Small scale pressuremeter for shallow foundation model tests

Pressiomètre à petite échelle pour essais sur modèle de fondation superficielle

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ABSTRACT

This paper presents a modified version of the small-scale pressuremeter prototype developed in the nineties. This new version has been used in a 1g scale model where shallow foundations were tested under centered and eccentric cyclic loading. Massifs of reconstituted Hostun sand and number 6 sand were used in France and Japan respectively with a density index of 0.8. The results obtained are compared with those deduced from reduced dynamic penetrometer and SPT models. The various possible applications are described and discussed in the light of the bibliography. The transposition, in accordance with the rules of similarity, of the analytical design methods is studied and criticized.

RÉSUMÉ

Ce papier présente une version modifiée du prototype de pressiomètre échelle réduite développé dans les années quatre-vingt-dix. La nouvelle version a été utilisé dans un modèle réduit 1g où sont testées des fondations superficielles sous charge cycliques centrées et excentrées. Des massifs de sable d'Hostun reconstitué à un indice de densité de 0.8 et de sable numéro 5 ont été utilisés respectivement en France et au Japon. Les résultats obtenus sont comparés à des résultats déduits de modèles réduits de pénétromètre dynamique et de SPT. Les différentes applications possibles sont décrites et discutées au regard de la bibliographie. La transposition, conforme aux règles de similitude, des méthodes analytiques de dimensionnement est étudiée et critiquée.

Keywords: shallow foundations, 1g testing, small-scale, laboratory, sand

1. Introduction

The conventional bearing capacity of a shallow foundation under vertical centered loading measured in a monotonic and a cyclic loading differs. This observation is of crucial importance to anticipate behavior of structure under cyclic loads with or without eccentricity. To build a dedicated database, small scale model tests have been chosen. To allow to accumulate tests without the difficulty of real scale and uncertainty and cost of centrifuge testing, 100 mm square footing have been tested on reconstituted sand container in 1g. This approach has shown its relevance either in 2D or 3D equipment (Meyerhof, 1956; Giraudet, 1965; Tcheng et Iseux, 1966; Lebègue, 1972; Bauer et al., 1981). However, the lack of characterisation of the soil in the container in the same way as in the real project weakens the power of the conviction. This paper presents an attempt to characterize the soil using equipments similar to that available on real construction sites.

2. Testing program

2.1. Test set up

Three main equipments have been used for reduced scale foundations loading testing. Two of them were used in RTRI facilities in Japan and one in UGE facilities in France.

The first one, originally dedicated to large triaxial testing, is a hydraulic frame improved by addition of a Tokyo university displacement control device. The soil mass is contained in a rigid tank with a height of 500 mm, and 990 mm long and 400 mm wide (Figure 1a).

The second one, consist of a electromechanic jack fixed on a beam that can accept two tanks 560 mm wide and 500 mm deep.

A third one shown on Figure 1b is a large metallic frame designed to accept 1.5x1.5 square container. The tank used is a metallic tank, 900 mm long, 450 mm wide, with a depth of 500 mm.

In either case, the depth of soil used in the tanks was 450 mm.

a)

b)

Figure 1. Global view of RTRI and UGE equipment

2.2. Soils

The sands used during these experiments are in Japan an industrial sand with a particle size distribution close to Toyoura sand with a mean grain size close to 200 µm and in France a Hostun HN31 sand (Figure 2).

These sands are poor graded sands, very uniform.

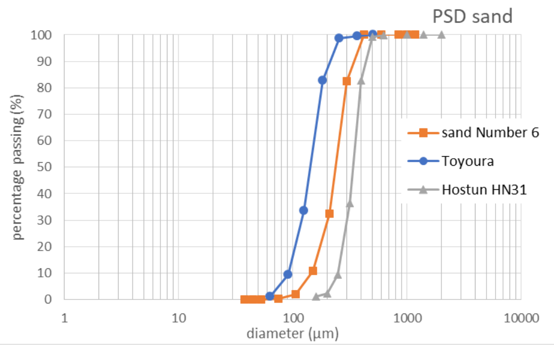

Figure 2. Particle size distribution

The particles density estimated by immersion in fluid is $\rho_s = 2.644$ g/cm^3

For N°6 sand minimum and maximum void ratio measured are e_{min} = 0.54 and e_{max} = 0.823 and for Hostun sand e_{min} = 0.532 and e_{max} = 0.893. At a relative density Dr=80% and a confining stress of $\sigma'_r = 49$ kPa the initial modulus of N°6 sand is estimated at $G_0 =$ 69.0191 MPa at a shear strain $\gamma = 0.0004192$ %.

Hereafter, relative density is expressed in percentage and equal to $D_r = 100 \times I_D$ where density index $I_D = \frac{e_{max}-e}{e_{max}-e_{min}}$ according to ISO 14688-2

Peak and residual friction angle have been estimated from direct shear tests at different relative density. N°6 sand has a 42° peak angle of friction and Hostun sand 40°. For comparison, Toyoura sand is reputed to have a 44° peak angle of friction at Dr=90%.

2.3. Sand containers

Density was controlled by levelling the sand surface on 9 points. Relative density presents an error less than 5 %. Figure 3 shows density profiles of 8 tanks.

The stage consisting in the creation of the mass of sand in the tank lasts half a day. Between each test, the container was moved to position the jack vertically above the test point.

This density was also checked by dynamic cone penetration tests (DPT) in tanks after the loading tests on the foundation (Figure 3). Not all tanks were checked. The DPT equipment called Panda (reference pour pub ?) was brought and operated by UGE technical staff.

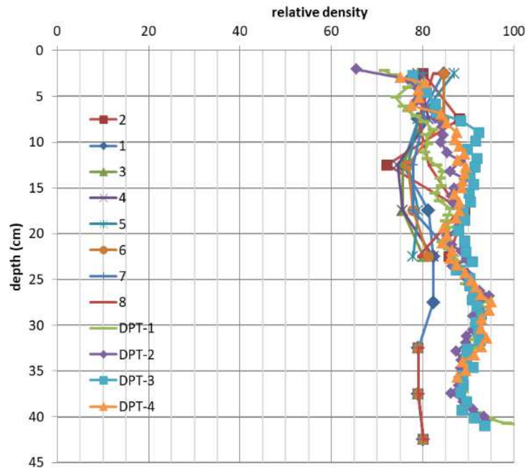

Figure 3. Density index profiles of tank 2

Correlation with the density index was used to derive density index profiles (Retamales, 2022). For the first tank the observed density is higher than the target density. An increase of relative density 5-10% (90-95 to 100%) may be observed in the lower part of tanks, which is expected given the procedure followed for the filling of the tanks.

2.4. Foundations

The foundations are square stainless steel plates, 100 mm wide and 25 mm thick. A ball joint connection was used between the footing and the system used to apply loading (Figure 4 and 5).

Figure 4. View of footing and displacement transducers, 2023 and 2024 equipment, in Japan

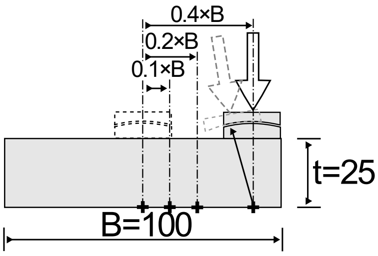

Figure 5. Foundation geometry and ball joint used

The radius of the sphere was chosen to be very close to the plate thickness t, in order to keep the point of load application at the base of the footing..

3. Plate loading test results

Tests were carried out either with a hydraulic system or an electromechanical device developed by the University of Tokyo with a DC motor and two clutches for reversing the direction of movement. Loading was carried out at a displacement rate deduced from a step test protocol (AFNOR, 2001). Since the maximum settlement is B/10=10 mm to be achieved in ten increments of five minutes each, the proposed speed is 0.2 mm/min.

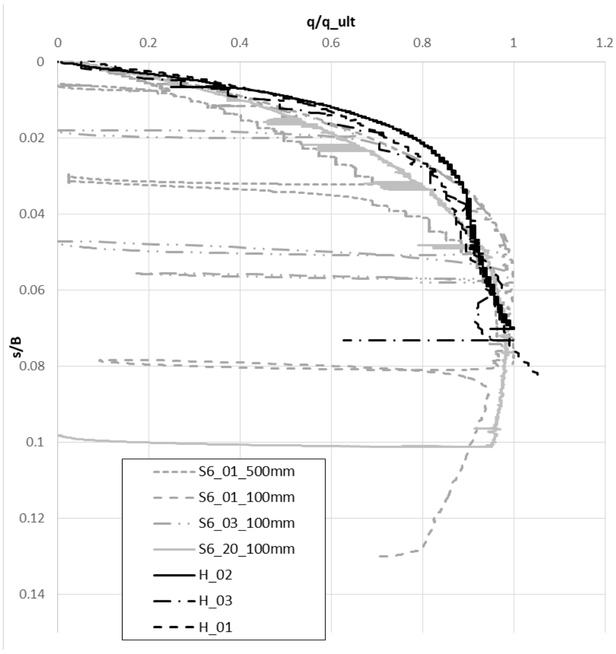

Figure 6. Load settlement curves on sand N°6 and Hostun sand.

These settings were kept for all tests.

Figure 6 shows several results obtained in RTRI and UGE facilities normalized by footing width for settlement and maximum force for axial force. Some control problem may be observed during attempts of step loading. It may be observed that one test on Hostun sand is performed on slightly more dense sand.

These tests offer the possibility to compare shape, embedment and inclinations in parametric studies.

4. Mini pressuremeter

4.1. Equipment

To characterize the sand mass the probe developed at LCPC by Beckerich (1997) and presented at ISP5 by Thorel et al. (2005) was adapted to have different slenderness ranging from 2 (diameter 10 mm and length 20 to 40 mm (Figure 7). It should be noted that this probe differs from the 32 mm pressuremeter also called mini-pressuremeter. A pressure volume controller injects silicone oil to expand the silicone membrane.

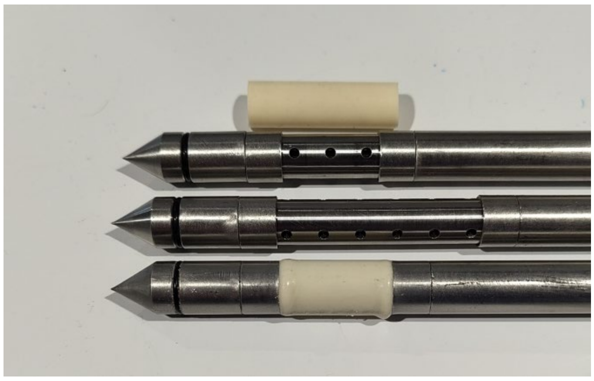

Figure 7. Mini pressuremeter probe.

Figure 8 shows the pressure loss calibration curve. The silicone membrane with a shore hardness of 40 has a slightly linear reaction.

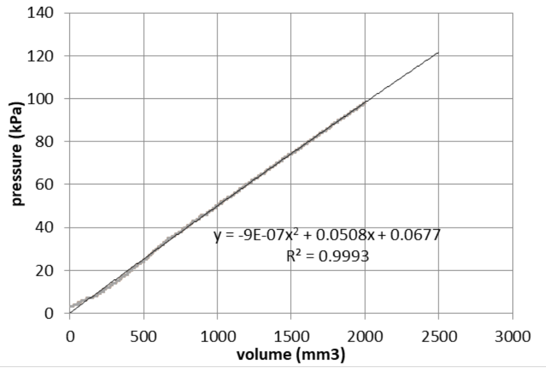

Figure 8. Pressure loss correction test.

4.2. Test results

For reasons of realism, the mini pressuremeter was placed by jacking the probe in the sand mass. Compare to placement in calibration chamber experienced by Doherty et al. (2016) with sand pluviation, or Thorel et al. (2005) in clay with preboring, the first part of the curve shows a small decrease of pressure needed to re-densify borehole walls. The conventional definition of the limit pressure, which relies on doubling the test cavity volume, was used to derive this parameter.

Figure 9 shows that between 5 and 10 cm under the footing a limit pressure of 35 kPa is observed.

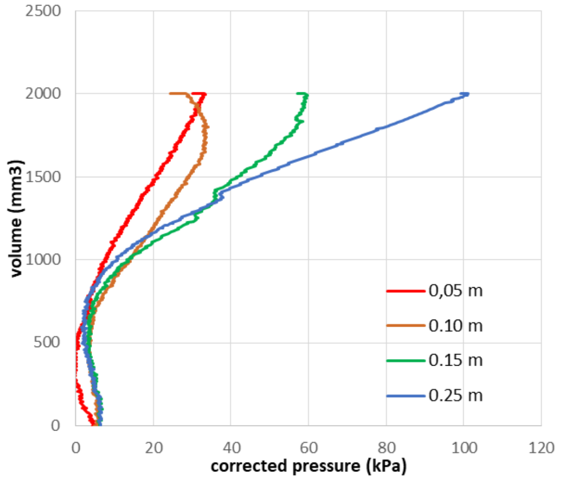

Figure 9. Mini pressuremeter expansion curves.

5. Other tests

A mini SPT equipment was tested (internal diameter 9 mm, external diameter 12 mm, hammer weight 56 g and drop height 74 mm) and gives an average N value of 24 at 10 cm depth.

Figure 10 shows the dynamic penetrometer profiles in several tanks either in RTRI and UGE facilities. An average value of q_d close to 0.5 MPa is observed.

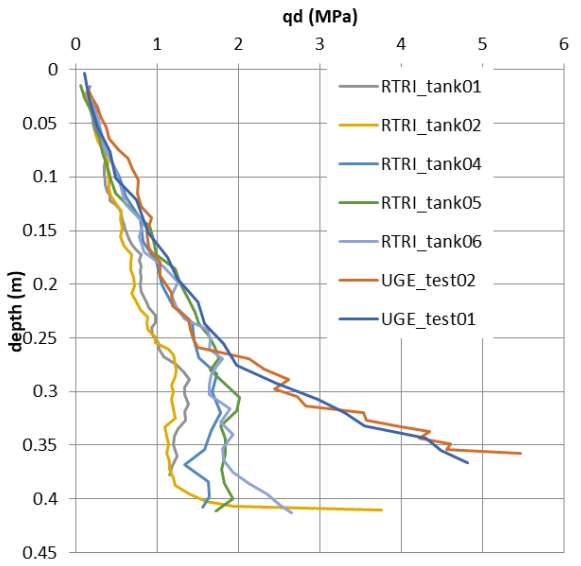

Figure 10. Panda light DPT profiles in N°6 and Hostun sands.

6. Discussion

Usually the relation in sand linking SPT and DPT to PMT are:

$$q_d \cong 7 \times p_\ell$$

$$N_{SPT} \cong 21 \times p_\ell$$

In 1g reduced scale model, the first relationship become

$$q_d \cong 14 \times p_\ell$$

And the second

$$N_{SPT} \cong 686 \times p_\ell$$

For the last relationship, the high value of correlation coefficient comes from the choice in geometry of the hammer. A much heavier one may have been chosen to respect scaling law.

It may be noted that Thorel et al. (2005) have measured a 1400 kPa limit pressure for a test at 10 cm depth in a 40g flight in centrifuge. These conditions corresponding to 4 m at prototype scale are thus close to 1000 kPa observed on Labenne site in medium dense sand.

The settlement of a shallow foundation is estimated according to the Ménard and Rousseau formula (1962), where for a square foundation λ_c =1.1 and λ_d =1.2, the reference radius is R_0 =0.3m and R =0.05m and for dense sand α= 0.5. So, if a homogeneous massif is considered, spherical domain and deviatoric domain moduli are identical and:

$$E_M = 0.22 \times \frac{q}{s}$$

Relation which is for Labenne site close to: 0.27.

For the small scale model an ultimate bearing capacity of 30 kPa was measured for a 0.39 mm settlement and a 365 kPa pressuremeter modulus. The relationship is modifies as follow:

$$E_M = 4.27 \times \frac{q}{s}$$

In the present case, the geometric scale factor (length of prototype/length of model) was close to 10. As the similitude law suggested by Iai (1989) has been widely adopted for the physical modeling of soil-structure systems, its applicability has been experimentally verified for various types of geotechnical structures. Many authors conducted monotonic load tests on model footings on the frictional or cohesive model under three different geometric scale factors (Meyerhof, 1956; Giraudet, 1965; Tcheng et Iseux, 1966; Lebègue, 1972; Bauer et al., 1981). Using "modeling of the model technique" which regards the largest physical model as a virtual prototype and compares the results of the physical models in terms of the virtual prototype scale, the experiment results indicated that Iai's similitude law is valid for use in the physical modeling of the behavior of the footings under monotonic conditions, and the discrepancy between the models was more likely related to the boundary effects of the model experiments rather than to the inconsistencies in the scaling relations.

7. Conclusions

This paper has presented experiment of footing of 1g small scale models on sand. To create a link between mechanical characterisation of the reconstituted sand mass and the foundation bearing capacity, as it is in everyday design practice with the direct design method, reduced scale testing equipements have been tested. Example of results of footing loading test and mini-pressuremeter, SPT and DPT have been presented and discussed. The small scale pressuremeter and SPT and Panda have shown their ability to characterise the homogeneity of the sand box and the limit condition at their lowest part.

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