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Penetration Resistance of Lubiatowo Sand in Calibration Chamber Tests

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Abstract

A series of cone penetration tests with standard cone and mini-cone were performed in the calibration chamber at Gdańsk University of Technology. Medium size, double wall chamber permits full control of stress and strain acting on the soil mass. The calibration chamber houses cylindrical soil sample 53 cm in diameter and 100 cm high. Lateral and vertical stresses, as well as strain conditions can be applied independently. The results of pushed-in model piles of small diameter, dilatometer tests, standard cone and mini CPT probe were analysed to determine size and boundary effects in penetration testing for calibration chamber and model sand. Free field values of cone resistance, independent of size and boundary effects, are proposed for Lubiatowo sand.

Key words: calibration chamber, CPT, DMT, model piles, sands

Notations

d – diameter of inclusion,
d_{50} – mean grain diameter,
$e_{\max}$ – maximum void ratio,
$e_{\min}$ – minimum void ratio,
k – imposed lateral stiffness,
p_a – reference pressure,
q_c – cone resistance,
$q_{c,ff}$ – free field cone resistance,
q_D – DMT blade resistance,
AR – lateral stress amplification ratio,
D_c – diameter of the chamber,
I_D – density index,
K_0 – earth pressure coefficient at rest,
K_m – earth pressure coefficient,
Q_c – normalized cone resistance,

R_d	–	diameter ratio,
U	–	uniformity coefficient,
ε_h	–	horizontal strain,
ε_v	–	volumetric strain,
σ_h	–	water pressure in the lateral cell,
σ_{n0}	–	initial normal stress,
σ_v	–	water pressure in the membranes,
σ'_{h0}	–	horizontal effective stress in-situ prior to penetration,
σ'_m	–	mean effective stress,
σ'_v	–	vertical effective stress,
σ'_{v0}	–	vertical effective stress in-situ prior to penetration.

1. Introduction

Modern in-situ test like CPTU, DMT or pressuremeter tests are more frequently used in soil investigation, reclamation works and quality control on the Baltic seashore and offshore. Interpretation of these tests in sandy soils needs some correlations to be established between the test parameters and the soil characteristics. They should be determined in model tests in well defined boundary conditions, on homogeneous and repeatable samples of reconstituted sand. As the real model of in-situ device is generally used, the tests should be realised in the container with a diameter sufficiently large to minimize the size effects. Flexible lateral boundary condition, used in a calibration chamber, permits the size effect to be reduced for a given soil specimen. The first calibration chamber was constructed in 1969 for Country Road Boards in Australia (Holden 1991). This double wall chamber, with independent stress control around the soil sample reconstituted in a reproducible and uniform way, was a turning point in physical modelling of deep foundations, calibration of in-situ devices and cone penetrometers, especially. A dry specimen with sand raining technique was formed and subjected to consolidation in K_0 conditions. The tests showed the importance of the stress level on the response of CPT, but also a strong size effect in dense sand for the tests at constant lateral stress. Larger calibration chambers were then constructed in the USA (Holden 1971, Veismanis 1974), Norway (Parkin and Lunne 1982), Italy (Bellotti et al 1982), Canada (Been et al 1987), the UK (Houlsby and Hitchman 1988), Japan (Iwasaki et al 1988) and France (Foray 1991). Besides these large diameter apparatus designed mainly for tests on quartz sands, some small size calibration chambers were constructed to study the response of more compressible or crushable sands (Bellotti and Pedroni 1991) or cohesive soils (Huang et al 1988, Kurup et al 1996).

Calibration chamber tests, performed on the different sands, showed (Schmertmann 1978, Robertson and Campanella 1983, Ghionna and Jamiolkowski 1991) a strong influence of sand compressibility on the cone resistance. The soil compressibility at elevated stress level should be considered, as presented by Colliat-Dangus

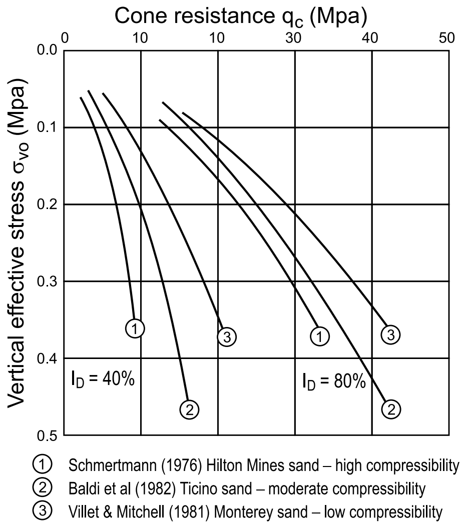


Fig. 1. Effect of sand compressibility on cone resistance (Lunne et al 1997)

et al (1988) in a series of triaxial CD tests realised at elevated, up to 15 MPa, confining stress. At given density index (Fig. 1) sands with higher compressibility would exhibit a lower cone resistance (Lunne et al 1997). The influence of the size and boundary conditions should be analysed for a given model sand, individually. Moreover, the extrapolation of calibration chamber results to real soil deposit should be done with caution even if similar sand is considered. The effect of ageing, bonding or cementation of real soil deposit cannot, however, be reproduced in calibration chamber tests (Parkin and Lunne 1982, Ghionna and Jamiolkowski 1991). It makes a major drawback for all laboratory tests on reconstituted samples.

The objective of this study is to perform and analyse a series of CPT tests in the calibration chamber at Gdańsk University of Technology (GUT), at different boundary conditions and stress levels in medium dense and dense sand, in order to propose or check some correlations concerning cone resistance in model sand. The parametric study of the cone resistance in a model sand includes the influence of stress level, density index and boundary conditions.

2. Calibration Chamber

2.1. General

A calibration chamber designed and developed at GUT (Bałachowski and Dembicki 2003), with intermediate dimensions between large size apparatus and small devices intended for cohesive soils, houses a soil sample 53 cm in diameter and 100 cm high (Fig. 2). It is a double wall chamber with independent pressure control in internal and external cells, which enables complex boundary conditions to be applied with pneumatic control system. Top and bottom membranes and the lateral cells are

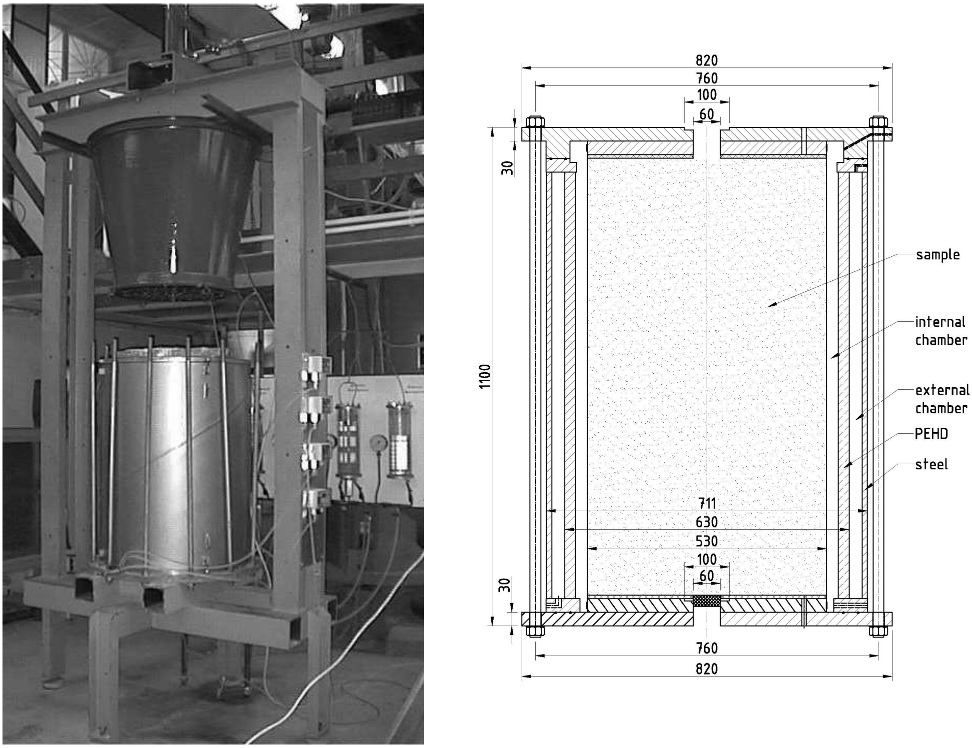


Fig. 2. View and cross-section of the calibration chamber at GUT (Bałachowski and Dembicki 2003)

filled with water. Volumetric changes in the top and bottom membranes and the internal chamber are measured with the position of the floating element in air-water columns equipped with micropulse transducers BTL2 (Fig. 3) and mounted on the control panel.

2.2. Soil Mass Formation

Clean, predominantly quartz, uniform (U = 1.41) fine sand with subrounded grains (d_{50} = 0.21 mm) is used. From wet laser analysis the material contains less than 0.3% of fines. The characteristic void ratios equal e_{max} = 0.807 and e_{min} = 0.528. According to suggestions given by Lunne et al (1997), the model sand appears to have a moderate compressibility.

Sand raining technique is used to reconstitute the sand specimen. Soil mass with I_D = 0.8 is formed with stationary sand spreader and storage bin is mounted on the frame (see Fig. 2). The storage bin is equipped with a system of two perforated bottom plates with 8 mm orifices, distributed in a uniform way. During pluviation the sand passes through diffuser sieves. For a given perforated bottom plate, the sand density is controlled by the distance between the diffuser sieves and the surface

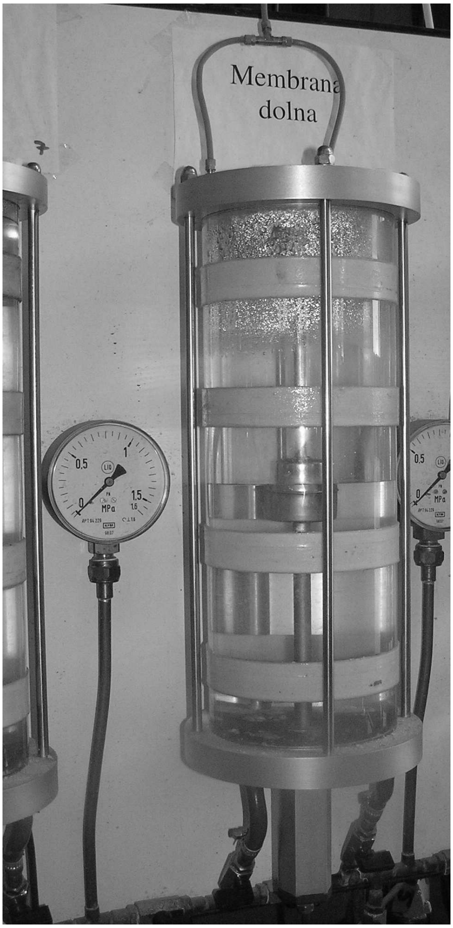


Fig. 3. Air-water column with BTL2 transducer

of the soil. In this study this distance was equal to 40 cm, kept constant during the chamber filling. The soil mass with $I_D = 0.4$ is formed using small traveling pluviator in the form of a flexible tube, as proposed by Fretti et al (1995). The double wire mesh was located just beneath the tube, to assure more regular and quiet sand flow. The sand was uniformly distributed over the sand surface using a small falling height of grains about 15 cm. Due to the considerable amount of manual work the preparation of medium dense specimens needs more time than dense sand mass. When the sand raining is completed, the membrane and upper plate are installed, the system is sealed, the cover is placed and the screws are fixed. The tests are made on dry sand. Vertical and horizontal stresses are applied to consolidate soil specimens. During this process volumetric changes in the upper and bottom membranes and in the internal chamber are measured. While the volumetric changes during consolidation are small in dense sand, they can be considerable in medium dense sand, especially where a high stress level is applied. The soil densification due to consolidation should be included in the analysis and the obtained penetration results should refer to the density index of the soil mass after consolidation. This index can reach 0.42 at small stress level and even 0.53 when a vertical stress of 400 kPa is applied (see Fig. 15). In order to include this effect $I_D = 0.5$ was chosen for the correlations presented in medium dense sand. Sand mass is consolidated with zero lateral strain condition and the obtained earth pressure at rest coefficient K_0 is close to 0.4. An overconsolidation can also be imposed, with *OCR* ratio up to 7 (Bałachowski et al 2005). Vertical working stress to be applied in the chamber is 500 kPa. After the test, the soil is removed from the chamber by the hole in the bottom plate and stored in the trolley.

2.3. Boundary Conditions

Independent pressure control can be applied in the internal and external chambers and in the upper and lower membranes. Flexible radial conditions with cushions on the bottom and on the top of the specimen are realised. Typical boundary conditions (Table 1) can be used in the constructed chamber. BC1 with constant lateral stress $\sigma_h =$ const and BC3 with zero lateral strain ($\varepsilon_h = 0$) are mainly used, as they simulate in-situ conditions, where overburden stress is generally constant. Imposed lateral stiffness ($k =$ const) condition (BC5) can be applied to simulate an infinite soil mass (as proposed by Foray 1991 and Ghionna and Jamiolkowski 1991), taking into consideration volumetric changes in the internal lateral cell or horizontal displacements on the sample boundary. In this chamber, the imposed lateral stiffness condition can be used, taking into account volumetric changes in the internal lateral cell.

Table 1. Boundary conditions to be applied

Boundary conditions	Horizontal	Vertical
BC1	$\sigma_h =$ const	$\sigma_v =$ const
BC2	$\varepsilon_h = 0$	$\varepsilon_v = 0$
BC3	$\varepsilon_h = 0$	$\sigma_v =$ const
BC4	$\sigma_h =$ const	$\varepsilon_v = 0$
BC5	lateral stiffness $k =$ const	$\sigma_v =$ const

Cone resistance measured in the constrained soil volume in the calibration chamber is generally lower than the corresponding free field value in unconfined half space. For a given sand and boundary conditions, chamber size effect is related (Parkin and Lunne 1982) to diameter ratio R_d:

$$R_d = \frac{D_c}{d}, \tag{1}$$

where:

D_c – diameter of the chamber,
d – diameter of the inclusion,

depending on stress state and sand density. It can be presented as a function of state parameter (Been et al 1986, 1987). According to general recommendations (Parkin and Lunne 1982) R_d ratio should be higher than 60 in dense sand at low vertical stress and higher than 30 in loose sand to minimize size effect in penetration tests with a standard cone. The real size effect observed in a given chamber depends on the model sand compressibility i.e. its mineralogy, granulometry and shape of grains (Schmertmann 1978, Robertson and Campanella 1983, Lunne et al 1997). The influence of size and boundary effects in quartz sands was studied by Parkin and Lunne (1982), Been et al (1987), Houlsby and Hitchman (1988), Ghionna and Jamiolkowski (1991), Mayne and Kulhavy (1991), Hsu and Huang (1998) and Salgado et al (1998). Small size and boundary effects are observed in Ticino sand, but they are very important, up to 3, for Hokksund sand in medium size chamber (Been et al 1987). A series of mini-cone penetration tests in centrifuge in dense Fontainbleau sand, with rigid wall containers of different size, show no size effect for $R_d > 40$ (Bolton and Gui 1993, Gui et al 1998, Bolton et al 1999). When a thick rubber membrane was used at the circumference of the small container, to simulate an infinite soil mass, no size effect in cone penetration in dense sand was recorded even for $R_d < 40$.

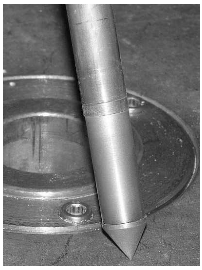
2.4. The Models used in Calibration Chamber Tests at GUT

The standard 10 cm^2 CPT cone and typical DMT blade were used in penetration tests. Geotech AB probe with wireless data transmission was used. Additionally, some penetration tests were performed with mini-CPT 13.4 mm in diameter. In this study R_d ratio is about 40 for mini-cone and about 14.8 for the standard cone. For the latter an important size and boundary effects should be expected, even for medium compressible sand. Some size and boundary effects would exist for the tests with mini-cone in dense sand, especially at small stress level. Two model piles of 20 mm and 32.8 mm in diameter instrumented with extensometer gauges at seven levels were also used. The model piles were pushed in the soil mass in the calibration chamber to study size and boundary effects in penetration testing. All models are presented in Fig. 4.

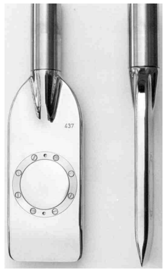
3. Cone Penetration Tests in Calibration Chamber

A series of CPT tests with standard cone and with mini-cone were realised in medium dense and dense Lubiatowo sand at BC1 and BC3 conditions at vertical effective stress level from 50 kPa to 400 kPa. The penetration tests were effected in normally consolidated sand mass.

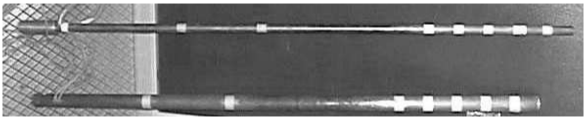
standard cone



mini-cone



DMT blade



model piles

Fig. 4. Models used in calibration chamber tests

3.1. BC1 Conditions

A series of tests were performed at constant lateral stress. A set of cone resistance at different vertical stress is given for medium dense (Fig. 5) and dense sand (Fig. 6). The measured cone resistance depends strongly on vertical effective stress applied in the chamber.

3.2. BC3 conditions

The mobilisation of cone resistance in dense sand at different vertical stress for zero lateral strain condition is given in Fig. 7. As the lateral stress increases in the lateral internal cell, the cone resistance at given vertical effective stress is found to be generally higher than in BC1 condition.

The evolution of the water pressure σ_h in the lateral cell, observed during penetration, can be described (Fig. 8) with lateral stress amplification ratio:

$$AR = \frac{K_m}{K_0}, \tag{2}$$

where:

$$K_m = \frac{\sigma_h}{\sigma_v}, \tag{3}$$

K_0 – earth pressure at rest coefficient,
K_m – earth pressure coefficient.

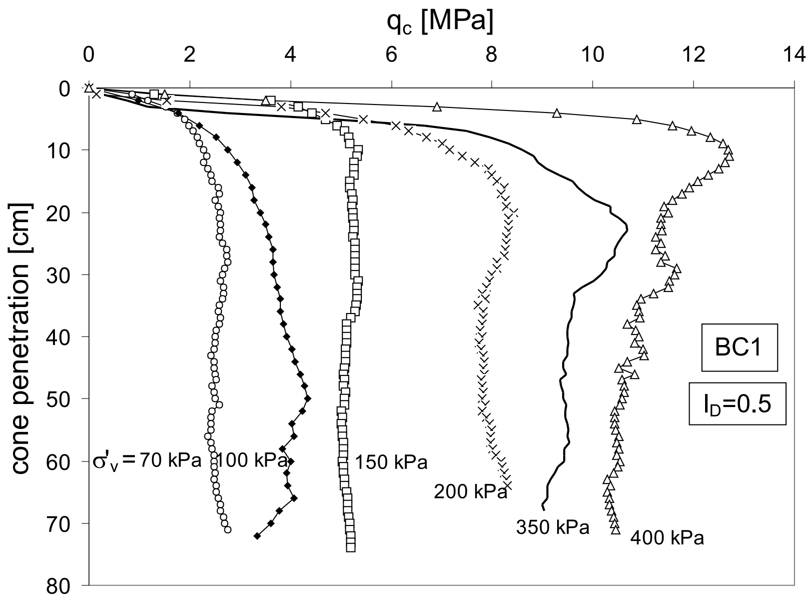


Fig. 5. Cone resistance in medium dense sand at BC1 condition

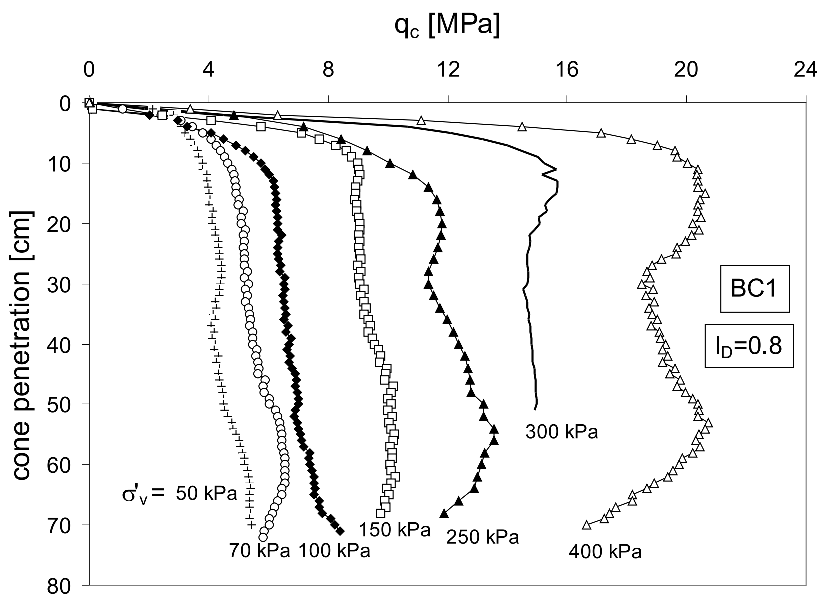


Fig. 6. Cone resistance in dense sand at BC1 condition

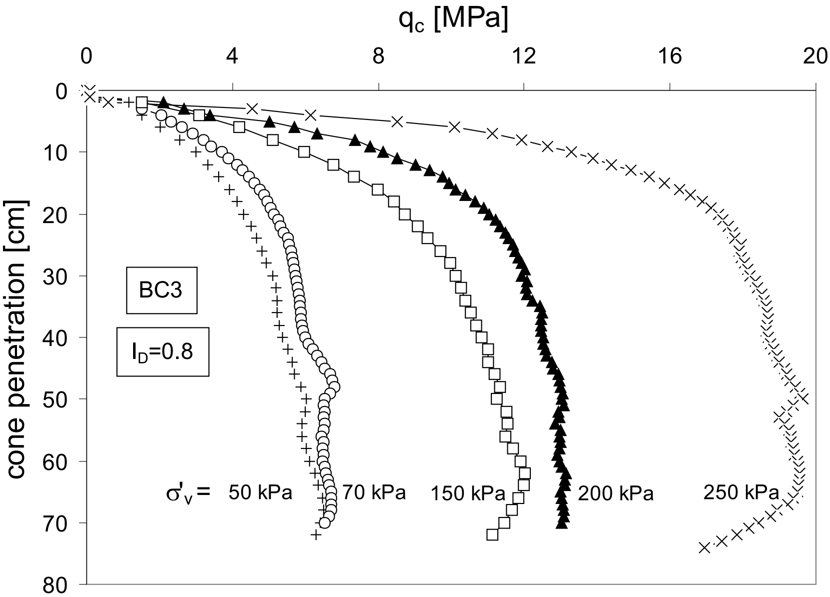


Fig. 7. Cone resistance in dense sand at BC3 condition

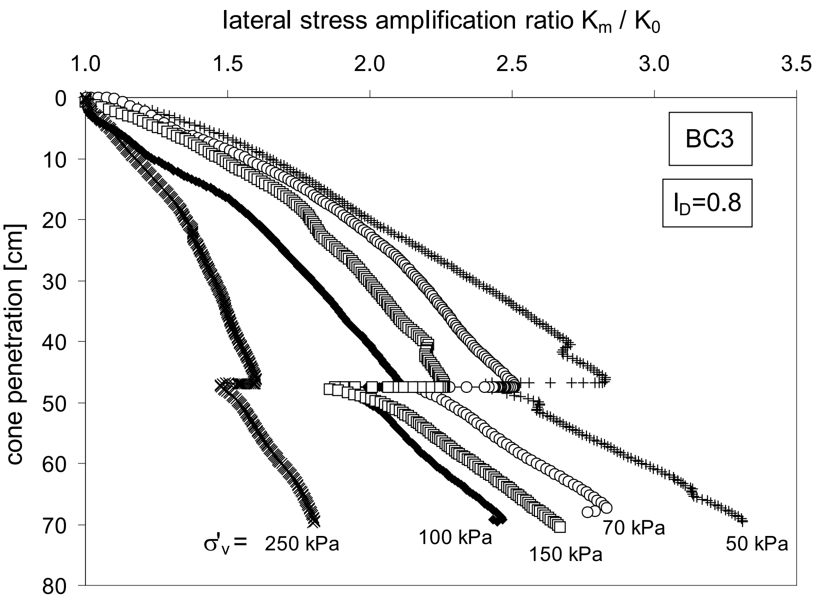


Fig. 8. Lateral stress amplification ratio in dense sand during cone penetration at BC3

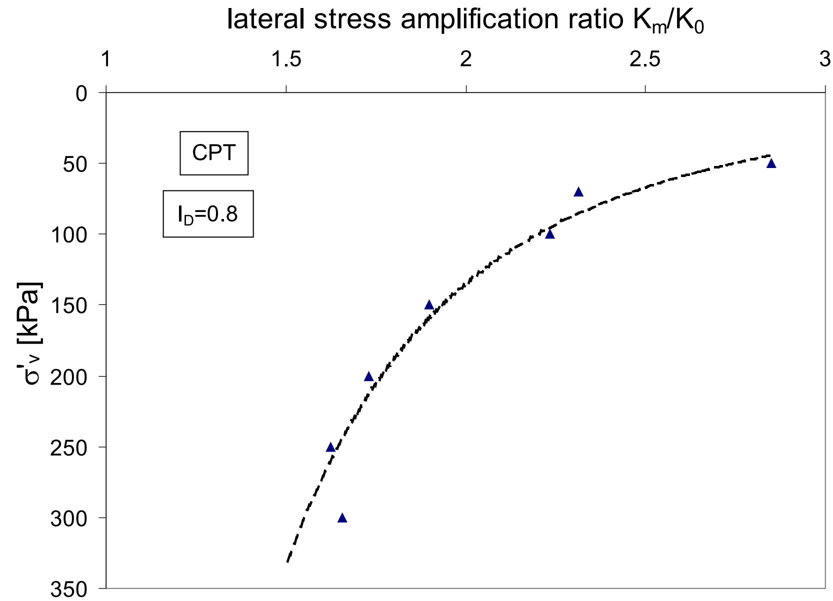


Fig. 9. Lateral stress amplification ratio in dense sand for different vertical effective stress

Due to limited 50 cm stroke of the loading piston the penetration was interrupted in order to screw the next rod. The load was released and a drop of lateral stress was observed at this depth.

Lateral stress amplification ratio for CPT at 50 cm of penetration is summarized in Fig. 9. An important lateral stress increase is also observed during the penetration of 32.8 mm model pile (Fig. 10). Lateral stress amplification ratio at 50 cm of pile penetration can reach 2.2 for $\sigma'_v = 70$ kPa, which is consistent with CPT data in Fig. 9. An example of lateral stress amplification factor at BC3 for the penetration of two model piles of different diameters (20 mm and 32.8 mm) pushed into dense sand from the surface of the soil mass, is presented in Fig. 11. For 20 mm model pile, only a small lateral stress increase is registered and a moderate size effect is observed.

4. Free Field Cone Resistance

The values of cone resistance for standard cone and mini-cone in dense sand at BC1 and BC3 conditions are summarized in Fig. 12. Highest cone resistance is measured for mini-probe in BC1 conditions. While larger q_c values are recorded in BC3 rather than BC1 conditions at given vertical effective stress, similar values of cone resistance are found for a given mean effective stress (Fig. 13). As suggested by Baldi et al (1986), Houlsby and Hitchman (1988) and Bałachowski (1995), the cone resistance depends not only on vertical effective stress, but also on the horizontal

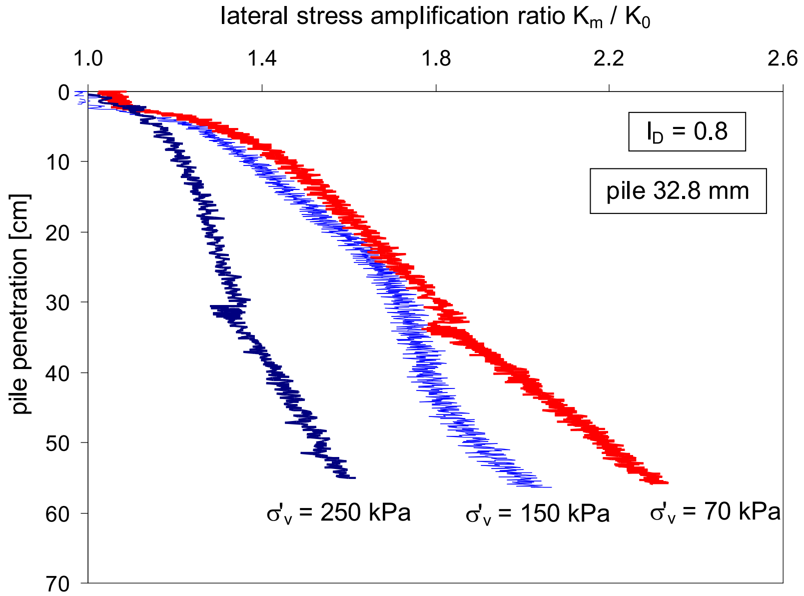


Fig. 10. Lateral stress amplification ratio for 32.8 mm model pile at different vertical effective stress

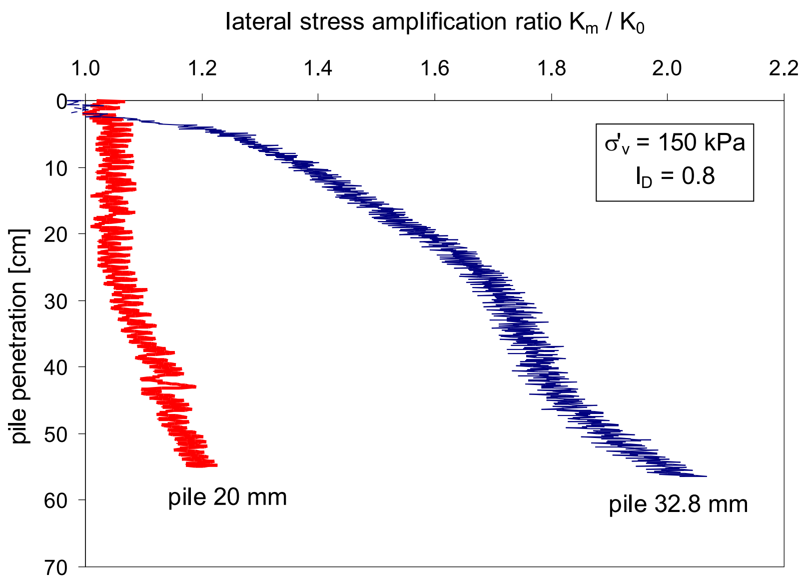


Fig. 11. Lateral stress amplification ratio for model piles (Bałachowski and Dembicki 2002)

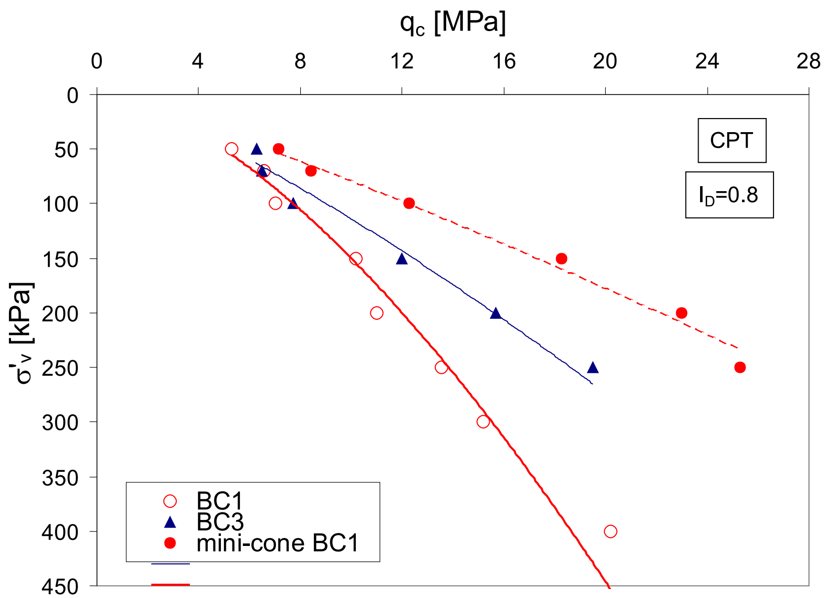


Fig. 12. Cone resistance for standard and mini CPT probes vs. vertical effective stress

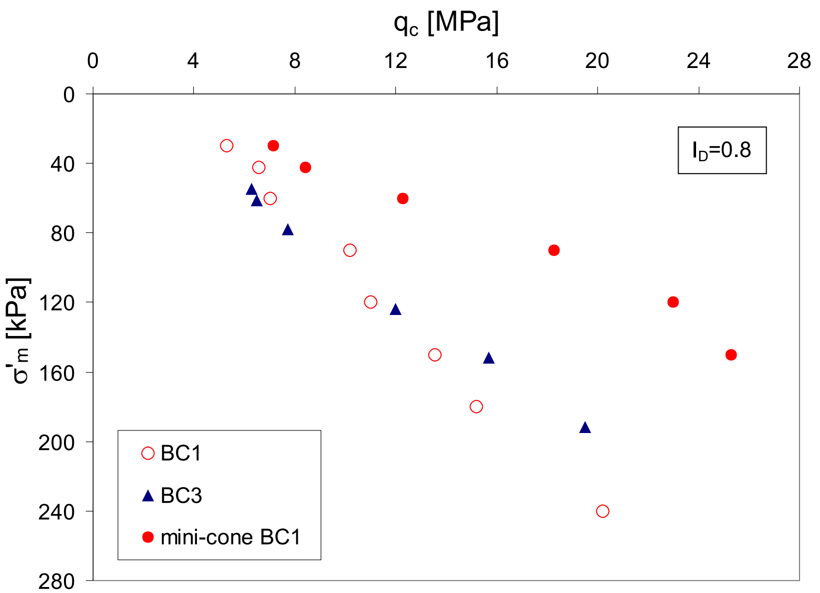


Fig. 13. Cone resistance for standard and mini CPT probes vs. mean effective stress

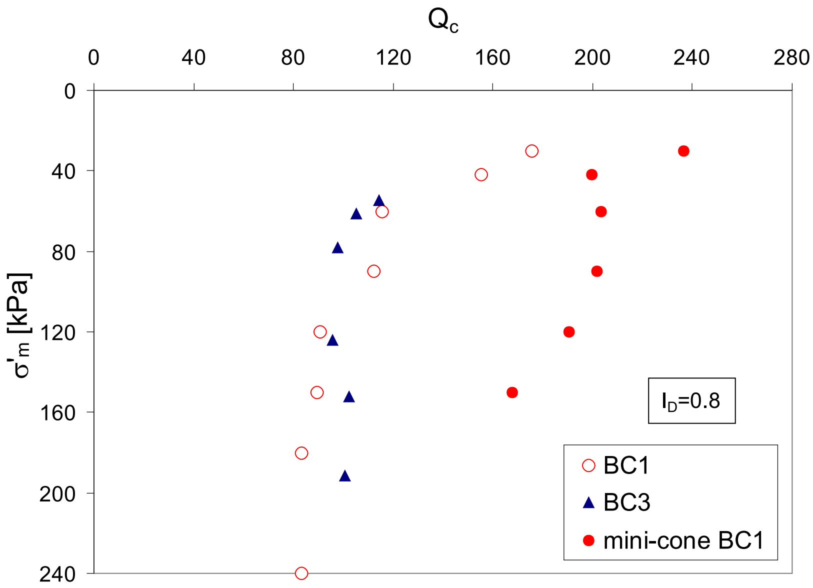


Fig. 14. Normalized cone resistance for standard and mini CPT probe

one. The difference between q_c values in BC1 and BC3 conditions is getting reduced when plotted as a function of mean effective stress. With such a presentation the influence of boundary effects in calibration chamber can be partially eliminated. The size effect between standard cone and mini-cone is, however, still well pronounced. In order to eliminate stress level influence on the cone resistance q_c, the normalized cone resistance can be defined:

$$Q_c = \frac{q_c - \sigma'_v}{\sigma'_v}, \tag{4}$$

$$\text{or } Q_c = \frac{q_c - \sigma'_m}{\sigma'_m}, \tag{5}$$

where:

σ'_m – mean effective stress,
σ'_v – vertical effective stress.

The last definition (Eq. 5) was used for the data in Fig. 14, where boundary effect for standard cone is getting reduced, but a distinct size effect can be noticed.

The cone resistance for standard cone, mini CPT probe and DMT blade resistance at BC1 conditions in medium dense sand are presented in Fig. 15. According to Jamiolkowski et al (2001) the blade resistance from dilatometer test q_D is slightly higher than corresponding cone resistance q_c:

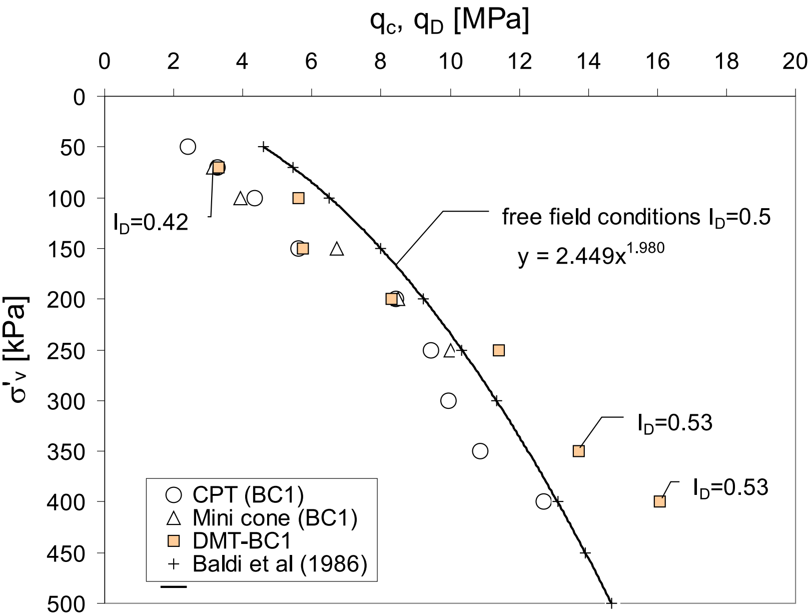


Fig. 15. Free field cone resistance in medium dense (I_D = 0.5) Lubiatowo sand

$$q_D = 1.21 \times q_c. \tag{6}$$

A correlation for free field values of cone resistance proposed by Baldi et al (1986) for medium compressible sands was analysed:

$$q_{c,ff} = 220 p_a \left(\frac{\sigma'_{v0}}{p_a}\right)^{0.065} \left(\frac{\sigma'_{h0}}{p_a}\right)^{0.44} \exp\left(2.93 I_D\right), \tag{7}$$

where:

- p_a – the reference pressure (1 kPa),
- σ'_{v0} – vertical effective stress in-situ prior to penetration,
- σ'_{h0} – horizontal effective stress in-situ prior to penetration,
- I_D – density index.

This correlation can be considered as a first approximation of free field values of cone resistance in Lubiatowo sand. At large overburden stress (Fig. 15) this correlation slightly overpredicts q_c values and underestimates q_D in medium dense Lubiatowo sand. At low stress level some size effect would be expected. Moreover, some differences in density index due to consolidation exist between the samples consolidated at small and large overburden stress (see Fig. 15).

In dense sand the cone resistance q_c at BC1 at BC3 conditions and the results from mini CPT probe at BC1 are presented in Fig. 16. The data from mini-probe

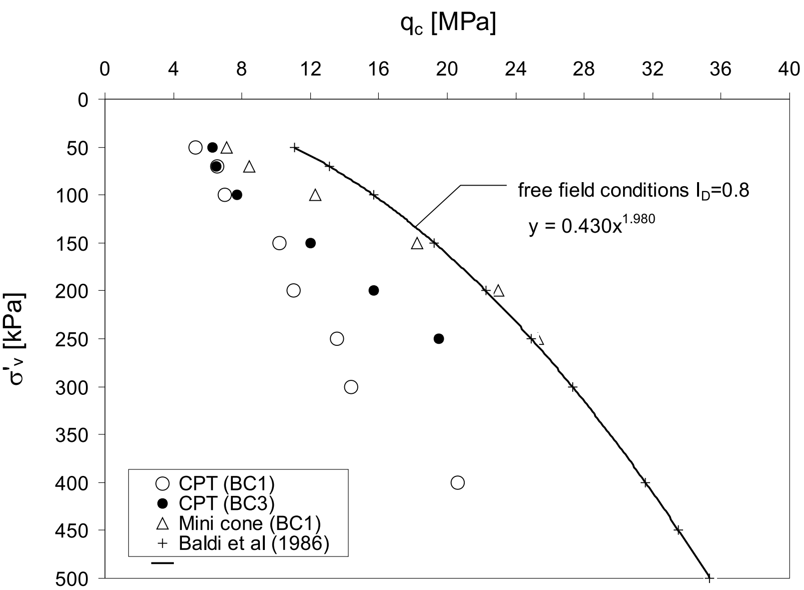


Fig. 16. Free field cone resistance in dense (I_D = 0.8) Lubiatowo sand

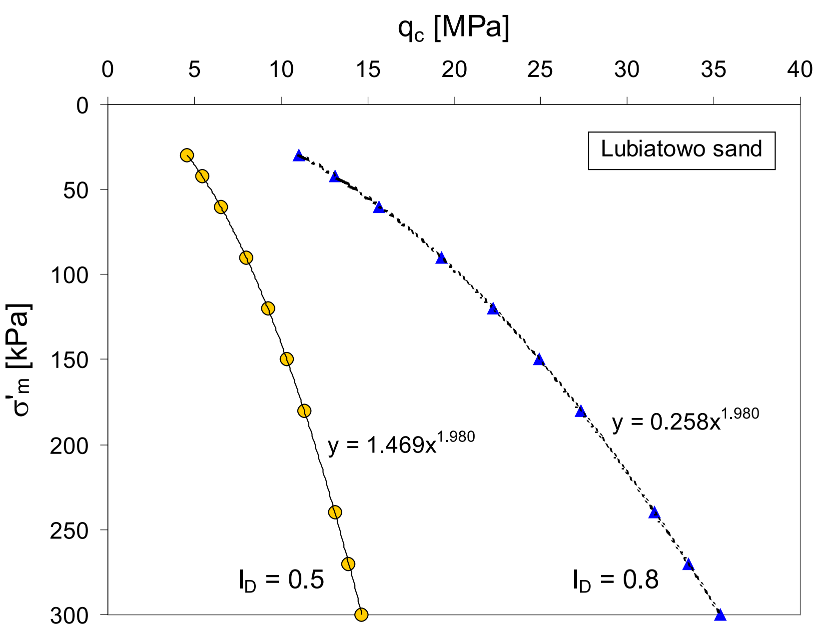


Fig. 17. Free field cone resistance in NC and OC Lubiatowo sand

and CPT at BC3 at large overburden stress are situated close to the proposition of Baldi et al (1986). An important size effect, larger than 2, observed at small overburden stress attenuates with stress level. It is considerably higher at BC1 than BC3 condition. Effective vertical stress, where only negligible size effect in cone penetration at BC3 condition would be expected, can be determined by an extrapolation of lateral stress amplification ratio (Fig. 9). With a rough estimation one can obtain $\sigma_v' = 700$ kPa.

The proposed free field cone resistance in medium dense and dense Lubiatowo sand is given in Fig. 17 as a function of mean effective stress. While the correlations in Figs. 15 and 16 can be used for normally consolidated (NC) sands, the proposal in Fig. 17 can be applied for both normally consolidated and overconsolidated (OC) sands.

5. Conclusions

A series of CPT was performed in the calibration chamber in GUT using different boundary conditions. The medium size of the chamber needs the size and boundary effects in penetration testing to be carefully analysed. Preliminary tests including standard and mini CPT probes and dylatometer tests show that Baldi et al (1986) correlation for medium compressible sand can be considered as a first estimation of free field values of the cone resistance in Lubiatowo sand. A small size effect is found for CPT and DMT in medium dense sand at BC1 condition. A considerable size effect is found for CPT tests performed in dense sand, especially at BC1 condition and low vertical effective stress. The results of mini-cone and CPT at BC3 condition approach Baldi et al (1986) correlation at larger vertical stress. This correlation forms a conservative bound for the cone resistance in fine to medium predominantly quartz sands on the Baltic seashore. Further research will eventually permit to refine this proposal. In overconsolidated sand the correlation normalized to mean effective stress should be used.

The constructed and developed calibration chamber enables a comprehensive physical modelling under controlled stress and boundary conditions. A wide range of confining stress to be used enables the proper modelling of soil rheology, including soil-structure interaction. Further research on the influence of size and boundary effects in penetration testing is necessary, especially with BC3 and BC5 conditions. The boundary effect could be reduced at these two conditions. The size effect will, however, still be present. The influence of size and boundary conditions would be less important in more compressible materials like silty or carbonated sands. After some modifications of pluviation technique, the calibration chamber can be used for these materials.

The calibration chamber tests are performed in fresh deposits. The influence of ageing, structural overconsolidation or cementation, which cannot be reproduced in laboratory conditions, restrains the direct application of these results to freshly deposited or unaged, clean, predominantly quartz sands. With a sound knowledge of geological history, the influence of these phenomena can be considered only qualitatively.

Acknowledgements

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References

Baldi G., Bellotti R., Ghionna V., Jamiolkowski M., Pasqualini E. (1986), Interpretation of CPTs and CPTUs – Part II: drained penetration in sands, *Proceedings of the Fourth International Geotechnical Seminar on Field Instrumentation and in Situ Measurements*, Nanyang Technological Institute, Singapore, 143–156.

Bałachowski L. (1995), Différents aspects de la modélisation physique du comportement des pieux: Chambre d'Etalonnage et Centrifugeuse, *Thèse de doctorat*, Institut National Polytechnique de Grenoble, France, 320 pp.

Bałachowski L., Dembicki E. (2002), Une étude de pieux foncés dans la chambre d'étalonnage, *Journées Nationales pour la Géotechnique et la Géologie d'Ingénieurs*, Nancy, 8–9 Octobre, CD ROM ISBN 2-85555-056-6.

Bałachowski L., Dembicki E. (2003), La construction d'une chambre d'étalonnage à l'Université Technique de Gdańsk, *Studia Geotechnica et Mechanica*, Vol. 25, No. 1–2, 21–26.

Bałachowski L., Kozak P., Dembicki E. (2005), Effect of preconsolidation on pile bearing capacity in model tests, *Studia Geotechnica et Mechanica*, Vol. 27, No. 1–2, 3–11.

Been K., Crooks J. H. A., Becker D. E., Jefferies M. G. (1986), The cone penetration test in sands: Part I, state parameter interpretation, *Géotechnique*, Vol. 36, No. 2, 239–249.

Been K., Jefferies M. G., Crooks J. H. A., Rothenburg L. (1987), The cone penetration test in sands: Part II, general inference of state, *Géotechnique*, Vol. 37, No. 3, 285–299.

Bellotti R., Bizzi G., Ghionna V. (1982), Design, construction and use of a calibration chamber, *Proceedings of the 2nd European Symposium on Penetration Testing*, Amsterdam, 24–27 May, 439–446.

Bellotti R., Pedroni S. (1991), Design and development of a small calibration chamber for compressible sands, *Proceedings of the 1st International Conference on Calibration Chamber Testing*, Clarkson Univ. Potsdam, USA, 91–99.

Bolton M. D., Gui M. W. (1993), The study of relative density and boundary effects for cone penetration tests in centrifuge, *Report CUED/D-SOILS/TR256,* Cambridge University, Engineering Department, UK., 31 pp.

Bolton M. D., Gui M. W., Garnier J., Corté J. F., Bagge G., Laue J., Renzi R. (1999), Centrifuge cone penetration tests in sand, *Géotechnique,* Vol. 49, No. 4, 543–552.

Colliat-Dangus J. L., Desrues J., Foray P. (1988), Triaxial testing of granular soil under elevated cell pressures, *Advanced Triaxial Testing of Soil and Rock,* ASTM STP 977, Philadelphia, 290–310.

Foray P. (1991), Scale and boundary effects on calibration tests, *Proceedings of the 1st International Conference on Calibration Chamber Testing*, Clarkson Univ. Potsdam, USA, 147–160.

Fretti C., Lo Presti D. C. F., Pedroni S. (1995), A pluvial deposition method to reconstitute well-graded sand specimens, *Geotechnical Testing Journal*, Vol. 18, No. 2, 292–298.

Ghionna V. N., Jamiolkowski M. (1991), A critical apparaisal of calibration chamber testing in sands, *Proceedings of the 1st International Conference on Calibration Chamber Testing,* Clarkson Univ. Potsdam, USA, 13–37.

Gui M. W., Bolton M. D., Garnier J., Corté J. F., Bagge G., Laue J., Renzi R. (1998), Guidelines for cone penetration tests in sand, *Proc. of International Conference Centrifuge'98*, Tokyo, Japan, Vol. 1, 155–160.

Holden J. C. (1971), Laboratory research on static cone penetrometers, *Internal report CE-SM-71-1*, University of Florida, Gainsville, Department of Civil Engineering.

Holden J. C. (1991), History of the first six CRB calibration chambers, *Proceedings of the 1st International Conference on Calibration Chamber Testing*, Clarkson Univ. Potsdam, USA, 1–11.

Houlsby G. T., Hitchman R. (1988), Calibration chamber tests of cone penetrometer in sand, *Géotechnique*, Vol. 38, No. 1, 39–44.

Hsu H.-H., Huang A.- B. (1998), Development of an Axisymmetric Field Simulator for Cone Penetration Tests in Sand, *Geotechnical Testing Journal, GTJODJ*, Vol. 21, No. 4, 348–355.

Huang A.-B., Holtz R. D., Chameau J.-L. A. (1988), Calibration Chamber for Cohesive Soils, *Geotechnical Testing Journal*, Vol. 11, No. 1, 30–35.

Iwasaki K., Tanizawa F., Zhou S., Tatsuoka F. (1988), Cone resistance and liquefaction strength of sand, *Proceedings of the 1st International Symposium on Penetration Testing* (I.S.O.P.T.), Orlando, 20–24 March, Vol. 2, 785–791.

Jamiolkowski M., Lo Presti D. C. F., Manassero M. (2001), Evaluation of Relative Density and Shear Strength of Sands from CPT and DMT, *Symposia in Honor of C. C. Ladd, Soil Behavior and Soft Ground Construction*, Geotechnical Special Publications, No. 119, 5–6 October, Cambridge, Massachusetts, USA, 37 pp.

Kurup P. U., Voyiadis G. Z., Tumay M. T. (1996), Calibration chamber studies of piezocone test in cohesive soils, *Journal of Geotechnical Engineering,* Vol. 120, No. 1, 81–107.

Lunne T., Robertson P. K., Powel J. J. M. (1997), *Cone Penetration Testing in Geotechnical Practice,* Blackie Academic and Professional, London, pp. 312.

Mayne P. W., Kulhawy F. H. (1991), Calibration chamber database and boundary effects correction for CPT data, *Proceedings of the 1st International Conference on Calibration Chamber Testing,* Clarkson Univ. Potsdam, USA, 257–263.

Parkin A. K., Lunne T. (1982), Boundary effects in the laboratory calibration of a cone penetrometer for sand, *Proceedings of the 2nd European Symposium on Penetration Testing*, Amsterdam, 24–27 May, Vol. 2, 761–768.

Robertson P. K., Campanella R. G. (1983), Interpretation of cone penetrometer test: Part I: Sand, *Canadian Geotechnical Journal*, Vol. 20, No. 4, 718–733.

Salgado R., Mitchell J. K, Jamiolkowski M. (1998), Calibration Chamber size effects on penetration resistance in sand, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 124, No. 9, 878–888.

Schmertmann J. H. (1978), Guidelines for Cone Penetration Test, Performance and Design, *Report, FHWA-TS-78-209*, US Federal Highway Administration, Washington, DC, 145 pp.

Veismanis A. (1974), Laboratory investigation of electrical friction cone penetrometers in sands, *Proceedings of European Symposium on Penetration Testing*, Stockholm, June 1974, 407–419.