

DYNAMIC TESTS IN REMOULDED LATERITIC SOILS

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SYNOPSIS

Dynamic penetration tests were carried out in three specimens of remoulded lateritic soil using a modified MacKintosh probe to determine whether any correlation exists between dynamic tests and those conducted with a constant rate of penetration apparatus. The results indicate that the load bearing capacity of the remoulded soils can be defined in terms of their dynamic resistance to penetration. The curves relating (i) dynamic resistance to depth of penetration and (ii) dynamic resistance to moisture content are similar to the corresponding curves obtained from the constant rate of penetration tests.

The effect of the velocity of penetration on the dynamic resistance is also discussed and a method of plotting the experimental results is presented which will detect any confining effect of the soil container or any inconsistency in the soil.

INTRODUCTION

Though dynamic penetration tests were originally confined to cohesionless soils, chiefly because of the difficulty in obtaining satisfactory undisturbed samples, they are now being increasingly employed to supplement the information on the soil between boreholes in $c - \phi$ soils. The reasons for this increasing use are

1. the comparatively high cost and difficulty in obtaining samples for triaxial and other laboratory tests, especially if they are from a great depth below the surface,
2. that the very light dynamic sounding equipment now available permits the tests to be carried out at very little cost and with comparative ease and speed.

The MacKintosh prospecting tool has been widely used in Malaysia as a dynamic sounding probe to supplement and fill-in the gaps between boreholes. A sounding with this probe is normally made near each borehole and the resistance, which is measured in terms of N_m , the number of blows per unit depth of penetration, is related to the soil properties of samples obtained from the boreholes. A grid of such soundings is normally made to provide some information about the soil profile over a large area.

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While N_m is a measure of the resistance of the soil, there is the need to relate it to bearing capacity (in terms of, say, ton/ft²). This paper presents a study of the relationship between N_m and bearing capacity as determined by a constant rate of penetration test in regard to remoulded lateritic clays from three building sites in Petaling Jaya. Many of the low hills in this area have been truncated and the spoil removed to fill the intervening valleys for domestic housing. Because the proposed buildings are to be light, reinforced concrete, framed structures in which the loads are transmitted by means of shallow footings to the recompacted soil, the effect of overburden pressure was not included in this study.

THE MACKINTOSH PROBE

The MacKintosh prospecting tool consists of a series of 1/2 in. diameter spring steel rods, each of length 4 ft, which can be connected together by means of screw couplings. The standard point is 1 in. in diameter and is 4-1/8 in. long. To test the validity of experimental results it is necessary to perform the tests using geometrically similar pointers of different sizes. A pointer in the form of a simple 60° cone was used instead of the standard MacKintosh pointer because the former was simpler to reproduce in different sizes. In the experiments described in this paper cones of base diameters 0.75, 1.0, 1.25, 1.75 and 2.0 in. were used.

A hammer of weight of 9.5 lb falling through a fixed height provided the driving energy. The greater part of the length of the hammer was tubular and was hexagonal in cross-section. A hexagonal nut inside the hammer, when screwed on to the topmost rod, enabled the hammer to slide along the rod through a fixed height of 18 in. The modified apparatus is shown in Fig. 1.

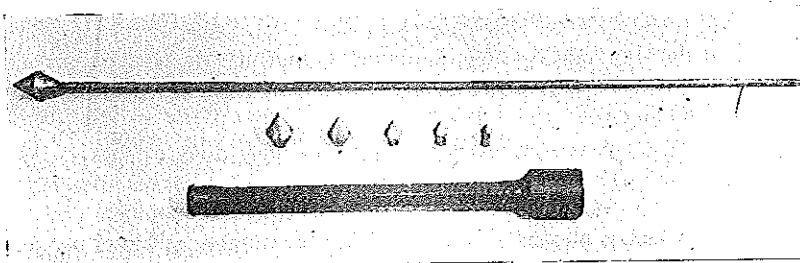


Fig. 1. The dynamic penetration test apparatus.

THE CONSTANT RATE OF PENETRATION APPARATUS

The constant rate of penetration apparatus (Fig. 2) was designed for laboratory use and consists essentially of an electrically operated mechanism

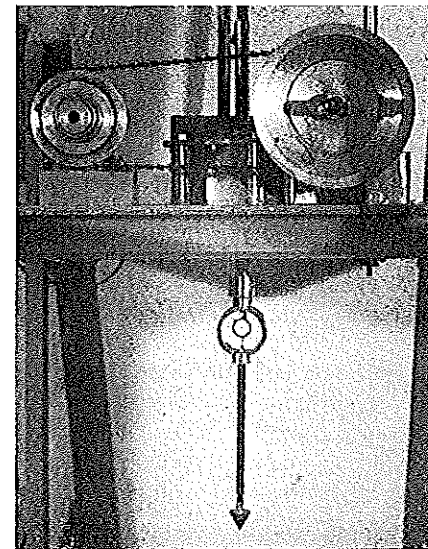


Fig. 2. The constant rate of penetration test apparatus.

driving a 60° cone vertically into a soil specimen at a constant rate of penetration. By a system of pulleys the constant rate of penetration can be increased to a maximum of 3 in./min; in the experiments described in this paper a constant rate of penetration of 2.34 in./min was used. The cone is connected to the lower end of a steel rod, and a proving ring attached to the top of the steel rod enables the driving resistance to be measured.

THE LATERITIC SOILS

The soils used in the tests were lateritic soils obtained from three different sites. Each soil specimen was remoulded at a series of different moisture

Table 1 Soil properties

Soil	Location	Plastic limit (%)	Liquid Limit (%)	S.G. of Soil particles	In situ dry density (lb/ft ³)
A	Economics building site, University of Malaya.	32.1	48.3	2.79	94.0
B	Paramount Gardens, Petaling Jaya.	40.2	82.0	2.70	94.5
C	Happy Gardens, Petaling Jaya.	42.3	87.0	2.71	90.4

contents and compacted by static pressure to its insitu dry density. The soil was compacted in four layers into 15 in. high steel moulds of section 18 in. \times 12 in. After compaction the soil was equilibrated for a period of 24 hours before testing.

Some of the properties of the three soil specimens are given in Table 1, and their grain size distributions are shown in Fig. 3.

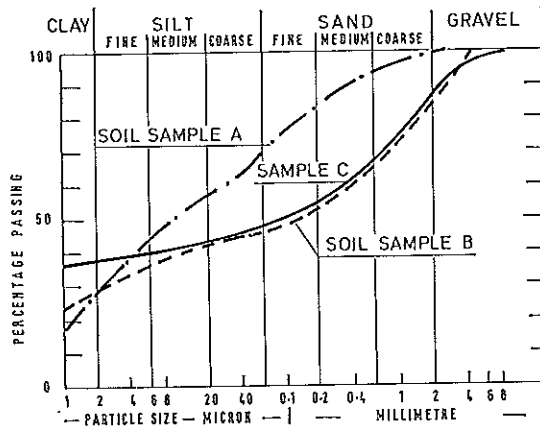


Fig. 3. Grain size distributions of soils used.

RESTRAINING EFFECT OF THE CONTAINER

The values of c and ϕ for lateritic clay change with variations in moisture content. It was, therefore, essential that throughout the entire process of remoulding, compaction and testing there be as little change as possible in the moisture content of the soil. For practical purposes it was necessary to reduce the quantity of soil to be handled for each test to a minimum in order to cut down the time required for the entire operation. This, in fact, meant that the container into which the soil was to be compacted in layers should be as small as possible without producing any restraining effect on the soil as the cone penetrated into it.

In their experiments on model piles with enlarged bases WHITAKER & COOKE (1961) used brass containers 14 in. in diameter. Their tests were on remoulded, brown London clay. The diameter of the pile shaft was 0.75 in. with bases of up to four times the shaft diameter. GOLDER & SKIPP (1957) in investigating the buckling of piles in soft clay drove metal rods of 0.125 in. diameter into clay contained in a cylinder of 4 in. internal diameter.

HAEFELI & FEHLMANN (1957) employed a consolidometer 250 mm in diameter containing clay at its liquid limit into which a pile of 50 mm diameter was introduced.

In general, the ratio of the container dimension to the diameter of the penetrometer used in experiments on sand are larger than on clay. STUART & HANNA (1961) used a steel box 40 in. \times 30 in. \times 25 in. deep. According to JAKY (1948), who computed the dimensions of the bearing bulb developed around the point of a pile driven into sand, the bulb in sand was approximately 25 times as large as in clay.

It has been shown (CHIN, 1969a) that, if penetration tests are carried out on the same remoulded lateritic clay with discs of different diameters, the plots of $q/\gamma d$ against Δ/d produce a single curve where Δ is the depth of penetration of the horizontal disc of diameter d at a bearing pressure q into a soil of density γ . This finding would seem to suggest a method by which any confining effect of the container, or any inconsistency in the soil properties, may be detected. If there were any inconsistency in the body of soil a single curve would not result. Inconsistencies in the body of soil can arise from

1. the closeness of the base and sides of the container, which prevents the free deformation and movement of the soil,
2. the driving of subsequent cones at too close a spacing, which causes deformation of the soil to take place in material which has already been disturbed by the penetration of previous tests.

The number of constant rate of penetration and dynamic tests to be made in the same soil required many moulds each containing soil of the same consistency. If the soil in each mould was not of the same consistency, then the plots of $q/\gamma d$ against Δ/d did not produce a single curve. The validity of all the test results was verified by means of this criterion; the constant rate of penetration tests were distributed over the multiple moulds to enable this verification to be made.

EXPERIMENTAL RESULTS

Variation of N_m with Depth of Penetration

An examination of the expression given by MEYERHOF (1951) for a deep foundation will show that, beyond a certain ratio of depth of penetration to size of foundation, D/B , the ultimate bearing pressure q_u becomes constant and does not further increase in magnitude with further increase in depth for

a constant soil profile. According to Meyerhof the ultimate bearing pressure of a deep foundation of width B is

$$q_u = c.N_c + p_o (N_q - 1) + \gamma \frac{B}{2} N_\gamma \dots \dots \dots (1)$$

where N_c , N_q and N_γ are the bearing capacity factors and p_o is the effective pressure of the overburden soil at the foundation level.

So long as D/B is greater than 5.0, the values of N_c , N_q and N_γ remain the same for a given soil; their magnitudes depend only on the angle of shearing resistance, ϕ , of the soil. TERZAGHI (1943) has shown that the effective pressure of the overburden soil approaches a constant value when the value of D/B becomes greater than 5.0. The value of q_u as given in Eq. (1) will, therefore, become constant beyond a certain depth of penetration. KERISEL (1961) and VESIC (1961) found this to be the case in their experiments in dense sand.

Figure 4 is a typical plot from the experimental results, giving the variation of N_m (expressed as blows/2 in. penetration) and the point resistance, P , with depth of penetration. It is clear that, in the constant rate of penetration tests, P initially increases with depth of penetration, reaches a maximum value and does not further increase with further increase in depth of penetration. It will be seen from Fig. 4 that N_m behaves in the same manner; the maximum value of N_m is reached at a greater depth of penetration for a greater diameter of the cone. This confirms that the dynamic penetration tests carried out on the same soil with cones of different diameters produce geometrically similar deformations.

The results of the tests are summarised in Table 2; N_m is the number of blows/2 in. penetration and A is the base area of the cone in in^2 .

Effect of v on N_m

It can be shown that the expression relating to the penetration of a cone into a $c - \phi$ soil is

$$P / \gamma d^3 = f (c / \gamma d, \phi, v^2 / gd) \dots \dots \dots (2)$$

where P is cone resistance, d is base diameter of the cone, v is velocity of penetration of the cone, g is acceleration due to gravity, γ is unit weight of the soil, c is soil cohesion, and ϕ is angle of internal friction of the soil.

Since N_m is a measure of the dynamic cone resistance, Eq. (2) can be written as

$$N_m / \gamma d^3 = f (c / \gamma d, \phi, v^2 / gd) \dots \dots \dots (3)$$

for a dynamic penetration test.

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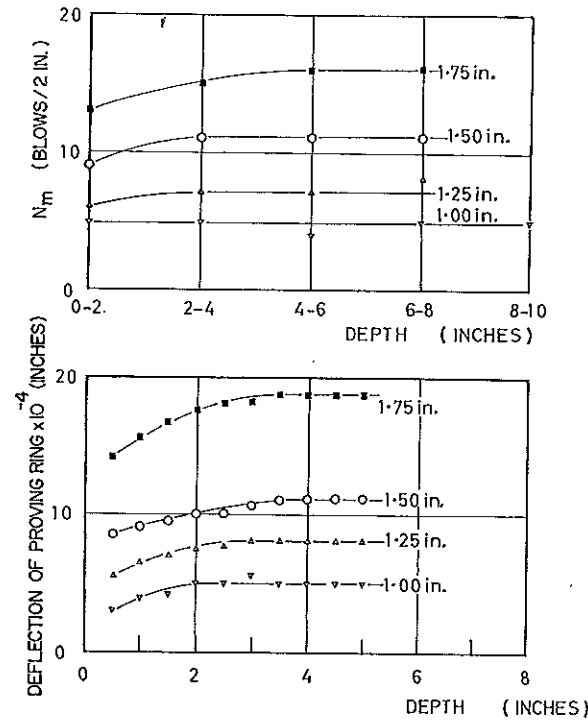


Fig. 4. Typical plot of the variation of N_m and P with depth of penetration.

For the MacKintosh probe each blow involves the dropping of a fixed weight hammer from a fixed height. The velocity of impact between the hammer and the anvil, therefore, is the same for each blow. The velocity of penetration of the cone into the soil, however, will depend on the weight of the rod and cone and on the magnitude of the cone resistance. Hence, if geometrically similar cones of different diameters are driven into the same soil using the same rod length, the velocity of penetration will increase with decrease in cone diameter. As indicated in Eqs. (2) and (3) the effect of the variation in the value of v on the magnitude of the ultimate dynamic cone resistance can be assessed when the plot of P/d^3 against $1/d$ obtained from the constant rate of penetration tests is compared with that of N_m/d^3 against $1/d$ obtained in the dynamic cone tests using the same soil. If these two plots

Table 2. Summary of Test Results

Location	Moisture Content (%)	1 in. dia		1.25 in. dia		1.50 in. dia		1.75 in. dia		Mean	
		q_u (ton/ft ²)	N_m/A (Blows/in. ²)	q_u (ton/ft ²)	N_m/A (Blows/in. ²)	q_u (ton/ft ²)	N_m/A (Blows/in. ²)	q_u (ton/ft ²)	N_m/A (Blows/in. ²)	q_u (ton/ft ²)	N_m/A (Blows/in. ²)
Economies Building Site (A)	20.5	32.1	16.5	32.8	19.2	33.3	22.1	34.3	30.4	33.1	22.1
	23.7	21.5	8.9	19.6	8.35	22.2	9.63	21.5	11.7	21.6	9.7
	26.3	14.9	6.36	15.3	6.7	16.5	6.2	15.35	6.25	15.0	6.3
	28.7	10.0	3.8	10.0	3.34	10.2	3.96	9.53	4.16	9.9	3.8
Paramount Gardens (B)	30.0	6.72	2.5	6.67	2.5	6.95	2.26	6.15	2.5	6.4	2.4
	16.0	37.8	16.5	35.8	17.6	39.8	17.6	42.8	21.4	39.1	18.3
	18.1	19.8	8.25	21.0	8.8	21.5	8.5	20.4	9.15	20.7	8.7
	20.0	14.9	6.35	15.25	6.3	14.6	6.2	14.2	6.45	14.8	6.2
Happy Gardens (C)	22.5	7.94	3.18	7.65	3.78	7.95	3.68	7.79	3.75	7.8	3.6
	25.3	4.48	1.9	4.77	2.1	4.65	1.98	4.87	2.08	4.7	2.0
	19.8	33.6	15.3	34.3	16.0	36.1	17.0	38.9	17.5	34.2	16.5
	22.3	20.1	7.65	19.55	8.8	20.6	8.5	17.2	8.5	19.3	8.2
	24.8	11.9	3.8	11.45	4.3	11.9	4.53	11.7	4.6	11.7	4.3
	27.2	6.72	3.18	7.16	3.34	7.28	3.12	7.3	3.3	7.1	3.3

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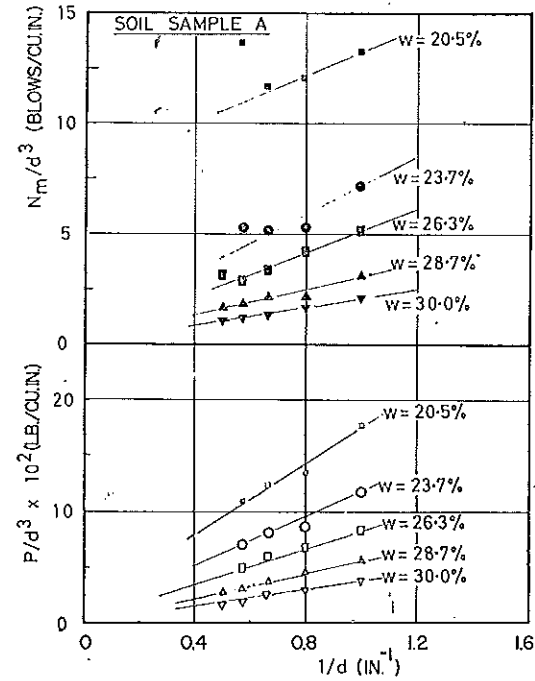


Fig. 5. Variation of N_m/d^3 and P/d^3 with $1/d$ for soil sample A.

are of the same function, e.g. if P/d^3 varies linearly with $1/d$ and N_m/d^3 also varies linearly with $1/d$, then the effect of the variation of ν on N_m is insignificant, since it has been shown (CHIN, 1969b) that the effect of ν on the value of P in a constant rate of penetration test is insignificant for values of ν as small as 2.34 in./min.

Figure 5 gives the plots of N_m/d^3 against $1/d$, and P/d^3 against $1/d$, for soil sample A. Figure 6 gives the corresponding test results for soil sample B. It is clear from these two figures that the relationship between N_m/d^3 and $1/d$ is reasonably linear, as is the relationship between P/d^3 and $1/d$. The variation in the velocity of penetration encountered in the dynamic cone tests has no significant effect, therefore, on the value of N_m . In these experiments only one length of rod was used and, therefore, any variation in the rate of penetration resulted from differences in soil resistance arising out of the different cone sizes. In the constant rate of penetration tests it

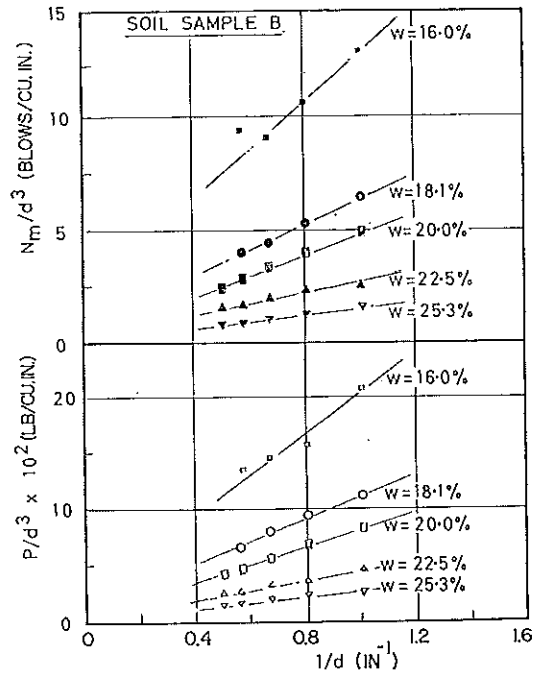


Fig. 6. Variation of N_m/d^3 and P/d^3 with $1/d$ for soil sample B.

was shown that the soil resistance to a 2 in. cone was four times that for a 1 in. diameter cone.

In the standard MacKintosh apparatus, though only one size of cone is used, soundings are frequently taken to a depth of 40 ft. Even if the soil does not change with depth, there will be a decrease in the velocity of penetration with increase in the length of the rods. This is because, in the deeper sounding, the same energy has to drive a greater length and, therefore, a greater mass, of rods. The weight of the rods in a sounding of 40 ft depth is ten times that when only one rod is used. If the values of N_m at depths of 4 ft and 40 ft are the same, it does not necessarily mean that the bearing capacities of the soil at these two depths are the same. Results of deep soundings have, therefore, to be interpreted with caution since the significance of the effect of the velocity of penetration has to be considered.

Relationship between N_m and Moisture Content

RUTLEDGE (1947, 1948) has observed that, on a semi-logarithmic plot of test data for saturated natural clay, the compressive strength - moisture content relationship is essentially a straight line parallel to the virgin part of the consolidation test curve. SEED & REESE (1957) also found this relationship to hold for reconsolidated, remoulded clay.

Figure 7 gives the semi-logarithmic plots of N_m and q_u against moisture content, w , for the three specimens of lateritic soil. In the case of soil specimen A each test specimen was prepared at a different moisture content but was compacted to the same dry density of 94.0 lb/ft³. In the case of specimens B and C the dry densities were maintained at 94.5 and 90.4 lb/ft³ respectively. These values of dry density are the *in situ* dry densities of the soils concerned.

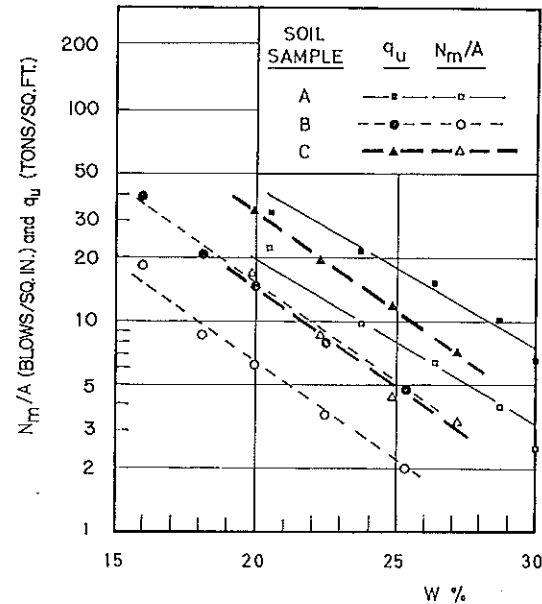


Fig. 7. Variation of N_m and q_u with moisture content.

It will be seen from Fig. 7 that the semi-logarithmic plots of N_m and q_u against percentage moisture content are parallel straight lines; that is, the

relationship between N_m and w is exponential. This is also true for the relationship between q_u and w .

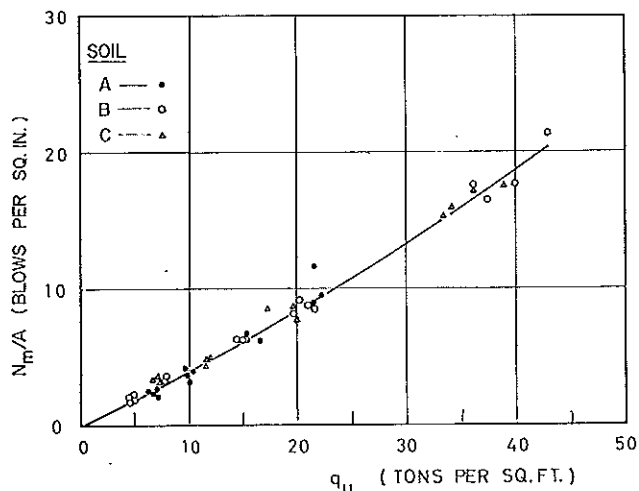


Fig. 8. Relationship between N_m/A and q_u on linear scales.

Relationship between N_m/A and q_u

The plots of N_m/A against q_u are given in Fig. 8. It is interesting to note that, in spite of the differences in their Atterberg limits, grain size distributions and dry densities, the three soil specimens produce a fairly common curve. It is evident from Fig. 8 that, for the soil samples used, some correlation between the constant rate of penetration test and the MacKintosh sounding test exists. The ultimate bearing capacity q_u (in ton/ft²) and the MacKintosh resistance N_m (measured in number of blows/2 in. penetration) are plotted on logarithmic scales in Fig. 9. A straight line relationship between N_m and q_u is more pronounced on the logarithmic scale than if plotted on a linear scale. This would seem to indicate that the relationship between N_m and q_u is not linear but has the form.

$$\log N_m = Y \log q_u - Z \dots\dots\dots (4)$$

where Y is the slope of the straight lines in Fig. 7 and Z is the intercept on the ordinate.

A single linear relationship results (Fig. 10) when N_m/A is plotted against q_u on logarithmic scales for the experimental results obtained with the dif-

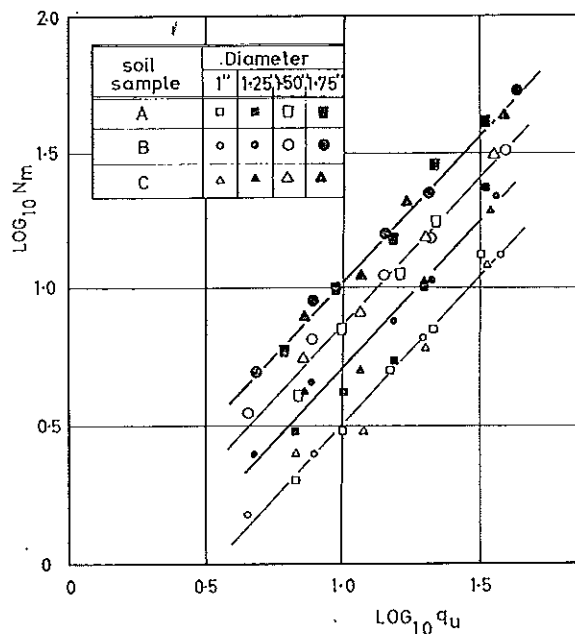


Fig. 9. Relationship between N_m and q_u plotted on logarithmic scales.

ferent sized cones. From Fig. 10 the relationship between N_m/A and q_u is deduced as

$$q_u^{1.60} = 2.692 (N_m/A) \dots\dots\dots (5)$$

where N_m is the number of blows/2 in. penetration, A is the base area of the cone in in², and q_u is the ultimate bearing capacity in ton/ft² as determined from a constant rate of penetration test.

CONCLUSIONS

From this experimental study it could be concluded that, for the soils concerned, there is a distinct correlation between the results obtained from the dynamic tests using the modified MacKintosh prospecting tool and the constant rate of penetration tests. This correlation manifests itself in the load-penetration curves and in the plot of N_m/A against q_u . The number of blows/unit depth of penetration is a distinct measure of the bearing capacity of each

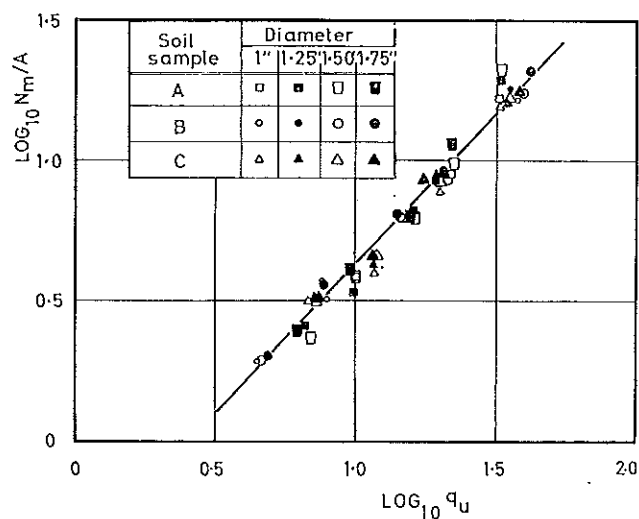


Fig. 10. Relationship between N_m/A and q_u plotted on logarithmic scales.

of the cohesive soils concerned; the relationship between these two is given by Eq. (5).

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