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ENGINEERING GEOLOGICAL INVESTIGATIONS FOR PILE FOUNDATIONS ON A DEEPLY WEATHERED GRANITIC ROCK IN HONG KONG

ÉTUDES GÉOLOGIQUES POUR FONDATIONS PAR PIEUX DANS UN GRANITE PROFONDÉMENT ALTÉRÉ À HONG KONG

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Summary

In weathered rocks the determination of founding depth for the foundations of major structures is complicated by the highly variable nature of the material. In order to ensure that the properties of the soil or rock mass on which the structure is to be founded correspond to those assumed in design it is necessary to unambiguously characterise the mass.

This paper outlines a scheme used to characterise the weathered rock mass and describes the engineering geological logging of the large diameter pile foundations on a deeply weathered granodiorite for a series of bridges in the New Territories, Hong Kong. The results of field and laboratory index and engineering tests carried out to determine the characteristics of the foundation rock are discussed. The drawbacks of use of RQD, percentage core recovery or probing tests as a means of determining and proving founding depth of piles are also discussed.

Résumé

La nature très variable des roches altérées rend compliquée la détermination de la profondeur de l'assise des fondations pour les structures importantes. Il devient nécessaire de définir clairement la couche de fondation, afin d'assurer que les caractéristiques des sols ou roches correspondent bien à celles préconisées au moment de l'étude.

Ce rapport donne un aperçu sur une méthode de caractérisation d'une masse rocheuse altérée et décrit l'enregistrement des paramètres géologiques lors de l'exécution de pieux de grand diamètre dans une granodiorite profondément altérée, pour une série de ponts dans les Nouveaux Territoires de Hong Kong. On examine les résultats d'essais de classification et d'essais mécaniques, exécutés sur place et en laboratoire, pour définir les caractéristiques des roches de fondation. On examine aussi les désavantages liés à l'utilisation du RQD, du pourcentage de récupération et des essais de perforation pour la reconnaissance et le contrôle de l'assise des pieux.

1. Introduction

In weathered rocks the determination of founding depth for major structures is complicated by the highly variable nature of the material. The variability results not only from the nature and intensity of discontinuities, but also from the degree and type of weathering, complicated by faulting, dyke intrusion and hydrothermal alteration.

In order to ensure that the properties of the soil or rock mass on which the structure is to be founded correspond to those assumed in design it is necessary to unambiguously characterise the mass. To do so requires a full appreciation and understanding of the role of the geological factors involved (Kulhawy and Goodman, 1980). Detailed geological mapping and description of the rock mass aided by suitable index testing on representative rock samples allows the information gained from a limited number of high quality, expensive or time consuming tests to be applied at other locations with similar rock mass characteristics.

A series of five bridges, founded on 148 large diameter piles, are being constructed as part of the New Territories Trunk Road, near North Tai Po, Hong Kong (Fig. 1). Advance piles were carefully logged and tested to obtain data and correlations to aid the engineer in arriving at a decision on a suitable founding depth.

This paper outlines the system, based on BS 5930, used to characterise the weathered rock mass and describes the engineering geological logging of the pile foundations on Tai Po granodiorite. The results of field and laboratory index and engineering tests carried out to determine the characteristics of the foundation rock are discussed.

2. Background

The rocks of Hong Kong, predominantly granites and volcanics, have been weathered by dominantly chemical processes under Hong Kong's humid subtropical climate (Ruxton and Berry, 1957) to great depths, typically

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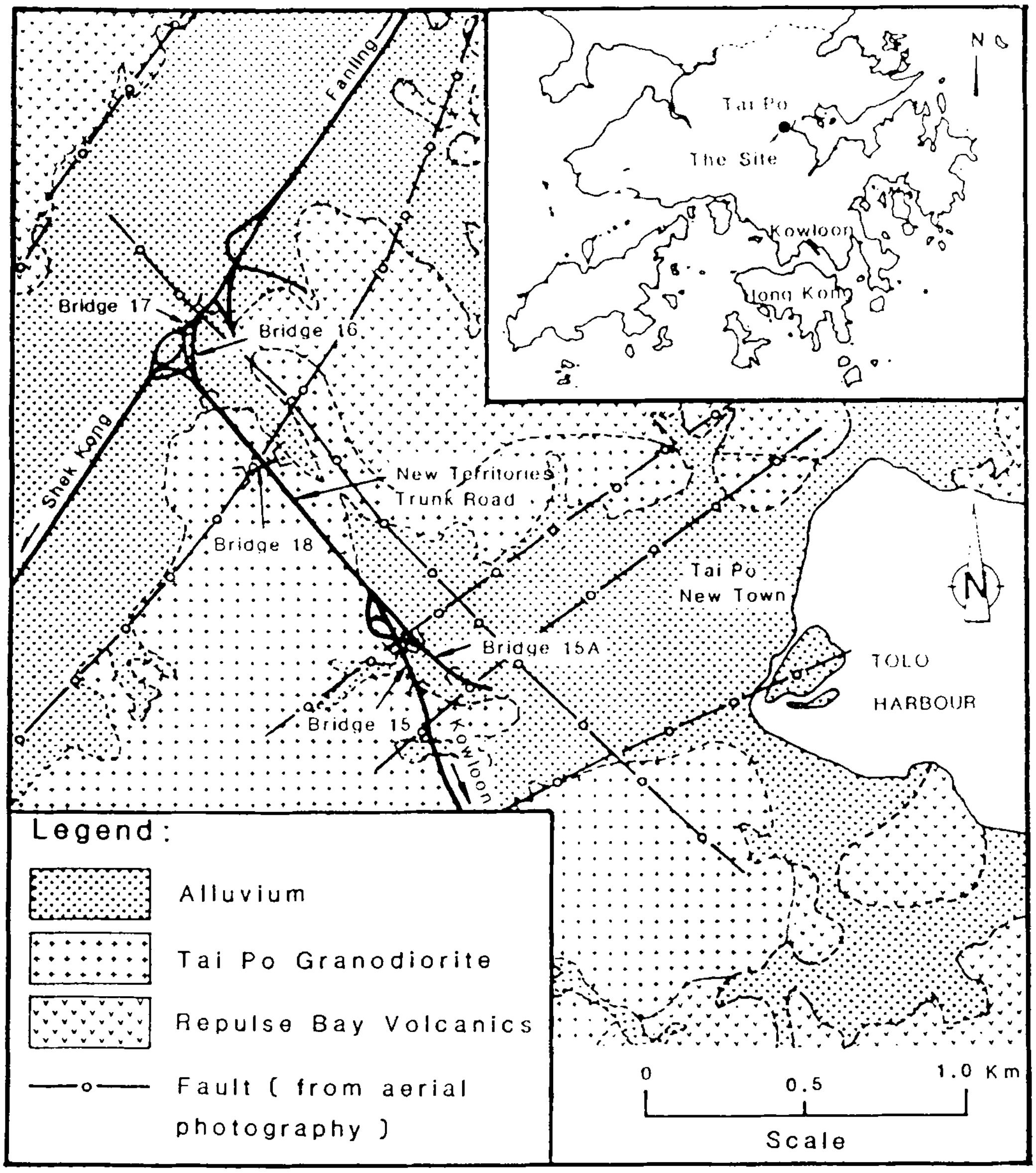


Fig. 1 : Site location and regional geology.

20 to 40 m whilst 90 to 100 m have been reported. The 'rockhead' is variable in most instances and where complicated by faulting, dyke intrusion or hydrothermal alteration the depth to rockhead may vary considerable within a distance of a few metres. Prediction of bedrock level during the preliminary site investigation stage based on a limited number of boreholes is difficult and requires the services of an experienced engineering geologist who is familiar with the site and the regional geological setting. In addition to irregular rockhead levels, corestones of up to 6-7 m diameter may be present within the weathered soil mantle. These may be mistaken for rockhead.

In Hong Kong, foundations for high load structures are traditionally taken to "massive crystalline rock in sound condition" using presumptive allowable bearing stress of 5 MPa in line with the Building (Construction) Regulations (Government of Hong Kong, 1976). RQD values of 75%, or core recovery of 85%, have commonly been specified to define founding depth.

In weathered rocks RQD does not provide a reliable measure of the engineering properties since it fails to take into account the extent and state of the weathered products, and in highly fractured rock may lead to unduly conservative founding depths being adopted. Core recovery is highly dependent on the drilling method and operator technique employed. With modern coring methods 100% core recovery can be obtained at any level within the weathered profile (Brand and Phillipson, 1984) and so the method is extremely limited as a guide to rock mass properties.

A commonly used procedure for formation of deep pile foundations in Hong Kong is to hand excavate the pile, placing concrete rings as the work progresses. The work is normally carried out by a husband and wife team using hand tools and pneumatic drill as necessary. This method, known locally as 'hand dug caissons', is particularly suitable for advance through highly variable weathered profiles which may include substantial quantities of core stones above the sound rock. The large exposure provided by these piles allows detailed mapping and testing to be carried out at or above the anticipated founding level.

3. Geological setting of the project

The New Territories Trunk Road passes through granitic rock terrain between North Tai Po and Lam Tsuen River (Fig. 1). The main rock type is Tai Po granodiorite which has intrused into volcanic and volcaniclastic rocks of Jurassic age (Repulse Bay Formation). Both Tai Po granodiorite and Repulse Bay volcanics were intruded by dolerite dykes at a later stage. Along the Lam Tsuen River valley these rocks are covered by variable thickness of recent and old alluvium.

The major structural features of the area are NE-SW and NW-SE trending faults. Dolerite dykes, up to 20 m thick, follow a NW-SE to NNW-SSE direction which is also the dominant direction of hydrothermal alteration and mineralization in the granodiorite in this region.

Tai Po granodiorite is extensively weathered and hydrothermally altered to depths of 30 m or more over the hills, whereas in the valleys fresh rock is typically reached at depths of 5 to 10 m. A complete weathering profile exists over the hills, where a thick weathered soil mantle overlies a relatively thin 2 to 6 m, transitional rock and soil zone of moderately weathered granodiorite and fresh rock. The weathering profile is locally complicated by faulting, hydrothermal alteration and later mineralization. A sketch showing a typical weathering profile underneath the centre line of one of the bridges is given in Figure 2.

4. Weathering grade classification

The engineering behaviour of rock masses is controlled by both the *mass* and the *material* properties of the rock. Rock mass characterisation for engineering purposes involves the description of the geological nature of the

Tab. 1 : Scale of weathering grades of rock mass in Tai Po granodiorite.

Term	Grade	Description
Fresh	I	No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.
Slightly weathered	II	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering.
	III_I	Less than 10 per cent of the rock material is decomposed and/or disintegrated to soil either as continuous or discontinuous seams.
Moderately weathered	III III_{II}	10 to 50 per cent of the rock material is decomposed and/or disintegrated to soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.
Highly weathered	IV	More than half of the rock material is decomposed and/or disintegrated to soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.
Completely weathered	V	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.
Residual soil	VI	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.

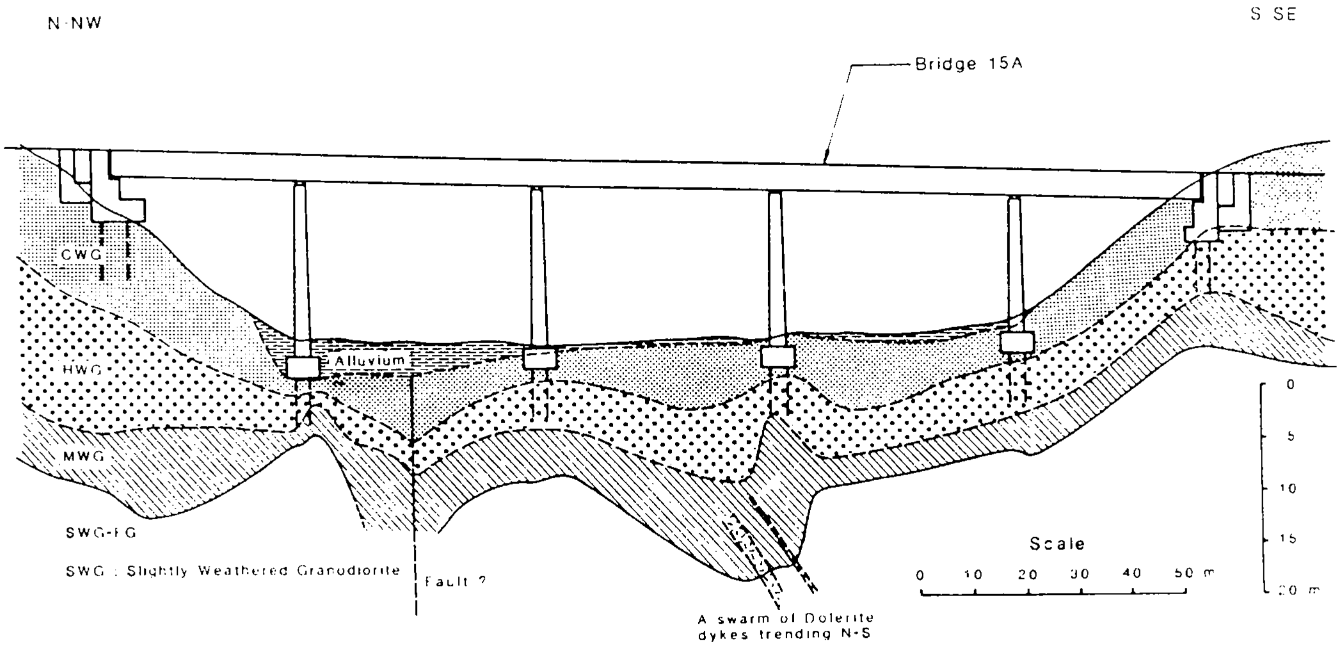


Fig. 2 : Weathering profile along centre line of bridge 15A.

Tab. 2 : Characterization of material state for weathered rock, Tai Po granodiorite.

Weathering state of rock material (after BS5930)		Rock / Soil	Description			Field recognition		Decomposition grade equivalent (GCO manual, 1981)
			Staining (discoloration)	Chemical state of mineral constituents (decomposition)	Physical state of rock fabric (disintegration)	Visual recognition	Approximate strength assessment	
Fresh	Fresh	rock	No visible sign of weathering.	Mineral constituents are fresh and sound.	Rock fabric is unfractured.	Fresh	Specimen requires many blows of geological hammer to fracture it. Very strong.	Fresh I
Discoloured	Partially discoloured	rock	Stained on the joint surfaces with yellowish brown discoloration penetrating into the rock material, the core may be fresher.	Plagioclases are partially gritty, biotite and hornblende are slightly decomposed.	Partial staining along microcracks.	Partial discoloration, fresher core.	Specimen requires many blows of geological hammer to fracture it. Very strong.	Slightly decomposed II
	Completely discoloured	rock	Completely stained yellowish brown throughout.	Plagioclases are partially decomposed throughout to gritty aggregates.	Microfractured throughout the fabric, grain boundaries tight.	Complete discoloration	Specimen can be fractured with more than one blow of hammer (upper range), cannot be scraped or peeled by penknife, pieces cannot be broken by hand (lower range). Weak to strong.	Moderately decomposed III
Decomposed	Highly decomposed	very weak rock / soil	Completely stained yellowish brown to yellowish orange brown.	Plagioclases are highly completely decomposed to gritty and soft aggregates.	Intensely microfractured, grain boundaries are open, may contain very closely spaced macrocracks.	Complete discoloration, partially friable. Ranges from very weak rock to weakly cemented cohesive soil.	Crumbles under firm blows with hammer, can be peeled with penknife, pieces can be broken by hand. Very dense soil, very weak rock.	Highly decomposed IV
	Completely decomposed	soil	The soil is completely stained yellowish brown to reddish brown, may be bleached.	Plagioclases are completely decomposed to soft aggregates, potash feldspars are partially decomposed and extremely fractured.	Grain boundaries are open. Fabric is intact.	Complete discoloration, some bleaching, friable granitic fabric is intact. Silty sand to clayey sandy silt.	Crumbles to individual grains (disintegrates) by finger pressure, easily peeled by penknife. Loose, very dense soil.	Completely decomposed V
	Residual soil	soil	The soil is completely stained deep reddish brown.	All feldspars are decomposed, quartz grains are reduced in size.	The material fabric is destroyed.	No granitic fabric. Sandy clayey silt to silty clay.		Soil VI

rock mass, including the petrographic properties and details of discontinuities and structures. This is then coupled with an assessment of the engineering properties of the rock mass, such as strength and deformation modulus.

Weathering grade classification is an extremely useful tool for description and characterisation of the rock in areas like Hong Kong. The assessment of weathering grade has become an important aspect of rock mass description for engineering purposes (Anon, 1970, 1972; Dearman, 1976). A scheme based on observation, supplemented by index tests, is considered the least liable to misinterpretation.

The scale of mass weathering grades used in this investigation follows the recommendation of BS 5390: 1981 (Table 1). The rock material classification has been based on Irfan and Dearman (1978) and Dearman (1984) (Table 2). The scheme does not take account of changes in rock strength or whether discontinuities are open or not, 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:

(a) the degree of discolouration
(b) the proportion of rock decomposed and/or disintegrated to soil, and
(c) the presence or absence of the original fabric.

The commonly used weathering grade system in Hong Kong (Geotechnical Control Office, 1984; Hencher and Martin, 1982) based originally on Moye (1955) has a rock material basis, in which the identification of any particular decomposition grade is based on examination of a hand specimen, usually in the form of a piece of drill core. Criteria for the grades relate almost exclusively to the strength and/or petrographic characteristics of the rock material. At a particular level in the weathering profile weathering tends to produce mixtures of rock materials weathered to different degrees (Fig. 3), and unless the proportions of these materials are clearly identified, the mass description resulting from examination of hand specimens can be in error.

The weathering grade system used is outlined in Tables 1 and 2. The moderately weathered rock mass grade has been separated into two subgrades, IIIi and IIIii, at about 10 per cent soil ratio. For foundation purposes grade IIIi rock, with less than 10 per cent soil, is a favourable foundation layer because of its high bearing capacity and low settlement properties.

5. Engineering geological logging and testing

Detailed logging of three pile foundations excavated in granodiorite was undertaken in terms of mass and material weathering grades and the properties of discontinuities. A series of index tests including N-Schmidt hammer value, point load testing and air percussion drilling were undertaken in these piles to establish site specific correlations. The results of the index tests were compared with the detailed logging of the piles and adjacent boreholes in terms of both material and mass properties, such as weathering and alteration state, discontinuities, RQD, fracture intensity and strength. The results are summarised in Table 3.

The field observations around the perimeter of the excavation and the results of the index tests have been shown on engineering geological logs together with type of excavation, rate of advance, groundwater conditions and other details. The log of one of the advance piles is given in Figure 3.

A series of laboratory tests were conducted to obtain quantitative data on the strength and deformation properties of the rock and to verify the assumed design parameters, which were based on tests carried out elsewhere on granitic rocks. The core samples representative of the range of weathered granodiorite material were selected for testing from the existing boreholes. The tests included uniaxial compressive strength, deformation properties (Young's modulus of elasticity, secant modulus and Poisson's ratio) and bulk density. Point load testing was carried out on the other halves of the cores tested in uniaxial compression.

6. The engineering properties of Tai Po granodiorite

6.1. Uniaxial Compressive Strength

Uniaxial compressive strength (UCS) of intact rock material is commonly used in empirical relationships to determine the allowable bearing pressures of the rock mass, modified by a factor, usually 0.2, to take account

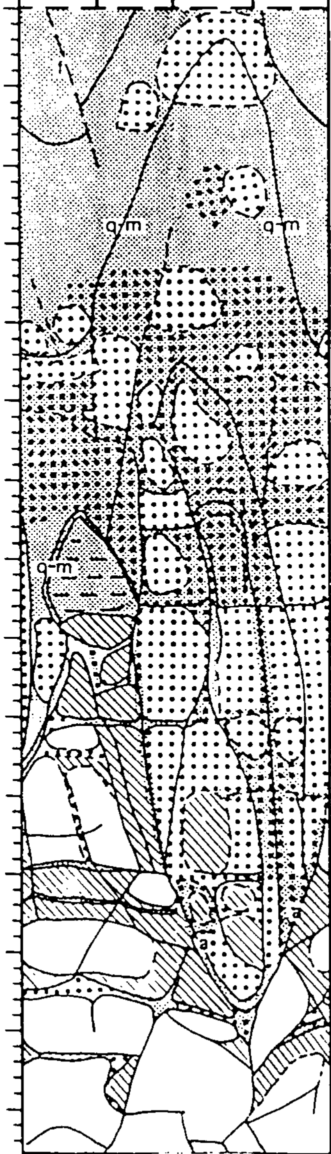
Tab. 3 : Material classification of weathered Tai Po granodiorite in terms of strength, deformation modulus, Schmidt hammer value and bulk density.

Material weathering grade	Dry bulk density T/m^3	Schmidt hammer value N	Point load strength PLS, MPa	Uniaxial compressive strength UCS, MPa	Tangent young's modulus E_t, GPa	Secant modulus (0-10 MPa) E_s, GPa	Poisson's ratio
Fresh	2.70-2.85	59-68	7-11	175-275	50-67	45-65	
Partially discoloured	2.65-2.76	45-68	5-9	125-225	30-60	30-55	0.13
Completely discoloured	2.60-2.75	25-50	0.25-6	5-150	5-35	2.5-35	
Highly decomposed	2.20-2.68	15-30	0.1-0.6	2.5-15	< 5.0	< 3.0	0.4
Completely decomposed	< 2.20*	< 18	< 0.1	< 2.5			

Mass Grade: Completely weathered granodiorite WV; Highly weathered granodiorite WIV; Moderately weathered granodiorite WIIIii; Moderately weathered granodiorite WIIIi; Slightly weathered granodiorite WII

Depth (m): 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30

Pile Log: 1, 2, 3, 4



Base of Caisson

Finish: CONCRETE RINGS; UNLINED; TOE-IN

W.L.

Finish	Fracture Index per m	N-Schmidt Hammer Value: Mean	St. Div.	Range
Concrete rings		19	±3.4	12-23
		16	±2.2	12-21
		15	±2.1	13-20
		16	±1.9	12-19
		21	±2.5	17-25
		20	±5.5	14-33
		19	±3.4	13-24
		19	±5.3	11-28
		13	±2.8	10-18
		21	±4.0	14-26
		17	±3.3	10-26
		20	±5.2	14-30
	4	34	±12.6	21-62
	4	40	±16.7	21-65
	3	45	±16.5	11-67
Unlined	3	55	±19.2	10-65
Toe-in	2	61	±5.1	48-68

Legend :

Material grades

- V Completely decomposed
- IV Highly decomposed
- III Completely discoloured
- II Partially discoloured
- I Fresh
- Joint
- q-m Mineralized quartz vein
- a Altered (kaolinized) clayey zone
- -f- Fault

Pile diameter : 1250 mm

Excavation method : Hand dug

Fig. 3 : Typical engineering geological log of pile.

of the nature of jointing in rock (d'Appolonia *et al.*, 1975).

In a more refined form the Canadian Foundation Engineering Manual (Canadian Geotechnical Society, 1978) also correlates uniaxial compressive strength of rock material to allowable bearing pressure by the formula:

$$q_a = UCS.K_{sp}.d$$

where K_{sp} is an empirical factor which depends upon spacing of discontinuities and includes a factor of safety of 3, and d is the depth factor.

Theoretical studies and in situ test results 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 and Davis, 1980). For foundations on rocks stronger than concrete, the ultimate bearing capacity of the rock imposes no practical limitation. The allowable bearing capacity is then governed by the bearing stress at which allowable settlements occur.

In this investigation 28 cores were selected from various materials in each weathered layer (grade). The boreholes were described and re-logged in standard engineering terms (BS 5930) for both material grade and mass grade. The cores were cut to a length/diameter ratio of approximately 2. The specimens were prepared and tested generally in accordance with the procedures given by ISRM (1978). However, the ends were disk ground rather than lapped, and steel end platens were not used during loading.

The results of uniaxial compressive strength testing are tabulated on Table 4. The strength results are consistently low by about 25% compared to other published work (Kulhawy, 1975; Dearman and Irfan, 1978a). This is considered due to the variations from the standard procedure mentioned above. Figure 4 groups the uniaxial compressive strength test results for each material weathering grade. Failures through pre-existing fractures and veins have not been included.

Fresh granodiorite is a very strong rock with UCS $\geqslant 150$ MPa. The strength is dramatically reduced when the material is completely discoloured by yellowish brown iron-oxide staining which indicates the migration of weathering agents along newly formed microcracks and opening grain boundaries. Advanced weathering decomposes the feldspars further and produces gritty to soft kaolinite aggregates. Fairly large pieces of rock or NX size cores can be broken by hand when the feldspars are highly decomposed and the rock material is friable.

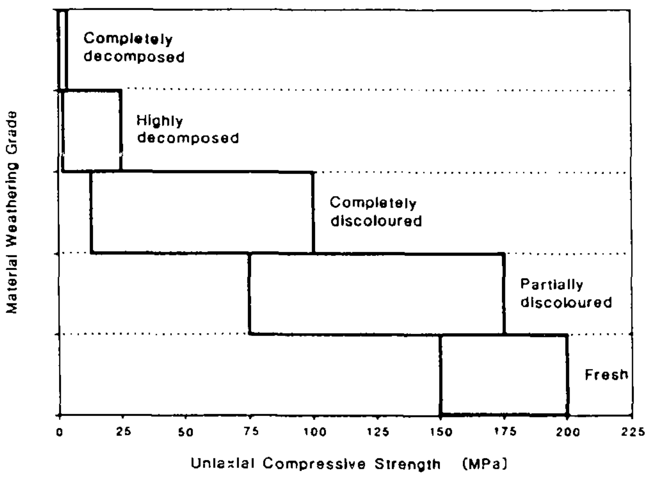


Fig. 4 : Ranges of uniaxial compressive strength in weathered granodiorite.

6.2. Stress-Strain Behaviour and Elastic Properties

Where deformation or settlement is the limiting criterion for a structure founded on a rock mass, the deformation modulus of the rock mass is the prime factor in the determination of this settlement. In situ deformation tests such as plate bearing, jack, or pressuremeter tests may be carried out to determine the deformation modulus of the rock mass. Modulus values determined from different tests are influenced by the test arrangement and by the volume of material tested. The volume effect, a manifestation of joints and cracks and the extent of weathering away from these discontinuities are major factors in reducing the modulus values. A number of in situ load tests must be carried out in order to reasonably assess the variability of elastic modulus of the rock mass and the effect of discontinuities on the mass deformation modulus in jointed rock masses weathered to different degrees. The tests are expensive and cumbersome to carry out but may yield excellent results.

In the absence of in situ test data, mass deformation properties may be determined from the laboratory tests on intact cores and an empirical/semi-empirical reduction factor used to take account of the jointing of the rock mass (Coon and Meritt, 1970; Bieniawski, 1975a; Kulhawy and Goodman, 1980).

The elastic properties used in settlement analysis are the tangent Young's modulus of elasticity or secant modulus of intact rock material, E_t or E_s, and Poisson's ratio v, usually determined on intact cores. In this investigation 14 compression tests were conducted to determine the elastic properties of granodiorite cores selected to represent a wide weathering spectrum. The tangent Young's modulus and Poisson's ratio were calculated from the first loading cycle at 50% of the ultimate stress. The secant modulus, E_s, was also calculated between 0 and 10 MPa stress range to define a deformation modulus for low stress levels relevant to the loads imposed by the structures on rock.

The results of stress-strain tests are given in Table 4. The ranges of secant modulus in the stress zone 0 to 10 MPa are shown in Figure 5 for each weathering grade based on laboratory test results.

The tangent modulus results and Poisson's ratio of granodiorite are comparable to other published data for granitic rocks (Kulhawy, 1975; Dearman and Irfan 1978a; Lama and Vutukuri, 1978). The tangent Young's modulus of fresh granodiorite is generally over 50 GPa. The value drops drastically below 30 GPa after the material is completely discoloured as a result of weathering and microfracturing. The modulus of deformation of the mass also drops rapidly with weathering.

Tab. 4 : Laboratory test results on granodiorite.

Borehole No.	Sample No.	Depth (m)	Material weathering grade	Mass weathering grade		Density T/m^3	Moisture content %	Compressive stress		Modulus of elasticity			Poisson's ratio	Failure mode
								at failure MPa	corrected MPa	tangent at 50°	secant at 0-5°	secant (0-10 MPa)		
S3	S3U1	18.80	Fresher core of moderately weathered granodiorite	1	II	2.70	0.20	93.2	93	—	—	—	—	
S3	S3U2	18.00	Completely discoloured granodiorite	3	II	2.67	0.32	64.6	64	25.3	29.9	30.3	0.129	Vertical fractures
S3	S3U3	16.35	Moderately discoloured granodiorite, epidote veined	2	II	2.83	0.15	84.3*	85*	31.8	34.0	29.4	0.136	One vertical fracture
S3	S3U4	14.90	Fresher core of moderately discoloured granodiorite	1	II	2.71	0.15	113.7	114	—	—	—	—	Cone and vertical
S3	S3U5	14.65	Completely discoloured granodiorite	3	II	2.70	0.15	79.4	80	—	—	—	—	Cone
S3	S3U6	9.15	Completely discoloured, highly decomposed granodiorite	4	III/IV	2.74	0.28	27.7	28	—	—	—	—	Cone
S3	S3U7	9.45	Completely discoloured granodiorite	3	III/IV	2.76	0.27	40.2	40	7.1	7.1	7.1	0.356	Cone
S3	S3U8	10.70	Highly decomposed granodiorite	4	III	2.68	0.55	20.0	20	2.9	3.1	3.1	0.267	Cone and fractures
S4	S4U1	26.65	Fresh granodiorite	1	I	2.78	0.03	156.4	156	55.1	47.9	38.5	0.291	Cone and fractures
S4	S4U2	26.45	Fresh granodiorite, epidote veined	1	I	2.77	0.05	112.1*	112*	59.0	51.9	45.5	0.352	Cone and fractures
S4	S4U3	25.25	Fresh granodiorite	1	I	2.78	0.04	164.1	164	67.2	58.2	47.6	0.336	Cone and fractures
S4	S4U4	23.90	Fresher core of slightly discoloured granodiorite	1	II	2.76	0.11	129.1	129	—	—	—	—	Cone
S4	S4U5	17.10	Fresh core of slightly discoloured granodiorite	1	II	2.85	0.08	181.8	183	62.3	59.8	47.6	0.270	Cone
S4	S4U6	5.00	Highly discoloured granodiorite	2	III	2.69	0.18	92.9*	93	40.5	47.5	55.5	0.202	Existing fractures
S4	S4U7	1.50	Completely discoloured granodiorite, epidote veined	3	III	2.69	0.38	87.2	87	—	—	—	—	Vertical fractures
S23	S23U1	34.40	Partially discoloured granodiorite	2	II	2.71	0.20	116.3	115	46.6	52.9	52.6	0.237	Cone and vertical
S23	S23U2	35.95	Highly discoloured granodiorite	2	II	2.71	0.18	74.2*	73*	—	—	—	—	Cone and vertical
S23	S23U3	33.90	Completely discoloured granodiorite	3	III	2.68	0.25	78.1	78	—	—	—	—	Cone and vertical
S23	S23U4	33.00	Completely discoloured granodiorite	3	III	2.60	0.31	51.1	51	—	—	—	—	Vertical fractures
S23	S23U5	26.70	Highly decomposed granodiorite	4	III/IV	2.52	0.29	14.2	14	—	—	—	—	Cone and disintegration
S23	S23U6	21.50	Completely discoloured, highly decomposed	3	IV	2.62	0.30	35.8	35	—	—	—	—	Cone and disintegration
S27	S27U1	32.30	Fresher core of slightly discoloured granodiorite	1	II	2.72	0.04	132.2	132	52.9	52.1	47.6	0.238	Cone and vertical
S27	S27U2	31.35	Fresher core of slightly discoloured granodiorite	1	II	2.66	0.14	80.9*	81*	—	—	—	—	Thro' vein
S27	S27U3	30.10	Moderately discoloured granodiorite	2	II	2.70	0.14	42.8*	43*	42.5	45.5	45.5	0.196	Thro' vein
S27	S27U4	28.70	Completely discoloured granodiorite	3	II	2.66	0.20	73.8	74	35.9	35.8	33.3	0.138	Cone
S27	S27U5	25.70	Highly decomposed granodiorite	4	III	2.45	0.38	18.5	17	—	—	—	—	Disintegration
S34	S34U1	12.50	Fresh granodiorite	1	II	2.75	0.06	108.5*	108*	—	—	—	—	Thro' vein
S34	S34U2	11.45	Fresh granodiorite	1	II	2.73	0.11	100.4*	101*	46.5	48.7	52.6	0.161	Thro' vein
S34	S34U3	11.95	Fresh granodiorite	1	II	2.73	0.11	166.3	167	60.7	65.5	55.6	0.231	

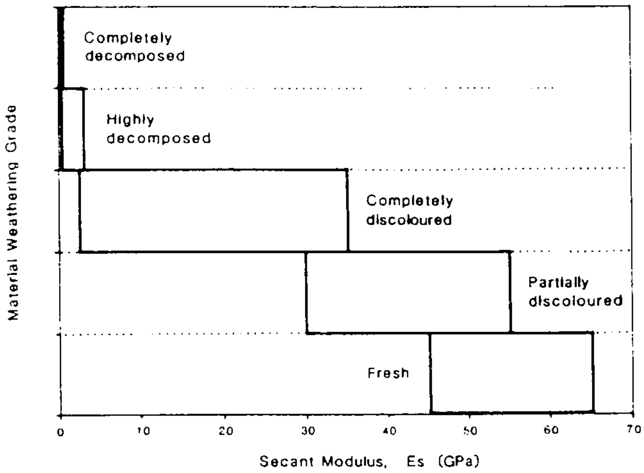


Fig. 5 : Ranges of secant modulus of deformation in weathered granodiorite.

6.3. Point Load Strength

Point load strength (PLS) testing on cores or irregular lumps provides a rapid index for the classification of weathered rocks provided samples are standardised in terms of size and shape (Broch and Franklin, 1972; ISRM, 1973). Data points where failure occurs through pre-existing discontinuities should be discarded. Point load strength has been widely used to estimate uniaxial compressive strength (D'Andrea *et al.*, 1965; Bieniawski, 1975b).

The point load strength was determined for various weathered rock materials obtained both from drill cores and the pile excavations. The results have been standardised to 50 mm diameter sample size. Figure 6 groups the results of point load strength testing of each weathering grade.

In the fresh and partially discoloured state the granodiorite is a very strong rock with PLS $\geqslant 5$ MPa. The PLS reduces rapidly when the material is completely discoloured. For highly decomposed material the PLS is generally less than 0.6 MPa when tested at natural water content. The test is insufficiently sensitive to accurately determine the strengths of decomposed material.

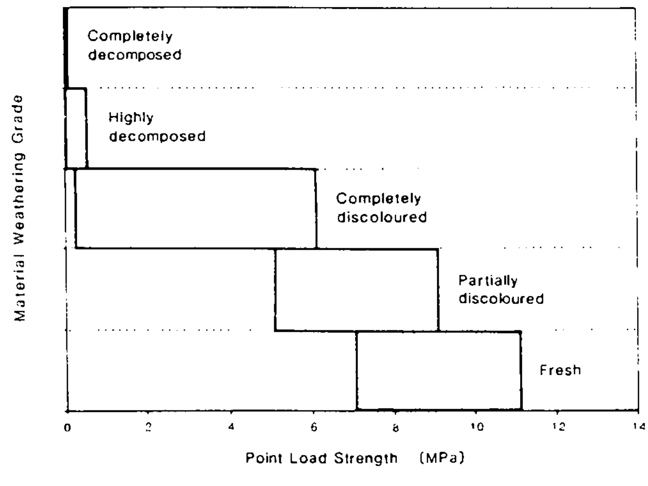


Fig. 6 : Ranges of point load strength in weathered granodiorite.

An empirical relationship between PLS and UCS, with a correlation factor of 24 for UK granites, has been shown by Dearman, Baynes and Irfan (1978). For this testing, correlation factors of between 12.5 and 25, with a mean of 17 were obtained. This is most likely to be due to the consistently lower strength values obtained using the incorrect compression testing procedure. The same discrepancy can be seen in the values published by Lumb (1983) tested in the same laboratory.

6.4. Schmidt Hammer Testing

The Schmidt hammer is a hand operated instrument used for the nondestructive determination of the unconfined compressive strength of rock or concrete. It is quick, simple, easy to operate and can be used to distinguish between different material weathering grades (Dearman and Irfan, 1978b; Hencher and Martin, 1982).

In this investigation the N-Schmidt hammer was used to classify the weathered rock material as well as to characterize the rock mass. Readings were taken at predetermined locations on the pile wall (Fig. 7). At each point, 10 readings were taken and the mean of the upper 50 percent of the readings was recorded.

For weathered material the ranges of N-Schmidt hammer values (SHV) are given for each weathering grade in Figure 8. In fresh rock the SHVs are above 60, whereas in discoloured rock a wide range of values was obtained depending on the state of decomposition of mineral constituents and microfracturing of rock texture. In materials weathered to very weak rock to soil range, SHV of 0 to 30 were obtained on freshly excavated surfaces.

In characterising the weathered mass, the results of the N-Schmidt hammer testing averaged over a 1 m pile ring are shown in Figure 9, for various mass weathering grades. This method of averaging results over a heterogeneous layer gave a very good indication of the rock mass grade.

The results are influenced by the decomposition state of the mineral constituents, wetness of the rock surface and tightness of discontinuities close to the point of testing.

6.5. Percussion Air Drilling

It is a common practice in Hong Kong to prove hand-dug piles by percussion drilling using a percussion air hammer to probe up to 4.5 m below the proposed founding depth. The suitability of the bedrock is determined by measuring the rate of penetration of a drill at 300 mm intervals. The test specification, the methods used during the test, the level of supervision and the criteria used to judge the results are not standardised, and may be further affected by variations in the drilling method and equipment. Variables to be considered are air pressure, hammer weight, length of rods, diameter and bit type of drills, number of changes of bit, the operator's methods, the level of site supervision and the rock conditions such as the type of rock, strength, weathering and alteration state, and jointing.

For this project, a 25 kg air drill fitted with a 38 mm freshly sharpened bit was set up vertically in the base of

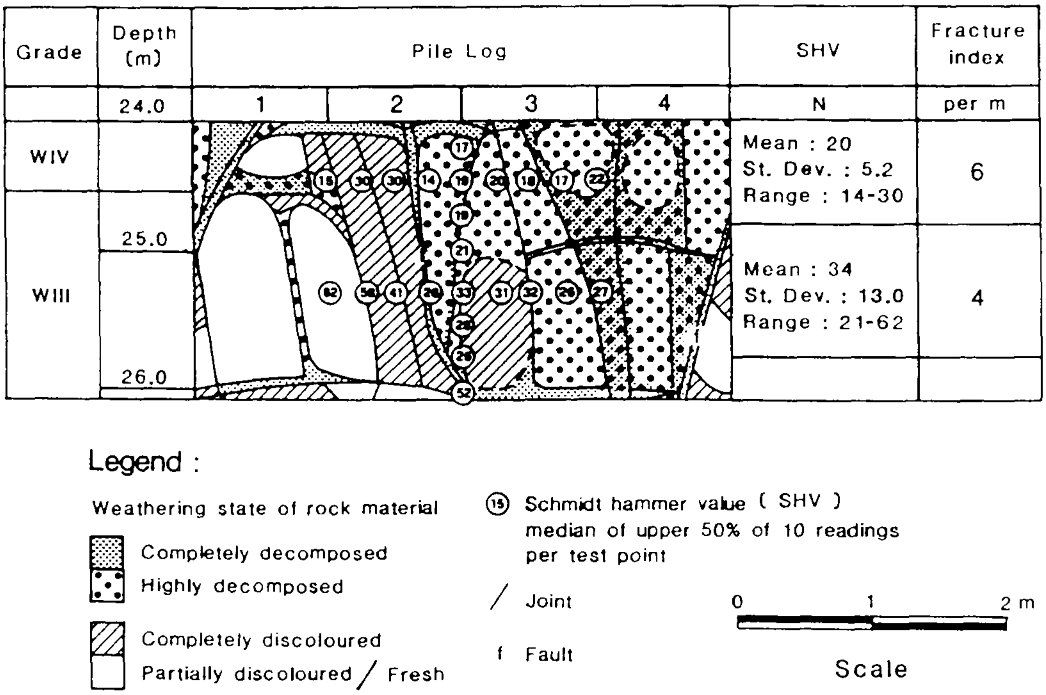


Fig. 7 : Schmidt hammer array for logging.

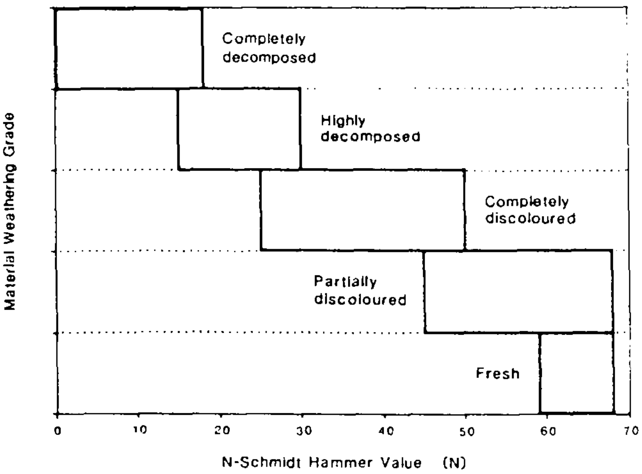


Fig. 8 : Ranges of Schmidt hammer values in weathered granodiorite (material).

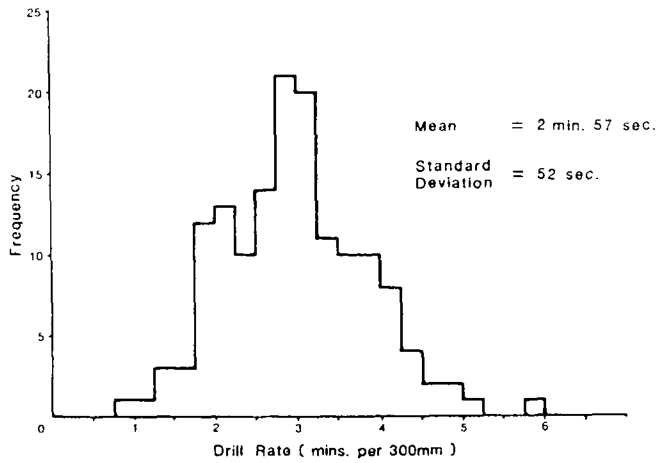


Fig. 9 : Ranges of Schmidt hammer values in weathered granodiorite (mass).

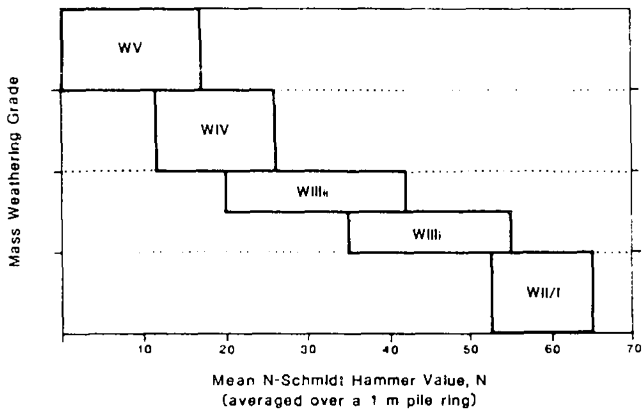


Fig. 10 : Variability of air drill rates in slightly weathered granodiorite.

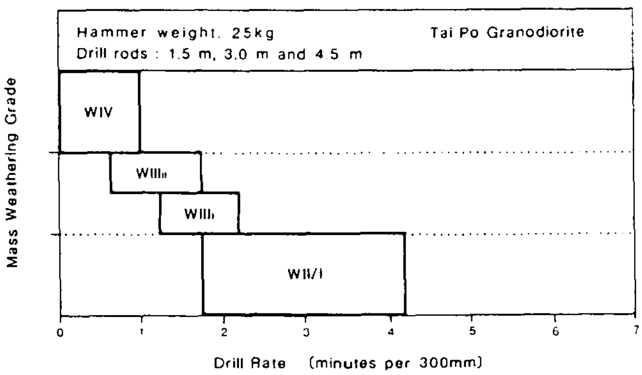


Fig. 11 : Ranges of air drill rates in weathered granodiorite.

the pile and allowed to operate under its own weight. The drill bit was changed at approximately 1.5 and 3.0 m penetration. The compressor pressure varied between 100 and 150 psi. The time taken to drill each 300 mm interval was recorded. The drill rate test results do not correlate with RQD or other indices related to the jointed nature of the rock, and cannot therefore be used as a criterion where founding depth is specified in terms of RQD. Thin zones of weathered or fractured material cannot be detected by this method as any variation in drill rate is masked by the general scatter in the results. Figure 10 gives typical variability noted in slightly weathered to fresh granodiorite. However, thicker zones, generally greater than 200 mm, of fractured rock or soil can be noted by significantly faster drill rates. Slightly faster drill rates were noted at each change of drill bit.

Despite the large number of variables affecting the rate of advance, it was found that as a means of distinguishing the overall rock mass quality the air drill rate was useful. Figure 11 shows the variation in average rate through differing weathering grades. In the highly weathered zone where the mass is composed of over 50 per cent soil, the air drill rate was generally less than 1 minute for 300 mm and in moderately weathered rock mass the rate was generally less than 2 minutes per 300 mm.

7. Rock mass properties

7.1. RQD and Fracture Intensity

The fresh granodiorite has moderately to widely spaced joint sets giving an equidimensional to rhomboidal rock mass blockiness. On weathering these are modified by rounding of the sharp edges and the eventual formation of spheroidal blocks. Further discontinuities including subhorizontal discontinuities are formed as a result of weathering, erosion and the associated stress release. Discontinuity surveys in the piles showed three dominant subvertical to vertical joint sets.

The jointing state of the rock mass can be assessed by RQD values determined on cores. The rock quality designation (RQD) is a quantitative index of rock mass quality based on core recovery by diamond rotary drilling (Deere, 1964). RQD does not take into account the discontinuity opening, continuity or condition, or the presence and character of any infilling material. Another major drawback of using RQD in the determination and verification of founding level is that rock with a fracture spacing greater than 100 mm is regarded as having excellent quality.

The normal practice in determining RQD (Deere, 1964) for use in foundation design is to adopt a standard core run of at least 1.5 m and core diameter of at least NX size. For the present investigation, RQD's were standardised to 1.5 m core runs and plotted against the fracture intensity for various weathering grades of Tai Po granodiorite (Fig. 12). There is a considerable scatter of results, and the values for slightly weathered granodiorite range from 60 to 100 per cent. The original logging during site investigation was not done on consistent lengths of core and the scatter was even greater (Fig. 13). It is apparent that it would be uneconomical to determine and verify the founding level of piles on rock based on minimum RQD specification of say 75 per cent. In some boreholes an RQD of 75 per cent would not have been reached even 10 m into slightly weathered rock.

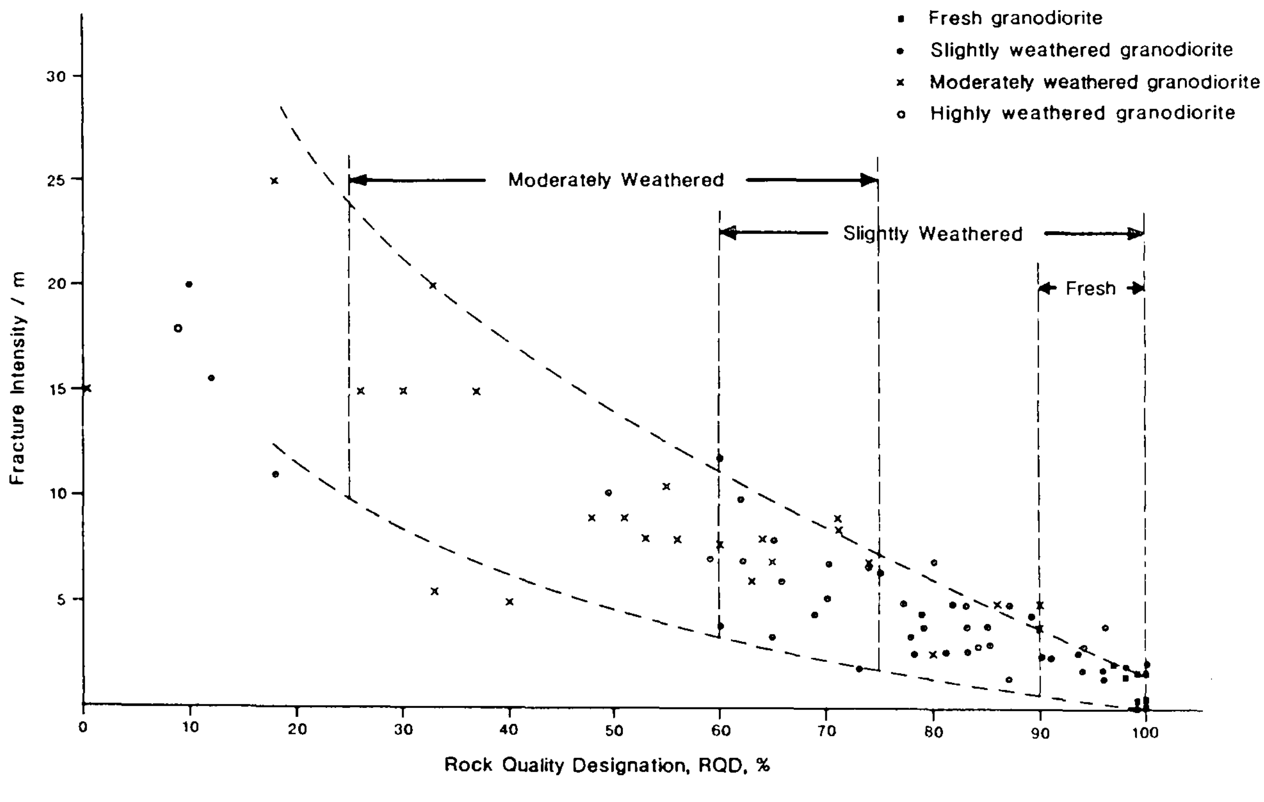


Fig. 12 : Fracture intensity versus RQD in Tai Po granodiorite (standardised).

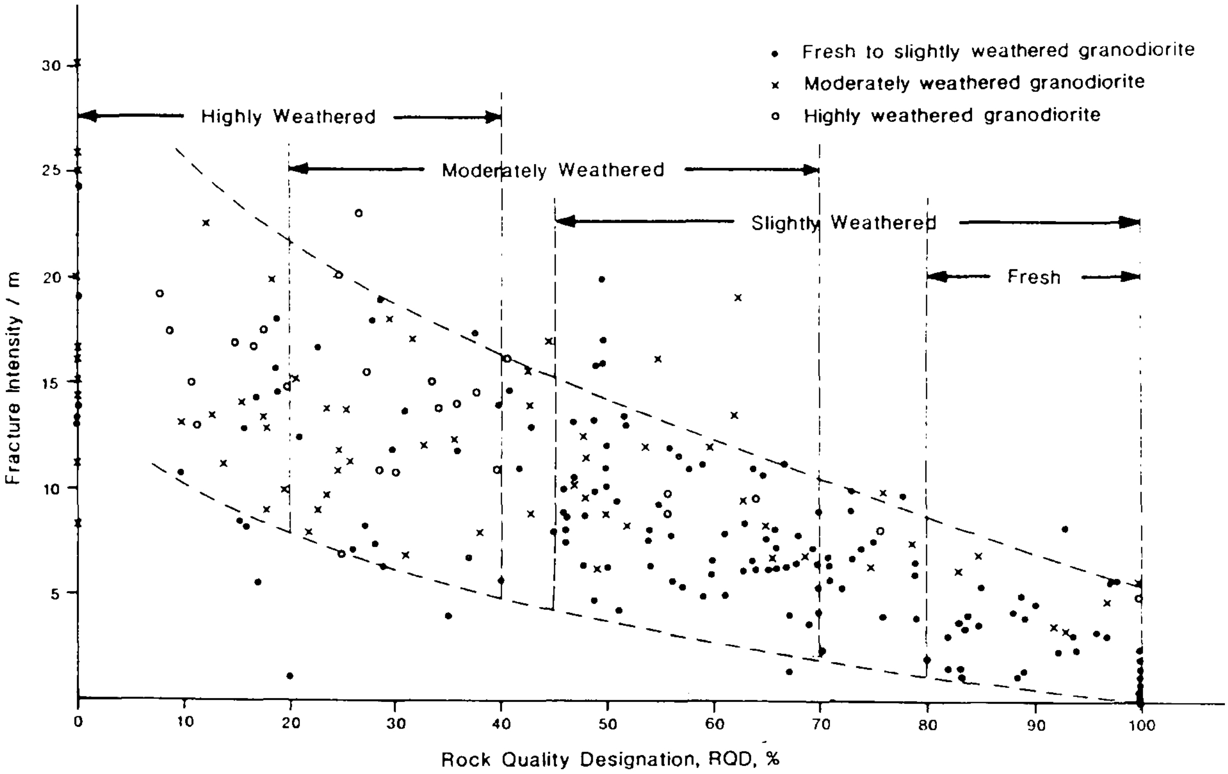


Fig. 13 : Fracture intensity versus RQD in Tai Po granodiorite (original data).

RQD has been used to determine the allowable bearing stress of foundations on rock (Peck *et al.*, 1974). RQD values and fracture frequency have been closely related to reduction factor or rock mass factor (Deere *et al.*, 1966). Woodward, Gardner and Greer (1971) in their textbook "Drilled Pier Foundations" suggest the use of reduction factor, E = Emass/Eintact vs RQD as an approximate way to obtain the rock mass modulus for settlement calculations.

Because of the inherent errors in using RQD for weathered rock, fracture intensity (number of fractures per metre) has been used to calculate the reduction factor for rock mass (Deere *et al.*, 1966; Hobbs, 1975).

A generalised classification of weathered granodiorite at Tai Po in terms of RQD and fracture intensity based on the borehole record is given in Table 5.

Tab. 5 : RQD classification of weathered granodiorite.

Mass weathering grade	Fracture intensity per m	RQD %	Rock mass Factor E_d/E_t (1)
Fresh granodiorite	1-3	90-100	0.8-1.0
Slightly weathered granodiorite	2-9	60-100	0.3-1.0
Moderately weathered granodiorite	6-15	25-75	0.1-0.5
Highly weathered granodiorite	over 12	0-50	0.1

(1) E_d is the deformation modulus computed from field seismic velocity. E_t is the tangent modulus of intact rock at 50% compressive strength (after Woodward, Gardner and Greer, 1971).

Tab. 6 : Estimated mass deformation properties.

Mass weathering grade		Material weathering grade	Tangent Young's modulus E_t, GPa	Rock mass factor	Mass deformation modulus E_m, GPa
I	Fresh granodiorite	Fresh	50 (60)	0.8 (0.8)	40 (48)
II	Slightly weathered granodiorite	Fresh partially discoloured	30 (30)	0.5 (0.5)	15 (15)
III_i	Moderately weathered granodiorite	Fresh discoloured (90%) decomposed (10%)	20 (15)	0.2 (0.2)	4 (3)

Values given in ( ) are the original design parameters based on test results elsewhere.

7.2 Rock Mass Modulus

A comparison of the rock mass factor and mass modulus of deformation values assumed in the original design of the piles for this project and those determined by the field and laboratory investigation during construction is given on Table 6. The rock mass deformation values based on laboratory determined intact elastic modulus and RQD are similar. Settlements of the piles were predicted using these mass modulii. The results of two pile load tests to be carried out on the site will allow a comparison of predicted and observed settlement to be made.

8. Conclusions

The system recommended for determining and proving founding level of piles on rock based on a descriptive weathering grade classification (after BS 5930) and quantitative index testing worked successfully during the construction of hand dug piles for the New Territories Trunk Road Bridge foundations at North Tai Po.

Index testing was useful to help characterize both the rock material and mass. A uniformity of description by unskilled personnel under the supervision of an experienced engineering geologist was obtained.

The system recommended can be applied to other project sites on granitic and volcanic rock terrain. Site specific correlations between index tests and rock grades may be obtained for each rock type.

RQD, percentage core recovery or the results of probing tests should not be used as the sole means of determining and proving founding depth of bored and hand dug piles in weathered rock as it may lead to erroneous but generally conservative founding depths being adopted.

Field and laboratory tests carried out on rock samples indicated that the range of values obtained were comparable to the original design parameters assumed to Tai Po granodiorite and other test results carried out elsewhere on granitic rocks.

9. Acknowledgements

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