Some Loading Tests to Failure on Piles

Quelques essais de charge de pieux poussés jusqu'à la charge limite

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Summary

The paper describes a number of loading tests on piles carried up to the ultimate load. Some of the piles were pre-cast and some cast in-situ. In every case the soil conditions and characteristics are given, thus enabling an estimate to be made of the theoretical maximum load.

Sommaire

Cette étude décrit de nombreux essais de charge de pieux poussés jusqu'à la charge limite. Quelques-uns des pieux étaient moulés d'avance et d'autres coulés «in-situ». Les conditions et les caractéristiques du sol sont données pour tous les exemples mentionnés ce qui permet de calculer la charge limite théorique.

General

There are at least 68 types of pile described in engineering literature.

The problem is to find a method of determining the bearing capacity of a pile when installed without recourse to a loading test on every pile. At present this is attempted by the use of a formula which is applied regardless of how the pile is formed. In the several formulae available one or other, and sometimes both, of the two most important variables, viz: soil properties and method of installation of pile, is neglected. It is evident that there will be no unique solution to this problem, and that a simple classification of piles into groups must be adopted.

It is suggested that the methods of piling should be divided into driven piles and bored piles, each of these groups being subdivided into pre-cast piles and piles formed in-situ. The main types are therefore

(1) *Driven pre-cast.* The most common form of pile which is driven to a set.
(2) *Driven in-situ.* Formed by driving a tube with a closed end to a set and filling with concrete as the tube is withdrawn.
(3) *Bored pre-cast.* Pre-formed piles dropped into previously made borings. Not a common type but useful on occasions.
(4) *Bored in-situ.* Piles formed by boring a hole and filling it with concrete.

Many intermediate types exist, such as open ended tubes driven into the ground, cored out and filled with concrete, but they can all be placed in one or other of the above categories for purposes of calculation of bearing capacity.

In addition to the classification of the type of pile it is necessary to divide the soils into different categories. It is suggested that four categories will be necessary, namely, clay, sand, gravel, and soft rock. In the first of these categories the piles will normally be friction piles, the main support coming from the friction on the embedded sides, and the point resistance being relatively small. In the last category the point resistance will be all important and any skin friction can be neglected. The middle categories will be intermediate between these two, but in general will tend towards the last; i.e. in compact sand and gravel the point resistance will be of much greater importance than the skin friction.

Test Data to be reported

In each of the tests to be described, the following data will be given when they are available. In some cases the complete information was not recorded as its importance was not realised at the time. The data have been collected from many loading tests carried out by the author's firm over a number of years.

(1) *Soil Properties.* Description of soil and brief statement of strata giving levels and groundwater level. Liquid and plastic limit, mechanical analysis, natural moisture content, shear strength. Results of penetration or other in-situ tests.

(2) *Type of Pile.* Whether driven or bored, pre-cast or in-situ. Size, shape, length and weight of pile and material of construction. Level of toe.

(3) *Driving Record.* Weight and type of hammer and packing. Drop used and method of release. Final set in inches per blow. Measured temporary compression.

(4) *Loading Test.* Load-time-settlement diagram (or ulti-

mate load and corresponding settlement taken from such a diagram). Method of applying load i.e. whether dead load, jacking against a reaction or jacking off other piles. Methods of measurement of settlement. Any extraction data available.

(5) *Calculated bearing capacity.* Results of calculations by dynamic and static methods as described below.

Estimation of Ultimate Bearing Capacity

The methods available for estimating the ultimate bearing capacity of a pile can be divided into two classes, namely, dynamic and static.

(1) *Dynamic Methods.* The dynamic methods include the use of pile driving formulae in which a calculation is made using the driving records of the pile and a formula which purports to give the resistance of the pile to driving. They also include dynamic sounding methods in which a small-scale pile is driven and the resistance of the large pile is obtained by extrapolation from these results. Perhaps it would be more correct to say that the small scale tests are used to determine the constants of the driving formula, these constants then being used in the calculation for the large pile.

It is quite probable that the small scale dynamic sounding will give a fairly reliable guide to the dynamic resistance to driving of the large pile. The relationship of the dynamic

Table 1 Test results (Résultat des essais)

Loc.	*Soil Properties*	*Type of Pile*	*Driving Record*	*Loading Test*	*Calculated Bearing Capacity*
A	S Stiff clay underlying 8′ of gravel. STL Pile penetrated 13′ into the clay. WL Above ground level. L.L. P.L. Nat. M/C 44%–63% 17%–19% 23%–25% SS 3,500 lb/ft².	D Composite driven pre-cast pile of concrete and steel tube. L Bottom end 22′ long. Dim Octagonal concrete 10″ side i.e. approx. 25″ Dia. W 6.85 Tons.	TH Drop Hammer W 3 Ton Dr 84″ Se 0.08″ TC 0.5″ N Helmet contained hardwood dolly and rope grommet.	ML By jacking against two adjacent piles. MSM By cm scale. UL From Load-Time-Settlement. Diagram—210 Tons S At max. Load—2″ EL 110 Tons	DF E.N.R. Hiley 200 Tons 275 Tons Faber Clay Faber Sand 42 Tons 132 Tons SF 212 Tons—Assuming skin friction in clay equals shear strength and skin friction in gravel equals ½ Ton/ft². Note: Assuming that point resistance is difference between total load and extraction load the value is 100 Tons. This corresponds to a K* of 18. * K is the ratio of point resistance per unit area to shear strength.
B	S London clay (Stiff fissured clay—see Cooling and Skempton 1942) under 13 ft. of Thames gravel and sand. Peat and soft clay above this. PTL Pile 1: Penetration in clay 9 ft. Pile 2: Penetration in gravel 4 ft. WL in the soft clay. SS of clay between 2,000 and 4,000 lb/ft². Mean value at toe of pile 2,600 lb/ft². DS point resistance 5,500 lb/in² in gravel and 500 lb/in² in London clay.	D Two driven pre-cast concrete piles. Pile 1 (in clay) Pile 2 (in gravel) L 45′ 37′ Dim 14″ × 14″ 16″ × 16″ W 4.26 Tons 4.6 Tons	TH Semi-automatic steam hammer. Pile 1 Pile 2 W 4 Tons 4 Tons Dr 42″ (approx.) 42″ (approx.) Se 0.5″ 0.1″ TC 0.4″ 0.4″ P Softwood packing in helmet.	ML By jacking against kentledge MSM By cm scale UL Pile I (in clay) 155 Tons St 1.6 cm UL Pile 2 (in gravel): Not reached Loaded to 200 Tons St 0.5 cm	E.N.R. Hiley DF Pile 1 110 Tons 104 Tons Pile 2 151 Tons 225 Tons Faber Pile 1 49 Tons (80 Tons max. possible) Pile 2 167 Tons SM Pile 1—Deep sounding 150 Tons osuming skin friction equals shear strength in clay and ½ Ton/ft² in gravel. Pile 2—Deep sounding > 600 Tons. SF Pile 1—96 Tons assuming friction as above and 9 × shear strength for point resistance. With the assumed values of skin friction the value of K calculated from the actual load lies between 21 and 47 depending on value of shear strength of clay.
C	S Laminated silty sandy clay (Bracklesham Beds) overlain by 60 ft of sand. PTL Piles penetrated 22′ into the clay. WL near surface: L.L. P.L. Nat. M/C 52%–68% 15%–22% 19%–26% SS Undrained 3,010 lb/ft². Drained c = 700 lb/ft². $\varphi = 25^\circ$.	D Two driven pre-cast concrete piles. Pile 1 (pre-stressed concrete): L 75′ extended to 98′ Dim 12″ × 12″ W 6.4 Tons (Pile 2 (pre-cast): L 55′ extended to 91′ Dim 16″ × 16″ W 11.4 Tons	TH Drop hammer. W 4 Tons. Pile 1 Pile 2 Dr 27″ 6″ (on friction winch) Se 0.06″ 0.24″ TC 0.50″ Not measured Both piles jetted down most of the way. Pile 1 then driven 7 ft. Pile 2 then driven 6 ft. N Some sand removed by jetting.	ML By jacking against water tank. MSM By dial gauges and cm scale. UL Pile 1 Pile 2 120 Tons 50 Tons St 3.1 in. 3.4 in.	E.N.R. Hiley Faber DF Pile 1 102 Tons 128 Tons 166 Tons Pile 2 23 Tons 73 Tons 87 Tons SF Pile 1: 130 Tons point resistance for K = 9. Skin friction by normal methods = 120 Tons. Making a total of 280 Tons. However, owing to jetting skin friction is probably very much lower. Pile 2: For $\varphi = 25^\circ$ point resistance according to Meyerhof's coefficients is only 4 Tons. Some skin friction present but owing to effect of jetting it is impossible to say how much.
D	S Thames gravel under water. PTL Penetration into gravel 10 ft. The tests are described in detail by Bishop, Collingridge and O'Sullivan (1948). The results of a deep sounding on the same site were given by Golder and Ward (1950).	D Two driven pre-cast concrete piles. L 30′ Dim 12″ × 12″		UL 115 Tons and 150 Tons.	DF E.N.R. Hiley Faber 82 Tons 72 Tons 70 Tons SM Deep sounding 130 Tons. SF 58 Tons for $\varphi = 40^\circ$ using Meyerhof's coefficient. For $\varphi > 40^\circ$ $N\gamma$ increases very rapidly.
E	S Soil gravel below water level. PTL Penetration into gravel not known.	D Driven pre-cast concrete piles. L 30′ Dim 14″ × 14″ W 2.6 Tons	TH Drop hammer W 2 Tons Dr Approx. 36″ Se 0.05″	ML By jacking against kentledge UL Failure not reached. Loaded to 200 Tons. St 0.8 cm.	DF E.N.R. Hiley Faber 69 Tons 112 Tons 91 Tons SF 67 Tons for $\varphi = 40^\circ$ 268 Tons for $\varphi = 45^\circ$
F	S Soft clay 25 ft. to 30 ft. deep over soft rock (Keuper Marl).	D Driven pre-cast concrete pile. L 50′ Dim 16″ × 16″ W 6 Tons	TH Drop hammer W 4 Tons Dr 48″ Se 0.08″ TC 0.5″	UL Under a load of 130 Tons failure was not approached. St Maximum 0.3″.	DF E.N.R. Hiley Faber 183 Tons 190 Tons 180 Tons

resistance to driving, whether measured on the full scale pile or obtained by extrapolation from small scale dynamic tests, to the ultimate bearing capacity of a pile under a static load, is however doubtful.

In a clay soil the calculated resistance to driving would not be expected to compare with the ultimate bearing capacity by anyone aware of the properties of clays. This is in fact the case, as the tests quoted below indicate. Most of the dynamic formulae are based on Newtonian impact mechanics which are not fundamentally applicable to the case of driving a rod into a solid material. The simplest of these formulae is probably the "Engineering News Record" (*Chellis*, 1946) formula. A better formula, which makes some attempt to allow for energy losses by observation of the movement of the pile being driven, is the Hiley formula (*Hiley*, 1930). A further formula is the Faber formula (*Faber*, 1947) in which a differentiation is made between frictional soils, such as sands and gravels, and clays. If the formulae are regarded as a means of assessing the static bearing capacity of the pile, then this differentiation is an advance. If, however, they are regarded as a means of calculating the ultimate resistance to driving, there seems to be no logical reason why the two cases should be separated.

In the case of gravels and coarse sands there seems some possibility on theoretical grounds of developing a dynamic formula in which the calculated driving resistance will have

Table 1 (continued/suite) Test Results (Résultat des essais)

Loc.	*Soil Properties*	*Type of Pile*	*Driving Record*	*Loading Test*	*Calculated Bearing Capacity*
G	S Thames gravel overlain by soft silty clay. Depth to gravel 40 ft. PTL Pile founded an top of gravel. WL In silty clay. DS Cone resistance at top of gravel 2,000 to 3,000 lb/in^2 rising to 6,000 lb/in^2—2 to 3 ft. into gravel.	D Driven in-situ tube pile—concreted. L 40′ Dia 18½″ W of tube—1.5 Tons	TH Drop hammer. W 2.8 Tons Dr 24″ winch operated Se 0.12″ TC Not measured.	ML By jacking against kentledge. UL About 110 Tons. St Only 2 cm.	DF E.N.R. Hiley Faber 60 Tons 122 Tons 100 Tons SM Deep sounding 240 Tons.
H	S London clay overlain by 15 ft. of Thames gravel. PTL Penetration into clay 18 ft. WL In the gravel. SS Not known precisely for clay approx. 3,000 lb/ft^2.	D Bored in-situ concrete. L 45′ Dia 25″		UL Loaded to 113 Tons. Failure not approached. St Total settlement 0.15″. Permanent settlement 0.07″.	SF Total load = 224 Tons assuming skin friction in gravel of ¼ Ton/ft^2 (less than for driven pile) and 3000 lb/ft^2 for clay and 9 × shear strength for point resistance. Total load = 157 Tons if shear strength of clay is 2,000 lb/ft^2. The actual measured load under which no settlement occurred is greater than the calculated load would be if the skin friction had been due only to the softened shear strength of the clay.
J	S London clay throughout. PTL Pile 1: Penetration in clay 30 ft. Pile 2: 31 ft. Pile 3: — SS Mean shear strength 2,700 lb/ft^2. The best representation of shear strength is given by a straight line joining the points 1,600 lb/ft^2 at depth 10 ft. and 3,800 lb/ft^2 at depth 40 ft. DS Gave a cone resistance carrying from 20,000 lb/ft^2 at depth 10 ft. to 80,000 lb/ft^2 at depth 40 ft.	D Bored in-situ concrete. (Three piles tested.) Pile 1 Pile 2 Pile 3 L — — 23′ Dia 24″ 18″ 24″		Pile 1 UL > 110 Tons (failure not reached) Pile 2 Pile 3 130 Tons 125 Tons Pile 1 St 0.12″ (permanent 0.03″) Pile 2 Pile 3 St 4″ 2″	SF Pile 1: 248 Tons assuming K = 9 Pile 2: 176 Tons Pile 3: 170 Tons assuming K = 9. Some plate tests were carried out at the bottom of the borings. The values of K for those tests were from 9 to 15. The values of K from the soundings were 10 to 30 with even higher values for greater than normal rates of penetration. SM Pile 2: Deep sounding 221 Tons of which 157 Tons is skin friction.
K	S London clay overlain by 22′ of Thames gravel with 18′ of soft clay and fill above. PTL Penetration in London clay—3 ft. WL Just below ground level.	D Bored in-situ concrete. Dia 16″		ML By jacking from adjacent piles. UL Probably little more than 60 Tons. St 0.8″.	SF Ultimate bearing capacity = 44 Tons assuming K = 9 and skin friction in gravel is ¼ Ton/ft^2 for bored piles.
L	S Thames gravel under 14 ft. of soft clay. PTL Pile 1 penetrated 4′ into gravel. Pile 2 also penetrated 4′ into gravel but was on a bulb or mat of gravel consolidated by injection of chemicals. WL 9 ft. below ground level. DS Cone resistance at level of pile toe.—4,000 lb/ft^2.	D Bored in-situ concrete. (Two piles tested.) Dia 18″		Pile 1 UL 70 Tons St 18″ Pile 2 UL Failure not reached at 100 Tons St 0.4″ Permanent 0.35″	SM Deep sounding gives 500 Tons for both piles. SF Pile 1: 60 Tons for $\varphi = 40°$ Pile 2: 111 Tons allowing for area of chemical bulb.
M	These results have been published by Glossop and Greeves (1946). PTL Pile founded in gravel below water level. DS Gave pressure of 2,000 lb/in^2 at level of the pile toe.	D Bored in-situ. Dia 16″		UL About 100 Tons.	SM Deep sounding 180 Tons. SF 46 Tons for $\varphi = 40°$ and using Meyerhof's coefficients.
	S = Soil, PTL = Pile Toe Level, WL = Water Level, SS = Shear Strength, DS = Deep Sounding	D = Description L = Length Dim = Dimensions W = Weight Dia = Diameter	TH = Type of Hammer, Dr = Drop, Se = Set, TC = Temporary Comp., P = Package, N = Notes	ML = Method of Loading, MSM = Method of Settlement Measurement, UL = Ultimate Load St = Settlement	DF = Dynamic Formulae SF = Static Formula SM = Static Method

Table 2 Comparison of Measured and Calculated Loads (Comparaison des charges mesurées et calculées)

Location	Type of Pile	Soil	Actual Ultimate Load from Test Tons	Load in tons estimated from					Remarks
				Driving Records			Deep Sounding	Static Formula and Soil Tests	
				E.N.R.	Hiley	Faber			
A	D–PC	C	210	200	275	42 clay/ 132 gen.	—	212	Assuming ½ ton/sq.ft. friction in gravel
B	D–PC	C	155	110	104	49 (Mx. Poss. 80)	150	96	
		G	>200	151	225	167	>600	—	Failure not approached
C	D–PC	C&S	120	102	128	166	—	130–250	Jetted 130 is resistance of clay 250 includes ½ ton/sq.ft. skin friction in sand
			50	23	73	87	—	—	Jetted in sand—not driven hard afterwards
D	D–PC	G	115&150	82	72	70	130	58	Assumes $\varphi = 40°$
E	D–PC	G	>200	69	112	91	—	67–268	For $\varphi = 40°$ and $\varphi = 45°$ Failure not approached
F	D–PC	SR	>130	183	190	180	—	—	Failure not approached
G	D–IS	G	110	60	122	100	240	—	Ultimate may be higher than value quoted—settlement only 1.7 cm for 16″ diameter pile
H	B–IS	C	>113	—	—	—	—	224	Approximative only—no precise information on strength of clay
J	B–IS	C	>110	—	—	—	—	248	
		C	130	—	—	—	222	176	
		C	125	—	—	—	—	170	
K	B–IS	C	60 +	—	—	—	—	44	Assumes ¼ ton/sq.ft. friction in gravel
L	B–IS	G	70	—	—	—	500	60	For $\varphi = 40°$
			>100	—	—	—	500	111	Chemical bulb-size assumed for friction calculation
M	B–IS	G	100	—	—	—	180	46	$\varphi = 40°$

D = driven B = bored PC = pre-cast IS = in-situ C = clay S = Sand G = gravel SR = soft rock

some definite relationship to the static ultimate bearing capacity, but so far this end has not been achieved.

(2) *Static Methods.* The static deep sounding method as developed in Holland has given good results in the sands of that country. It has been found that the resistance per unit area is the same for a small diameter probe pushed into the sand at a steady rate as for a large diameter pile. This result would be expected from theory and has been proved to be true by loading tests on piles. An attempt has been made in England to extend this method to the gravels which are common there. The difficulty met with is that the size of the probe is comparable to the size of the stones which occur in the gravel. This results in a much more erratic record of resistance with depth and so far the approach which has been developed is to regard the low points as the value applicable to a pile which is large in diameter compared with the stones. This approach is probably conservative and is used for design purposes. In the results quoted in this paper, however, an attempt has been made to interpret the mean value.

Certain formulae exist from which it is possible to calculate the bearing capacity of a foundation. Most of these refer to shallow foundations and they must be extended if the case of a pile, which is a deep foundation, is to be included. Such an extension has recently been made by *Meyerhof* (1951).

In general these formulae consist of three terms. The first is a factor N_c multiplied by the cohesion of the soil; the second is a factor N_q which is multiplied by the vertical pressure at the level of the bottom of the foundation, and the third is a factor $N\gamma$ which is multiplied by the weight of the soil and the half-width of the foundation. These factors depend on the angle of international friction of the soil.

For a deep foundation, the following possibilities exist:—

(*a*) *Cohesive materials*

$$q = c \cdot N_c + \gamma \cdot D$$

where N_c is approximately 9 from theoretical considerations and some small scale tests. To the above the skin friction on the shaft of the pile must be added. This is normally taken as the embedded area times the shear strength of the soil.

(*b*) *Frictional materials.* For frictional materials with a deep foundation the only important term is the $N\gamma$ term. The values given by *Meyerhof* (1951) for $N\gamma$ with different values of φ are given below in Table 3. These values refer to a strip footing, and for a circular footing Meyerhof introduces an empirical shape factor which is 2.1 for driven piles and 1.3 for bored piles. *Meyerhof*'s values of $N\gamma$ for a surface footing are practically identical with those given by *Terzaghi*.

Table 3 $N\gamma$ terms by *Meyerhof*, 1951

φ	$N\gamma$ *Surface*	$N\gamma_d$ *Depth*
10	0.7	2
20	5	18
25	11	50
30	24	150
40	110	2,000
45	300	8,000

Finally, the most reliable method of estimating the ultimate bearing capacity of a pile is by means of a loading test to failure on the pile itself. Failure can be defined as the continued penetration of the pile under no increase in load. It is important to realise that time is a factor in this condition and that for this reason the failure load cannot be estimated from a load-settlement curve alone. A load-time-settlement curve of the type shown in Fig. 1 is required.

The settlement and bearing capacity of a group of piles may be very different from that of a single pile for many reasons. This is not considered in this paper.

Test Results

The results of loading tests on piles at twelve different locations are given in Table 1.

Table 2 gives a comparison of the measured and calculated loads. In this table the value K is the ratio of point resistance per unit area to shear strength. Fig. 2 shows a grading curve for the Thames gravel in which many of the tests were carried out, and in Fig. 3 is given the result of a deep sounding in the same soil.

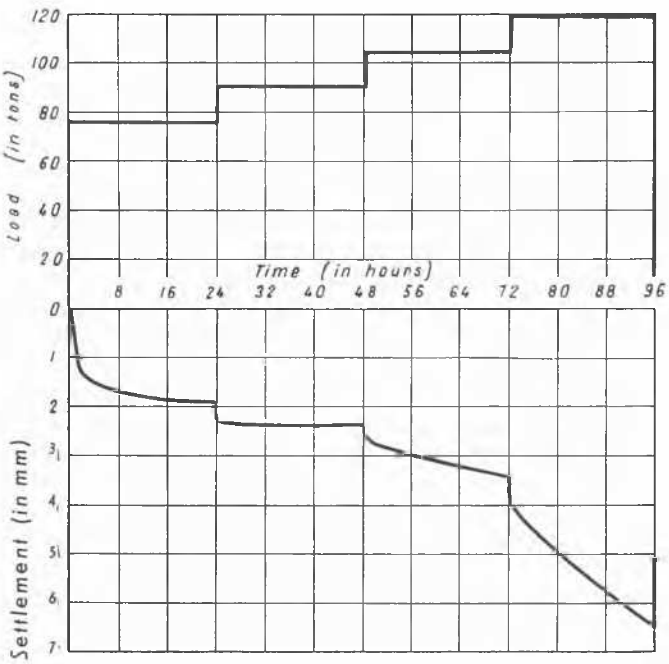

Fig. 1 Typical Load-Time Settlement Curve
Courbe typique montrant la relation de la charge avec le temps et le tassement

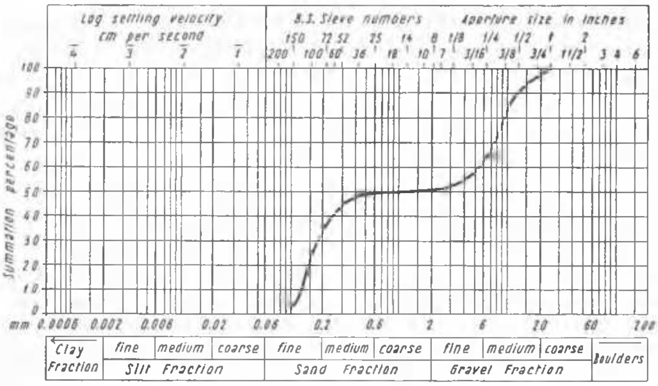

Fig. 2 Typical Grading Curve for Thames Gravel
Courbe granulométrique pour le gravier de la Tamise

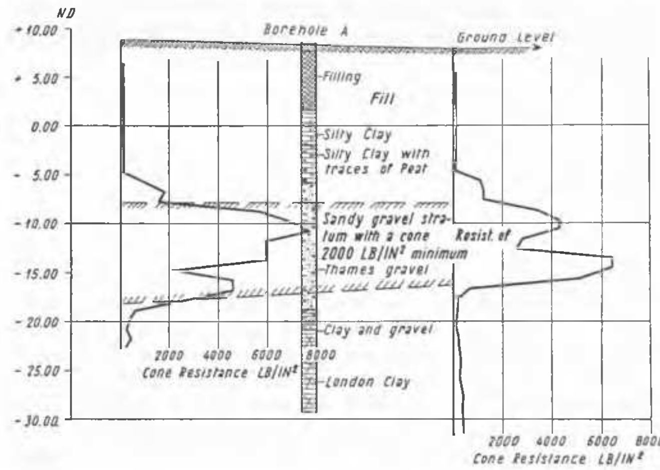

Fig. 3 Typical Deep Sounding Results in Thames Gravel
Résultats des essais de pénétration statique dans le gravier de la Tamise, méthode hollandaise

Discussion and Conclusions

The following tentative conclusions are advanced as a result of a study of the test figures given.

(1) None of the dynamic formulae considered can be relied upon to give an answer which is correct. The answer can vary from about one third to nearly twice the correct value, but tends to be lower rather than higher. These formulae therefore tend to cause uneconomical design but may sometimes be unsafe.

(2) The deep sounding method as used in Holland cannot be applied to gravels without modification owing to the marked variations in resistance which are obtained. This is no doubt due to the cone lodging on stones comparable in size with itself. In clays the point resistance is not the most important factor, but the application of this method to these soils deserves further investigation.

(3) In gravel the point resistance depends so critically on the value of the angle of internal friction when this is greater than 40° that the static formula can only be used to give a safe lower limit of bearing capacity. This is not so for sands in which the angle of friction is lower. Although the theoretical

approach is the same for both these materials they must be treated separately in practice because of this important difference.

(4) The static formula may prove fairly reliable in clays when a better estimate of the skin friction in gravel overlying the clay becomes available.

(5) Some indication is given by the results that the value of *K*, which relates the point resistance to the shear strength of the clay, is greater in practice than the theoretical value of 9. Further testing is necessary to clear up this point. However, the suggestion is made that the value will in fact be found to lie between 10 and 20. The higher values ranging up to 30 or 40 obtained by the deep sounding device may possibly be explained by vacuum effect when the cone leaves the sounding tube. This is particularly noticeable with the modern hooded type of cone.

(6) The tests give some evidence that the allowable shear strength to be used in calculating the skin friction on the shaft of a bored pile in clay is about half the unconfined compression strength. Differences of opinion on this point are likely to exist as it is probable that the value obtained in practice will always be a function of the technique of construction of the pile.

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