Soil Modulus from Large Scale Plate Load Test

A. Holeyman[1)], M. Bottiau[2)]
and K.R. Massarsch[(3)]

ABSTRACT

An innovative plate load test has been developed, using the 44 tons (400 kN) load provided by a piling rig. Because of the large size of the loading plate (80 ft^2 or 7.5 m^2), the depth of influence typically extends to about 20 ft (6 m). The piling rig is instrumented with sensors to measure the load-deformation characteristics of the soil. Simple charts are produced for the interpretation of field data in terms of reaction modulus.

An extensive field testing programme was performed to check the relative soil improvement achieved by sand column installation. The comparison between moduli determined empirically from CPT and PMT tests and those measured by the large plate load test shows good agreement and that settlements can be significantly reduced by the installation of displacement-type sand columns.

INTRODUCTION

The efficiency of stone columns is affected by the method of installation. One common method uses an uncased hole which is created by water-jetting and the column is constructed by compacting the stone which is added from the ground surface, by a vibrator. Displacement type stone columns are installed in a similar way as conventional driven in-situ cast piles. A steel casing is driven into the soil, and then gradually withdrawn while the stone, gravel or sand is effectively compacted by a free-fall hammer. In this way the strength and the shape of the shaft can be controlled.

1) Deputy General Manager, 2) Project Engineer, 3) General Manager Technology; S.A. Franki, 196, Rue Grétry, 4020 Liège, BELGIUM

The compaction effect of stone column installation is normally checked by conventional geotechnical methods such as static cone penetration tests (CPT), Standard Penetration Tests (SPT) or pressuremeter tests (PMT). However, it is difficult to predict from tests performed at individual points the load-deformation behaviour of the soil mass which is reinforced by the stone columns.

In order to overcome this difficulty, a method has been developed, where the weight of the piling rig is utilized to carry out a large-scale plate load test, called a static ground test (SGT). SGT can be interpreted in two ways, either as a qualitative test, where only the increase in soil stiffness is measured, or as a quantitative test, where the actual modulus of subgrade reaction, k, is determined. As a static ground test is carried out within a short time period (several minutes), excess pore water pressure can not dissipate in cohesive soils and thus the soil will be tested under undrained loading conditions. However, as the stone columns consist of granular material, these will be loaded under drained conditions.

An extensive field testing programme was performed on a site in the vicinity of Antwerp, Belgium in order to assess the performance of displacement-type sand columns. The objective was to measure in the field the effect on the soil stiffness of sand columns and of a surface fill with varying thickness. This paper summarizes the test results which allowed us to compare the soil stiffness deduced from the literature, and from conventional geotechnical investigations, with that determined from large-scale plate load tests (SGT).

The successive stages of the field testing programme involved :

a) initial CPT and PMT,
b) static ground testing (SGT) of the virgin site, of the virgin site covered with a 16 in (40 cm) sand layer, of the same site after installation of 26 ft (8 m) deep sand columns and after placement of an additional 12 in (30 cm) lift of sand,
c) CPT, PMT in and between sand columns and small-scale plate load tests.

Fig. 1.a illustrates the grid pattern of the 19 sand columns and fig. 1.b the execution of the test programme.

GEOTECHNICAL CONDITIONS

The geotechnical conditions on the site are characterized by an about 2.6 ft (0.8 m) thick fill on an at least 42.3 ft (13 m) deep deposit of grey sandy clay. The water table is about 6.5 ft (2 m) deep.

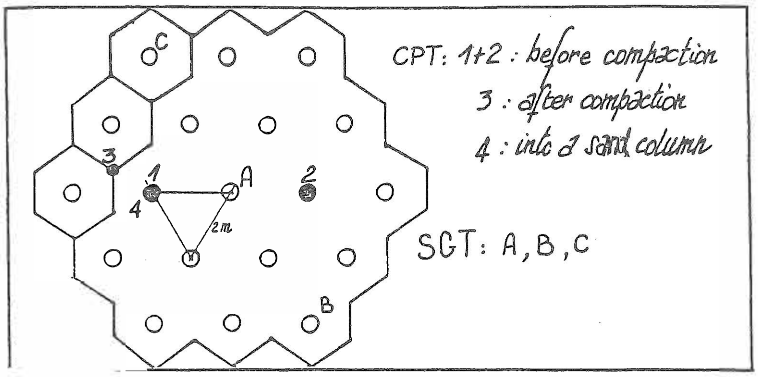

Fig. 1.a. Grid pattern of stone columns

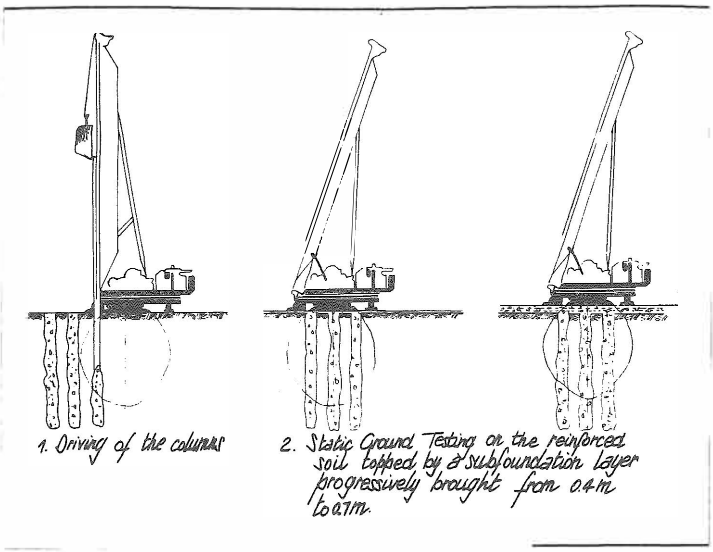

Fig. 1.b Field test programme

The geotechnical properties of this clay are well known in Belgium, belonging to the Rupelian formation (tertiary). The clay is stiff, overconsolidated and fissured with a water content close to the plastic limit. The cone resistance (electric cone) varies typically between 100 psi (1) and 400 psi (4 MPa) and the sleeve resistance yields an average friction ratio of 5.5 %.

Van Burm and Maertens (6) report the following typical properties :

Classification : CH PI = 52

2 μm fraction ≈ 49 % w_ℓ = 81 %, w_p = 29 %

The top layer (about 8 m) of this clay has the following strength and deformation properties :

Undrained shear strength :
Unconfined compression test : 75 kPa (10.9 psi)
Field vane test : 250 kPa (36.3 psi)
Elastic (Young's) Modulus :
E_{50}: Undrained conditions : 20 MPa (2003 psi)

In an overconsolidated clay, the installation of displacement-type sand columns will have little effect on the soil sitffness. Thus the soil improvement will be mainly caused by the reinforcing effect of the compacted stone columns. Previous investigations have shown that in soils with a friction ratio exceeding 3 %, no compaction effect can be achieved (2).

LARGE-SCALE PLATE LOAD TEST

The Franki walker-type rig moves by alternate movements of a large central plate and two adjacent smaller plates (beams), cf. Fig. 2. The weight of the rig (400 kN, 89 klb) is transferred by the 7,5 m^2 (78 ft^2) large central plate to the soil. Because of the comparatively large size of the plate, the depth of influence typically extends to about 6 m (19.38 ft). For the test programme, a conventional piling rig was instrumented to measure continuously the load-deformation characteristics of the soil. Displacement transducers were used to measure the relative displacement between the soil below the central plate and reference beams. Thus the relative displacements between the central plate and the adjacent beams is recorded. A pressure transducer measured the oil pressure applied to the jacks and thus a value proportional to the contact pressure at the central plate-soil interface, cf. Fig. 2. The test provides load-settlement curves for a stress range of 0 to 50 kPa (0-7.25 psi).

An example of such a curve is shown in Fig. 3. It should be noted that when the applied pressure reaches the maximum contact pressure, the reference beams are lifting off the ground, which results in large apparent deformations, cf. Fig. 3. These curves allow to determine the apparent secant modulus k_T for a contact stress range e.g. 20-40 kPa (2.9 - 5.8 psi), as illustrated in Fig. 4.

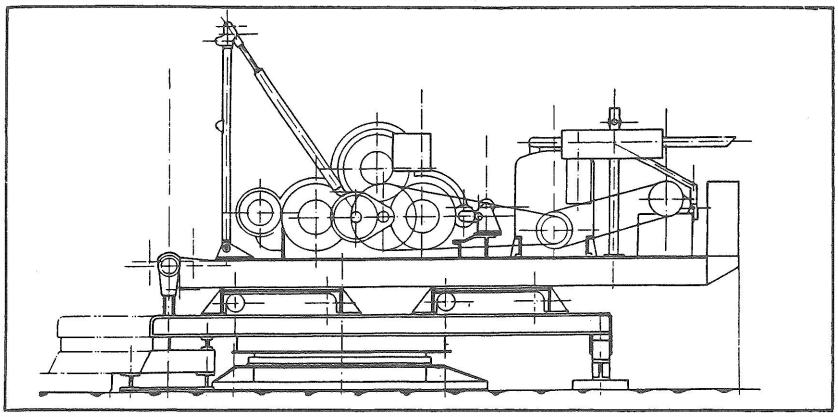

Fig. 2.a Sketch of walking mechanism of the piling rig.

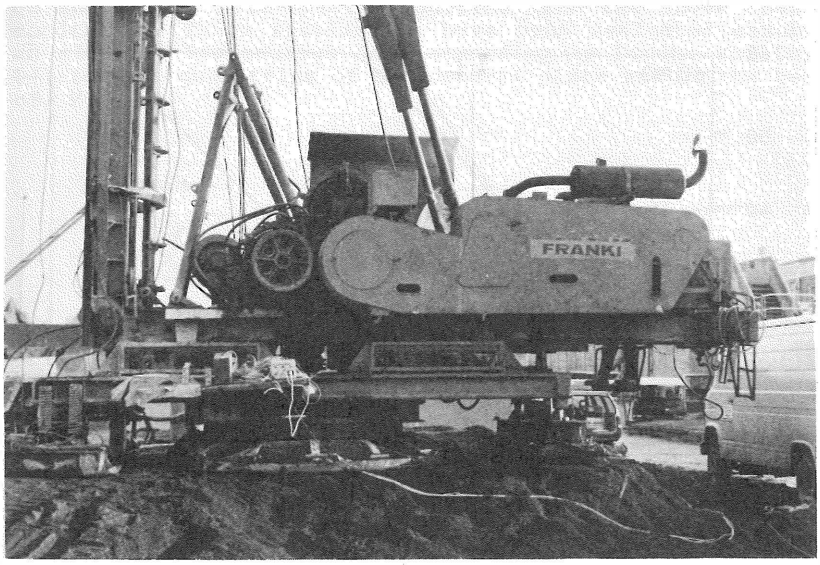

Fig. 2b Franki rig instrumentation with data acquisition system

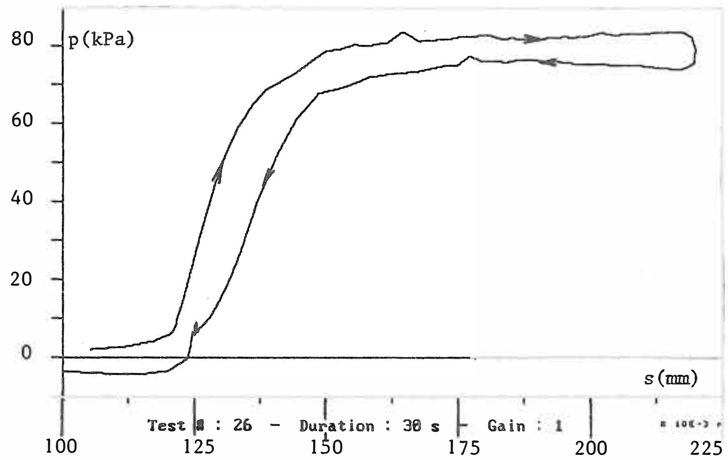

Fig. 3. Stress-displacement curve (1 kPa = 0.145 psi, 1 mm = 0.0394 in)

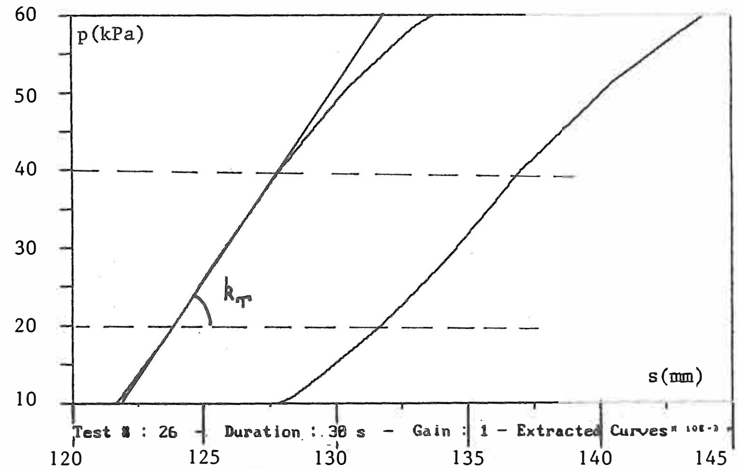

Fig. 4. Enlarged section of the load-deformation curve for detailed data interpretation (1 kPa = 0.145 psi, 1 mm = 0.0394 in)

where: $$k_T = \frac{\Delta p}{\Delta S} \qquad (1)$$

with Δp : increase of contact pressure
ΔS : increase of differential settlement between central plate and beams.

Also the flexibility of the rig chassis needs to be taken into account, either by structural analysis of the steel frame or, preferably by field calibration.

Test Interpretation

The absolute displacement of the central plate can either be measured (with respect to a reference point) or estimated using theory of elasticity. The relative movements between the central plate and the beams can then be determined by comparing the two loading conditions as illustrated in Fig. 5:

a) the weight of the rig is fully carried by the small beams without contact pressure below the central plate

b) the weight of the rig is fully carried by the central plate while the beams begin to lift off the ground

This analysis involves the calculation of settlements on an elastic half-space, resulting from the above loading conditions. These settlements have been evaluated assuming an elastic, homogeneous soil according to Poulos and Davis (4) for the geometries of the central plate and of the beams cf. Fig. 6.

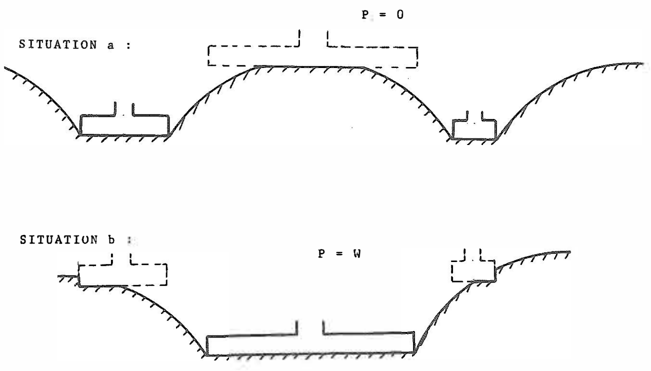

Fig. 5. Loading conditions below rig.

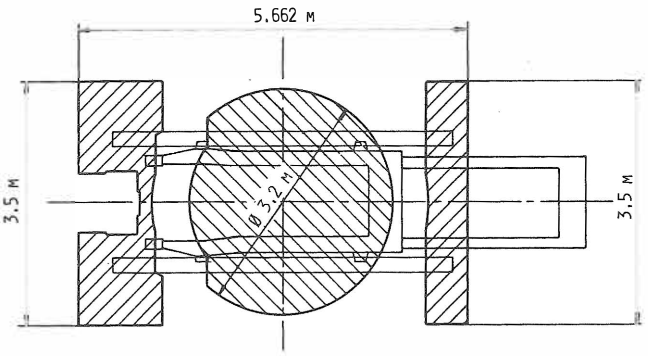

Fig. 6. Geometry of the central plate and outer beams.

The results are presented in a simple diagramme for deduction of the plane strain modulus E' :

$$E' = \frac{E_s}{1 - \nu_s^2} \tag{2}$$

with E_s : Young's modulus
ν_s : Poisson's ratio

Fig. 7 shows in full line the relation between E' and k_T assuming rigid plates. Alternatively, the coefficient of subgrade reaction k for the central plate can be determined from :

$$k + k_R = \beta \; k_T \tag{3}$$

with : β = 1.19 for the rigid plate of the Franki rig.
k_R = stiffness of the rig frame, as determined from field calibration.

For a circular rigid footing of radius R, the coefficient of subgrade reaction can be expressed by

$$k = \frac{2E'}{\pi R} \tag{4}$$

DETERMINATION OF COEFFICIENT OF SUBGRADE REACTION, K

Static load tests (SGT) were performed during the following stages of the test :

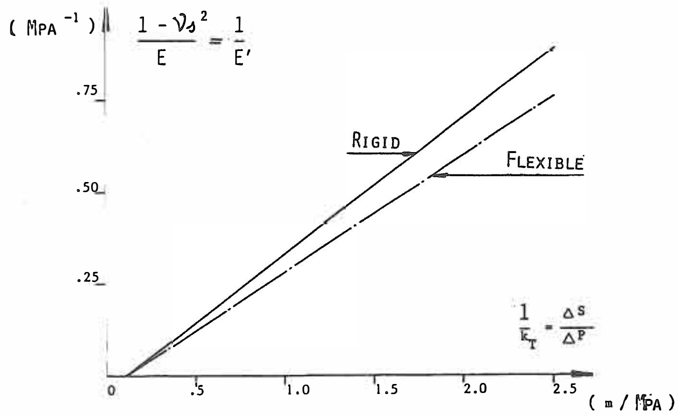

Fig. 7. Chart for direct interpretation of field tests. (1MPa = 145 psi, 1 m = 0.328 ft)

1 - on virgin soil
2 - on virgin soil provided with a 1.31 ft (0.4 m) thick fill.
3 - after installation of 26 ft (8 m) deep sand columns with a diameter of 1.62 ft (0.5 m) and a spacing of 6.46 ft (2 m).
4 - thickness of the surface layer of 2.30 ft (0.7 m).

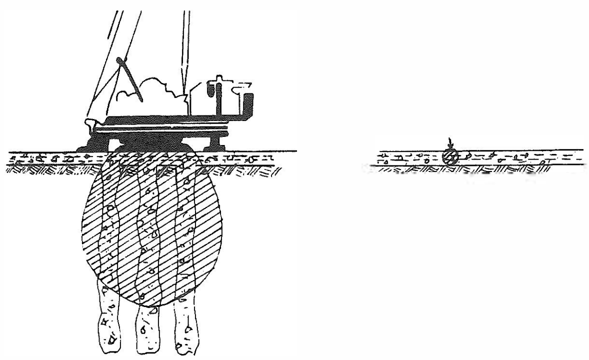

Fig. 8. Comparison of the bulbs of influence

Tables 1 and 2 present the k-values as measured during the sucessive stages of the test programme in three different locations, cf. Fig. 1a. The average values $\bar{k}$ are also given.

TABLE 1 : k-Value in Virgin Soil
($1\ N/cm^3 = 3.68\ lb/in^3$, 1 m = 3.28 ft)

Location	Virgin Soil k (N/cm^3)	Virgin Soil $\bar{k}$ (N/cm^3) (1)	after placement of 0.4 m fill k (N/cm^3)	after placement of 0.4 m fill $\bar{k}$ (N/cm^3)
A	5.71, 5.21	5.46	5.40, 5.38	5.25
B	5.31, 5.31 4.52, 4.32	4.87	5.13, 5.13 5.71	5.32
C	4.27, 5.00 5.67, 5.59	5.13	5.50, 5.60 5.60	

(1) average

TABLE 2 : k-Values after Installation of Stone Columns
($1\ N/cm^3 = 3.68\ lb/in^3$, 1 m = 3.28 ft)

	Fill 0.4 m		Fill 0.7 m	
Location	k	$\bar{k}$ (1)	k	$\bar{k}$ (1)
	N/cm^3	N/cm^3	N/cm^3	N/cm^3
A	5.38 5.26 8.64 5.38	6.03	7.57	7.57
B	7.66 6.72	7.19	8.21 6.29	7.25
C	5.50 6.48 6.48 6.26 7.73 7.89	6.72	-	-

(1) average

Comparison of test results

As previously mentioned, SGT allows one to monitor the increase in stiffnes of the overall soil mass achieved by the installation of the sand columns. Table 3 summarizes the average k-values for the different stages of the test programme.

These values are graphically represented in Fig. 9 as a function of the thickness of the fill and clearly show the influence of sand column installation as well as of the layer thickness.

TABLE 3 : Summary of SGT Measurements
($1\ N/cm^3 = 3.68\ lb/in^3$, 1 m = 3.28 ft)

	$\bar{k}$ values (N/cm^3)			
Location (1)	non-compacted soil (2)	non-compacted soil + 0.4 m (3)	stone columns 0.4 m fill (4)	stone columns 0.7 m fill (5)
A	5.46	5.32	6.38	7.92
B	4.87	5.55	7.54	7.60
C	5.13	5.25	-	-
Average value	5.15	5.37	7.07	7.76

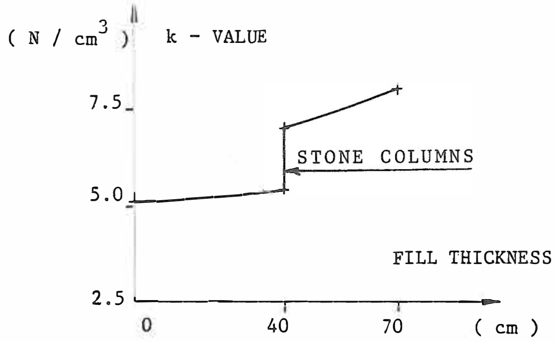

Fig. 9. Effect of reinforcement on measured k-value.
(1 cm = 0.394 in, $1\ N/cm^3 = 3.68b/in^3$)

CONE PENETRATION TESTS (CPT)

CPT tests were performed in the virgin soil as well as after installation in a stone column cf. Fig. 10. CPT 4, performed in the center of a stone column confirms that the length of the columns was about 26 ft (8 m). The CPT performed in the clay a few days after installation of the stone columns did not show any discernable change.

Although a drained long term settlement parameter cannot be directly correlated to an undrained strenght parameter such as q_c, empirical correlations have been developed for overconsolidated clays. However, as will be shown below, the scatter is large and this approach is thus not recommended for design purposes.

CPT tests were used to calculate the settlement for contact pressures of 1.45 psi (10 kPa) and 3.63 psi (25 kPa), respectively. The coefficient of subgrade reaction was then determined within this stress range.

The vertical stress σ_z along the axis below a circular rigid plate in a homogeneous, isotropic, elastic, half-space, can be expressed by :

$$\sigma_z = \frac{p_{av}}{2} \frac{1 + 3\zeta^2}{(1 + \zeta^2)^2} \tag{5}$$

where p_{av} is the average contact pressure and
ζ is $\frac{z}{a}$ with z the depth where the stress is evaluated and a the radius of the plate

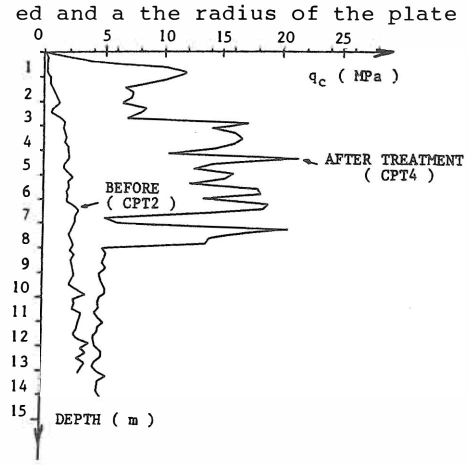

Fig. 10 CPT profiles before and in stone column, after treatment. (1 m = 3.28 and 1MPa = 6890 psi)

The settlements can then be estimated according to Terzaghi (5) :

$$\frac{\Delta h}{h} = \frac{1}{C} \ln \frac{p_o + \sigma_z}{p_o} \qquad (6)$$

where $\frac{\Delta h}{h}$ is the relative settlement

p_o is the geostatic stress at the depth considered

σ_z is the incremental stress due to the applied pressure

C is the compressibility constant which can be expressed by Cassan, (1)

$$C = \alpha \frac{q_c}{p_o} \qquad (7)$$

with q_c : cone resistance measured by CPT.

α : empirical coefficient depending on soil type.

The coefficient of subgrade reaction can then be calculated from the following ratio

$$k = \frac{p_1 - p_2}{\Delta h_1 - \Delta h_2} \qquad (8)$$

where p_1 and p_2 are the average stress of 1.45 psi (10 kPa) and 3.63 psi (25 kPa) respectively, acting on the central plate,

Δh_1 and Δh_2 are the respective settlements, resulting from the action of these two stresses

This k-value can be compared with the secant modulus determined from SGT measurements (cf. equations (1), (2), (3) and (4)).

Table 4 gives the k-values determined from CPT according to the above described procedure. As could be expected CPT3 did not show any increase in the cone resistance of the clay between stone columns. Table 4 also gives the k-values in the center of a sand column.

PRESSUREMETER TESTS (PMT)

Two pressuremeter tests were performed on the site in the clay before sand column installation as well as at the center of a sand column after treatment, respectivelly. The PMT results are summarized in Table 5.

TABLE 4 : Estimate of k-Values from CPT Tests
1 kPa = 0.145 psi, 1 N/cm^3 = 3.68 lb/in^3,
1 cm = 0.393 in)

			p_1 (kPa)	p_2 (kPa)	Δh_1 (cm)	Δh_2 (cm)	k (N/cm^3)
before	CPT 1	α = 3	10	25	0.428	1.170	2.03
		α = 4	10	25	0.321	0.878	2.71
installation	CPT 2	α = 3	10	25	0.920	2.100	1.27
		α = 4	10	25	0.690	1.580	1.69
after installation	(1) CPT 4	α = 1.5	10	25	0.129	0.297	8.93

(1) Test in stone column.

TABLE 5 : PMT in Virgin Soil and in a Column

	Virgin Soil			In Sand Column		
z (m)	E (kPa)	p_ℓ (kPa)	E/p_ℓ	E (kPa)	p_ℓ (kPa)	E/p_ℓ
1.0	3100	270	11.5	8500	960	8.85
2.0	5700	680	8.4	5700	710	8.03
3.0	5500	540	10.2	19100	1100	17.36
4.0	8000	800	10	9600	1100	8.72
5.0	15400	860	17.9	8700	1100	7.90
6.0	18000	970	18.6	8600	1100	7.92
7.0	14600	1100	13.3	14800	1100	13.45
8.0	13700	1100	12.5	14100	950	14.80

Following the recommandations suggested by Ménard (3) the settlement of the circular central plate can be calculated under a vertical stress, which yields the following estimates of the coefficient of subgrade reaction:

non compacted soil : k = 5.71 N/cm^3 (21.0 lb/in^3)
compacted stone column : k = 23.31 N/cm^3 (87.8 lb/in^3)

PLATE LOAD TEST (PLT)

A conventional plate load test (PLT) was also performed on the surface of the sand layer. The area of the plate was

750 cm^2 (11508 sq in). Since the depth affected by the test is on the order of 70 cm (27.5 in), only the sand layer has been actually tested.

Table 6 gives the plate settlements s for different stress levels p and the k-values determined at each stress level.

TABLE 6. Results of Conventional Plate Load Test ($1\ N/cm^3 = 3.68\ lb/in^3$, 1 mm = 0.039 in)

p (kPa)	s (mm)	k_{750} N/cm^3
50	0.99	50.5
100	2.86	35.0
150	4.69	32.0
200	6.54	30.6
250	8.33	30.1
300	10.00	30.0
350	11.86	29.5
400	13.53	29.6
450	15.05	29.9

COMPARISON OF TEST RESULTS

The average k-values obtained from SGT, CPT and PMT methods for the non compacted soil are :

$\bar{k} = 5.15\ N/cm^3$ ($18.95\ lb/in^3$) (SGT)

$\bar{k} = 2.37\ N/cm^3$ ($8.72\ lb/in^3$) (CPT)

$\bar{k} = 5.71\ N/cm^3$ ($21.01\ lb/in^3$) (PMT)

The obtained values are thus within the same range and globally of the order of $k = 5\ N/cm^3$ ($18.4\ lb/in^3$). This value corresponds to an E_s value of about 14 MPa (96.56 kpsi), which is in good agreement with the values (5-20 MPa 34.5 - 137.8 kpsi) in the literature for the Boom clay (6).

For the compacted soil, the results of the conventional plate load test can be used to assess the k-value of the larger plate load test, using Terzaghi's (5) recommended scaling law for sands :

$$k = k_1 \frac{(B+1)}{2B} \quad (9)$$

with k_1 : coefficient of subgrade reaction, determined for a 1 ft diameter plate
B : actual plate diameter in feet

CPT, PMT and PLT performed in the sand column give the following k-values for the large plate (k_{75000}) and for the 750 cm^2 (115.8 sq in) plate (k_{750}), respectively.

TABLE 7 : k-Values for Compaction Stone Column
(1 N/cm^3 = 3.68 lb/in^3)

	k_{75000} (N/cm^3)	k_{750} (N/cm^3)
CPT	13.9	-
PMT	23.3	10.1
PLT	≃ 10	30

In spite of the variation of individual values, it can be concluded that a k-value of 20 N/cm^3 (73.6 lb/in^3) is representative for the 7.5 m^2 (78 ft^3) large central plate of the Franki rig, resting on the compacted sand.

CONCLUSIONS

By instrumenting a Franki piling rig it is possible to determine in-situ the coefficient of subgrade reaction below a 7.5 m^2 (78 ft^2) plate. The modulus values estimated from CPT and PMT tests agreed well with those of the large scale plate load test in the stiff to very stiff sandy clay. The k-value for the large plate was on the order of 5 N/cm^3 (18.4 lb/in^3), which is also in good agreement with the extensive data published.

SGT tests performed after installation of the displacement-type sand columns indicate that the k-value can be increased to 8 N/cm^3 (29.4 lb/in^3), which corresponds to an increase of 60 %.

The test performed in the stone columns and in the fill material indicate that the k-value of the 7.5 m^2 plate resting on a uniformly compacted fill is on the order of 20 N/cm^3 (73.6 lb/in^3).

ACKNOWLEDGEMENTS

The concept of large-scale plate load tests using a

Franki piling rig was initiated by Prof. B. Broms who also made valuable suggestions during the implementation of the project. His assistance is gratefully acknowledged.

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