# Mechanical Properties of the Gravel of Santiago

## Propriétés Mécaniques du Sol Gravier de Santiago

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SYNOPSIS The construction of important civil works in the urban area of Santiago (Chile) have forced soil engineers to perform several test and field measurements for the purpose of widen their knowledge on the behavior of gravelly soils. Results obtained from field tests on undisturbed gravel samples, such as, triaxial tests and vertical-horizontal plate bearing tests are presented. Also, field observations of existing slopes, horizontal displacements and settlement measurements of structures are included. The dynamic behavior of the gravel was studied by means of refraction survey and cyclic plate bearing tests. Values of the cohesion, angle of internal friction, elastic modulus and creep function have been derivated from the analysis of test results. Functions relating the shear modulus vs cyclic shear strain and cumulative strain vs number of stress cycles are proposed for the gravel.

## INTRODUCTION

The construction of important civil works such as deep excavations near existing structures, foundation systems for tall buildings and for the Santiago subway located on fluvial gravel have forced soil engineers to measure the mechanical properties of that soil. Field tests have been performed upon undisturbed samples due to the large particle size of the soil (about 0.25 m diameter).Triaxial and plate bearing tests were the most important measurements performed in order to get the mechanical soil properties. Plate bearing tests were run including the application of cyclic loads. Besides, refraction survey to obtain shear and crompressional wave velocities at different depth were conducted.

Santiago gravel is a coarse granular material from fluvial origin, without cementation. Particles are subrounded, tough and exhibit good gradation with maximun sizes of about 0.25m. Content of fines from an integral sieve analysis varies between 2 to 4%. Shear strength and rigidity increase with depth. Accordingly,it is possible to distinguish two depositions with an horizontal contact surface between them, typically at 4 to 7 m depth. First deposition underlies this contact surface and contains plastic fines which improve the mechanical properties of the soil. Above the contact surface gravel exhibits the same grain size distribution and compactness, but fines have a low plasticity which decreases strength and rigidity of the soil. This upper strata is called second deposition. Overlying the second deposition a superficial layer of a medium to stiff silty - clay, 1.5 to 3 m thick, covers the main area of the city. The water table is deeper than 20 m and bedrock normally is over 80 m.

Index properties of the Santiago gravel are presented in Table I.

TABLE I

Index Properties of the Santiago Gravel

| Index Properties | Integral Sieve Analysis Size N° (ASTM) | % Finer |
|---|---|---|
| $\gamma_d$ = 21.1 kN/m3 | 10" | 100 |
| | 4" | 86 |
| w = 4 to 6% | 3" | 75 |
| | 2" | 62 |
| | 1" | 40 |
| | 1/2" | 29 |
| $G_s$ = 2.7 | # 4 | 20 |
| $w_L$ = 20 to 35% | 16 | 14 |
| | 50 | 7 |
| $I_p$ = 5 to 20 | 200 | 3 |

## SHEAR STRENGTH

Results from in-situ triaxial tests performed on undisturbed samples, 0.80 m diam x 1.60 m height, trimmed by hand in the first deposition are shown in Fig. 1. These tests were run using effective confining stresses similar to those expected near excavations for underpinning and near subway tunnels. Details of these tests are presented by Kort et al (1979). Results show high values for both cohesion, c', and angle of internal friction, $\phi'$, mainly due to a high interlocking. For an axial strain equal to 0.7% values of $c'_{peak}$=36.3 kPa and $\phi'$=45° are obtained.

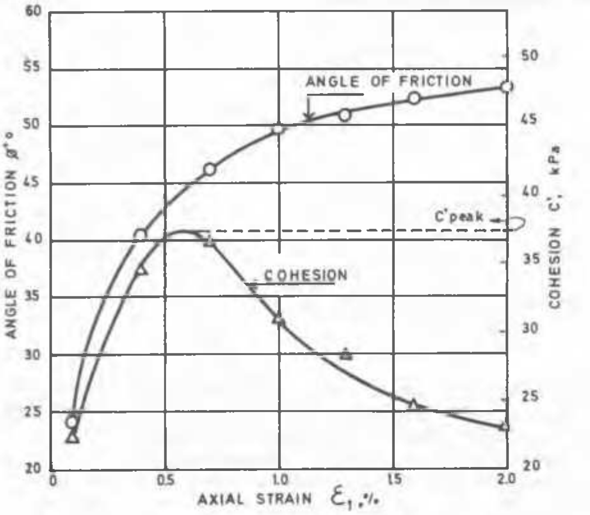


Fig. 1 Cohesion and Angle of Friction from Triaxial Tests vs $\varepsilon_1$ - First Deposition

The strength parameters are greatly dependent on the strain level in the soil, which has to be taken into account according to the analyzed problem. For example in the stability analysis of steep slopes, cohesion plays an important role. In those cases it is wise to keep the movilized cohesion, in any point inside the soil mass, below $c'_{peak}$; otherwise brittle failure may occurs. Back analysis of slope failures, both for static and seismic conditions, gave values of $c'_{peak}$ - $\phi'$ fairly close to those presented in Fig. 1.

The peak cohesion for the gravel of the second deposition has been estimated on about 15 kPa by using back analysis of slope failures that involved only soil of that deposition. An angle of friction equal to 45° was assumed for the soil becouse grain size distribution and compactness are similar to those measured for the first deposition.

## STRESS-STRAIN BEHAVIOR FOR STATIC LOADS

The gravel stress-strain behavior at different depths was studied by using plate bearing tests, triaxial tests on undisturbed samples and building settlement measurements. The variation of the elastic modulus with depth for static loads, $E_{EST}$, is shown in Fig. 2. Values plotted on this figure exhibits a definite trend with a moderate dispersion in spite of being obtained from different type of tests. Almost linear behavior was observed for the pressure range reported in Fig. 2, but some tests exceeded the limit pressure for the linear behavior (for plates 0.80 m diam. this limit pressure was around 2000 kPa).

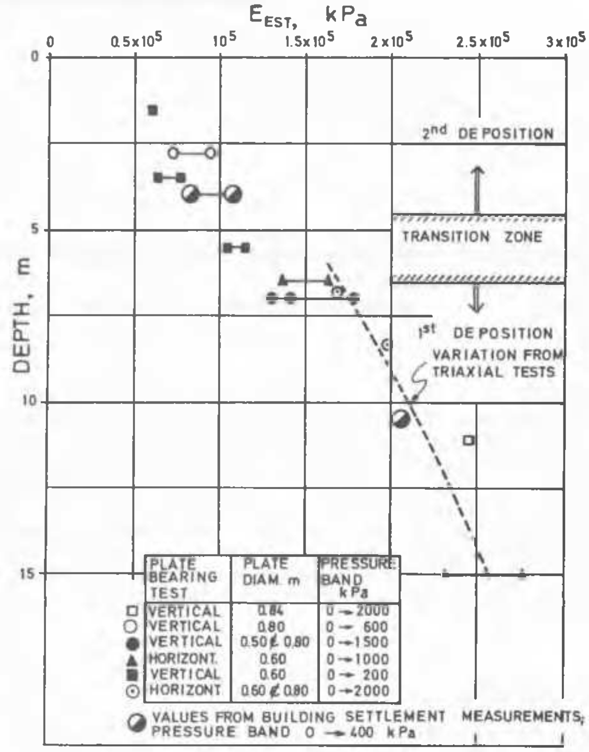


Fig. 2 Elastic Modulus for Static Loads

Lateral displacements measured on the walls of the subway tunnels were analyzed by means of a soil-structure interaction model. Elastic modulus selected from Fig. 2 were used in the model for prediting those desplacements, which compared very well with the measured ones. (Ortigosa el al, 1973).

That because of Santiago gravel is a very good bearing-capacity soil, during the last decade allowable contact pressures for foundation design have been continuously increased up to 1180 kPa. In spite of this high pressures, shallow foundations still have large safety factors against shear failure and reasonable elastic settlements. However, plate tests have shown elastic displacements ending more or less 2 hours after the application of the load increments. Beyond this time development of creep settlements make necessary to investigate this phenomenon before going to higher allowable pressure values. Creep results from plate tests performed on the first and second gravel depositions are presented in Fig. 3. From this figure it is possible to define a creep coefficient, $m_\alpha$, which can be expressed as:

$$m_\alpha = 0.0027\ \sigma_E / p_a \qquad (1)$$

where $\sigma_E$ is the static contact pressure at the foundation level and $p_a$ is the atmospheric pressure.

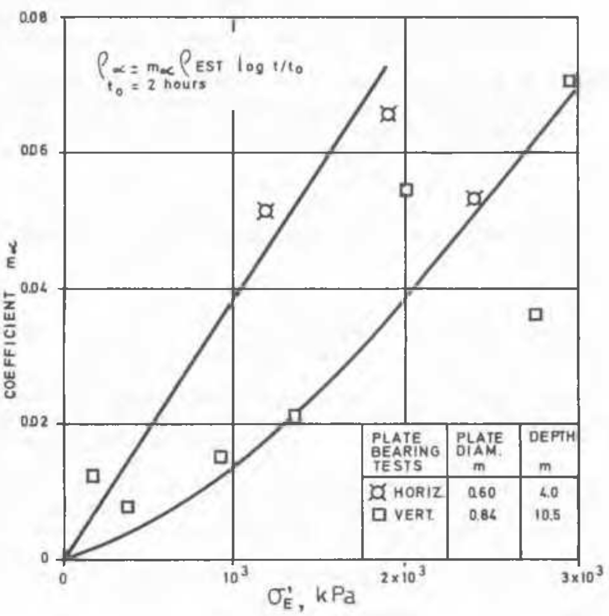


Fig. 3 Creep Displacement Coefficient $m_\alpha$ from Plate Bearing Tests

By using $m_\alpha$ values from eq. (1) with $\sigma_E/p_a = 15$ and assuming an structure lifetime equal to 30 years, a creep settlement 20% of the elastic settlement, $\rho_{EST}$, is obtained. Elastic settlements are computed using equations from the theory of elasticity and values of elastic moduli from Fig. 2. This percentage of creep deformation can be considered below the allowable limit for reinforced concrete structures if the stress relaxation with time is considered in the concrete material.

## STRESS-STRAIN BEHAVIOR FOR CYCLIC LOADS

Cumulative settlements induced by cyclic loads, $\rho_c$, are expressed in terms of $\rho_{EST}$ and the ratio $\sigma_c/\sigma_E$ ($\pm\,\sigma_c$ represents the stress variation at the foundation level due to cyclic loads acting on the structure). The equation relating the forementioned parameters uses the so called cyclic displacement coefficient, $m_c$, which was obtained for the Santiago gravel by means of plate bearing tests. This coefficient is a function of the number of load cycles, N , applied to the soil:

$$\rho_c = m_c \, \rho_{EST} \, \sigma_c / \sigma_E \qquad (2)$$

$$m_c = 0.29 + 0.42 \log N \qquad (3)$$

Eq. (2) was used by Ortigosa (1980) in order to analyze the effect of differential settlements upon reinforced concrete frames subjected to earthquake loads. In this analysis $m_c$ was increased by 50% to take into account cyclic shear stresses induced at the foundation level which are not present in the plate bearing tests. Results of this analysis show that the effect of the cumulative settlements can be of some importance for the structure safety when the seismic event exhibit high peak accelera - tions. This detrimental effect can be almost completely neglected by including foundation beams which conect column footings.

The data used to define the gravel shear modulus for cyclic loads were obtained by means of refraction survey. For this purpose, it was assumed that typical refraction survey induces in the soil shear strains not greater than $\gamma_c \approx \pm\,10^{-4}\%$. For larger strains, results from cyclic-load plate tests were used by applying a cyclic deviator stress $\pm\,\sigma_c$ around a static stress $\sigma_E$. Values of $\pm\,\gamma_c$ were estimated as a weighted average of the shear strains induced within the pressure bulb of the plate.

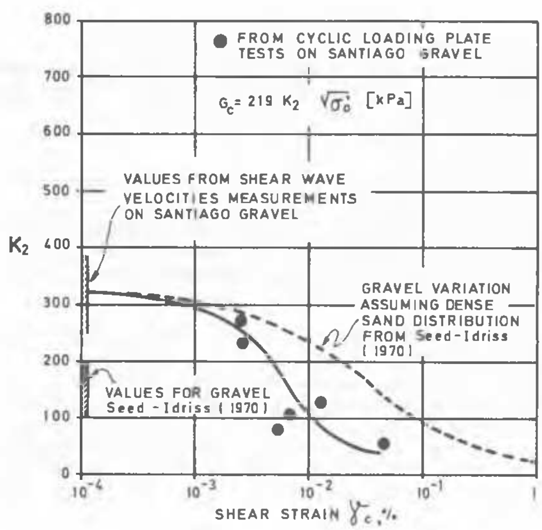


Fig. 4 In-Situ Shear Moduli for Cyclic Loading-First Deposition

In Fig. 4 the classical coefficient $K_2$ defined by Seed and Idriss (1970) has been plotted as a function of $\gamma_c$. The coefficient $K_2$ relates the cyclic shear modulus, $G_c$, with the equivalent isotropic stress, $\sigma'_o$, acting at a certain depth in the gravel deposit. This relation can be written as:

$$G_c = 219 \, K_2 \sqrt{\sigma'_o} \qquad (4)$$

where $\sigma'_o$ and $G_c$ are expressed in kPa.

Results obtained for the Santiago gravel are compared with those reported by Seed and Idriss, for coarse granular soils, as shown in Fig. 4.

This comparison is established only for small strains ($\gamma_c = \pm 10^{-4}\%$) because no data have been reported in literature for larger strains in coarse gravelly soils. To overcome this problem, common criteria to extend results from seismic refraction measurements to larger strains, uses the attenuation curves obtained for dense sands through laboratory tests (Seed and Idriss,1970). This criteria overestimates $G_c$ values when applied to the Santiago gravel as it is shown on Fig. 4. However, more reaserch and field measurements must be done to confirm this conclusion for others coarse gravelly soils.

## CONCLUSIONS

Field tests and measurements performed on the Santiago gravel during the last decade have made possible to obtain reliable soil properties to be used in complex problems, such as soil structure interaction, foundation design for tall buildings, slope stability for deep excavations and underpinning of existing structures.

Also, available data have permitted a rational increase of allowable bearing pressures for foundation design, together with feasible predictions for slope stability and earth pressure problems.

The use of plate bearing tests with diameter over 0.60 m has proved to be suitable for getting soil properties in coarse gravelly soils under static loads. Cyclic tests were also useful for getting dynamic properties of this type of soil.

Triaxial tests show reliable results when using undisturbed gravel samples. The experience gained from this tests has shown that difficulties presented during the tests can be overcome more easily than most people commonly think. Tests can be rather easily performed as long as soil exhibits some amount of cohesion and confining pressures are kept below 80 kPa.

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