Performance of a large plate on a group of stone columns

La performance d'une grande plaque sur un groupe de colonnes de pierre

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ABSTRACT: Large size rigid reinforced concrete plate-load test data is evaluated to determine the performance of stone columns proposed for the foundation of a power plant. Soil profile consists of a top soil of sands, silts and transition soils up to a depth of 5.5 m - 6.5 m, and below coral limestone exists. Soil investigations indicated that the soil in the upper horizon has low shear strength and relatively high compressibility. Also, a large size zone load test, at an elevation approximately 1 m above the proposed foundation level was conducted using a 5 m^2 plate and it was observed that shear failure and excessive settlements occurred before reaching the design load. Hence, ground improvement technique by stone columns was selected to upgrade the foundation soil. The stiffnesses of improved and unimproved soils were compared. Settlement reduction and stress concentration ratios were also evaluated as a function of applied normal load.

RESUME: Les résultats des essais du chargement sur une plaque de béton-armé de grande dimension, sont évalués pour calculer la performance des colonnes de pierre proposées pour la fondation d'une centrale d'énergie. La section du sol se compose du sable pour la partie supérieure, du limon et du matériel de transition jusqu'à une profondeur de 5.5 à 6.5 m et du coral calcaire au fond. Les recherches du sol ont montré que le sol a une petite resistance de découpage avec relativement grande compressibilité dans les couches supérieures. Aussi, les essais du chargement zonal de grande dimension, à une distance approximativement 1 m du niveau proposé de la fondation, sur une plaque de 5 m^2 de surface ont montré qu'il y a des défaillances de découpage et des échouements excessifs avant d'atteindre le poids projeté. La rigidité des sols améliorés et non améliores ont été comparées. Le pourcentage de la réduction d'echouement et du concentration de la pression sont aussi évalués en fonction de la charge perpendiculaire appliquée.

1 INTRODUCTION

The structure hall planned is composed of 5 turbine units in a row and a turbine hall covering all. Turbine units will be on raft foundations of 21 m x 9 m that will carry a maximum load of 250 kPa. The turbine hall will be covering 45 m x 24 m and its foundation will be of strip footings loaded up to 250 kPa. The foundation level is approximately 2 m below the ground surface. SPT, CPT and a large zone load test with laboratory works showed that the soil in the upper horizon has low shear strength and relatively high compressibility. Therefore, a ground improvement work became a necessity. Among three, the others being deep excavation and soil substitution, ground improvement technique by stone columns was selected to upgrade the foundation soil.

2 SOIL PROFILE

The top soil reaching a depth of 5.5 m - 6.5 m from the ground surface can mainly be divided into three layers: 1) The covering sands down to 0.6 m - 2.8 m, 2) partly clayey silts down to 4.0 m - 5.75 m and 3) transition soils below the silts. Throughout all these layers, small coral pieces or pieces of shells, snails, etc., were found embedded in varying quantities. Below these, coral limestone exists. The ground water level ranges between 1.25 m and 2.5 m (Figure 1).

Sands have a gradation from coarse to fine and contain partly silts. Silts mostly contain fine sands and partly clay admixtures are also observed. Especially these layers are of soft consistency. SPT and CPT showed low penetration resistances. SPT values varied from 8 to 15 and CPT point resistances changed between 1 - 2 MPa with an average friction ratio of 4 % (Figure 2). The values in Figure 2 were plotted by superposing all the test data and borders were marked to show minimum and maximum values. However, although there seems to be a great discrepancy for friction ratio values, it was observed that for the upper horizons, they were around 4 % in great majority. Transition soils consist of silt with a very high content of coral pieces. These pieces are of different sizes, e.g., small pieces as well as coral fragments of several centimeters. Coral limestone was observed down to the last exploration depth of 26.5 m. Due to the natural growth, water movements and destruction by organisms, the coral limestone strata are very inhomogeneous, and it was possible to penetrate by CPT cone.

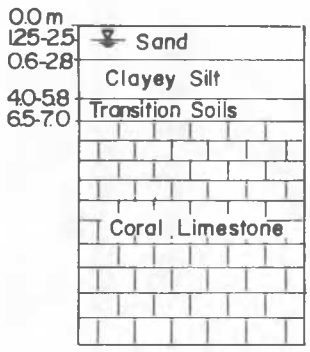

Figure 1. Soil Profile

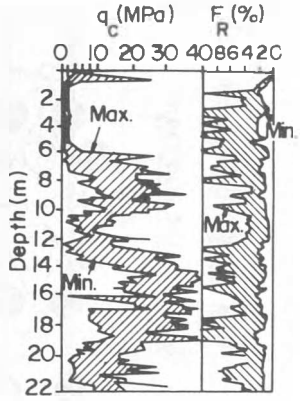

Figure 2. Variation of CPT data

3 FIELD AND LABORATORY WORKS

Field work can be divided into two lines of work that were carried out concurrently and independently from each other: Static penetration soundings and exploratory drilling. Totally 35 CPT were performed up to 29 m depth. 6 boreholes were drilled to depths ranging from 21 m to 26 m. Generally, all boreholes were cased down to a depth of approximately 17 m. Representative disturbed and undisturbed samples were taken from the upper overburden layer and 8 direct shear, 7 Atterberg limits, 7 moisture content and 7 sieve/sedimentation analyses were performed (Table 1). Moreover, samples taken from coral limestone were subjected to uniaxial compression tests.

Table 1. Laboratory test results and SPT values.

Borehole	Depth (m)	SPT (no.)	c (kPa)	φ (°)	w_P (%)	w_L (%)	w (%)
No.1	3.30-3.60		8	14	25	40	27
	3.60-4.05	14					
	4.80-5.25	13					
	6.30-6.75	17					
No.12	2.00-2.30		25	11	NP	-	26
	2.30-2.75	15					
	3.50-3.80		20	11.5	29	45	34
	5.30-5.75	26					
No.14	2.10-2.30		7	12	27	41	30
	2.30-2.75	5					
	3.00-3.50		6	16	28	40	30
	3.50-3.95	13					
	4.50-4.75		4	12	27	39	27
No.18	2.30-2.75	9					
	3.50-3.70		30	6	30	47	31
	3.80-4.25	23					
No.29	2.30-2.75	8					
	3.30-3.75	10					
	4.55-5.00	17					
No.31	3.40-3.85	7					
	4.20-4.65	9					

4 LOADING TESTS

A large size zone load test at an elevation approximately 1 m above the proposed foundation level was conducted using a 5 m^2 rigid R/C plate. Shear failure was observed before reaching 300 kPa and excessive settlements occurred under low loads. Hence, it was decided that the proposed structure could not be constructed on this ground.

A second load test on the same size plate covering 4 stone columns was conducted up to a total load of 3000 kN. 1 m diameter columns were arranged in a triangular grid of 1.2 m spacing all founded on coral limestone which exists at 6 m to 9 m depth.

Load test set-up is shown in Figure 3, and the load test results are given in Figure 4.

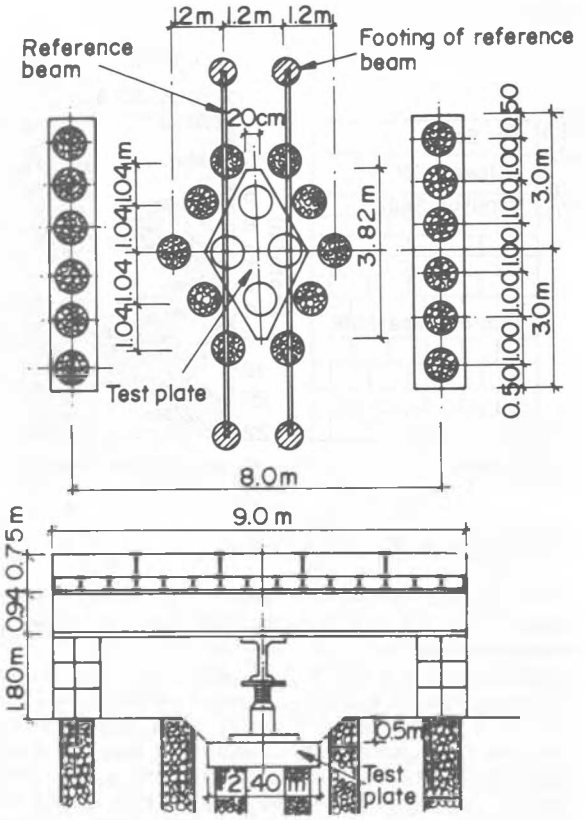

Figure 3. Test set-up.

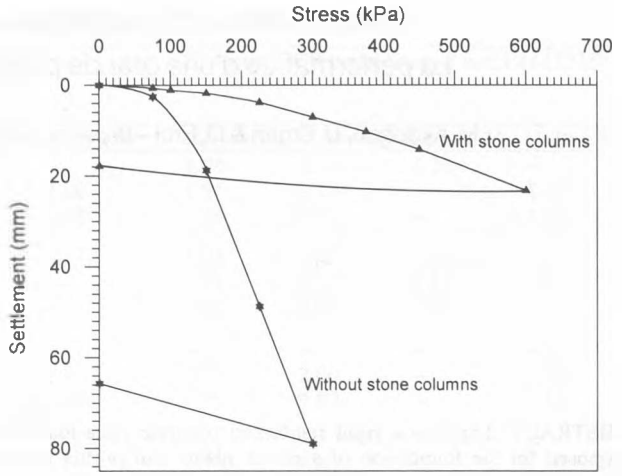

Figure 4. Load test results.

5 EVALUATION OF TEST RESULTS

To determine the extent of improvement by stone columns, the values of modulus of elasticity (E) were found before and after the ground improvement. This has been enabled by back-calculation of modulus of elasticity values from elastic settlement formula with the use of load test results.

The influence factor in elastic settlement formula shows some differences depending on shape of loaded area and influenced depth. Because of this, the calculations were not depended on only one source, however, different influence factors were also used.

Figure 5 shows the variation of modulus of elasticity with the applied stress for different influence factors which appear in different references. The moduli in the applied stress range greater than 150 kPa may be disregarded due to bearing capacity failure in the test without stone columns. Poisson's ratio was taken as an average value of $\upsilon = 0.3$. In addition, elastic moduli were also calculated for $\upsilon = 0.25$ and $\upsilon = 0.35$, and it was seen that the results are not too much different than the values for $\upsilon = 0.3$. Figure 6 shows the variation of modulus of elasticity with Poisson's ratio (Craig, 1987).

In the calculation stage, 5 m^2 rigid R/C plate was assumed as a rectangle of 3.0 m x 1.67 m.

An important point to note is the type of compressions and the moduli under the plate. The time-settlement data reveal that the immediate settlements are roughly 50 % of the total settlements, in both improved and unimproved cases and the consolidation settlements are nearly completed at the end of 3 - 5 hours of waiting times during the loading stages. Therefore, the measured moduli should be regarded as drained moduli.

The equilibrium method, proposed by Murayama and Ichimoto (1982), was used in finding the stress concentration ratio due presence of stone columns.

Calculated moduli of elasticity may be compared in the cases with and without improvement in the applied stress level of 75 kPa. The values are roughly 180 MPa and 45 MPa respectively resulting in a modulus improvement factor of 4.

Variation of stress concentration ratio with stress is given in Figure 7 after using improved ground settlement values in Murayama and Ichimoto's (1982) equilibrium method. It is seen that it increases with increasing applied stress, and the values are 6 and 16 for the stress levels of 75 kPa and 150 kPa respectively. However, it is presented in literature that this ratio varies generally between 2.0 - 6.0 (Mitchell 1981), values as high as 9.0 were also obtained. Highly rigid test plates are believed to be partly responsible for the higher values obtained (Bergado et al. 1994).

Ratio of the settlements in the improved and unimproved soils (settlement reduction ratio) varies with the stress level (Figure 8). A value of 0.26 is measured at the level of 75 kPa and 0.10 at

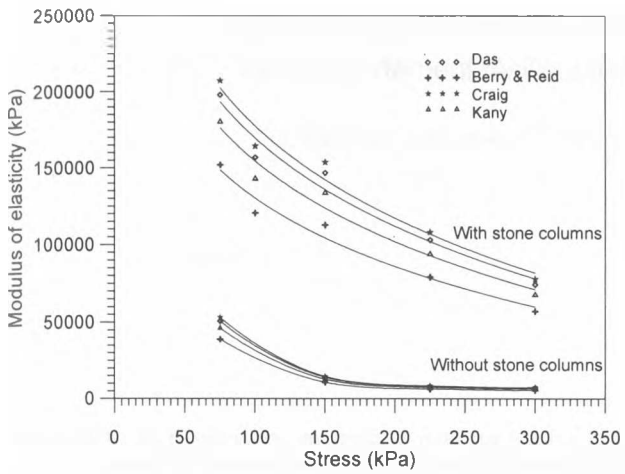

Figure 5. Variation of modulus of elasticity with stress.

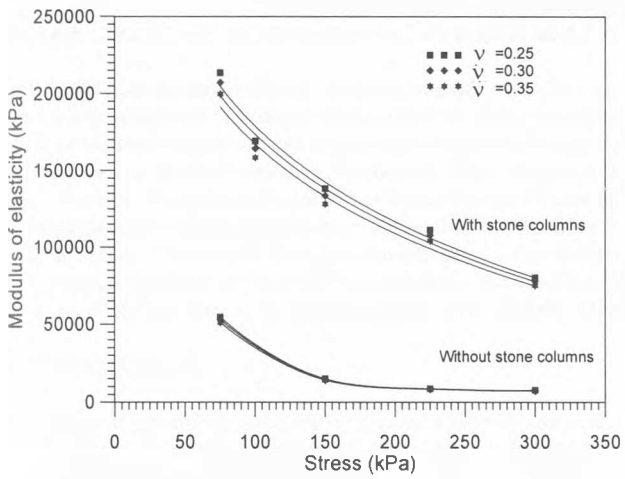

Figure 6. Variation of modulus of elasticity with Poisson's ratio.

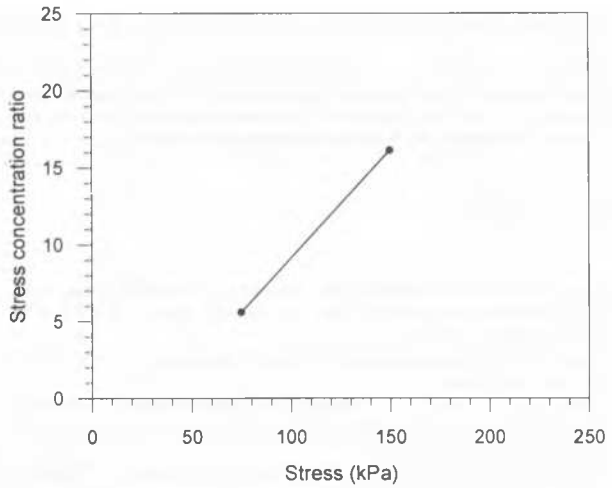

Figure 7. Variation of stress concentration factor with stress.

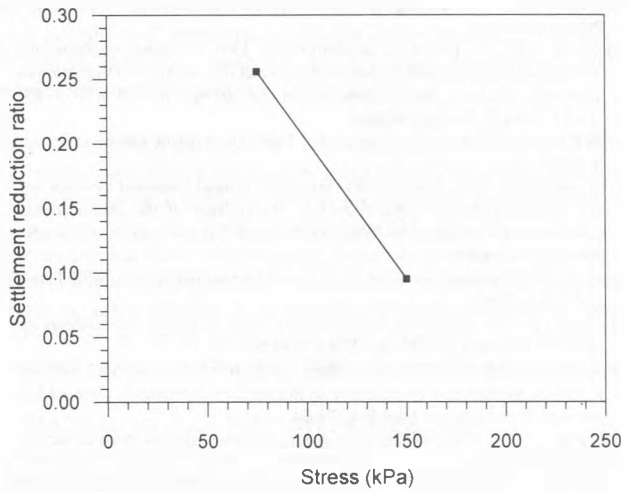

Figure 8. Variation of settlement reduction ratio with stress.

150 kPa. Comparisons at higher levels should not be made due to yielding of the unimproved soil. The area replacement ratio for the configuration in Figure 3 is 63 %. The angle of shearing resistance and the ratio of modulus of elasticity of column material to that of the soil have been taken as $\varphi = 42.5° - 45°$ and $E_p / E_s = 10$ respectively.

The references that can be found in the literature for finding the settlement reduction ratio values are given in Table 2 and the assumptions based on are as follows :

Van Impe - De Beer (1983 and 1989) started from the consideration that columns are at the limit of equilibrium and deform at constant volume. They accepted a rigid-plastic rheologic behavior for the stress-strain behavior of the column and considered the problem two-dimensional (a gravel wall). Priebe (1993) used incompressible column material within the elastic half space with no change in lateral stress with depth and no peripheral shears. In Balaam and Poulos (1983), analytic expressions from elasticity for estimating the settlement of rigid foundations supported by clay reinforced by stone columns have been developed from the analysis of the cylindrical units. Weighed modulus method mentioned in Greenwood-Kirsch (1983) is based on the simple ratios of modulus of elasticity values and plan areas of ground and column.

Bearing capacity of the plate on the improved and unimproved ground was also computed and compared using the general bearing capacity equation. The shear strength parameters for the unimproved ground were selected on the basis of the laboratory test results given in Table 1. Equivalent cohesion and friction angle for the improved ground were determined following the two propositions by Enoki (1991) and Priebe (1993), and the parameters and the bearing capacities are summarized in Table 3.

Table 2. Settlement reduction ratios in the literature.

Source	Settlement reduction ratio
Van Impe - De Beer	0.12
Priebe	0.11
Balaam - Poulos	0.18
Balaam (1978)	0.18
Greenwood - Kirsch	0.15
For the case presented	0.10 - 0.26

Table 3. Strength parameters and bearing capacities for improved and unimproved ground

Type of ground	Cohesion (kN/m^2)	Friction angle (°)	q_{ult} (kN/m^2)	Reference
Unimproved	7.0 (16)	12.9 (12.9)	194 (320)	-
Improved	1.4 (3.1)	34.5 (34.5)	1357 (1491)	Enoki (1991)
Improved	2.6 (6)	33.5 (33.5)	1274 (1505)	Priebe (1993)

The values appear in parentheses correspond to the second set of unimproved soil parameters. The measured bearing capacity values in Figure 4 are comparable to the calculated figures in Table 3.

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