The Use of Penetration Tests for Determining the Bearing Capacity of Piles

L'emploi des essais de pénétration dans la détermination de la force portante des pieux

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Summary

A soil penetration test was carried out with a Mega pile and the resistances obtained throughout the length of the pile are compared with the resistances estimated by extrapolation of values obtained with a penetrometer of Belgian type. The average values of minimum soil resistances of cone penetration and the average values of skin friction are used for a gravelly sand stratum 2·5 m thick. The linear relationship between the skin friction on one hand and the circumferences of the penetrometer tube and the pile on the other, is assumed for extrapolation. It was found that total ultimate resistances are in a good agreement.

These results are used for the interpretation of data obtained from penetrometer tests for estimating the ultimate bearing capacity of several prestressed concrete piles, on which loading tests have been carried out.

The part depending on skin friction as obtained by pile loading test i.e. from the graph of penetrometer test, is subtracted from the estimated total ultimate resistance of the pile by the load settlement diagram and thus the ultimate point bearing capacity is obtained.

The load bearing capacity of the piles has been compared with their average resistance, calculated from the resistance to cone penetration. An estimated factor of safety of 2 was calculated from penetrometer tests.

Sommaire

Un essai de pénétration fut effectué avec un pieu Méga et les résistances obtenues pendant l'enfoncement du pieu furent comparées aux résistances obtenues par extrapolation des chiffres donnés par un pénétromètre du modèle belge. Les valeurs moyennes du frottement latéral ont été appliquées à une couche de sable graveleux d'une épaisseur de 2,5 m. La relation linéaire entre le frottement latéral d'une part et les circonférences du tube du pénétromètre, et du pieu d'autre part, a été admise pour l'extrapolation. On a trouvé un bon accord entre les résistances totales à la limite de rupture.

Ces résultats ont été utilisés pour l'interprétation des résultats donnés par des essais au pénétromètre destinés à évaluer la charge portante limite de plusieurs pieux en béton précontraint, sur lesquels des essais de chargement avaient été effectués.

La charge portante limite en pointe est obtenue de la façon suivante : la partie correspondant au frottement latéral trouvée à partir de l'essai de chargement d'un pieu c'est à dire déduite du graphique d'un essai au pénétromètre, est soustraite de la résistance limite totale du pieu estimée d'après le graphique des enfoncements en fonction des charges.

La charge en pointe des pieux a été comparée avec la valeur moyenne, calculée à partir de la résistance en pointe du pénétromètre. Un facteur de sécurité évalué à 2 fut calculé à partir des essais au pénétromètre.

Introduction

While solving various soil mechanics problems in their country, the authors have been unable to obtain any data regarding density of sandy layers by means of classic sounding methods. For this reason they have introduced penetration tests by static deep-sounding penetrometer of Belgian-Dutch type and since 1957 they have used it as a supplement to boring tests.

All their field tests were carried out with tubes and a cone of 36 mm diameter. The base area of the solid 60 degree cone was 10 sq. cm. The periphery of the tube was 11·2 cm and the penetrometer capacity was 10 tons.

On the site of the Belgrade Fair (Universal Hall 1) a test with a Mega pile was carried out in addition to the penetration tests.

Borehole tests revealed that the soil under this hall consists of made ground and agricultural upper layers followed by clayey silt and silty sand with more or less organic silt in the lower strata. Average thickness of this very compressible layer is between 12·0 and 16·0 m. Under this layer there are sandy gravel layers of different thicknesses extending to the maximum thickness of 8·0 m. Under these strata there is a layer of green greyish marly tertiary clay, well-compacted and of very great thickness. The elevation of ground is about 75·50 m.

The soil on the present site of Belgrade Danube River Port is of typical alluvial character, with deposits which are underlain by tertiary marly clay. According to sounding data, from above and under the ground-level there is fill of heterogeneous composition under which follow the layers of silty sand and of silt blended with fine sand. These layers are over 16 m thick, and beneath them are layers of fine sand with silt. The distribution of these materials is irregular. Dense sandy gravel deposits occur at a depth of about 20 m.

The soil in which the main sewer is constructed consists of tertiary and quaternary products. Near the sewer are layers of tertiary clay at the depth from 16·0 to 18·0 m benaeath the ground. Over the tertiary clay there are alluvial sediments from the river Danube, represented by different layers of sand and gravel. The gravel appears in lentils and in very thin layers like the silt.

All Hoyer piles, which are further discussed in this paper, are made of prestressed concrete.

The penetration test with the Mega pile

This test was carried out by the static method with hydraulic pump and hydraulic jack. The reaction was taken by a wooden box filled with gravel. The wooden box rested on a raft of steel beams. Pressures were read by a manometer for each 30 cm of penetration, i.e. in the middle and at the top of each element of square cross section 30 × 30 cm and 60 cm long (Fig. 1).

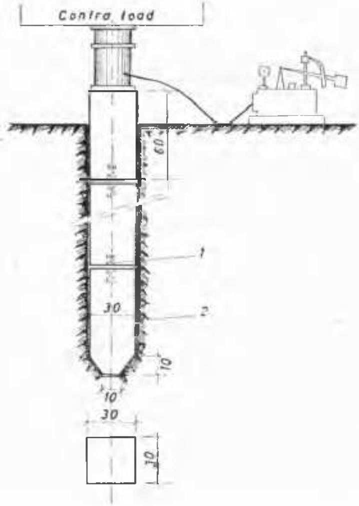

Fig. 1 Penetration of Mega pile. 1. wedge; 2. pile element.
Enfoncement d'un pieu « Mega ». 1. cale; 2. élément du pieu.

The ultimate soil resistance to penetration by the Mega pile represents the total resistance, which consists of the point resistance and skin friction between the soil and the pile. In Fig. 2 we have a diagram of the variations of the total penetration resistance down to a depth of about 60 m, where further penetration ceased.

Further penetration soundings revealed that the sandy gravel layers through which the Mega pile was driven, were of a medium to well compacted nature (see Fig. 3). The estimated extrapolated resistances on the Mega pile are obtained from the cone penetration resistance and from the total skin friction of the sounding P_2. They are both plotted in the diagram (Fig. 2).

Both graphs of resistances, to the depth where the Mega pile reaches the sandy gravel layer, take similar courses and the values of resistances are approximately equal at least for most of the depth. The relative variations of the resistance values obtained by penetration of the Mega pile as a function of depth, are smaller than those obtained by the penetrometer. In the upper sand layer the values of the total Mega pile penetration resistance are smaller than those estimated by extrapolation. This difference is mainly due to the fact that the values of cone penetration resistance are greater than the values of point penetration resistance of the Mega pile. Variability of soil resistance to penetration as related to the cone diameter i.e. the pile-point, is obvious.

For the 2·5 m thickness of alluvial deposit the average value of cone penetration resistance for both the adjoining penetration soundings P_1 and P_2 (Fig. 3) is approximately 112 kg per sq. cm $\left(\dfrac{109 + 116}{2}\right)$.

Referring only to the minimum values of cone penetration resistance for the 2·5 m thick stratum, the medium value for both penetration soundings, is approximately 82 kg/cm² $\left(\dfrac{72 + 91}{2}\right)$

If these values (112 and 82 kg/cm²) are extrapolated to the Mega pile point area the point penetration resistance of approximately 101 tons $\left(\dfrac{112 \times 900}{1\,000}\right)$ resp. approximately 74 tons $\left(\dfrac{82 \times 900}{1\,000}\right)$ will be obtained.

The calculated average value of total skin friction on the 2·5 m chaft-length for both penetration soundings is 2 955 kg. Assuming that the skin friction on the penetrometer tube and pileshaft in the soil is proportional to their peripheries the resistance of skin friction of the Mega pile, will be 32 tons $\left(\dfrac{2\,955 \times 120}{11{\cdot}2 \times 1\,000}\right)$.

The total calculated resistance of a Mega pile is :

(a) For the mean value of cone penetration resistance and total skin friction

$$N_m = 101 + 32 = 133 \text{ tons}$$

(b) For the average value of minimum cone penetration resistance and total skin friction

$$N_{1m} = 74 + 32 = 106 \text{ tons}$$

The total penetration resistance obtained by the penetration of a Mega pile over the larger part of the considered 2·5 m thick layer, was approximately 111 tons (Fig. 2).

The value of 32 tons should be regarded as the contribution of the skin friction of the Mega pile although its skin fric-

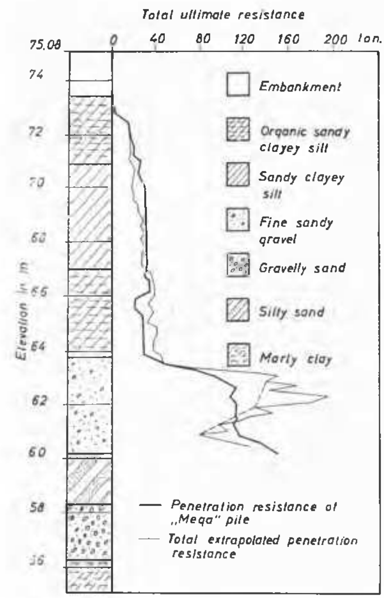

Fig. 2 Diagram of penetration resistance of Mega pile and total extrapolated penetration resistance.
Diagramme de résistance à la pénétration du pieu Méga et résistance totale à la pénétration après extrapolation.

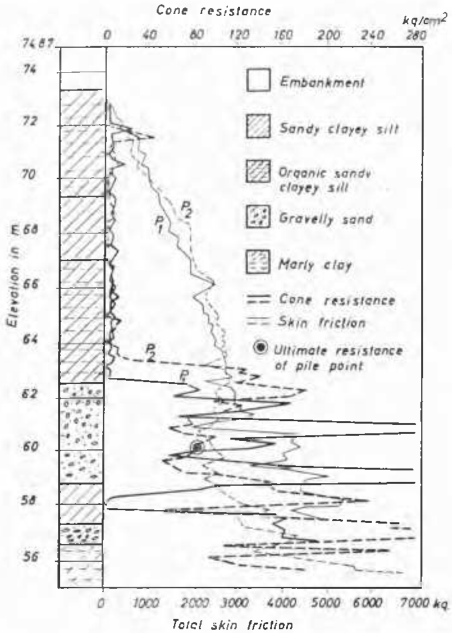

Fig. 3 Diagrams of soil penetration resistance for borings P_1 and P_2. Belgrade's fair-grounds, Hall I.
Diagramme de pénétration aux forages P_1 et P_2 (terrain de Foire de Belgrade, Hall I).

tion is greater than the skin friction of the smoothly polished metallic penetrometer tube and then the Mega pile point penetration resistance would be : 111 — 32 = 79 tons. The specific point penetration resistance amounts to approximately 88 kg/cm² $\left(\frac{79 \times 1\,000}{900}\right)$.

It can be seen from this analysis that a better agreement of ultimate resistances is obtained when the average values of the minimum cone penetration resistances are taken as 79 tons and 74 tons. The total ultimate resistances in this case with skin friction are in good agreement (106·0 tons and 111 tons).

In accordance with the above results, we can extrapolate the 36 mm diameter cone penetration resistance and the total skin friction, for estimating the ultimate pile resistance, the pile point being supported by the sandy gravelly layer at sufficient depth.

Pile loading tests

The authors selected a few piles on which to carry out pile loading tests. They also undertook field tests in the vicinity with a penetrometer.

The pile loading tests were not carried out to soil failure except for the pile No. 1 which was driven at the site of the river port on the Danube. The ultimate resistances were estimated according to the method described by C. Van der Veen. For estimating the contribution of skin friction the method described by A. F. Van Weele is used, except for the pile No. 20 only, for which the skin friction was determined by extrapolation.

During the loading test of pile No. 20, with the base at an approximate elevation of 60·0 m (Fig. 3) the maximum load was raised to 98 tons. The estimated ultimate bearing capacity of the pile was 135 tons.

The skin friction, extrapolated for the pile from the data obtained from a penetrometer test is 33·0 tons. The total settlement of the pile top is about 5 mm of which permanent settlement is about 2 mm and elastic settlement about 3 mm.

This pile is in the sandy gravelly layer, and has a length of 3·5 m. All other specifications are given in the table 1.

Pile No. 1 was driven through the upper strata and then through sand, which in its upper layers was of a medium dense character. The pile point, according to the penetration resistances, was in a layer of well compacted sand at the depth of about 5·0 m.

About 2·0 m apart from the pile the cone penetration test was performed. Fig. 4 shows the variations of the cone penetration resistances as a function of depth, which attained about 22·6 m. The dotted line near the pile toe is the envelope of minimum values of cone penetration resistance.

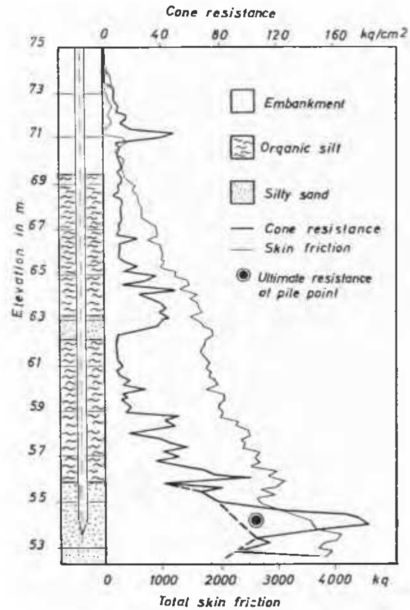

Fig. 4 Diagram of soil penetration resistance. Belgrade, Danube's port.
Diagramme de pénétration (Belgrade : port sur le Danube).

For the loading test of this pile a load of about 200 tons was used. To carry out the test a hydraulic pump and a hydraulic jack of 200 tons total capacity were employed. Pressure readings were made by manometer and for measuring the settlements of the pile top the comparators of 5/100 mm sensibility were used. The settlement was also checked by levelling. The load was applied in increments of about 15 tons. The unloading and re-loading was done by loads of 61, 92, 123 and 181 tons. The load-settlement record as presented

TABLE 1

Number	HOYER'S PILE					RESISTANCE FROM PILE LOADING TEST				ESTIMATED RESISTANCE FROM DATA OF PENETRATION			DATA OF PENTRATION		POINT RESISTANCE FROM PILE LOADING TEST R_{p_2} IN K_g/Cm^2	R_{p_1}/R_{p_2} IN %	LOCATION AND NUMBER OF TEST PILE
	Area of point in Cm^2		Length in m.	Free length in m.	Length of point in m.	Max. load in ton.	Total ultimate resistance in ton	Skin friction in ton	Point resistance in ton	Point resistance in ton	Skin friction in ton	Total ultimate resistance in ton	Skin friction in ton	Point resistance R_p in Kg/Cm^2			
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1	35 × 35	=1255	15	–	045	98	135	33	102	100	33	133	2.66	82	83	99	BELGRADE, FAIR - GROUNDS, HALL I, NO. 20
2	40×40	= 1600	21.28	4.17	0.55	186	220	(55) 44	(165) 176	142	55	197	3.80	89	(103) 110.	(86) 81	BELGRADE, DANUBE'S PORT, NO. 1
3	30×30	=900	10	–	0.40	100	130	(27) 20	(103) 110	135	27	162	2.50	146	(114) 124	(128) 118	BELGRADE, PRINCIPAL COLLECTOR TO DANUBE, NO. 27
4	30×30	= 900	8.80	–	0.40	137	165	(16) 23	(149) 142	100	16	116	1.50	111	(165) 158	(67) 70	BELGRADE, PRINCIPAL COLLECTOR TO DANUBE, NO. 180

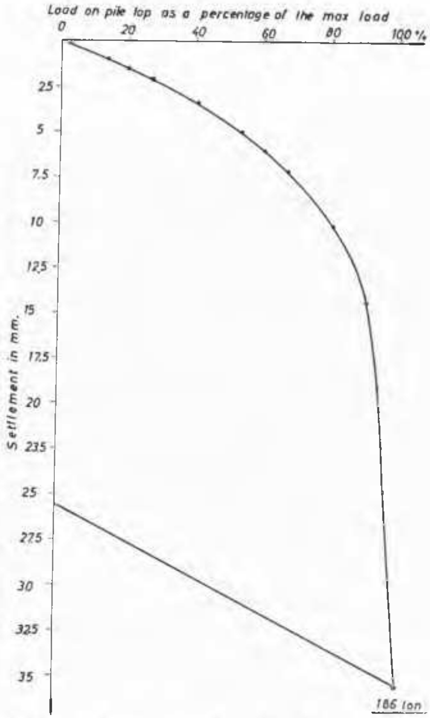

Fig. 5 Load-settlement curve of pile No. 1 (Belgrade, Danube's port).

Diagramme des enfoncements en fonction des charges pour le pieu n° 1 (Belgrade : port sur le Danube).

by Fig. 5 with hysteresis loops, omitted for better inspection, but these did occur during the unloading and re-loading. For loading increments of 61 and 123 tons the duration of load was prolonged up to 24 hours. Under a load of 186 tons a rapid increase of settlement indicated soil failure. According to Terzaghi, at a settlement of 1/10 pile side length the soil failure under the pile point should occur. In our case the measured settlement of 35·6 mm is close to the limit value of $1/10 \times 400 = 40$ mm. Fig. 6 shows the diagram for determination of the total ultimate resistance, which is 220 tons (see Table 1).

During construction of the main sewer which carries waste water to the river Danube, the loading test on piles No. 27 and No. 180 was carried out in a similar manner.

The pile No. 27 with its top at an elevation of 68·46 m under the sewer was driven through medium and fine sand of medium density, the pile point being at an elevation of 58·46 m in a layer of well compacted medium and fine sand. A field penetration test was also made by deep-sounding penetrometer at 3·25 m from the pile, measured across the sewer normal to its centre line.

During the load test on this pile after the loading increments of 32 and 70 tons, the load duration was prolonged up to 24 hours. The maximum load was 100 tons. Under this load the settlement of pile top was about 13 mm. Fig. 7 (a) shows the estimated ultimate resistance of 130 tons. Other data are given in the Table 1.

The pile No. 180 with its top on elevation of 69·5 m under the sewer was driven through loose, medium and fine sand. The pile point is on elevation of 60·70 m in the same layer of sand which is dense.

A field penetration test was also performed at 3·50 m from the pile and across the sewer.

The loading test was carried out in a similar manner to that on pile No. 1. The maximum load was 137 tons and the total settlement of the pile top under this load was 8·54 mm. Fig. 7 (b) shows the estimated ultimate resistance of 165 tons.

The use of soil penetration resistance

The shape of the sliding areas around the pile base depends on the angle of internal friction and pile diameter. For the interpretation of penetration data, the limit of sliding areas is assumed, which is at $4d$ over the pile base and under it at $2d$, d being the equivalent diameter of pile. The assumed length of the sliding areas has a total length of 6 d. The medium values of minimum cone penetration resistances are calculated for this length from the correspondent diagrams of penetration. These values are then extrapolated similarly as for a Mega pile, on the other piles. The estimated ultimate resistances for these piles are given in Table 1.

The contribution of skin friction was estimated, assuming that the skin friction is proportional to the peripheries of the penetrometer tube and pile. Average value of skin friction for the assumed length of sliding areas was obtained.

Except for the pile No. 180 (Table 1) the values of skin friction obtained by A. F. Van Weele's method are smaller than the estimated values obtained by extrapolation (piles No. 1 and No. 27). In column 17 of table 1 is given the ratio of the specific point resistance obtained from penetration

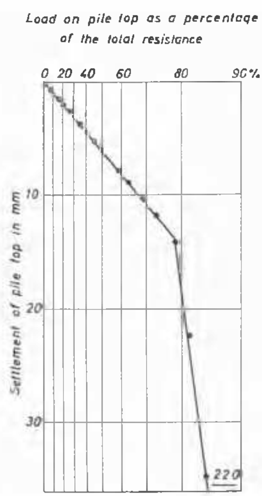

Fig. 6 Determination of total ultimate resistance according to the method described by C. Van Der Veen for pile No. 1. (Belgrade, Danube's port).

Détermination de la résistance limite totale d'après la méthode de C. Van der Veen, pour le pieu n° 1 (Belgrade : port sur le Danube).

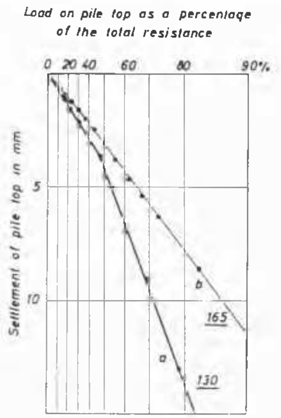

Fig. 7 Determination of total ultimate resistance according to the method described by C. Van Der Veen for : (a) pile number 27 ; (b) pile number 180. Belgrade, the main sewer to the Danube.

Détermination de la résistance limite totale d'après la méthode de C. Van der Veen pour : (a) le pieu n° 27 ; (b) le pieu n° 80 (Belgrade, collecteur principal vers le Danube).

data and from loading test. The lower limit of these ratios is 70 per cent and the upper one is 118 per cent.

The values of point resistance obtained after the values of skin friction have been reduced from the total ultimate resistances and are given in the brackets in table 1, column 10. The values of skin friction are given in column 12 in the same table. The lower ratio limit of the specific resistances from column 17, in this case, is about 67 per cent and the upper one is about 128 per cent.

Factor of safety

According to Bullen (1959) the linear ratio of the load and settlement of the pile point is extended to the limit (yield point), which is located approximately at the load of 50 per cent of the ultimate point resistance. According to the results of loading tests by Plantema (1948), the limit of linear ratio limit is at 60 per cent. The estimated limit from the load-settlement curve for pile No. 1, if the first curvature is neglected, this is approximatively 50-60 per cent. If 60 per cent of the ultimate pile resistance is adopted as the allowable load, the factor of safety would be 1·67. For the limit of 50 per cent, the factor of safety is 2. The upper limit of ratio of the specific resistances (see Table 1, column 17) for the pile No. 27 is 118 per cent (128 per cent respectively). The factor of safety for the allowable load, which is obtained from the ultimate pile point resistance, from the penetration data, should be : $1{\cdot}67 \times 1{\cdot}18 = 1{\cdot}97$ or $1{\cdot}67 \times 1{\cdot}28 = 2{\cdot}14$. For the limit of 50 per cent, the factor of safety is : $2 \times 1{\cdot}18 = 2{\cdot}36$ or $2 \times 1{\cdot}28 = 2{\cdot}56$.

Conclusions

From the above examples, in spite of their scarcity it may be concluded that the cone penetration resistance obtained by penetrometer with 36 mm diameter cone and tubes may be used with extrapolation for piles of much greater diameters. In the same manner it is possible to estimate the contribution of skin friction, with a sufficient accuracy, for the ultimate resistance of pile, assuming that the skin friction is proportional to the peripheries of penetrometer tube and pile. By the interpretation of penetration data, for the soil resistance according to the method discussed by the author it could be seen that the factor of safety of 2 is sufficient (for the adopted limit of the linear load-settlement ratio of 60 per cent). This item is important only if the allowable bearing capacity of piles is estimated against the ultimate point resistance. If we take skin-friction into consideration, the factor of safety should be greater than 2, in correlation with the compressibility of upper layers through which the pile is driven on its way to the bearing layers of sand or sandy gravel, and with due allowance for the negative values of pile skin friction.

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