2 FOUNDATION DESIGN OF CAISSONS ON VOLCANIC ROCKS : A TECHNICAL REVIEW

T.Y. Irfan

This report was originally produced as GCO Report No. 13/83

CONTENTS

1. INTRODUCTION

Design allowable bearing stress for deep foundations on rock can be selected on the basis of any of four different procedures: building codes, empirical rules, rational methods based on bearing capacity and settlement analysis and field load tests.

Building codes are traditionally conservative and in many structures little financial gain would be achieved by adopting higher allowable stresses. However, in the case of structures imposing large loadings such as bridges, dams, multistorey buildings, considerable saving could be achieved by increasing the allowable bearing stresses based on rational methods of bearing capacity and settlement analysis. Theoretical considerations and field test results suggest that the ultimate bearing capacity of the rock mass is unlikely to be reduced much below the uniaxial compressive strength of the intact rock, even if open vertical joints are present. In most cases where the rock is stronger than the concrete, the allowable bearing stress is governed by the bearing stress at which allowable settlement occurs.

To evaluate the allowable bearing stress and settlement of foundations on discontinuous rock, the rock mass must be characterized. Characterization of rock masses for foundation analysis and design can only be done with a full appreciation and understanding of the role of the geologic factors involved (Kulhawy & Goodman, 1980). Some of the factors that are considered in the characterization of rock masses for foundations are:

(a) rock type,
(b) discontinuities (orientation, spacing, continuity, openness, infilling),
(c) structure of the rock mass,
(d) rock material properties (strength, deformation modulus),
(e) weathering and alteration,
(f) groundwater,
(g) durability,
(h) in situ stresses, and
(i) geometry of imposed structure in relation to the geological structure.

Volcanic deposits usually give rise to extremely variable foundation conditions due to wide variations in strength, durability and permeability. Generally, the older volcanic rocks do not present many problems in foundation engineering unless they are weathered. Hong Kong volcanic rocks are geologically very old deposits; ash and lava deposits have been welded together, hardened under great pressures, slightly metamorphosed and transformed into very strong rocks with strengths in excess of 150 MPa in the fresh state. However, these rocks have been weathered extensively by dominantly chemical processes under Hong Kong's sub-tropical climate over a very long period of time. Weathering significantly reduces the strength and deformation properties of the rock.

In this report, allowable bearing stresses and settlements of the caisson foundations on weathered Hong Kong volcanic rocks have been computed, with particular reference to the fresh to moderately weathered rock mass grades, by various well-established and widely used foundation design methods. The volcanic rocks have also been characterized in terms of mass and material properties for the purposes of foundation analysis and design.

Detailed discussion of various methods of calculation of allowable bearing stresses and settlement and the effect of weathering on the rock mass properties have been given in "Foundation Design of Caissons on Granitic Rocks: A Technical Review" (Irfan & Powell, 1982).

2. DESCRIPTION AND CLASSIFICATION OF VOLCANIC ROCKS

2.1 General

The engineering behaviour of rock masses is controlled by the mass and material properties of the rock. Therefore, 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 structure, coupled with an assessment of the engineering properties of the rock material such as strength and deformation modulus. A detailed semiquantitative or quantitative description of rock material properties supplemented by description of discontinuities and weathering state provides one of the best engineering geological schemes for grading rock masses which then can be used in the preliminary design of foundations.

2.2 Geological Description of Volcanic Rocks in Hong Kong

Volcanic rocks of Hong Kong are grouped under the Repulse Bay Formation by Allen and Stephens (1971). The Repulse Bay Formation consists of a succession of tuffs, agglomerates, ignimbrites and mainly acid lavas deposited subaerially with several intercalated units of sedimentary rocks. These rocks have been taken to a few kilometres within the crust, hardened, folded, faulted and slightly metamorphosed by a granitic batholith.

The volcanics have been subdivided on the basis of major lithotypes into the following classes :

(1)	Undifferentiated volcanic rocks	R_B
(2)	Sedimentary rocks and water-laid Volcani-clastic rocks	R_{Bs}
(3)	Acid lavas	R_{Bv}
(4)	Mainly banded acid lavas, some welded tuffs	R_{Bvb}
(5)	Coarse tuffs	R_{Bc}
(6)	Agglomerates	R_{Bag}
(7)	Dominantly pyroclastic rocks with some lavas	R_{Bp}

The pyroclastics and lavas are similar in composition to granites. The term pyroclastic is used for rocks formed by all mechanisms of dispersal of debris extruded from a volcano which are distinct from lava flows and explosive pyroclastic erupts (Allen & Stephens, 1971). The common names used for pyroclastic rocks are : fine tuff, coarse tuff, lapilli tuff, tuff breccia, pyroclastic breccia and agglomerate.

The volcanic rocks of the Repulse Bay Formation are well-jointed. Generally three or more sets of joints occur giving the rock mass a blocky-tabular structure. Joint spacing commonly ranges from 200 to 600 mm (moderately jointed, ISRM 1978) but in the vicinity of fault and shear zones a joint spacing of 20 -200 mm is more common. Coarse tuffs may have widely-spaced joints, 600 - 2 000 mm, and may produce corestones when weathered.

Weathering is generally shallow in the volcanics with an average depth in the order of 10 metres except along the fault and shear zones and also in certain kinds of tuffs and sedimentary rocks where it may extend much deeper, to depths of 40 m or more. The depth of weathering may vary considerably over short distances due to the variations in grain size, mineral content, jointing pattern and hydrological conditions. The change from fresh volcanic to completely weathered volcanic is generally gradual and corestones are rare.

2.3 Scale of Weathering Grades in the Rock Mass

Rock can weather by chemical decomposition or by physical disintegration. Generally both mechanical and chemical effects occur together, but one or the other may be dominant depending on the climatic region during active weathering. In Hong Kong, rocks are weathered to great depths by dominantly chemical processes (decomposition) under a humid sub-tropical climate over a long period of time.

A scale of weathering grade in a rock mass may be erected on the relative intensity of decomposition and disintegration. The scale of weathering grade used in this report is the one recommended by BS 5930 (BSI, 1981) (Tables 1 and 2). This scheme, unlike earlier schemes, does not take account of changes either in rock strength or whether or not discontinuities are open as these are distinct aspects of the rock mass which are dealt with when the rock mass is described fully. The scheme is based on the visual recognition of the degree of discolouration, the proportion of rock decomposed and/or disintegrated to soil and the presence or absence of the original rock fabric. The rock mass grade can then be based on the weathering grade supplemented by determination of discontinuity and material strength grades. This in turn can be used to estimate the modulus of deformation of the rock mass.

3. ENGINEERING PROPERTIES OF VOLCANIC ROCKS

3.1 General

There is very little published data on the engineering properties of the fresh and weathered volcanic rocks of the Repulse Bay Formation. Published laboratory test data compiled by Lama and Vutukuri (1978) on the strength and deformation properties of volcanic rocks are given in Table 3. Figure 1 is an engineering classification of the volcanics such as dacite, andesite and rhyolite in terms of Young's modulus and uniaxial compressive strength.

Most of published test data on strength and deformation properties of volcanics are from relatively recent tuff deposits of USA and Japan with very low densities of 1.6 to 2.0 mg/m^3 and strengths of 3 to 40 MPa.

3.1.1 Hong Kong Data

Repulse Bay Volcanics are much older in age. The have been compressed under great pressures and metamorphosed to some degree with densities in excess of 2.5 Mg/m^3 and strengths of 100 MPa or more in the fresh state. Available test data compiled from various Public Works Department (P.W.D.) projects on the strength and deformation properties of the Repulse Bay volcanic rocks is tabulated in Table 4. The validity of some of the test results are questionable, particularly in the case of fresh volcanics where very low compressive strength and tangent Young's modulus values are obtained. Failures through the discontinuities may have been included in these results and the testing method is unlikely to have met the ISRM standards.

3.2 Classification of Hong Kong Volcanic Rocks in Terms of Strength and Deformation Properties

Tables 5 and 6 set out the classification of weathered Hong Kong volcanic rocks in terms of material properties such as the uniaxial compressive strength and the tangent Young's modulus and the rock mass properties such as the joint intensity and the RQD. The corresponding rock mass factors, (Hobbs, 1974) given in Table 6 is based on Deere et al (1966), and Coon and Merritt (1970).

In Hong Kong, volcanic rocks are closely to moderately jointed, and have lower than usual RQD's, usually in the range of 50-75% when fresh.

The mass deformation modulus to be used in the settlement analysis has been calculated for each rock mass class from the following formula :

$$j = \frac{E_m}{E_i}$$

where j is the rock mass factor,
E_i is the deformation modulus of the intact rocks, and
E_m is the deformation modulus of the rock mass

Lower-bound and mean values of uniaxial compressive strength and tangent Young's modulus have been selected as the design parameters (Table 7) for each rock mass class (foundation layer) and used in the calculation of bearing stresses and settlements of the pile and caisson foundations on volcanic rocks.

The rock mass modulus and the effect of weathering on mass engineering properties are further discussed elsewhere (Irfan & Powell, 1982).

4. ALLOWABLE BEARING STRESSES AND SETTLEMENTS FOR FOUNDATIONS ON VOLCANIC ROCKS

4.1 Allowable Bearing Stress and Settlement

Theoretical considerations suggest that the ultimate bearing capacity of rock is unlikely to be reduced much below the uniaxial compressive strength of the intact rock, even if open vertical joints are present (Poulos & Davis, 1980). On the basis of available data, an allowable bearing stress in the order of 0.3 X unconfined uniaxial compressive strength would appear to be quite conservative for all rocks except swelling shales. Reference to the theoretical solutions show that such values generally include a factor of safety of at least 3 in fractured or closely-jointed rocks and 12 or more for intact rocks. Thorne (1980) recorded end-bearing stresses of 0.3 to 4 x uniaxial compressive strength in actual field tests carried out in different rock types and in most cases no failure occurred (Table 8).

In a case where the rock is stronger than the concrete, the bearing capacity of the rock imposes no practical limitation on the foundation and the allowable bearing capacity is then governed by the bearing stress at which allowable settlement occurs.

There are various ways of determining allowable bearing stress of foundations on rock. In this review, the allowable bearing stresses for Hong Kong volcanic rocks are determined by using the following methods :

(i) Building codes - presumptive bearing stress
- empirical bearing stress

(ii) RQD method

(iii) Canadian Foundation Engineering Methods

(iv) Settlement

A brief description of these methods is given in Irfan and Powell (1982).

4.2 Calculation of Allowable Bearing Stress

4.2.1 Building Codes

Presumptive allowable bearing stresses specified by various building codes and authorities differ in their recommendation for the same kind of rock (Table 9). No specific values are given in these codes for volcanics except that the rocks are classified in general terms such as massive crystalline rocks in sound condition, foliated metamorphic rocks in sound condition, sedimentary rocks, shattered rocks, weathered rocks, etc.

Widely- to very widely-jointed (joint spacing of 600 mm or greater), fresh to slightly weathered volcanics can be considered as massive crystalline rocks with presumptive bearing stresses of 5 to 10.7 MPa. Fresh volcanics with medium-spaced joints (joint spacing of 200 to 600 mm) can be considered as "foliated rock in sound condition" or stronger sedimentary rock with presumptive bearing stresses of 2 to 6 MPa. When the rock is closely jointed or highly fractured, or moderately to highly weathered, then the presumptive allowable bearing stresses specified range between 0.5 and 3 MPa, but are generally in the range of 1 to 1.5 MPa.

Uniform Building Code (1964) and Dallas Code (1968) relate the allowable bearing stress to the uniaxial compressive strength of the intact rock material by the following formula :

Allowable bearing stress = K x Uniaxial compressive strength of core

Table 10 gives allowable bearing stresses calculated for Hong Kong volcanic rocks by using lower-bound and average uniaxial compressive strength values and K = 0.2 as specified by these codes for each grade of rock mass.

4.2.2 RQD and Allowable Bearing Stress

Allowable bearing stress of volcanic rocks calculated for a range of RQD values using the empirical correlation between the allowable contact stress and RQD suggested by Peck et al (1974) are tabulated in Table 11. The allowable bearing stress for the fresh to slightly weathered volcanic rocks determined by this method is over 6.5 MPa even for an RQD value as low as 50%. In the moderately weathered volcanics with very low RQD's this value may reduce to 1 MPa. If design is based on these values settlement of the caisson foundations is not expected to exceed 12.5 mm (Peck et al, 1974).

4.2.3 Canadian Foundation Engineering Method

Allowable bearing stresses of Hong Kong volcanic rocks have also been calculated by using the following formula (Canadian Geotechnical Society, 1978):

$$q_a = K_{sp} \cdot q_u \cdot d$$

where q_a is the allowable bearing stress,
q_u is the average uniaxial compressive strength of rock cores,
K_{sp} is an empirical coefficient depending on the discontinuity spacing and including a factor of safety of 3, and
d is the depth factor and equal to :

$$d = 0.8 + \frac{H_s}{D} \leqslant 2$$

where H_s is the depth of socket, and
D is the diameter of the socket.

In the calculation of K_{sp}, a joint spacing of 300 mm and joint thickness of 1 mm for fresh to slightly weathered rocks and 5 mm (or less than 25 mm if filled with soil or rock debris) for moderately weathered rocks, have been adopted. In moderately weathered volcanic rocks the material around the joints may be extremely weak or completely weathered to soil. In the weathering grade classification, moderately weathered rock may contain up to 50% decomposed or disintegrated material (BSI, 1981). For the purposes of this study the cut-off point is taken at 10% soil within the zone of moderately weathered rock. This approximately corresponds to 25 mm of decomposed or disintegrated material along the joints in the weathered rock mass. For very closely-jointed rocks or for rocks weathered to higher degrees or where the rock is of very low strength, in situ pressuremeter tests are recommended.

Minimum allowable bearing stresses calculated by the Canadian Foundation Engineering method is over 10 MPa for the fresh to slightly weathered volcanic rocks (Table 12). For moderately weathered volcanic rocks, the allowable bearing stress ranges from 1.3 to 3.5 MPa depending on the strength of intact rock.

4.2.4 Settlement

Settlement in rock foundations can be calculated by elastic analysis, from pressuremeter test results or from plate load tests. If deformation or settlement is the limiting criterion of a structure imposed on a rock mass the deformation modulus of the rock mass is the prime controlling factor in defining this settlement. Therefore, the determination of the rock mass modulus is an important part of the investigation, design and construction process. A large number of in situ pressuremeter tests must be carried out to assess the variability of elastic modulus of the rock mass and the effect of discontinuities on the mass elastic modulus. In situ plate load tests can be used to assess the settlement, but the results depend on the plate size, tests are expensive and difficult to carry out properly, and results are frequently variable.

In the absence of in situ test data settlements can be computed by elastic theory from the properties determined in the laboratory and an empirical rock mass factor (or reduction factor) to take into account the jointed state of the rock mass using the following formula :

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

where s is the settlement,
q is the uniform load per unit area,
E_m is the deformation modulus of the rock mass under the pile,
I_s is the depth reduction factor,
r is the radius of pile, and
ν is the Poisson's ratio.

Various empirical and semi-empirical methods of estimating rock mass modulus have been suggested in the literature (Coon & Merritt, 1970; Woodward, et al,1972; Bieniawski, 1975; Hobbs, 1974; Kulhawy & Goodman 1980; Thorne, 1980). In the present investigation, the rock mass moduli have been computed from the lower-bound and average elastic moduli of the intact volcanic rock in each foundation layer, using various rock mass factors corresponding to a range of RQD classes and joint intensities as discussed in Section 3.2.

Table 13 gives the maximum and average computed settlements of caisson foundations on weathered volcanic rocks under different bearing stresses. A depth reduction factor of I_s = 0.85 for ν = 0.25 has been adopted to allow for an average embedment of the foundation by approximately one diameter (Burland, 1970).

In computing the settlement, it is assumed that all the load is transferred to the base and the reduction of the load due to the rock socket effect has been neglected for the overlying rock. Elastic shortening of the concrete shaft of caisson is not included in these calculations. For hand-dug caissons a considerable proportion of the load is expected to be taken up by side friction. In addition, for rock-socketed piles lateral dilation of the caisson into the socket may take the majority of the remaining load, and relatively little load will be transferred to the base. These latter two relatively little load will be transferred to the base. These latter two mechanisms could be expected to reduce settlements of the caisson.

Settlement may also result from the presence of debris at the bottom of a caisson excavation and a careful inspection is therefore necessary to eliminate this possibility.

Figure 2 shows the range of maximum expected values of settlement against allowable bearing stress for a caisson diameter of 2 m in various foundation layers. The maximum settlement for caisson diameters of 1 to 4 m is less than 12.5 mm in fresh to slightly weathered volcanics for a high bearing stress of 15 MPa, even in highly jointed rock mass with average RQD of 50% or joint intensity of 5-8 joints per metre (Table 13).

In moderately weathered volcanics settlement will depend on the degree of weathering of the rock material, the amount of decomposed or disintegrated soil along discontinuities, the deformation modulus of the rock and the jointing in the rock mass. Allowable bearing stresses of up to 3 MPa will not lead to any excessive settlements in these rocks except in highly fractured zones.

Higher bearing stresses may be used in moderately weathered rock masses with high strengths and favourable joint orientation and spacing. Higher allowable bearing stresses should be based on detailed laboratory and field investigations to determine the mass and material properties of the rock.

4.3 Rock Socket

In the case of foundations on rock, 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).

Shaft resistance values of caisson foundations socketed into rock calculated by the Draft Australian Piling Code (Pells et al, 1978) and Coates (1967) formulae using the compressive strength of the concrete as the controlling strength are in the region of 1 to 2 MPa depending on the type of concrete used. Higher shaft resistance values have been reported in the literature for weaker rocks such as sandstone and shales (Horvath et al, 1980; Thorne, 1980, and Table 8). Further discussion of the effect of rock sockets and the calculation of shaft resistance are given in Irfan and Powell (1982).

If the caissons are to be founded in rocks weathered to higher degrees or in extremely fractured zones, detailed laboratory and field investigation should be undertaken to determine the mass and material properties of the rock.

For hand-dug caissons a considerable proportion of the load will be taken by side friction on the shaft of the caisson. In rock-socketed caissons, side friction coupled with the lateral dilatation of the pile into the socket will lead to reduced load being transferred to the base, and hence reduced settlements.

There is relatively very little local data on the engineering properties of Hong Kong volcanic rocks. Engineering parameters of volcanic rocks used in this report are based on limited Hong Kong data, published results elsewhere in the world and the author's own experience. Further testing will greatly assist to advance the state of the art in volcanics.

Where preliminary design is based solely on borehole data, the importance of accurate and detailed logging of boreholes, particularly in areas where variable rock conditions are likely to be encountered, by a qualified engineering geologist must be emphasized. During construction, the rock in caisson excavations should be visually inspected and described by experienced personnel to verify the rock conditions assumed in the structural foundation design. In situ techniques such as Schmidt hammer, core drilling and percussion air drilling can be used to provide quantitative evidence of in situ rock conditions (Irfan & Powell, 1983).

5. CONCLUSIONS

The current local practice of using a presumptive stress of 5 MPa for fresh to slightly weathered volcanic rocks is overconservative. Considerable saving could be achieved in the case of structures imposing large loadings by increasing the allowable bearing stress based on engineering geological characterization of the rock mass and rational methods of bearing capacity and settlement analysis.

Variable foundation conditions exist in Hong Kong volcanic rocks due to differences in mode of origin, weathering and structure.

Allowable bearing stresses for Hong Kong volcanic rocks have been determined by various conventional design methods available, using conservative geotechnical parameters. Settlement of volcanic rock foundations under various end-bearing stresses have also been calculated (Table 13). Settlement predictions by this methods are upper-bound values, as conservative parameters for elastic modulus have been adopted and the effect of the rock socket in reducing the end-bearing stress and settlement has been ignored.

A range of bearing stresses may be used for caisson foundations in the Repulse Bay Volcanics depending on the rock mass properties existing at a particular site. Figure 2 gives the expected range of maximum settlements in various volcanic rock foundations for a 2 m diameter caisson under different bearing stresses.

In fresh to slightly weathered volcanics an allowable bearing stress of 10 MPa will produce settlements of less than 12.5 mm, according to elastic theory. In moderately weathered volcanics end-bearing stresses of up to 3 MPa could be used without carrying out in situ testing. These allowable bearing stresses are lower-bound values, and if required for a particular project, higher values may be justified when sound design procedures are used with rock mass properties obtained on a site specific basis.

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LIST OF TABLES

Table 1 - Scale of Weathering Grades of Rock Mass

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. Al 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 2 - Description of Weathering Grades of Rock Material

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 3 - Laboratory Mechanical Properties of Volcanic Rocks
(Lama & Vutukuri, 1978) (Sheet 1 of 2)

Rock	Location & Description	Density, ρ (g/cm^3)	Modulus of Elasticity, E (GPa)	Modulus of Rigidity, G (GPa)	Poisson's Ratio, υ	Compressive Strength, σ_c (MPa)	Tensile Strength, σ_t (MPa)	Remarks	Reference
Tuff	USA, McDowell Dam, Ariz., lithic tuff, fg, altered, salt matrix	–	1.38	–	–	15.86	–	grain = 1–2 mm	BRANDON, 1974
	USA, AEC Nevada test site, lapilli tuff, glass with volcanic constituents	–	9.71*	–	0.10*	–	–	variable pore size	YOUASH, 1970
	Mexico,	1.9	1.7*	–	–	–	–	seismic	ULLAO, 1964
	Soledad Dam	2.2	2.7*					field	ULLAO, 1964
	USA, Green Peters Dam	–	7.4	0.19	–	23.0	–	c = 7.0, Ø = 24°	CORNS & NESBITT, 1967
	USA, Green Peters Dam	–	7.0	0.20	–	22.0	–	c = 7.7, Ø = 19°	CORNS & NESBITT, 1967
	USA, AEC Nevada test site	–	3.72	–	0.19	11.31	1.17	–	STOWE & AINSWORTH, 1968
	Japan, Seikan Tunnel	2.19	–	–	–	21.9	2.2	–	MOCHIDA, 1974
	Japan, Seikan Tunnel, tuff-breccia	2.36	–	–	–	28.6	3.9	–	MOCHIDA, 1974
	Japan, Shimane Nuclear Plant	2.4	3.0	–	0.2	–	–	c = 0.4, Ø = 30°	KITAHARA et al, 1974
	Japan, Shimane Nuclear Plant	2.4	2.0	–	0.2	–	–	c = 0.3, Ø = 35°	KITAHARA et al. 1974
	Japan	–	7.14	–	–	34.7	–	–	YAMAGUCHI, 1968
	Japan, Aoishi	–	68.0	–	–	33.8	4.31	sandy	YAMAGUCHI, 1968
Tuff	Japan, Aoishi	1.91	76.0	–	–	36.0	4.31	sandy	YAMAGUCHI, 1968
	Japan, Aoishi	–	70.0	–	–	34.2	4.45	sandy	YAMAGUCHI, 1968
	Canada, Macleod Mine	–	63.43	–	0.24	–	–	–	HERGET, 1973
	Canada, Macleod Mine	–	74.5	–	0.25	–	–	–	HERGET, 1973
	Canada, Macleod Mine	–	61.4	–	0.26	–	–	–	HERGET, 1973
	Canada, Macleod Mine	–	71.7	–	0.28	–	–	–	HERGET, 1973
	USSR, geosynclines + breccias	2.49	22.7	–	0.13	–	–	p = 8.31	BELIKOV, 1967
	Hungary, rhyolite,								
	\#1	2.08	–	–	–	9.4	–	p = 30.0, w = 16.5	MARTOS, 1965
	\#2	2.08	–	–	–	9.8	–	p = 30.0, w = 16.5	MARTOS, 1965
Tuff	Canada, Kirkland, Ont.	2.78	86.87*	32.41*	–	289.58	–	p = 1.5	WINDES, 1949
	USA, Howard Prairie Dam, Ore., lithic	1.45	1.38	–	0.11	3.65	–	p = 42.9	USBR, 1953
	USA, Oak Spring Formation, Nev., bedded	1.6	4.2*	2.1*	–	–	–	p = 37.0	ROBERTSON (Unpublished)
	USA, Oak Spring Formation, Nev., bedded	–	4.9	0.17	0.08	–	–	tension	ROBERTSON (Unpublished)
	USA, Oak Spring Formation, Nev., bedded	–	7.6	3.4	0.11	–	–	compression	ROBERTSON (Unpublished)
	USA, Nev., welded	2.2	10.2*	4.1*	–	–	–	p = 14.0	ROBERTSON (Unpublished)
	USA, Nev., welded	2.39	3.65	–	0.19	11.3	1.17	p = 19.8	STOWE, 1969
	Canada, Noranda Mines, Ont., silicified	2.74	81.5*	37.7*	0.07*	–	–	–	BIRCH & BANCROFT, 1939

Table 3 - Laboratory Mechanical Properties of Volcanic Rocks
(Lama & Vutukuri, 1978) (Sheet 2 of 2)

Rock	Location & Description	Density, ρ (g/cm^3)	Modulus of Elasticity, E (GPa)	Modulus of Rigidity, G (GPa)	Poisson's Ratio, υ	Compressive Strength, σ_c (MPa)	Tensile Strength, σ_t (MPa)	Remarks	Reference
	USA, AEC Nevada test site, red to red yellow	1.92	3.45	–	0.24	9.65	–	w = 19.3	Cording, 1967
	—, yellow	2.0	15.6	–	0.09	35.3	–	w = 17.5	Cording, 1967
	—, red & yellow	1.60	6.34	–	0.15	22.3	–	w = 4.6	Cording, 1967
	USA, NTS-E Tunnel, porous, cemented	1.61	5.03	–	0.21	24.1	1.45	–	Miller, 1965
	Canada, Lakeshore	–	76.53	31.03	0.23	262.92	105.49	–	Morrison, 1970
	Canada, Helen Mine, vertical	–	82.74	–	0.27	155.13	17.31	–	Morrison, 1970
Rhyolite	Japan, Seikan Tunnel	2.42	–	–	–	85.4	5.6		Mochida, 1974
	Japan, Taguchi	–	26.0	–	–	–	–	–	Iida et al, 1960
Dacite	S. Africa, Pangolapoost Dam	2.67	–	–	–	112.5	–	p = 6.06	Phelines, 1967
	USSR, geosynclines, porphyrites	2.62	41.0	–	0.22	–	–	p = 3.06	Belikov, 1967
Ignimbrite	New Zealand, Maraetai Dam	1.83–2.16	–	–	0.15–0.30	–	–	p = 6.4–15.6	James, 1955
	wet	–	1.7–6.7	–	–	7.74–33.8	0.86–2.65(R)	–	James, 1955
	dry	–	2.34–7.7	–	–	13.8–46.9	0.89–3.5(R)	–	James, 1955

Table 4 - Engineering Properties of Hong Kong Volcanic Rocks

Rock Type	Description[1]	Uniaxial Compressive Strength, UCS (MPa)	Modulus of Elasticity, E (GPa)	Density, d (Mg/m³)	Locality	Reference
Lapilli Tuff	MDV	24	4.5	2.54	Tsing Yi	
"	SDV	67	15.8	2.69	"	
"	F	70	14.2	2.72	"	
"	SDV	71	18.0	2.72	"	
"	MDV	57	11.3	2.70	"	
"	SDV	56	11.8	2.73	"	
"	F	79	14.8	2.76	"	
"	F	84	18.7	2.77	"	
"	F	45	12.8	2.73	"	
"	MDV	38[2]	9.1[2]	2.65	"	PWD Contract
"	F	81	23.8	2.71	Lantau	No. 512 of
"	SDV	94	20.6	2.69	"	1980
"	SDV	81	22.0	2.69	"	
Medium to						Lantau Fixed Crossing
Coarse Tuff	SDV	126	24.4	2.69	Ma Wan	Project,
"	SDV	70	27.7	2.67	"	Phase II
"	SDV	9[2]	7.0[2]	2.68	"	(Mott, Hay
"	SDV/MDV	55	6.0	2.62	"	& Anderson,
"	SDV	87	19.5	2.69	"	1981)
"	SDV	98	27.8	2.67	"	
"	F	140	19.3	2.70	"	
"	F	68	15.7	2.71	"	
"	MDV	39	16.2	2.64	"	
"	SDV	28[2]	11.6[2]	2.67	"	
"	F	111	18.4	2.67	"	
"	F	168	21.7	2.70	"	
"	F	144	25.0	2.70	"	
Pyroclastic	SDV/F	146	79.0	2.75	New Territories	Future
"	SDV	198	59.0	2.65	"	Increase of
"	SDV/F	109	20.4	2.71	"	Water Supply
"	SDV	155	31.8	2.71	"	from China -
"	MDV	25	25.5	2.68	"	Stage I -
Lava Tuff	SDV	66	18.0	2.72	"	Western
Pyroclastic	MDV/SDV	51	1.7	2.75	"	Aqueducts
"	MDV	39	9.5	2.63	"	(Charles
Lapilli Tuff	MDV	123	15.4	2.69	"	Haswell &
"	SDV	112	25.0	2.72	"	Partners,1983)
Coarse Tuff	MDV	166	40.0	2.74	"	
Tuff	SDV	142	69.0	2.69	"	

Notes : (1) Material description : F - fresh volcanic, SDV - slightly decomposed volcanic, MDV - moderately decomposed volcanic.
(2) Failure through joint (?)

Table 5 - Classification of Weathered Volcanic Rock Materials in Terms of Strength and Deformation Modulus

Mass Weathering Grade	Uniaxial Compressive Strength, UCS (MPa)	Tangent Young's Modulus, E_t (GPa)	Poisson's Ratio, ν
Fresh Volcanics(1)	75 to 200	25 to 75	0.09
Slightly Weathered Volcanics	50 to 150	15 to 50	to
Moderately Weathered Volcanics	12.5 to 75	5 to 25	0.28
Note : (1) These ranges cover a wide spectrum of volcanic rocks in Hong Kong.			

Table 6 - Rock Mass Properties of Volcanic Rocks

Mass Weathering Grade	Joint Frequency per m	RQD (%)	Rock Mass Factor, j
Fresh Volcanics	1 1 - 5 5 - 8	90 - 100 75 - 90 50 - 75	0.8 - 1.0 0.5 - 0.8 0.2 - 0.5
Slightly Weathered Volcanics	1 1 - 5 5 - 8	90 - 100 75 - 90 50 - 75	0.8 - 1.0 0.5 - 0.8 0.2 - 0.5
Moderately Weathered Volcanics	5 - 8 8 - 15 > 15	50 - 75 25 - 50 0 - 25	0.2 0.1 0.1

Table 7 - Summary of Engineering Properties of Volcanic Rocks Used in the Calculation of Allowable Bearing Stresses

Mass Weathering Grade	Uniaxial Compressive Strength, UCS		Tangent Young's Modulus, E_t		Rock Mass Factor, j	Rock Mass Modulus, E_m		Poisson's Ratio, ν
	Mean (MPa)	Lower bound (MPa)	Mean (GPa)	Lower bound (GPa)	Lower bound	Mean (GPa)	Lower bound (GPa)	
Fresh	125	75	40	25	0.8 0.5	32 20	20 12.5	0.25
Slightly Weathered	100	50	25	15	0.8 0.5 0.2	20 12.5 5	12 7.5 3	0.25
Moderately Weathered	35	12.5	12.5	5	0.2 0.1	2.5 1.25	1.0 0.5	0.25

Table 8 - Socket Adhesion and Uniaxial Compressive Strengths and Achieved End-Bearing Stresses of Various Rock Foundations (Thorne, 1980)

REF. NO.	LOCATION	ROCK TYPE	UNCONFINED COMPRESSION (Qu)	ROCK SOCKET MAX. ATTAINED STRESSES				LAB. SECANT	CALC. FIELD MODULUS	
				ADHESION		END BEARING		MODULUS		
			MPa	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[(4)] 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 9 - Presumptive Allowable Bearing Stresses (MPa) for Rock Specified by Various Building Codes and Authorities

Foundation / Reference		Massive crystalline rock in sound condition (granite, basalt, gneiss)	Foliated metamorphic rocks in sound condition (slate, schist)	Sedimentary rocks in sound condition	Badly fractured rocks, or broken rocks, or partially weathered rocks except argillaceous rocks	Heavily shattered or weathered rocks	Notes
NAVFAC, USA	1971	6 to 10	3 to 4	1.5 to 2.5	0.8 to 1.2		Increase by 10% for each 300 mm embedment
Canadian Geotechnical Society	1978	10	3	1 to 4	to be assessed by examination in situ		
EOCA	1968	10	4	2.5 to 4	1.0		
National Building Code, USA	1967	10	4	1.5			
Uniform Building Code of USA	1964	$0.2q_u$	$0.2q_u$	$0.2q_u$	$0.2q_u$		
Los Angeles	1965	1.0	0.4	0.3			Earthquake area
CP 2004 (BSI, 1972)	1972	10	3	2 to 4		to be assessed after inspection	may need alteration upwards or downwards
US Bureau of Reclamation	1965	10.7	3.8		1.1		
Dallas	1968	$0.2q_u$	$0.2q_u$	$0.2q_u$	$0.2q_u$		Increase by 1/3 if foundation relatively dry
New York City	1970	6	6	2 to 4	0.8		Increase by 10% for each 300 mm embedment
Hong Kong	1976	5	1 to 3			to be assessed after inspection	
Sowers	1979	> 10 (RQD = 90%)	1. 5 to (RQD = 50%)		0.5 - 1.2 (N > 50)		

Legend : q_u = uniaxial compressive strength of intact rock sample
N = SPT N-value

Table 10 - Allowable Bearing Stresses for Volcanic Rocks Determined by Building Codes, 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)
Fresh Volcanics	75	125	15	25
Slightly Weathered Volcanics	50	100	10	20
Moderately Weathered Volcanics	12.5	35	2.5	7

Table 11 - Allowable Bearing Stresses for Volcanic Rocks Based on the RQD[(1)] Method

Rock Type	RQD (%)	Allowable Bearing Stress, q_a (MPa)
Fresh Volcanics	75 - 100 75 - 90 50 - 75	20 12 6.5
Slightly Weathered Volcanics	90 - 100 75 - 90 50 - 75	20 12 6.5
Moderately Weathered Volcanics	50 - 75 25 - 50 0 - 25	6.5 3.0 1.0

Notes : (1) RQD for use from this table should be the average within a depth below foundation level equal to the width of foundation, provided the RQD is fairly uniform within that depth.

(2) These values should not be increased for embedment into rock.

(3) If the design is based on these values, the settlement of foundation is not expected to exceed 12.5 mm, even for large loaded areas.

Table 12 - Allowable Bearing Stresses Determined by the Canadian Foundation Engineering Method Using Calculated K_{sp} Values for Pile Diameters of over 300 mm

Mass Weathering Grade	Uniaxial Compressive Strength		K_{sp}	Allowable Bearing Stress	
	UCS-min (MPa)	UCS-av (MPa)		q_a-min (MPa)	q_a-av (MPa)
Fresh Volcanics	75	125	0.23	17.3	28.8
Slightly Weathered Volcanics	50	100	0.23	11.5	23
Moderately Weathered Volcanics(3)	12.5	35	0.10	1.3	3.5

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

(2) Joint spacing > 300 mm, and

(3) Joints filled with up to 25 mm of decomposed or disintegrated material.

Table 13 - Settlement of Volcanic Rock Foundations under Various Foundation Loads (End-Bearing)

Fresh Volcanic ν = 0.25 I_s = 0.85 Ei_{min} = 25 GPa Ei_{av} = 40 GPa

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 Coefficient	j	0.8				0.8				0.8				0.8			
Settlement (mm)	max.	0.16	0.23	0.31	0.47	0.31	0.47	0.63	0.94	0.63	0.94	1.25	1.88	0.94	1.41	1.88	2.82
	av.	0.10	0.15	0.20	0.29	0.20	0.29	0.39	0.59	0.39	0.59	0.78	1.17	0.59	0.88	1.17	1.76
Reduction Coefficient	j	0.5				0.5				0.5				0.5			
Settlement (mm)	max.	0.25	0.38	0.50	0.75	0.50	0.75	1.00	1.50	1.00	1.50	2.00	3.00	1.50	2.25	3.00	4.51
	av.	0.16	0.23	0.31	0.47	0.31	0.47	0.63	0.94	0.63	0.94	1.25	1.88	0.94	1.41	1.88	2.82
Reduction Coefficient	j	0.2				0.2				0.2				0.2			
Settlement (mm)	max.	0.63	0.94	1.25	1.88	1.25	1.88	2.50	3.76	2.50	3.76	5.01	7.51	3.76	5.63	7.51	11.27
	av.	0.39	0.59	0.78	1.17	0.78	1.17	1.56	2.35	1.56	2.35	3.13	4.69	2.35	3.52	4.69	7.04

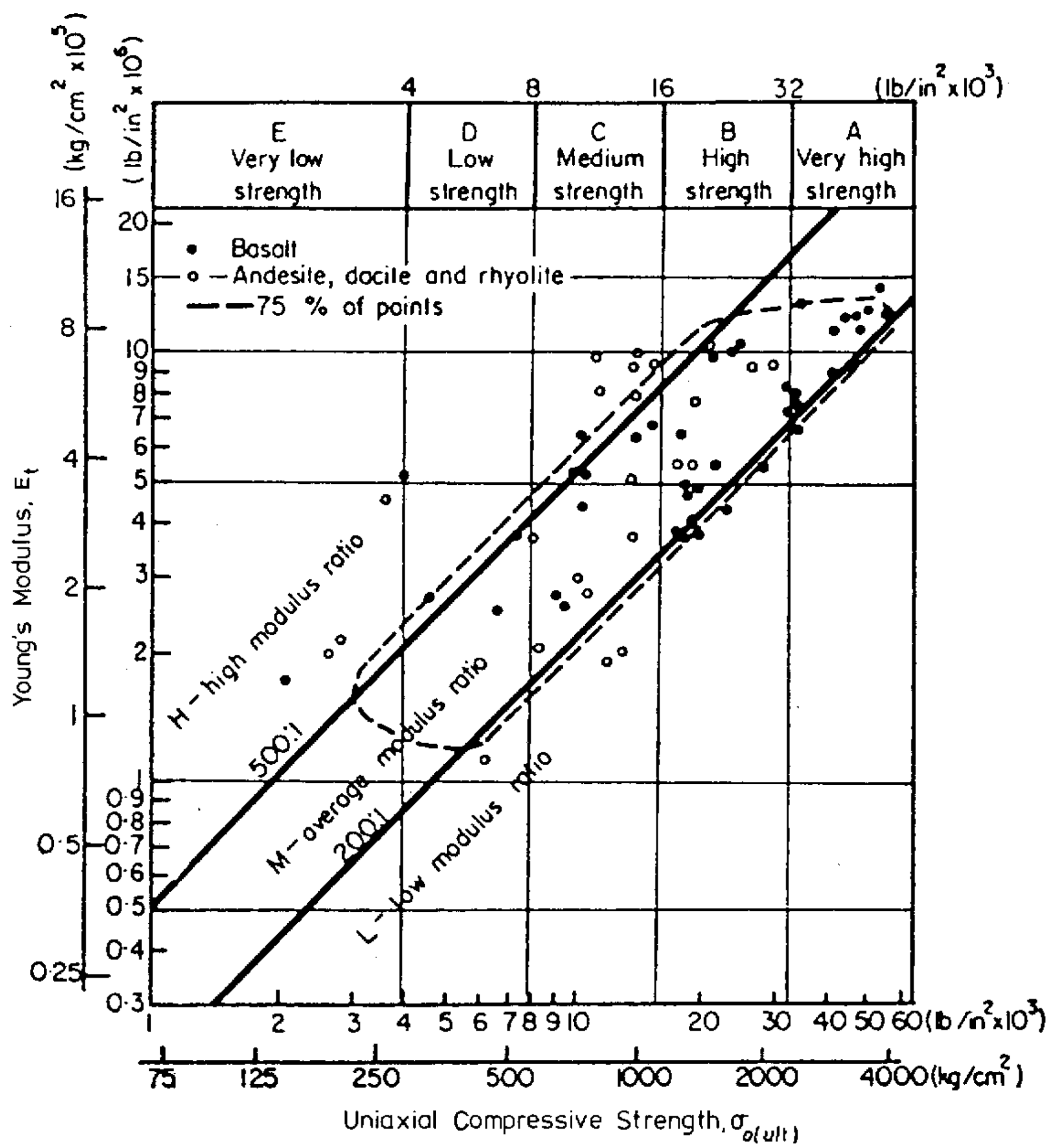
Slightly Weathered Volcanic ν = 0.25 I_s = 0.85 Ei_{min} = 15 GPa Ei_{av} = 25 GPa

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 Coefficient	j	0.8				0.8				0.8				0.8			
Settlement (mm)	max.	0.26	0.39	0.52	0.78	0.52	0.78	1.04	1.56	1.04	1.56	2.09	3.13	1.56	2.35	3.13	4.69
	av.	0.16	0.23	0.31	0.42	0.31	0.47	0.63	0.94	0.63	0.94	1.25	1.88	0.94	1.41	1.88	2.82
Reduction Coefficient	j	0.5				0.5				0.5				0.5			
Settlement (mm)	max.	0.42	0.63	0.83	1.25	0.83	1.25	1.67	2.50	1.67	2.50	3.34	5.01	2.50	3.76	5.01	7.51
	av.	0.25	0.38	0.50	0.75	0.50	0.75	1.00	1.50	1.00	1.50	2.00	3.00	1.50	2.25	3.00	4.51
Reduction Coefficient	j	0.2				0.2				0.2				0.2			
Settlement (mm)	max.	1.04	1.56	2.09	3.13	2.09	3.13	4.17	6.26	4.17	6.26	8.34	12.52	6.26	9.39	12.52	18.78
	av.	0.63	0.94	1.25	1.88	1.25	1.88	2.50	3.76	2.50	3.76	5.01	7.51	3.76	5.63	7.51	11.27

Moderately Weathered Volcanic ν = 0.25 I_s = 0.85 Ei_{min} = 5 GPa Ei_{av} = 12.5 GPa

Radius	(m)	0.5				1				2				3			
Bearing Stress	(MPa)	1	3	5	7.5	1	3	5	7.5	1	3	5	7.5	1	3	5	7.5
Reduction Coefficient	j	0.8				0.8				0.8				0.8			
Settlement (mm)	max.	0.16	0.47	0.78	1.17	0.31	0.94	1.56	2.35	0.36	1.88	3.13	4.69	0.94	2.82	4.69	7.04
	av.	0.06	0.19	0.31	0.47	0.13	0.38	0.63	0.94	0.25	0.75	1.25	1.88	0.38	1.13	1.88	2.82
Reduction Coefficient	j	0.5				0.5				0.5				0.5			
Settlement (mm)	max.	0.25	0.75	1.25	1.88	0.50	1.50	2.50	3.76	1.00	3.00	5.01	7.51	1.50	4.51	7.51	11.27
	av.	0.10	0.30	0.50	0.75	0.20	0.60	1.00	1.50	0.40	1.20	2.00	3.00	0.60	1.80	3.00	4.51
Reduction Coefficient	j	0.2				0.2				0.2				0.2			
Settlement (mm)	max.	0.63	1.88	3.13	4.69	1.25	3.76	6.26	9.39	2.50	7.51	12.52	18.78	3.76	11.27	18.78	28.16
	av.	0.25	0.75	1.25	1.88	0.50	1.50	2.50	3.76	1.00	3.00	5.01	7.51	1.50	4.51	7.51	11.27
Reduction Coefficient	j	0.1				0.1				0.1				0.1			
Settlement (mm)	max.	1.25	3.76	6.26	9.39	2.50	7.51	12.52	18.78	5.01	15.02	25.03	37.55	7.51	22.53	37.55	56.33
	av.	0.50	1.50	2.50	3.76	1.00	3.00	5.01	7.51	2.00	6.01	10.01	15.02	3.00	9.01	15.02	22.53

LIST OF FIGURES



Legend :

E_t = tangent modulus at 50% ultimate strength.

Classify rock as AM, BH, BL, etc.

Figure 1 - Engineering Classification for Intact Rock-
Basalt and Other Flow Rocks (Deere & Miller, 1966)

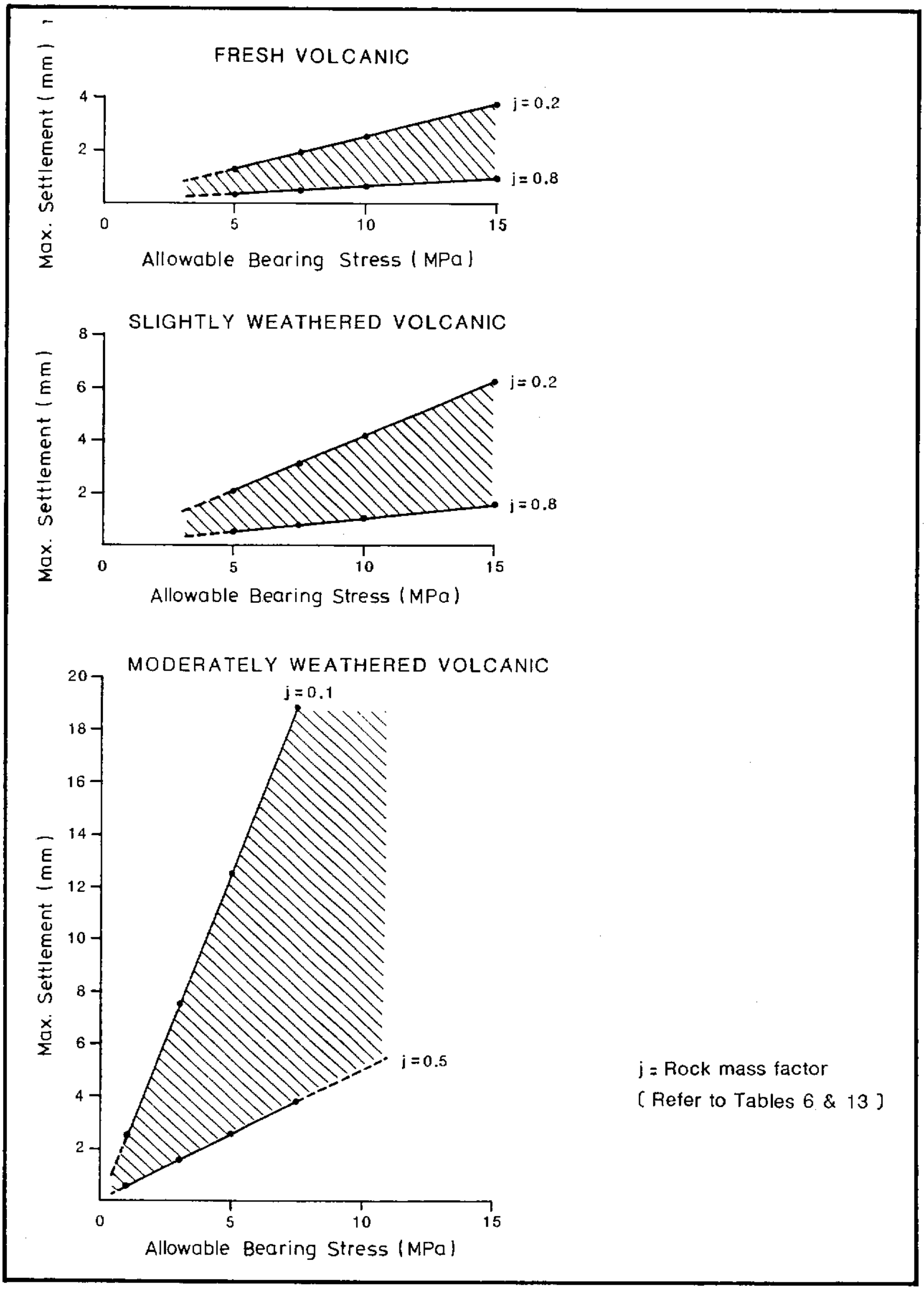


Figure 2 - Maximum Settlement Versus Allowable Bearing Stress
for a Caisson Diameter of 2 m