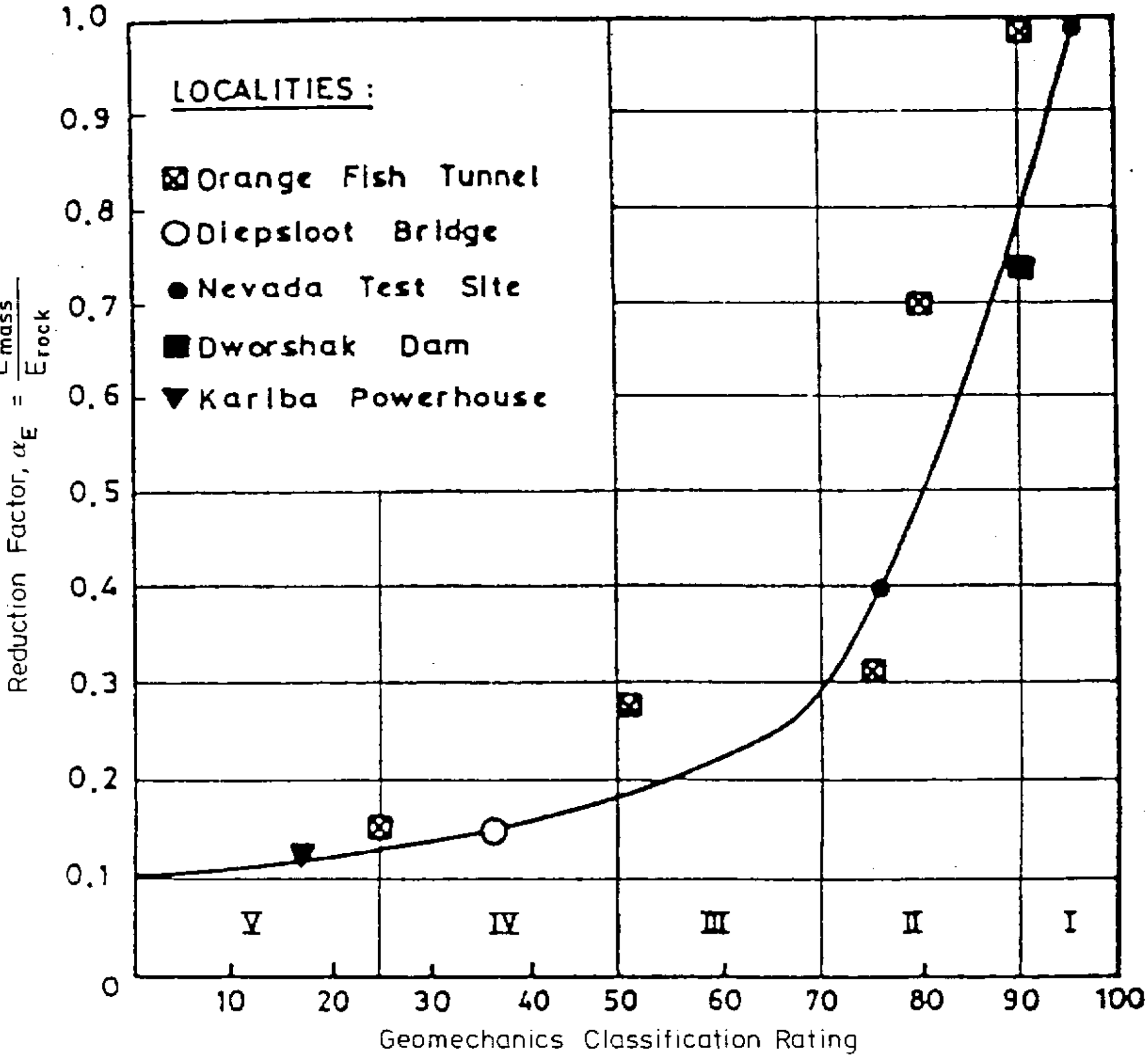

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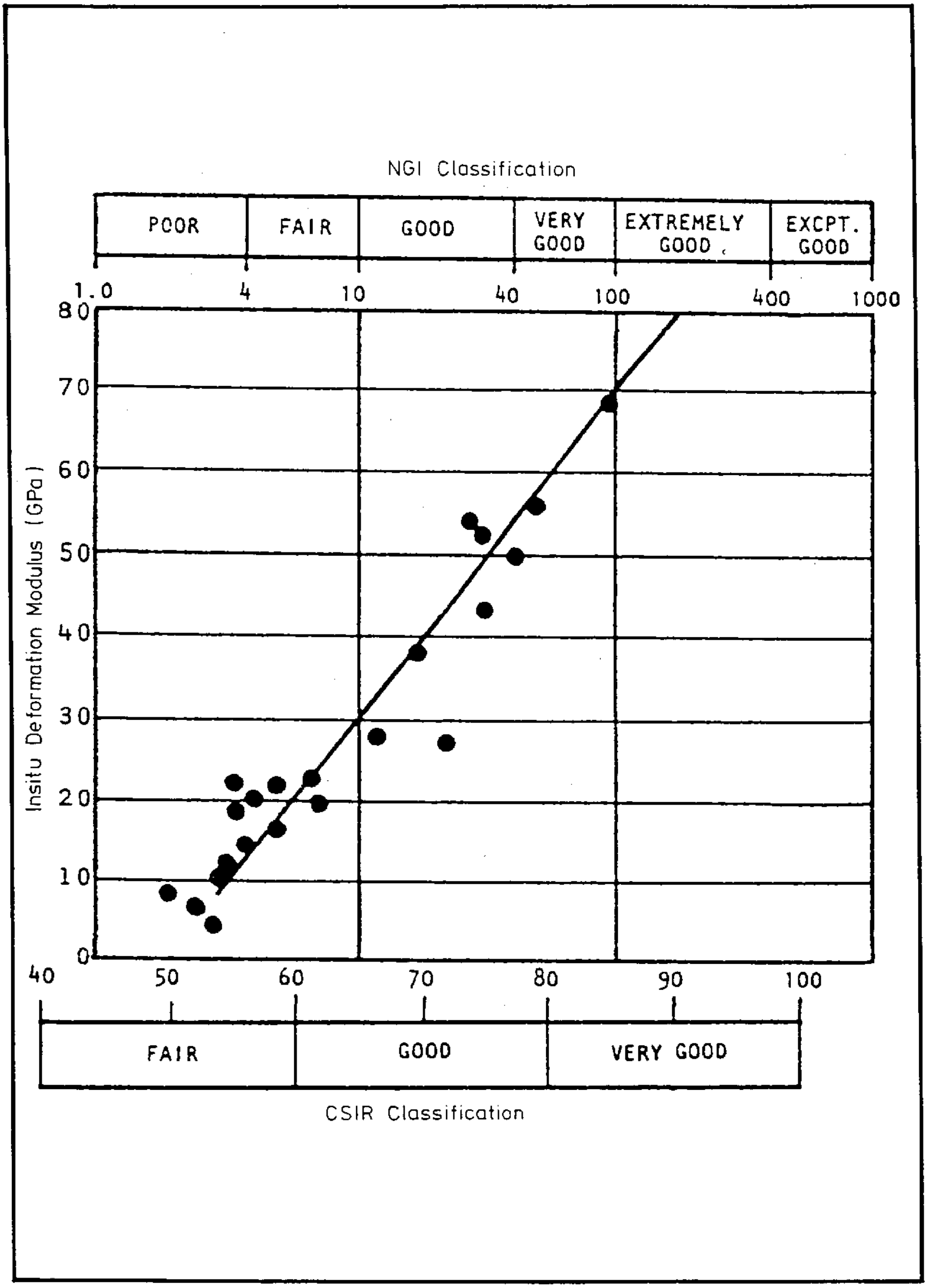
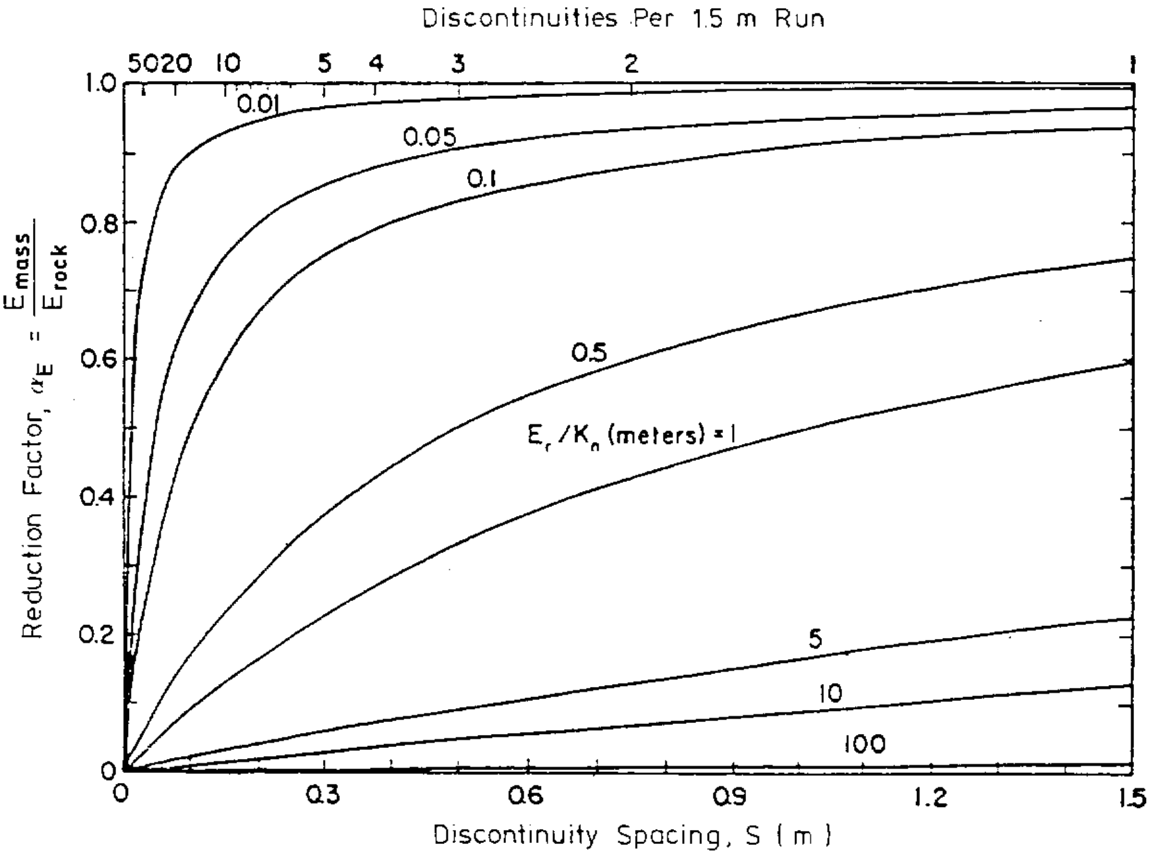


Figure 4 - Relationship between Insitu Deformation Modulus and Rock Mass Rating (Bieniawski, 1978)



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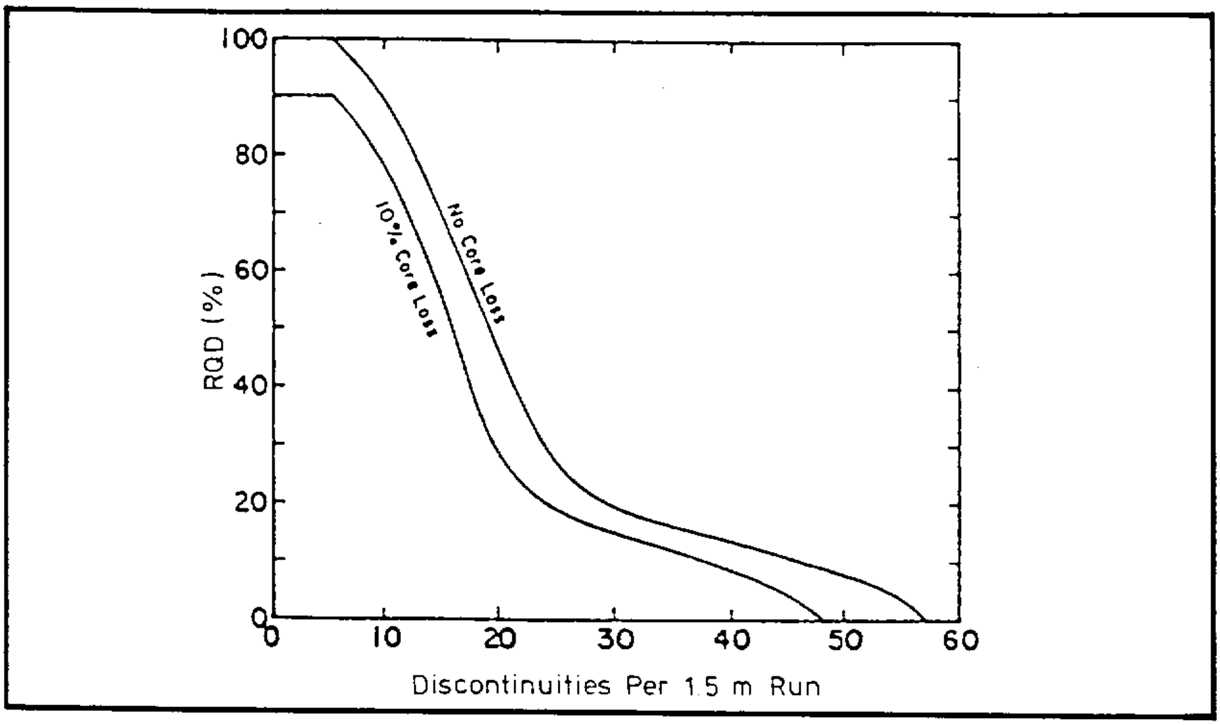

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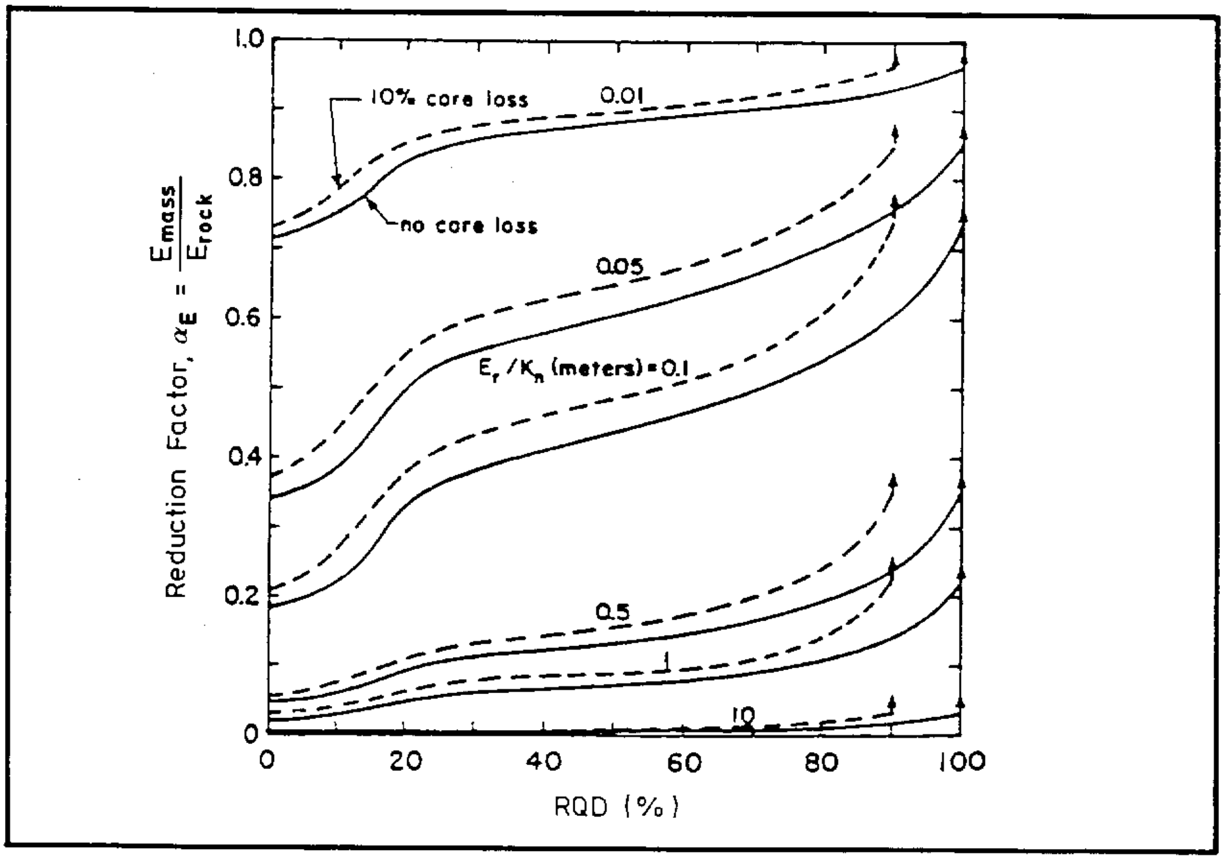
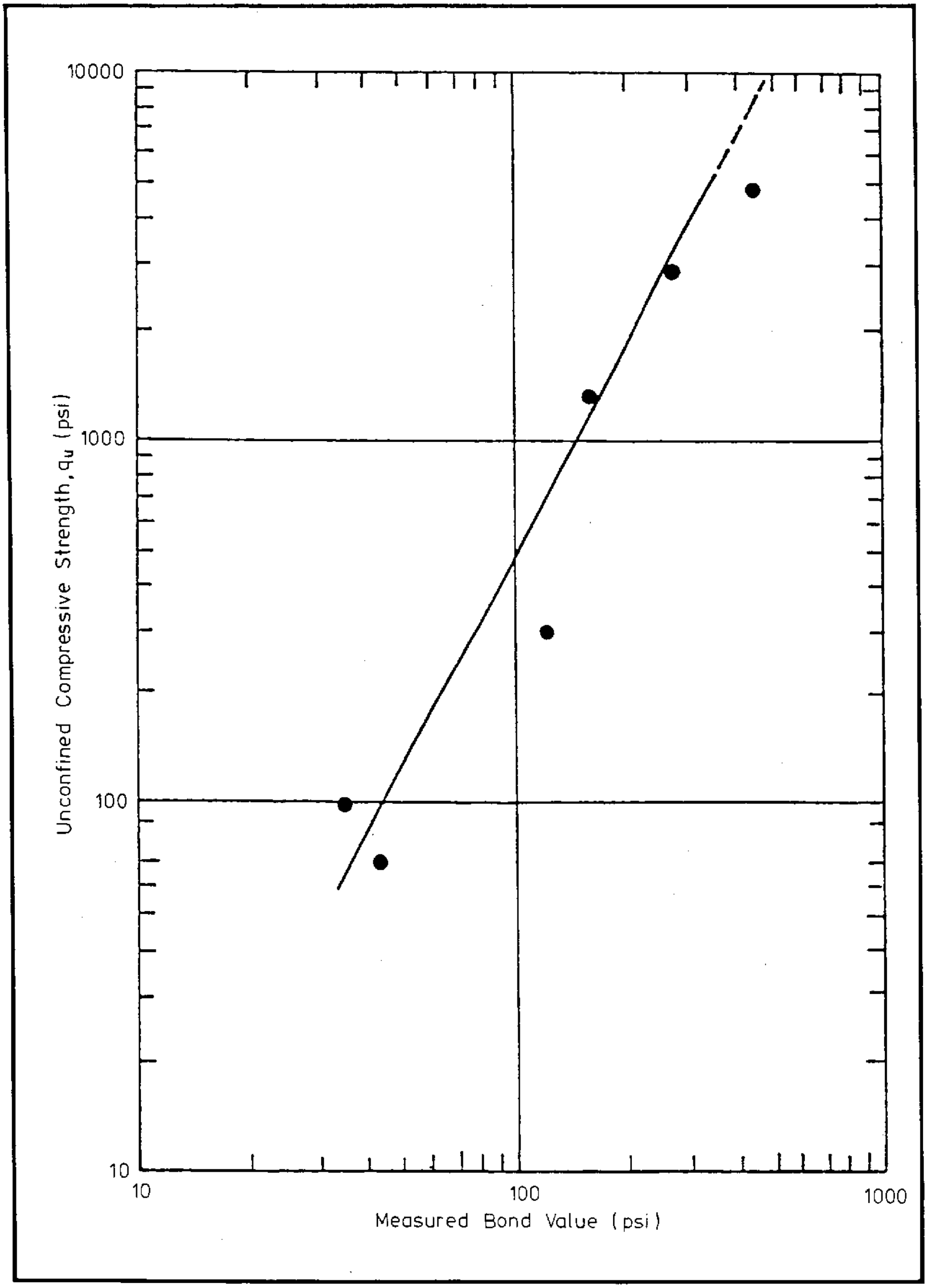


Figure 7 - Modulus Reduction Factor versus RQD (Kulhawy, 1978)



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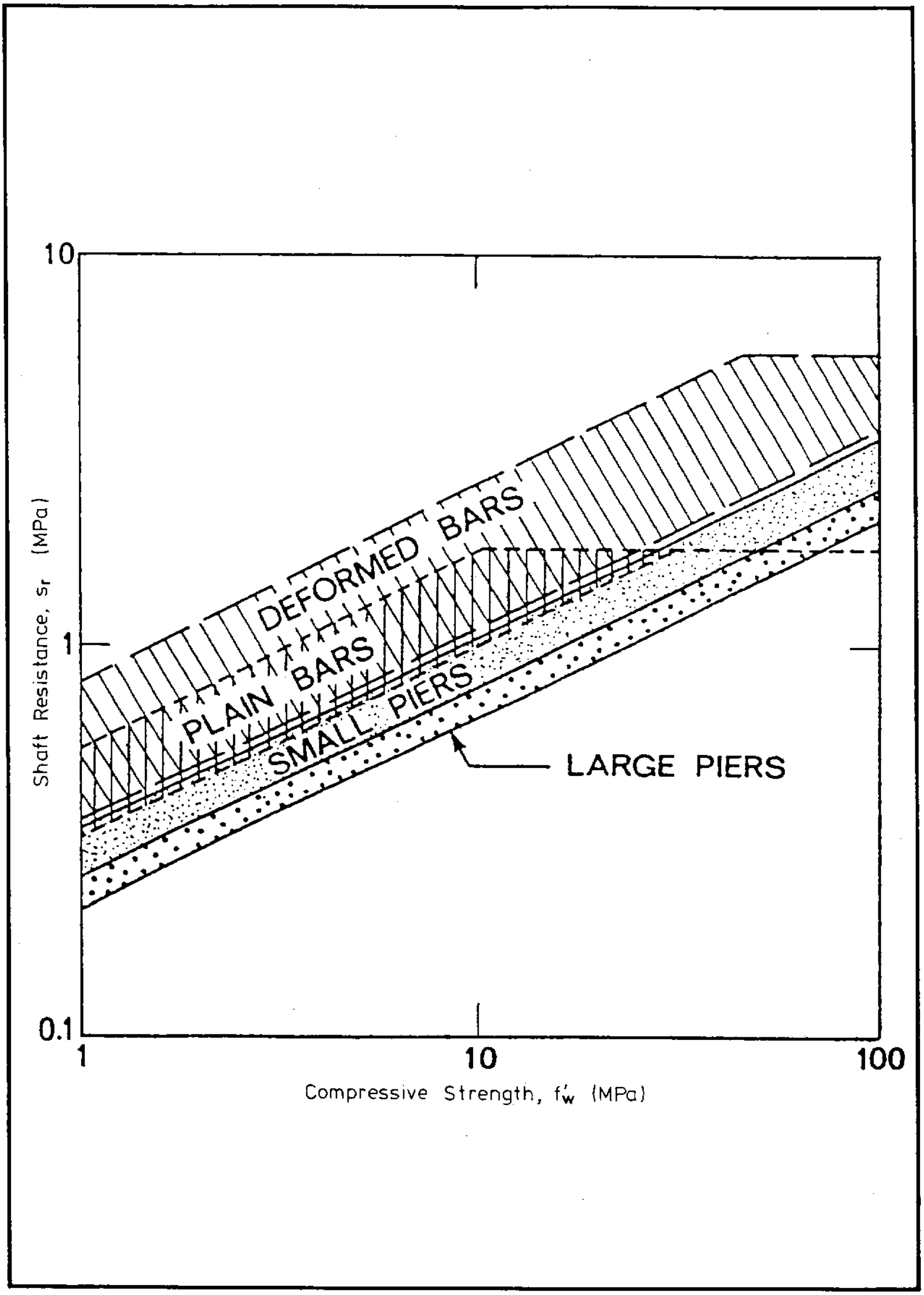

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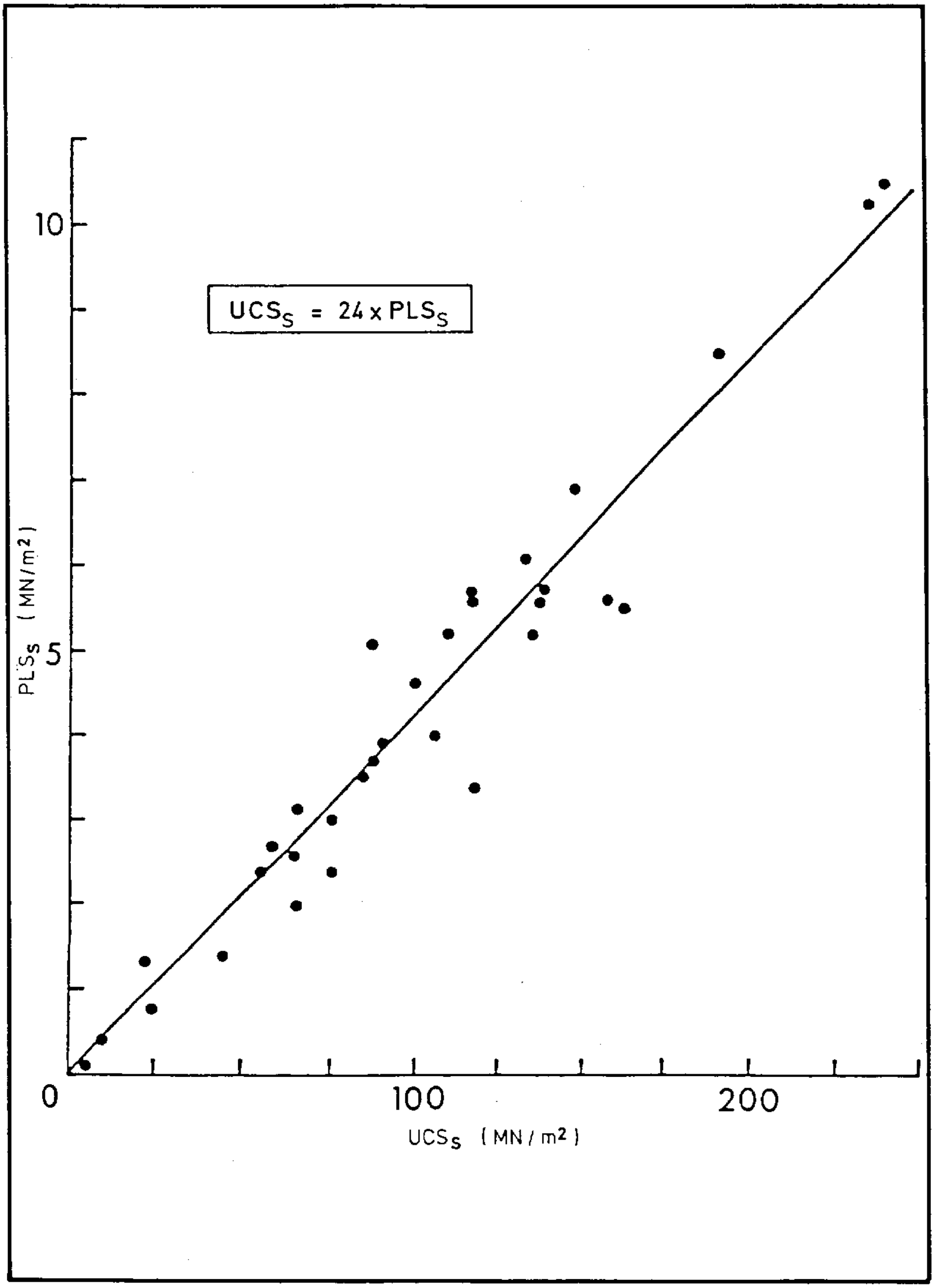

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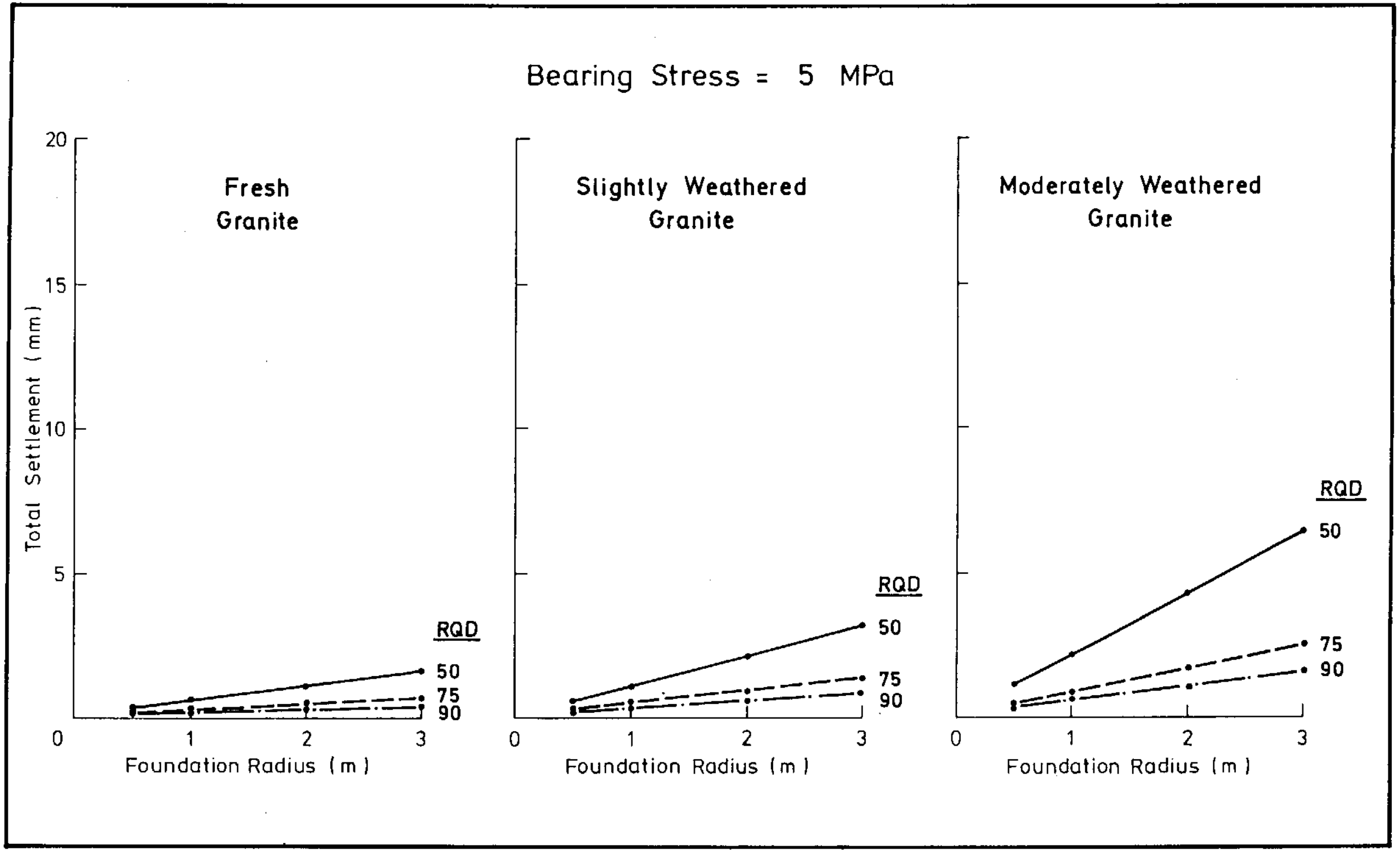

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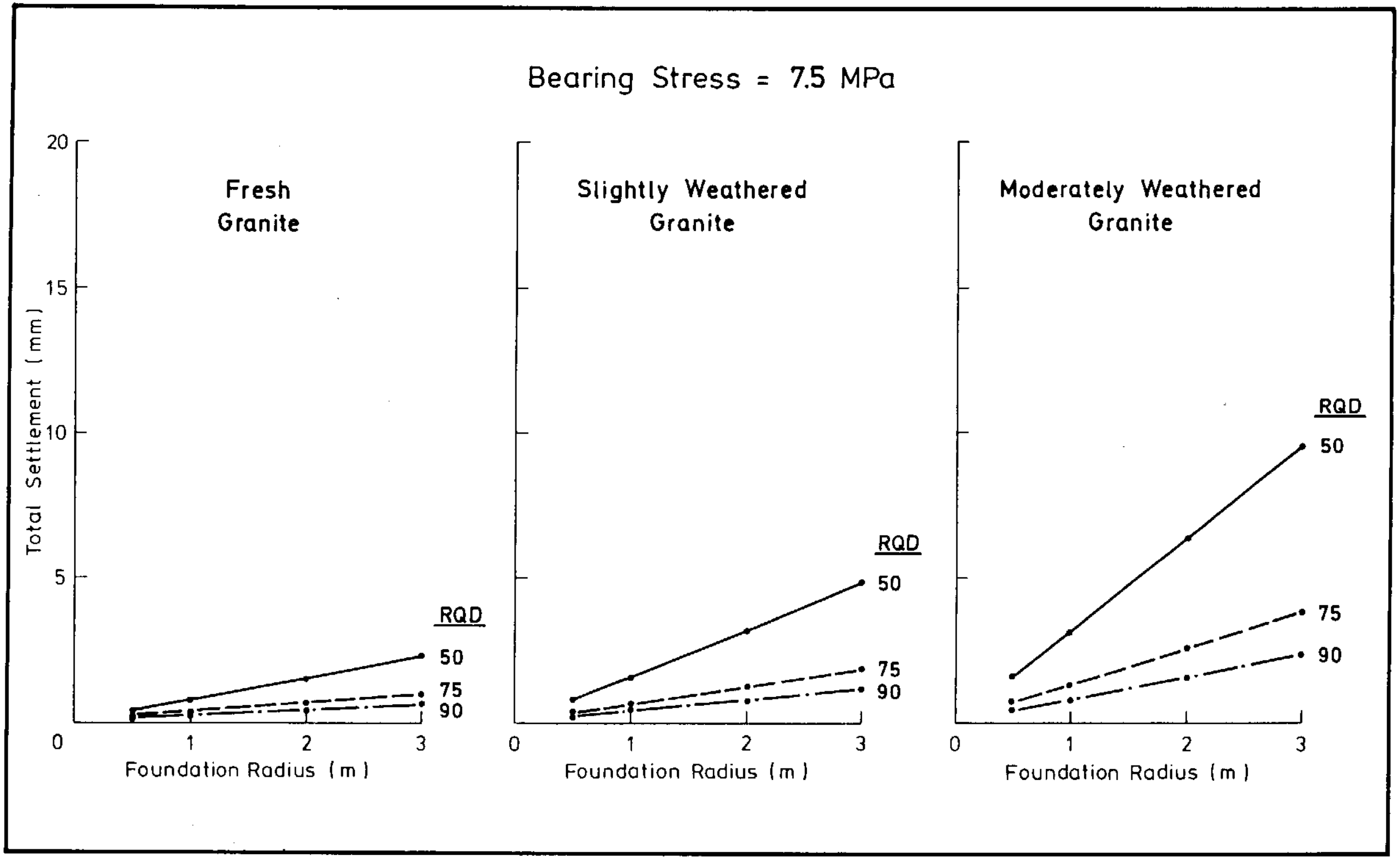

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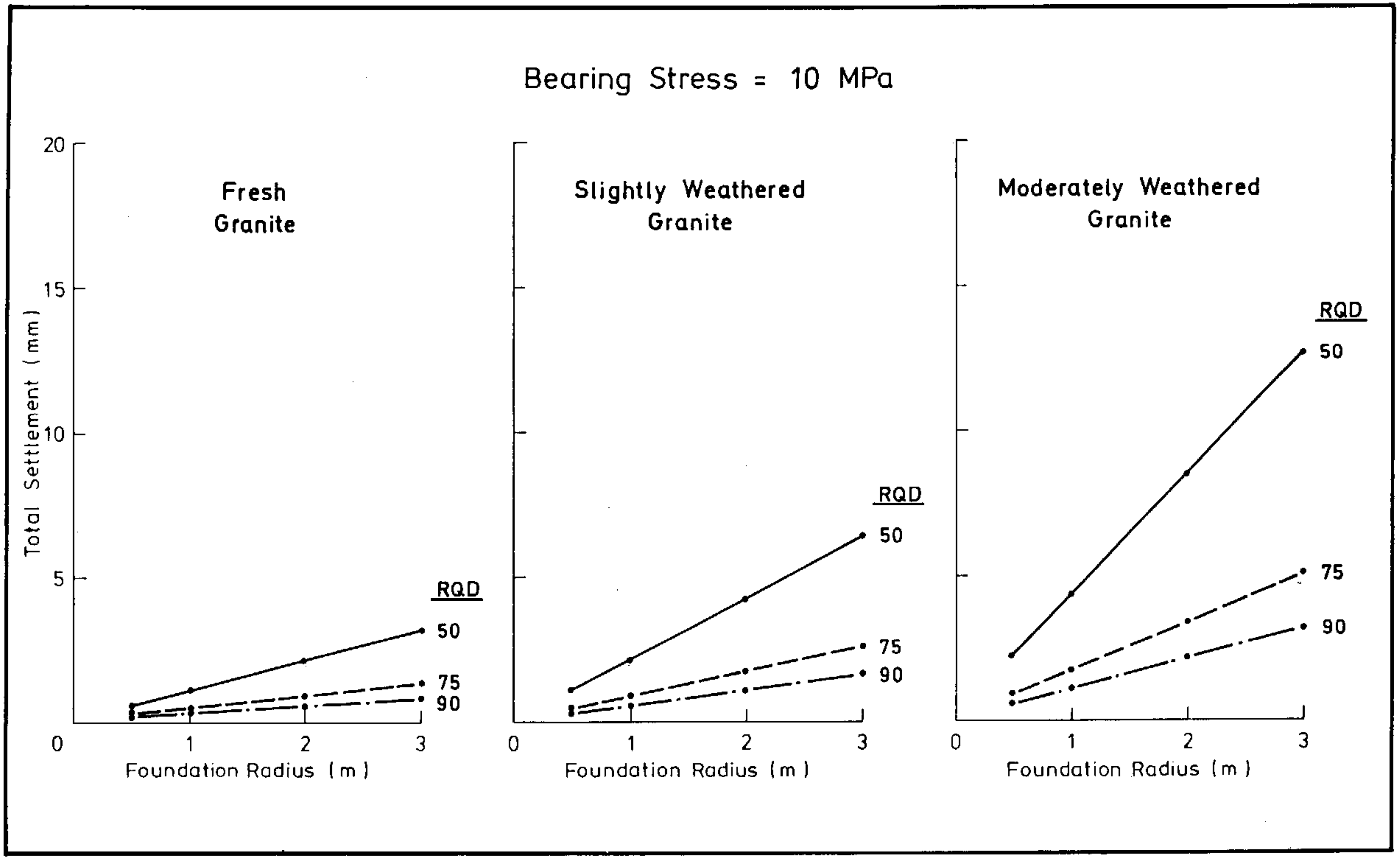

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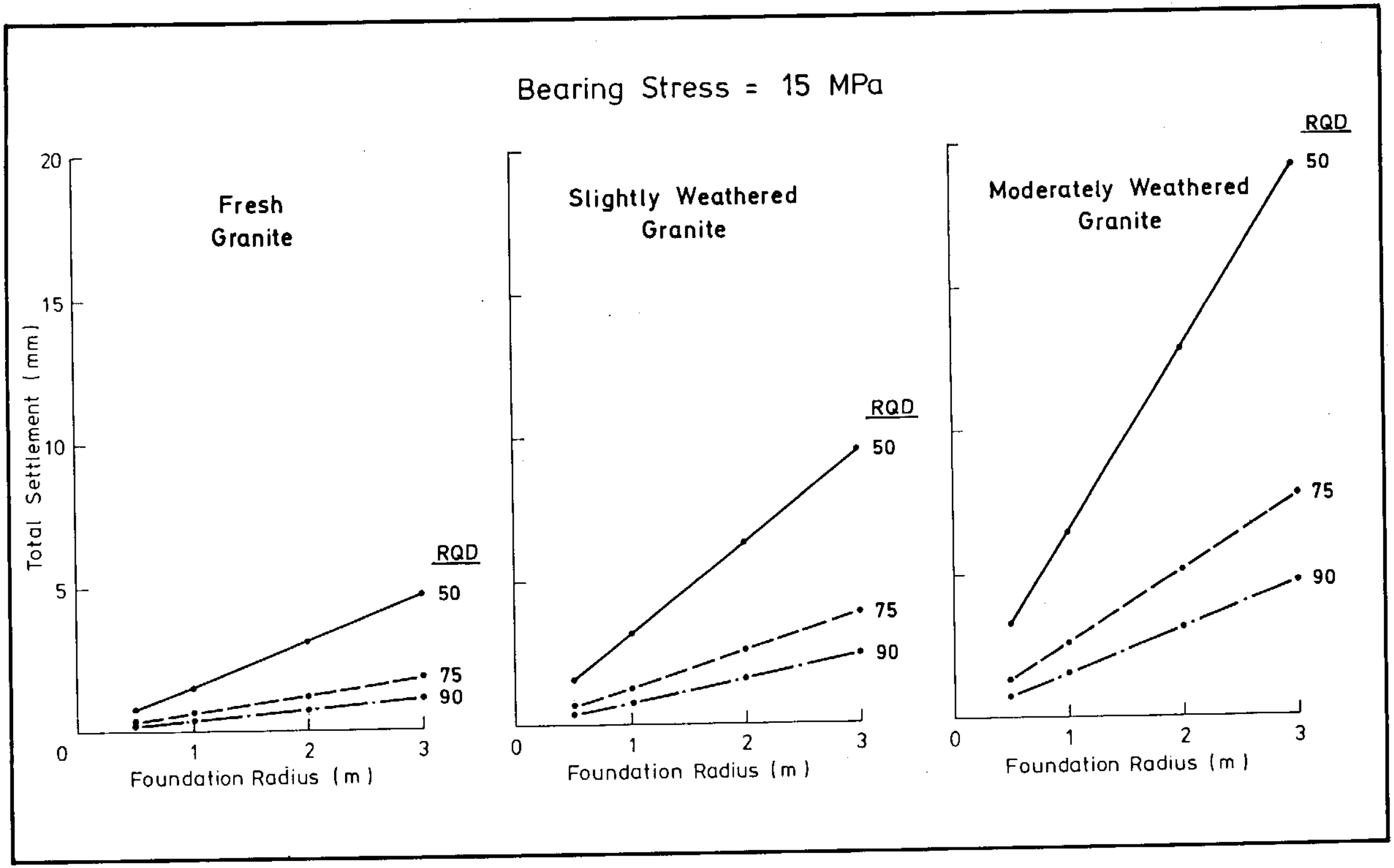

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