PROCEEDINGS OF THE

30 th YEAR ANNIVERSARY SYMPOSIUM OF THE SOUTHEAST ASIAN GEOTECHNICAL SOCIETY

DEEP FOUNDATIONS, EXCAVATIONS, GROUND IMPROVEMENTS & TUNNELLING

Bangkok, THAILAND
03 – 07 November 1997

Sponsored by

ASIAN INSTITUTE OF TECHNOLOGY 1959

Asian Institute of Technology

Southeast Asian Geotechnical Society

SOUTHEAST ASIAN GEOTECHNICAL SOCIETY
FOUNDED IN 1967

PROCEEDINGS OF THE

30 th YEAR ANNIVERSARY SYMPOSIUM OF
THE SOUTHEAST ASIAN GEOTECHNICAL SOCIETY

DEEP FOUNDATIONS, EXCAVATIONS, GROUND IMPROVEMENTS & TUNNELLING

Bangkok, Thailand
03 – 07 November 1997

Sponsored by

The Southeast Asian Geotechnical Society (SEAGS)

The Asian Institute of Technology (AIT)

PREFACE

The year 1997 coincides with the thirteenth year anniversary of our Southeast Asian Geotechnical Society. During the last thirty years, remarkable progress has been made in Southeast Asian Geotechnical Engineering Education, Research and Practice. The 30th Anniversary event this year will mainly concentrate on four areas in which Geotechnical Engineering Practice is very vibrant in the member countries of our society. These topics are Deep Foundations, Deep Excavations for Underground Utilities, Ground Improvements and Tunnelling for Rapid Transit Projects.

This volume contains the contributed papers and country reports as received prior to the symposium. For ease of reference, the papers are arranged into the following themes: 1. Ground Improvement; 2.Deep Excavation; 3. Piled Foundation; and 4. Tunnelling. Under the section Ground Improvement, there are thirteen papers and Deep Excavations contains four papers. Four papers are included in Piled Foundations and the last section, Tunnelling, presents six papers.

The editors would like to express their sincere thanks to the general committee members of the Southeast Asian Geotechnical Society. Sincere gratitude is likewise expressed to the AIT Faculty and Staff.

A.S.Balasubramaniam
D.T.Bergado
Lin Der Guey
S.Shibuya
T.H.Seah
N.Phien-wej
P.Nutalaya

November 1997

FACE STABILITY DURING SLURRY-SHIELD DRIVING AND SLURRY-FILLED TRENCH EXCAVATION

by

Takeshi Hosoi
Director & General Manager, Civil Engineering Design Dept.,
Nishimatsu Construction Co., Ltd., Japan
Hitoshi Mori
Chief Engineer, Technical Research & Development Institute,
Nishimatsu Construction Co. Ltd., Japan

SYNOPSIS: The increase in pore water pressure at the cutting face during slurry-shield driving and slurry-filled trench excavation has a great influence on the excavated face stability. Excess pore water pressure during slurry-shield driving may be caused mainly by ground water flow due to slurry pressure. On the other hand, the increase in pore water pressure during slurry-filled trench excavation may be caused mainly by cyclic shear stress introduced into the cutting face due to dynamic cyclic excavation.

1. Introduction

In slurry-shield tunneling and slurry-filled trench excavation, the face (the cutting face during shield driving and the excavated trench wall during trench excavation) is stabilized by the slurry pressure so as to resist pore water pressure and earth pressure.

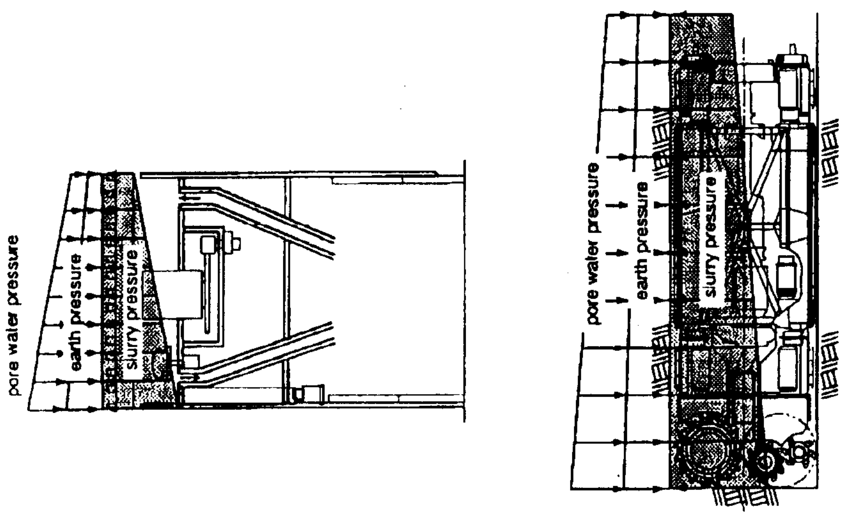
For face stabilization, it is essential to evaluate the mobilized earth pressure and the pore water pressure in the ground near the face.

The authors have focused on the pore water pressure, especially the increase in pore water pressure, and carried out field measurements of and laboratory tests on the increase in pore water pressure.

Based on the results of measurements and laboratory tests thus carried out, considerations to the increase in pore water pressure were made to improve the face stability.

2. Mechanism for Face Stability

At the cutting face during slurry-shield tunneling and slurry-filled trench excavation, the slurry pressure resists the mobilized earth pressure and the pore water pressure as shown in Fig. 1.



a) slurry-shield

b) trench excavation

Fig1. Face stability of slurry-shield and trench excavation

Slurry pressure to be set up is defined as shown in Formula (1).

Slurry pressure to be set up =

Pore water pressure + Mobilized earth pressure + Additional pressures .... (1)

In the above formula, the additional pressure is defined empirically by incorporating the possibility of errors in setting up the pore water pressure or the earth pressure, and variations in slurry pressure due to mechanical losses (for slurry-shield) or scattering of slurry density (for slurry-filled trench excavation).The values of 0.2 to 0.5 kgf/cm^2 are generally adopted, taking into account past experiences.

Recently, the increase in pore water pressure at the cutting face during slurry-shield driving and slurry-filled trench excavation has been observed at the site, and it has been confirmed that the excess pore water pressure has a great influence on the face stability and that the increase in pore water pressure should be taken into account when setting up the slurry pressure.

3. Field Measurement of Pore Water Pressure during Slurry-Shield Driving[1)]

A slurry-shield of 10.2 m in diameter and with an earth cover of 10 to 15 m was driven as part of the extension work for the Fukuoka City Subway from Hakata Station to Fukuoka Airport.

The relationship between the increase in pore water pressure and the face stability was examined on the basis of measurements and reviews conducted by Matsushita et. al. (reference 2).

The location of pore pressure meters at the measured section together with soil conditions are presented in Fig. 2.

Fig. 3 illustrates the variations in pore water pressures at each stratum of the measured sections during shield driving.

As can be seen in Fig. 3, the pore water pressure (P4 and P5) in the decomposed granite stratum (permeability coefficient of 1 x 10–4 cm/sec) shows a gradual rise along with approach of the shield to the measured section. The pore water pressure in the center of the shield (P5) reaches 1.875 kgf/cm^2 just before shield passage and the effective slurry pressure is extremely small at 0.025 kgf/cm^2 while the set slurry pressure is 1,900 kgf/cm^2. However, no such increase in pore water pressure (P1, P2 and P3) was observed in both the alluvial sandy soil (permeability coefficient of 1 x 10^{-3} cm/sec) and the diluvial sandy soil (permeability coefficient of 1 x 10^{-3} cm/sec).

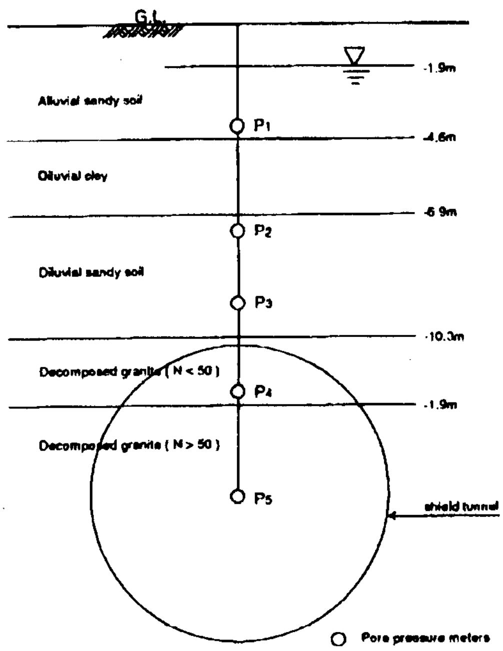


Fig2. Soil conditions and locations of pore pressure meters

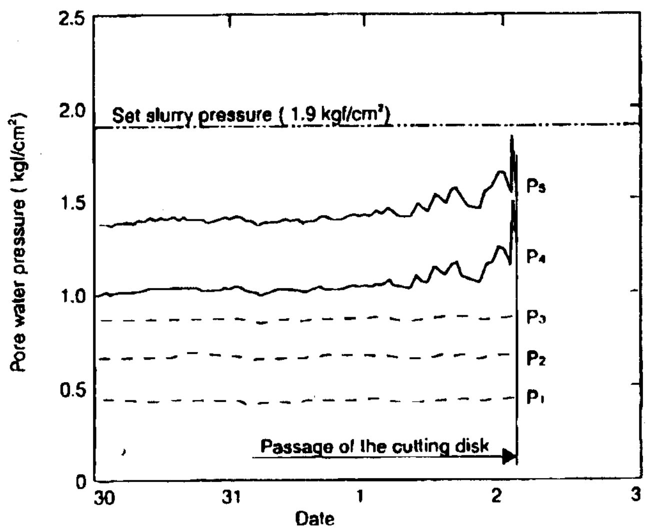


Fig3. Variation in pore water pressure in each stratum of the measured section during shield driving (modified,original in reference 2)

4. Field Measurement of Pore Water Pressure during Slurry-Filled Trench Excavation[3)]

A trial excavation of slurry-filled trenches (50 to 150 m in depth, 2.1 m in thickness) was carried out using the reverse-circulation type excavator having two drum cutters (with two horizontal axes, EMX-240, made by TONE) with the aim of developing a deep diaphragm wall construction system.

Fig. 4 shows the arrangement of the elements of diaphragm walls and trenches. The soil profile at the measured section is also shown in Fig. 4. An alluvial clay layer of about 30 m in thickness is located at the top of the ground, being underlain by a diluvial sandy layer (Narita layer).

The pore water pressure was measured at trench ⓑ of element No. 1 equipped with the pore pressure meters, U1 (GL-36 m), U2 (GL-42 m) and U3 (GL-47.7 m), as shown in Fig. 4.

The relationship between variations in pore water pressure and excavated depth is shown in Fig. 5. As can be seen in the figure, the pore water pressure increases gradually along with the approach of the excavator.

In order to confirm the cause for the increase in pore water pressure, the following tests were carried out in the excavated trench as shown in Fig. 6.

① At point A (the excess pore water pressure of 4 tf/m^2 at the excavated level of GL-47.7 m, the distance between the drum cutter of excavator and the pore pressure meter U3 (GL-

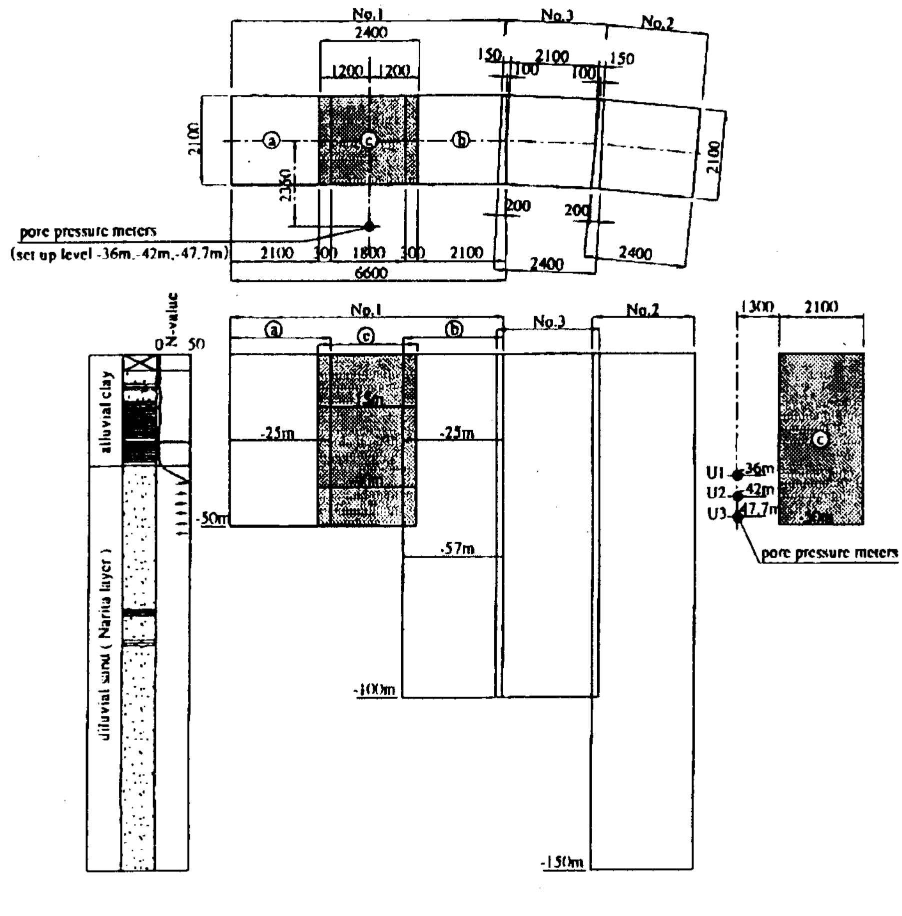


Fig4. Trial slurry-filled trench excavation

47.7 m) is lowest), excavation by drum cutters stopped suddenly. Just after the stoppage, the excess pore water pressure dispersed immediately as shown in Fig. 5.

② At point B, rotation of the drum cutters started without excavation (no loading), lifting up the excavator 70 cm above the excavated level (drum cutter level at GL-47.0 m). Variations in pore water pressure during rotation of the drum cutters was not observed.

③ The excavator was lowered down to the last excavated level (GL-47.7 m) and the cutter thrust was increased to 15 tf. Variations in pore water pressure, however, were not observed.

④ At point C, excavation by the drum cutters started again. Then, a gradual increase in pore water pressure was observed, as can be seen in Fig. 5.

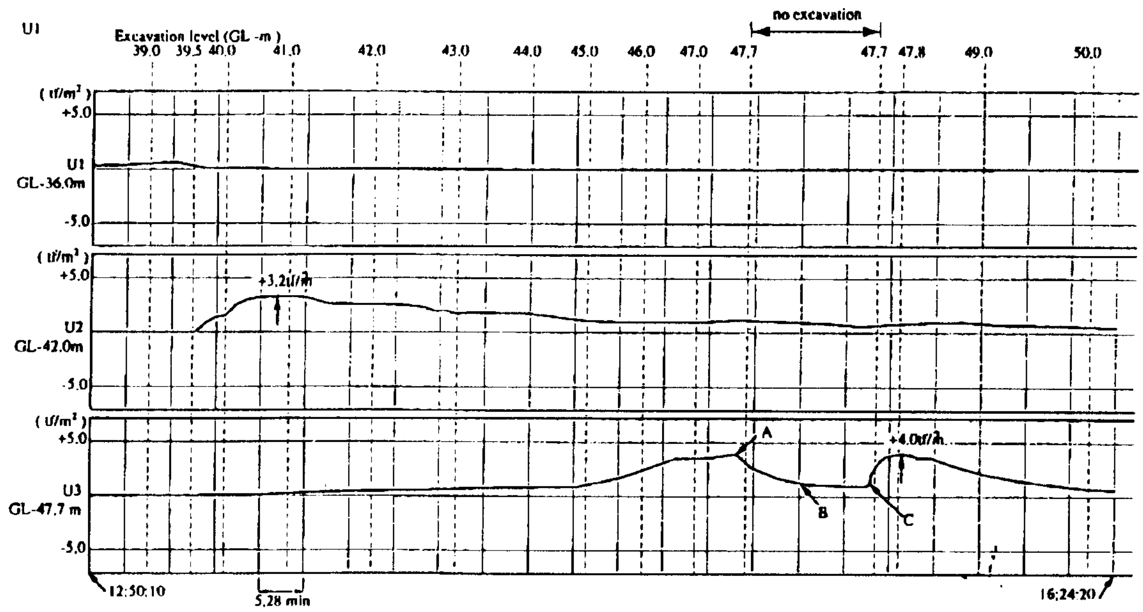


Fig5. Variation of pore water pressure during trench excavation

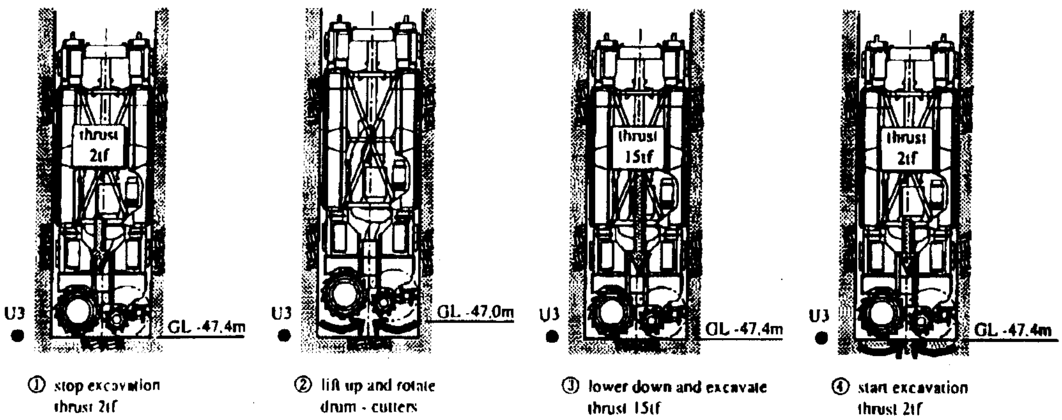


Fig6. Test excavation in the trench

Judging from these tests results, the increase in pore water pressure may be caused mainly by excavation by the drum cutters.

Fig. 7 presents the relationship between the maximum excess pore water pressure (ΔUmax) and the effective overburden stress(σ') under which ΔUmax was mobilized during excavation of trenches ⓐ and ⓑ shown in Fig. 4.

According to Fig. 7, it is confirmed that ΔUmax has a close relationship with σ' (r = 0.983) and that ΔUmax is approximately 10% of σ'.

Fig. 8 illustrates the relationship between the cutter torque (excavated resistance) and the excess pore water pressure. The relationship in the following formula is obtained from Fig. 8.

$\Delta U_{max} = 2.11\ (T/L^2) + 1.41,\ s = 0.65,\ r = 0.709$.........(2)

where,

ΔUmax: max. excess pore water pressure (tf/m³)

T: cutter-torque at ΔUmax (t-m)

L: distance between the center of the drum cutters and the pore pressure meter (m)

s: deviation

r: correlation coefficient

According to Formula (2), the pore water pressure increases with excavated resistance.

5. Experimental Study on the Increase in Pore Water Pressure

5.1. Study Approach

Judging from those field measurements mentioned above, probable causes for the increase in pore water pressure may be as follows:

① Infiltration (permeation) of slurry into the excavated face and consequential ground water flow

② Cyclic excavation by cutter bits and consequential introduction of cyclic shear stress to the ground

In shield driving, a common rotation speed is about 0.5 rpm and an excavation cycle using cutter bits is about 1 rpm for a 2-bit arrangement for one pass. On the other hand, in trench excavation by the drum cutters, a common rotation speed of drum cutters is about 25 rpm and an excavation cycle by use of cutter bits is about 50 rpm for a 2-bit arrangement for one pass. When comparing excavation by the shield machine with that by the trench excavator, the former would be static whereas the latter would be dynamic.

When these excavation conditions and field measurements are taken into account, it would be reasonable to assume as follows: The increase in pore water pressure during shield driving may occur mainly due to infiltration of slurry into the excavated face, while during trench excavation, is mainly due to dynamic cyclic shear stress.

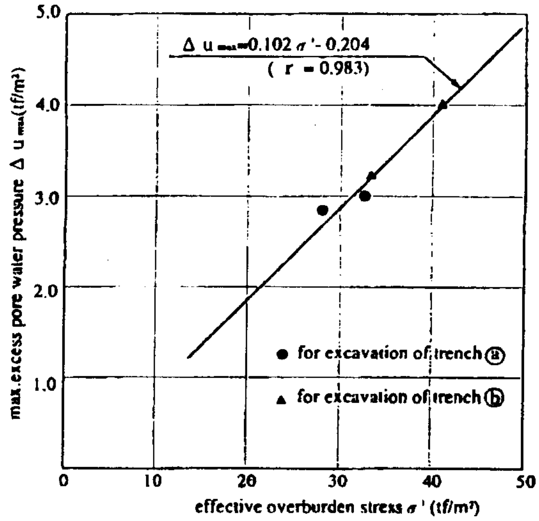


Fig7. Relationship between max. excess pore water pressure ΔUmax and effective overburden stress σ'

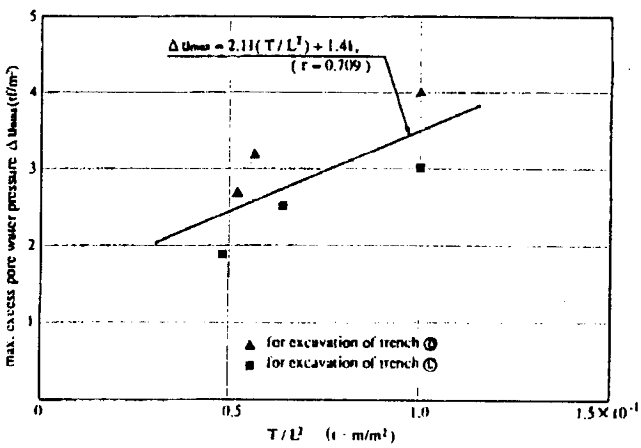


Fig8.Relationship between max. excess pore water pressure ΔUmax and cutter-torgueT

Based on the assumptions thus obtained, the following experimental studies were carried out:
① Slurry-infiltration test using a shield model
② Cyclic shear stress test using on-site soil in which trench excavation was carried out

5.2. Experimental Study on the Increase in Pore Water Pressure during Shield Driving[1]

Distribution of pore water as well as the discharge of ground water accompanying the shield driving were measured using the shield model shown in Fig. 9. Equipment used in these experiments has a mechanism similar to that of an actual slurry-typed shield.

Pore pressure meters were arranged at the side wall of the soil tank as shown in Fig. 10.

Fig. 11 presents the relationship between ground water flow and driving distance during shield driving in Toyoura standard sand, using 12% bentonite slurry at the driving velocity of 1 cm/min and the rotation velocity of 1 rpm with varying slurry pressures (Toyoura standard sand: $e = 0.792$, $\gamma d = 1.566$ g/cm^3, $k = 1.45 \times 10^{-2}$ cm/sec). The ground water pressure around the shield was 0.1 kgf/cm^2.

As can be seen in Fig. 11, when the slurry pressure is set at 0.1 kgf/cm^2, the differential pressure becomes zero and no ground water is observed. Meanwhile, when the slurry pressure is set at 0.2 to 0.4 kgf/cm^2, ground water discharge increases with an increase in slurry pressure.

Fig. 12 illustrates the distribution of pore water pressure during driving obtained from the above-mentioned shield driving test. As can be observed in Fig. 12, the excess pore water pressure increases with an increase in the slurry pressure in Toyoura standard sand.

The distribution of pore water pressure in Toyoura standard sand using 12% bentonite slurry and 1:3:9 slurry is presented in Fig. 13. The figure shows that the excess pore water pressure at the cutting face reduces when 1:3:9 slurry is used.

From Figs. 11, 12 and 13, the following is thus concluded :
① The increase in pore water pressure during driving may be caused mainly by ground water flow.
② The pore water pressure increases with an increase in slurry pressure.
③ The flow velocity of ground water can be controlled by

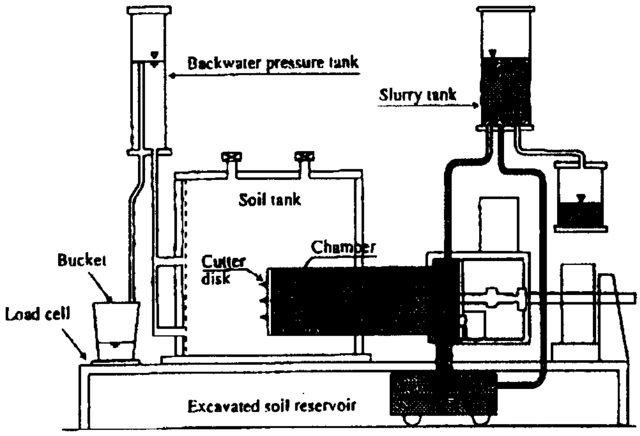


Fig9. Shield Model

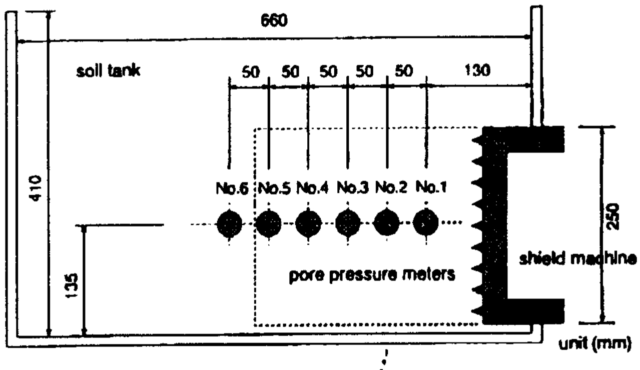


Fig10. Arrangement of pore pressure meters at the side wall of the soil tank

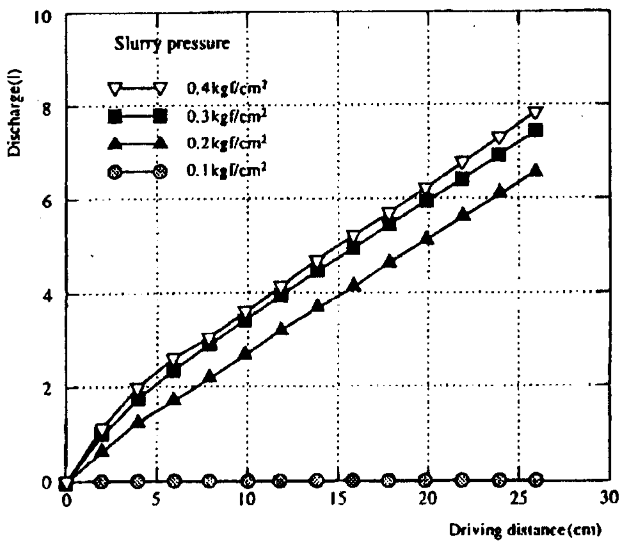


Fig11. Relationship between ground water flow and slurry pressure

changing the slurry seepage velocity through adjustment of slurry pressures and properties.

5.3. Cyclic Triaxial Tests for Trench Excavation[3)]

Cyclic triaxial tests were carried out using on-site sandy soil in which the trench excavation mentioned above was performed.

Fig. 14 illustrates the relationship between $\tau d/\sigma'$ and $\Delta U/\sigma'$, which is obtained from the cyclic triaxial tests. From the figure, the following formula is derived:

(formula) $$\frac{\Delta U}{\sigma'} = 5.16\frac{\tau d}{\sigma'} - 0.368 \quad \cdots\cdots\cdots(3)$$

where

ΔU: max. mobilized excess pore water pressure

σ': confining pressure (overburden pressure)

τd: cyclic shear stress

According to Formula (3), the following can be concluded:

① The excessive pore water pressure increases with an increase in cyclic shear stress, that is, in the excavation torque of the drum cutters.

② The excess pore water pressure increases as the effective overburden pressure increases.

③ The excess pore water pressure is not observed when the cyclic shear stress is small ($\tau d/\sigma'$ is less than about 0.1).

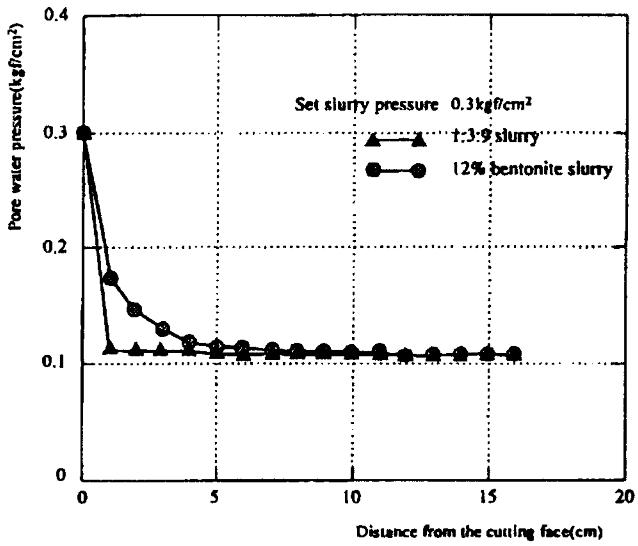


Fig13. Distribution of pore water pressure in front of the cutting face with varying slurry properties

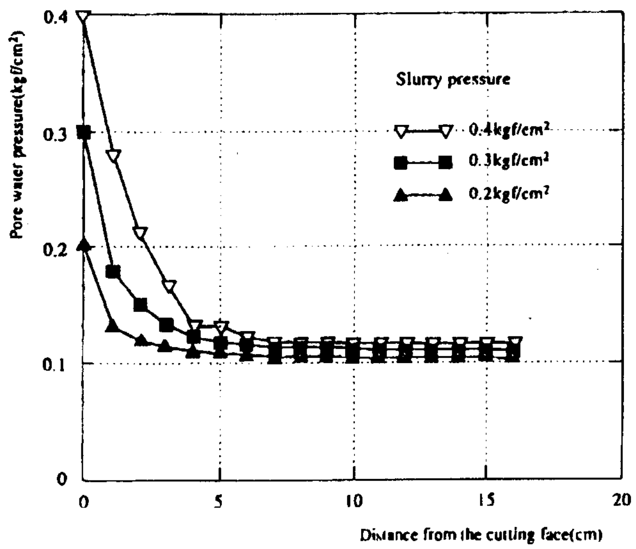


Fig12. Distribution of pore water pressure in front of the cutting face with varying slurry pressure (Toyoura standard sand)

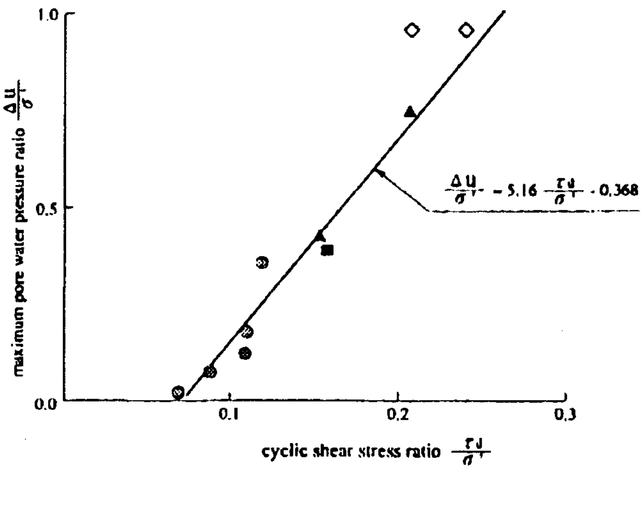


Fig14. Cyclie triaxial test

6. Conclusion

Based on the results of field measurements and laboratory tests thus made on the pore water pressure during slurry-shield driving and slurry-filled trench excavation, the following conclusions can be drawn:

1) The increase in pore water pressure at the cutting face during slurry-shield driving and slurry-filled trench excavation has a great influence on excavated face stability.
2) The excessive pore water pressure during slurry-shield driving may be caused mainly by the ground water flow due to slurry pressure.
3) The flow velocity of ground water can be controlled by changing the slurry seepage velocity through adjustment of slurry pressure and properties. That is, an effective measures to improve the face stability during slurry-shield driving is to control the slurry seepage velocity.
4) The increase in pore water pressure during slurry-filled trench excavation may be caused mainly by dynamic cyclic excavation by cutter bits and by the consequential cyclic shear stress introduced into the excavated cutting face.
5) The maximum pore water pressure ($\Delta Umax$) during trench excavation is expressed as follows:

   $$\Delta Umax = 2.11\ (T/L^2) + 1.41$$

   That is, the excess pore water pressure increases with excavated resistance, namely the cutter torque.
6) An effective measure to improve face stability during trench excavation is to excavate smoothly, that is, to control thrust loading and cutter torque.

References

1) A. Mori, K. Kurihara and H. Mori, 1994, "A Study on Face Stability during Slurry-type Shield Tunneling", Proceedings of International Symposium on Underground Construction in Soft Ground, New Delhi, pp. 261-254
2) Y. Matsushita, F. Yamasaki, Y. Okamoto, T. Hashimoto and H. Imanishi, 1992, "Ground Deformation of Decomposed Granite Soil and Important Factors of Driving Process of Large Diameter Shield Tunnels", Proceedings of Symposium on Underground Construction in Soft Ground, pp. 261-266
3) T. Hosoi, K. Saitoh and T. Nishi, 1994, "An Excavation of Deep and Thick Diaphragm Wall (in Japanese)", Journal of JSSMFE, Vol. 42, No. 3, pp. 41-44

GROUND IMPROVEMENT
HONG KONG COUNTRY REPORT

by

Geoffrey W Lovegrove
MA BAI MSc DIC CEng FICE FHKIE, Director, Mott Connell Limited, Hong Kong

ABSTRACT

More land for development is created in Hong Kong by reclamation than by any other means. In this decade alone, about 1,900 ha of reclamation has been formed for airport, port, housing, transport and other infrastructure uses. Much of this has been done using hydraulically placed sand fill, which has been densified in critical areas after placing to prevent settlement within the fill during the life of the reclamation. Most of Hong Kong's reclamation is formed on areas underlain by soft silty clay deposits of either marine or alluvial origin. Depending on the development programme, this material has either been dredged or left in place and treated to limit consolidation after construction.

This *country report* describes the principal methods of ground improvement used on major reclamation projects in Hong Kong and refers briefly to studies undertaken to determine the effectiveness in Hong Kong conditions of techniques used in other countries.

INTRODUCTION

Reclamation is Hong Kong's principal means of creating flat land for development. About 1,900 ha has been reclaimed during the past 7 years for airport, port, highway, rail, oil storage depots, landfills and new towns. Until the mid 1980s most reclamation was formed using rock and decomposed rock but, with the exception of Chek Lap Kok airport, nearly all recent reclamation has been formed using hydraulically placed sand. This has been dredged from borrow areas in Hong Kong waters and from the estuaries and rivers of the Pearl River Delta. The airport reclamation (940ha) was formed using nearly 120M m^3 of soil and rock quarried from Chek Lap Kok Island (levelled to become part of the airport platform) and nearby smaller islands and 76M m^3 of dredged sand.

The majority of reclaimed areas are underlain by marine and alluvial deposits. Most of the marine deposits and much of the alluvial material comprises silty clay, very soft to soft in the case of the marine deposits and soft to firm in the case of the alluvium. Consequently all reclamations have the potential to settle significantly during and after construction unless either the soft material is removed by dredging or measures are adopted to improve the consolidation characteristics of the foundation. A characteristic of reclamation formed on soft foundations is settlement control using vertical drains and surcharge. Over 15,000 km of vertical drain has been installed in the past 7 years to treat nearly 600 ha of ***drained*** reclamation. The soft deposits have been removed before forming the remaining 1300ha of ***dredged*** reclamation so that construction of infrastructure can commence immediately after the reclaimed platforms are formed.

Over 250M m^3 of hydraulically placed sand has been used to form these reclamations. Below sea level, this has either been bottom dumped (where water depths allow this), pumped into the sea or tipped from purpose built "pelican conveyor" barges, a design peculiar to southern China. Much of the material placed in this manner is loose and would densify if subjected to dynamic loading such as piling vibrations. (Hong Kong is located in a low seismic risk area and suffers only infrequent low intensity seismic events). Therefore, reclamation areas sensitive to settlement are identified during design and the sand fill is densified by vibro-compaction and less often by dynamic compaction.

Environmental pressure to reduce the amount of dredging for reclamation has led to alternatives being sought for increasing the shear strength of soft foundation materials. This has led to an increased use of vertical drains and surcharge. In addition, several studies have been carried out to ascertain whether ground improvement by mixing in situ soft soil with lime or cement is appropriate to Hong Kong conditions (eg Howley 1996). However, although studies have reported favourably on technical grounds, cost has prevented such methods from being used in Hong Kong.

VERTICAL DRAINS AND SURCHARGE

The use of vertical drains and surcharge is well understood and practice in Hong Kong follows that used successfully in other parts of the world. Several points highlighted by Hong Kong experience are worth noting.

Typical post construction settlements (residual settlements) chosen by designers in Hong Kong vary from 50mm to 200mm. The height and/or duration of surcharge period increase as specified residual settlements reduce in magnitude. Thus, the final choice of residual settlement adopted in design of reclamation must be based on a clear understanding by owners of the consequences of adopting a higher (therefore cheaper) threshold for residual settlement. The designer is usually under intense pressure from two sources to authorise removal of surcharge. The *owner* wants it removed to shorten construction time and make land available to end users with consequential cost benefit. The *contractor* wants it removed as soon as possible, either to recover or to give him float in his programme. Therefore reliable methods are required to ascertain when settlements have reached a stage at which residual settlement will not exceed the design limits. Major reclamation practitioners in Hong Kong have developed rapid analytical techniques based on curve fitting observed settlements to the total calculated vertical strain of foundation strata. Failure to estimate the magnitude of residual settlement reliably can lead to excessive settlement during the life of the reclamation with high consequential maintenance costs. Finite difference analysis was used on West Kowloon, North Lantau Expressway and Tung Chung New town reclamations to estimate consolidation settlement of the foundation from excess pore pressures generated during loading. This was done using a range of parameters (coefficient of consolidation and compressibility ratio) and the results compared with actual settlements measured by extensometer (Figure 1). This allowed realistic estimates of in situ soil parameters to be made which then were used (Figure 2) to predict total and target settlements in the form of time/settlement curves for given sets of ground conditions and residual settlement limits. Thus the earliest time compatible with residual settlement requirements when surcharge should be removed could be estimated.

Great care must be taken to place the first layers of fill so that the upper part of the foundation is not disturbed, creating mud waves. Bulk movements such as these will remould the in-situ material, thus reducing the coefficient of consolidation and lengthening consolidation times. Also bulk movements will move initial settlement marker plates set on the seabed. These identify the zero settlement level prior to the start of reclamation which is important for curve fitting analysis of settlement data. The plates are carefully surveyed in place so that they can be found subsequently by drilling through the completed reclamation. If these plates are lost through mass movement of foundation material, then the origin of the settlement time curve is also lost and curve fitting as a method of predicting surcharge removal becomes less accurate. Reliable methods of placing plates on the seabed have been developed for water depths up to 20m and also of subsequently relocating them when filling is finished. Other settlement monitoring equipment (such as extensometers installed in fill and foundation layers) is installed after reclamation rises above high water level and drilling rigs can be mobilised to site. By this time the risk of losing or damaging the equipment during filling is greatly reduced.

Recent reclamation projects constructed as part of the Airport Core Programme (Chek Lap Kok Airport platform, Tung Chung New Town, North Lantau Expressway, West Kowloon Reclamation, Central Reclamation Stages 1 and 2) have been carried out on fast track programmes. There has been a lot of construction vessel traffic in the reclamation areas in addition to dredging and filling and, as a result, there has often been a significant amount of fine material in the water column close to dredging and filling areas. Experience has shown that substantial sedimentation can occur in these situations with material accumulating in low areas during construction and creating layers and lenses of mud which become trapped in the reclamation. These will consolidate relatively quickly during surcharging and, if they are not known about, can distort time/settlement plots generated during monitoring. An engineer experienced in curve fitting analysis for certain sets of local conditions can quickly see such anomalies and order confirmatory cone penetration testing to determine the extent of fill contamination.

DEEP COMPACTION

Compaction of granular fill materials carried out in Hong Kong falls into the two categories - deep compaction using vibrating probes and dynamic compaction by dropping weights. Both techniques have been used successfully in many parts of the world. Two types of vibrating probe have been used recently in Hong Kong - a water assisted down-hole vibrator jetted into the sand fill, and a cruciform pile (roughened and with slots and holes cut in its fins) with a top mounted vibrator. Both methods have been successful but vibrating probes tend to be used more frequently.

Vibrating Probe

A down hole vibrator (Figure 3) is typically 3m long, 400 to 500mm diameter with 150 to 200mm fins at its lower end. Extension sections are attached to the vibrator unit to add weight and protect power and water supply lines. The densification process entails inserting the probe with water flush under its own weight to the bottom of the sand fill at which stage vibration is commenced. The probe is then withdrawn slowly, with water flush and vibration remaining in operation, pausing every half metre for pre-determined times or until power consumption reaches a certain maximum value. The horizontal spacing between probe locations, vertical intervals, vibrating duration and maximum power at each interval are determined by site trials carried out in advance of the main compaction work. These variables will depend on the quality of sand (fines quantity and angularity), the type of probe and end result specified. Probe spacings at West Kowloon and Tung Chung New Town reclamations varied from 2.8 to 4.5m depending on the standard of densification required, with maximum power consumption of about 200 amps and time intervals at each pause in withdrawal of 40 to 60 seconds. Depressions form around probe holes as the sand densifies and these are kept filled with sand from the reclamation.

Vibrating piles (Figure 4) used in Hong Kong comprise cruciform units up to 20m long and 2m wide. The pile is vibrated into the fill and retracted in stages using a variable frequency vibrator mounted at the top of the pile. The frequency, rate of withdrawal and spacing between probe locations are determined by prior site trials. Piles can be grouped in units of up to four, the controlling factor being the weight and vibration power available. Multiple pile units have been used but are practical only if the thickness of material to be densified is 10m or less.

Quality control for both techniques is achieved by Cone Penetration Testing (CPT) carried out before and after vibration (Figure 5). Densified sand ages due to pore pressure dissipation, creep and possibly cementation, therefore it is common to specify that check CPTs should be carried out a minimum time after vibration. Common Hong Kong practice is to specify testing seven days after compaction. The effectiveness of vibration is very dependent on sand quality. Contamination of the fill, either by fine material brought to site as part of the sand supply or, by silt trapped in the fill during reclamation, can seriously reduce the densification obtained. That is not to say that the fill is unacceptable, but other techniques such as surcharging for short periods might be necessary to give a fill material which will not strain vertically under either applied static or dynamic load.

The most frequent location of silt contaminated fill is in the lowest 1 to 2 metres of a reclamation because sand placed on very soft silty clay will mix with the top of that material. Also, if the marine deposits have been dredged prior to filling, a semi fluid sediment up to several metres thick can collect in the bottom of the dredged void before filling commences and this is very difficult to remove completely. Vibro compaction will be of little use in such conditions and short surcharge periods are a more realistic solution (in addition to vibration of non-contaminated fill).

Shelly sands have their own specific characteristics which can vary greatly according to the size, shape and quantity of shell fragments present. In general, the presence of shells can dampen out vibrations within the sand and cause a reduction of compactive effort. In addition shelly particles do not flow into dense configurations as easily as mineral sand due to their shape. This makes determination of acceptance criteria difficult.

CPT tip resistances after compaction specified in Hong Kong range from 7.5 to 15MPa. Lower values are chosen for urban road networks and higher values for foundations for culverts, etc. Acceptance tests are carried out typically at 1 test per 2,500m^2 of treated area for the lower range of specified tip resistance. This is increased (up to 16 times) in areas of great sensitivity where higher acceptance criteria are specified.

Dropping Weight

This technique is used less frequently in Hong Kong than vibro-compaction. It has been used on reclamations for oil tanks (totalling about 17ha) and a small area (less than 2ha) at Chek Lap Kok airport. Dynamic replacement, with associated dynamic compaction, has been carried out using a 35t weight falling through 20 to 40m on grids between 6.4 and 11m spacing with a total energy generated between 290 and 500 t-m per m^2. A crater formed at each location in the first pass was filled with rockfill and pounding/feeding of rockfill continued until the required improvement had taken place.

CPTs (Figure 6) showed 10 to 15 times improvement in the fill to a depth of 7m (q_c up to 30MPa), 1.5 times improvement to 20m (q_c 10 to 14 MPa), with little perceptible improvement in tip resistance below that. Vibrations measured on sandfill 11m from 700 t-m pounding were 13 to 22 mm/s and 33m from 1400 t-m pounding were 12 to 14 mm/s peak particle velocity.

OTHER TECHNIQUES

The feasibility of using deep chemical mixing (DCM) in Hong Kong as a means of improving the shear strength of very soft to soft marine silty clay has been studied. There is no mineralogical or chemical reason why DCM should not be successful. Unconfined compressive strengths of between 0.7 and 1.5MPa could be expected with the addition of 150 to 250 kg/m^3 OPC and 1.25 to 3.7MPa with the same quantities of special cements. These strengths make it viable for example to build seawalls on marine deposits improved by DCM, albeit at a relatively high cost.

ACKNOWLEDGEMENTS

The author is indebted to the following organisations for their assistance in providing information on current ground improvement practice in Hong Kong:

Organisation	*For information on*
Airport Authority Hong Kong and Maunsell Consultants Asia Ltd	940ha of recent reclamation
Binnie Consultants Ltd	50ha of recent reclamation
Maunsell Consultants Asia Ltd	100ha of recent reclamation
Mott MacDonald Hong Kong Ltd	580ha of recent reclamation
Scott Wilson (HK) Ltd	350ha of recent reclamation
Bachy Soletanche Group	vibro compaction
B & B Ground Treatment Ltd	vibro compaction
Penta Ocean Construction Co Ltd	vibro compaction
Penta-Wai Kee Joint Venture/ Scott Wilson (HK) Ltd	dynamic replacement
Mott MacDonald Hong Kong Ltd/ Swedish Geotechnical Institute	deep chemical mixing
Geotechnical Engineering Office	deep chemical mixing

Sub-layer Settlement Prediction

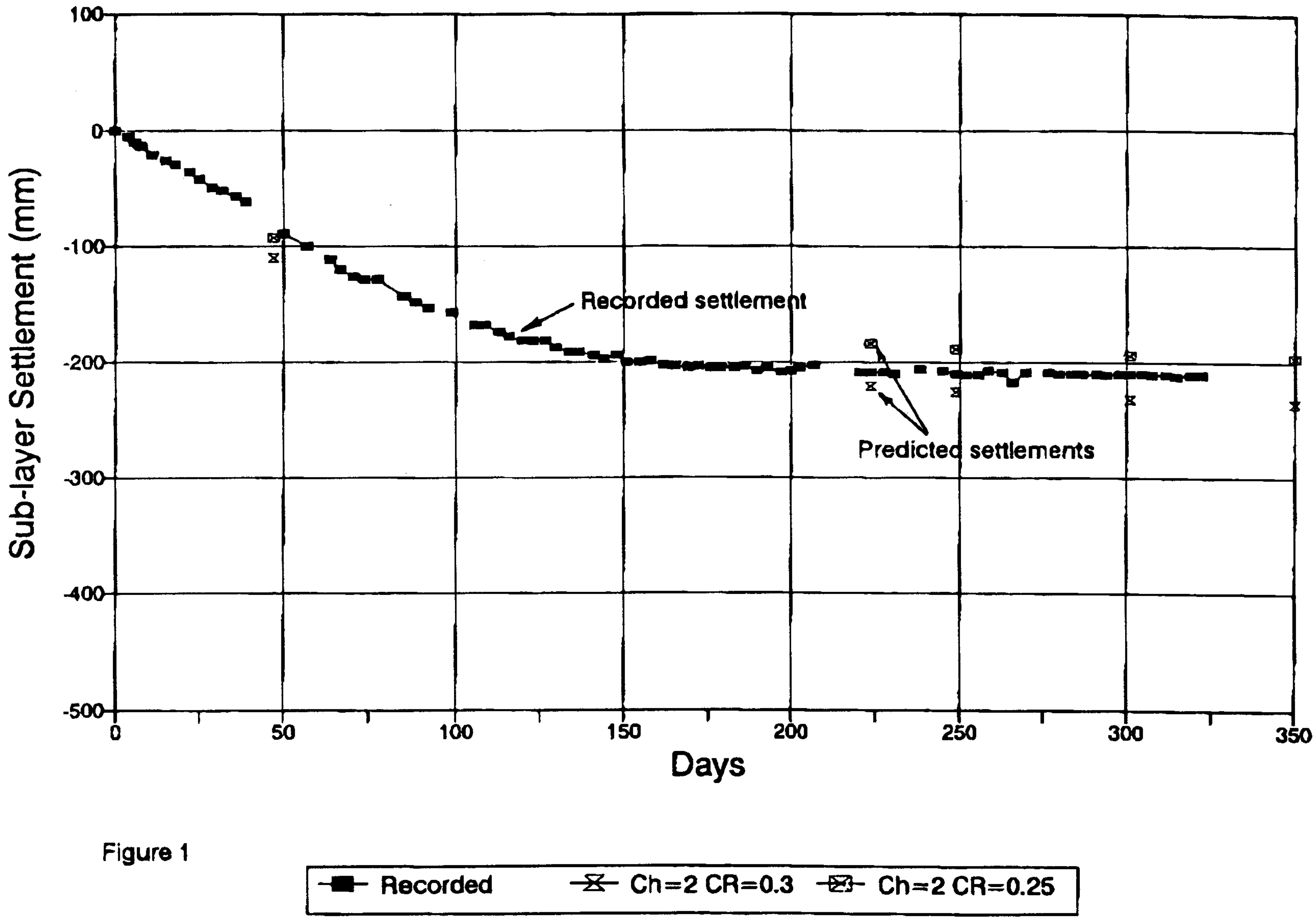
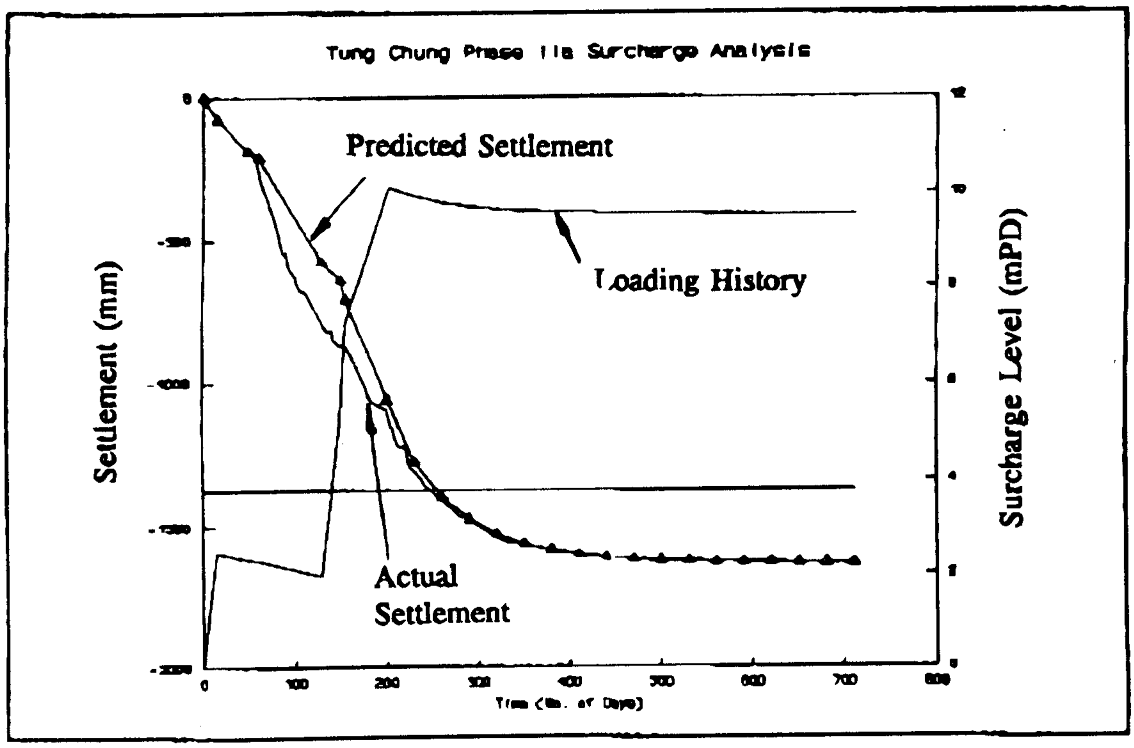


Figure 1

Figure 1

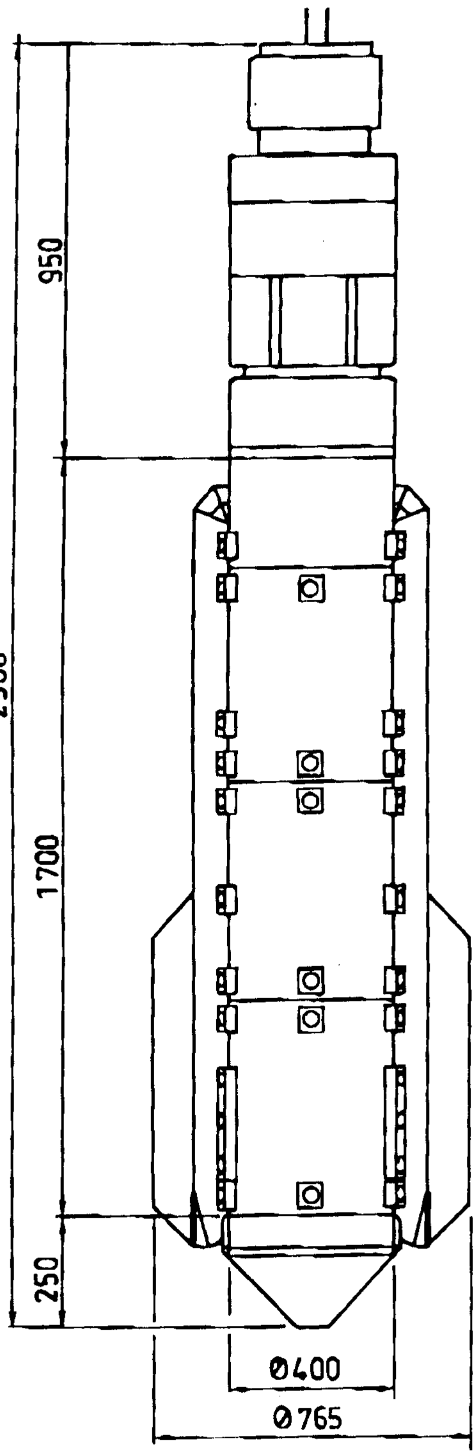


Cv = Ch = 3.00 CR = 0.300

Back Analysis of Actual Settlements

Figure 2

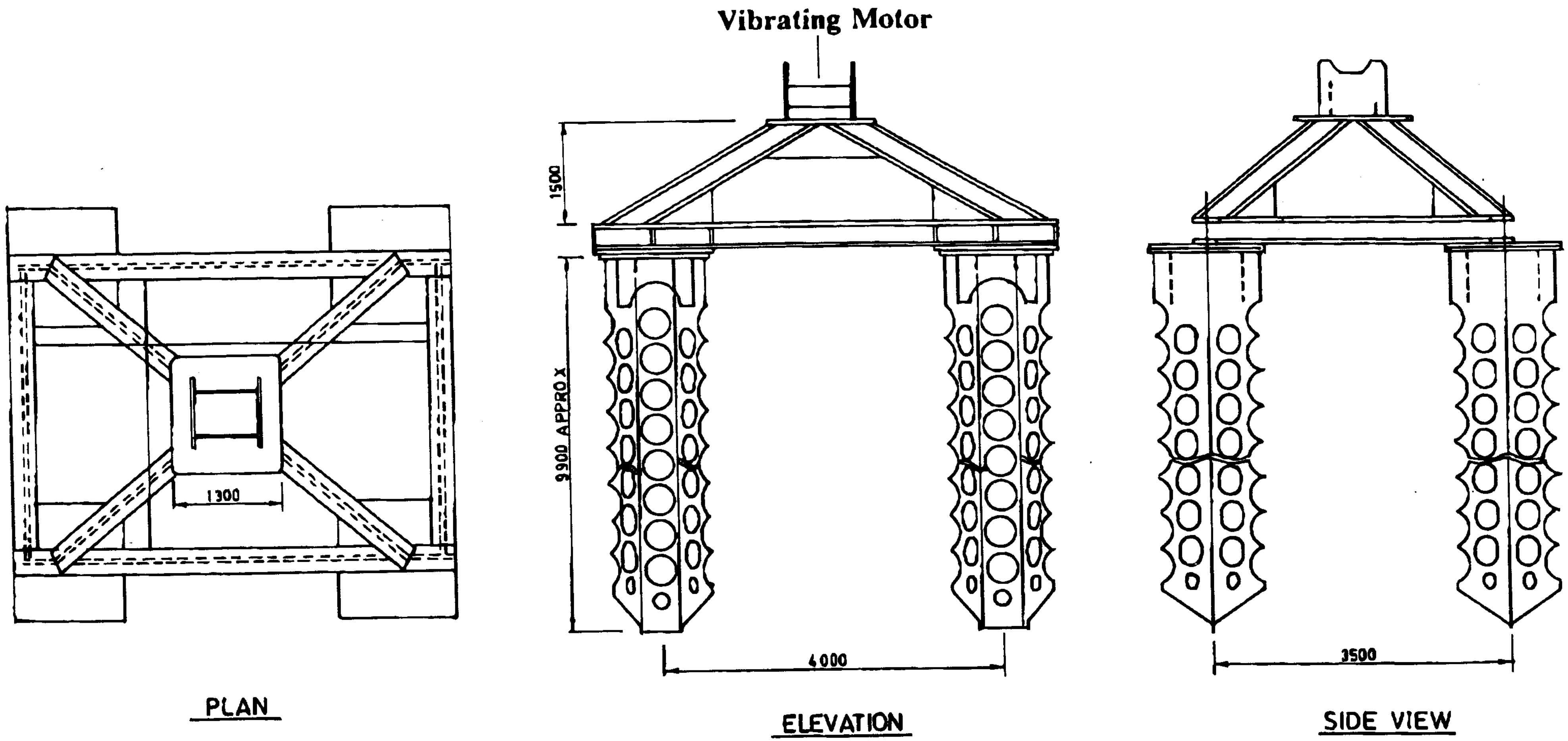
Vibrating Probe



Frequency	30 Hz
Centrifugal Force	290 KN
Amplitude	25 mm
Drive	Electric Motor
Motor Capacity	150 KW
Motor - Voltage	380 V
Motor - Rating	60 cps
Motor - Speed	1775 rpm
Power Cable	3 x 70 + 1 x 35 mm^2 Length 50m
Service Weight	2450 kg
Transport Weight	2600 kg

Figure 3

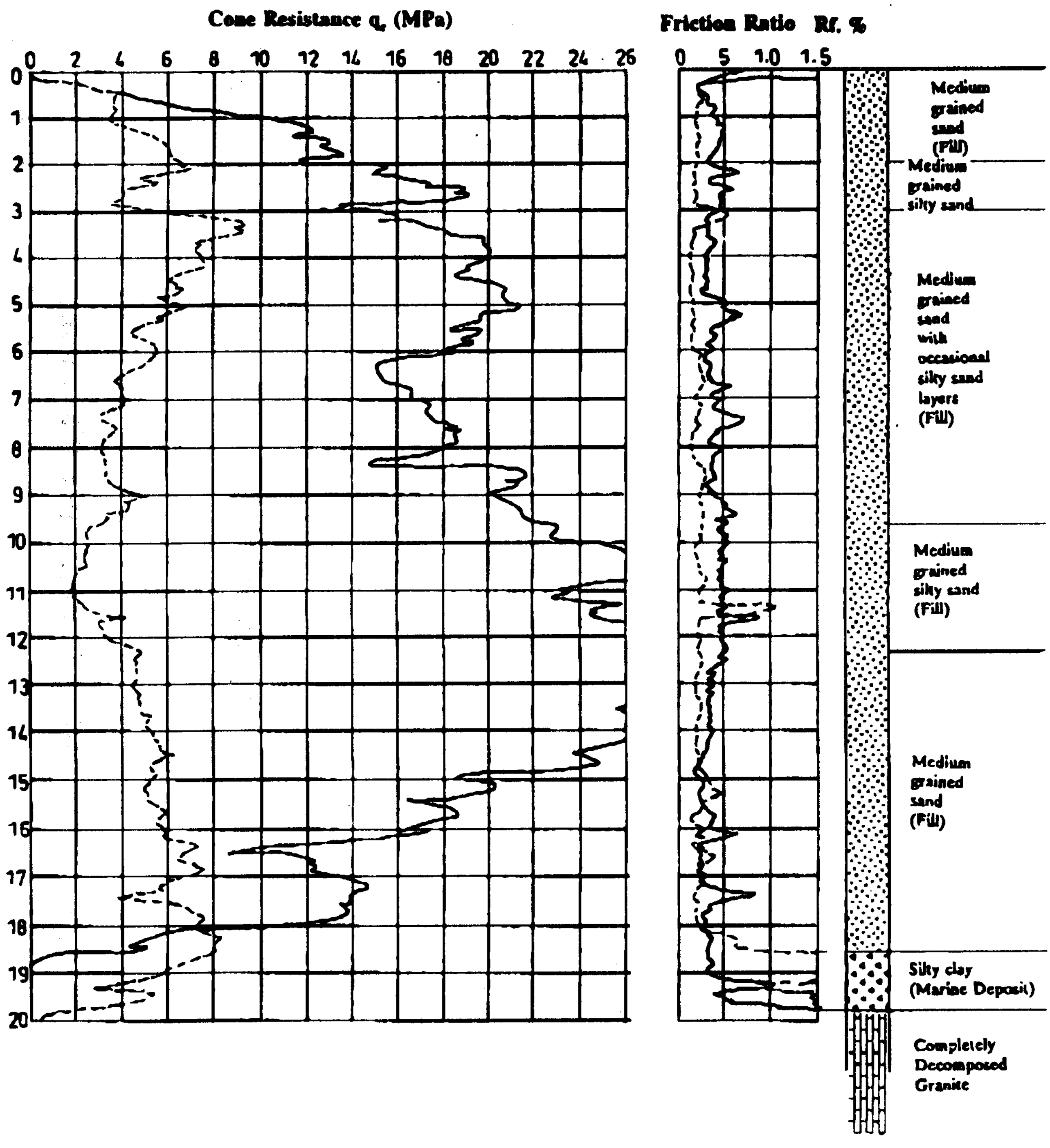
Courtesy of
Keller Grundbau GmbH



Multiple Vibrating Compaction Pile Probe

Figure 4

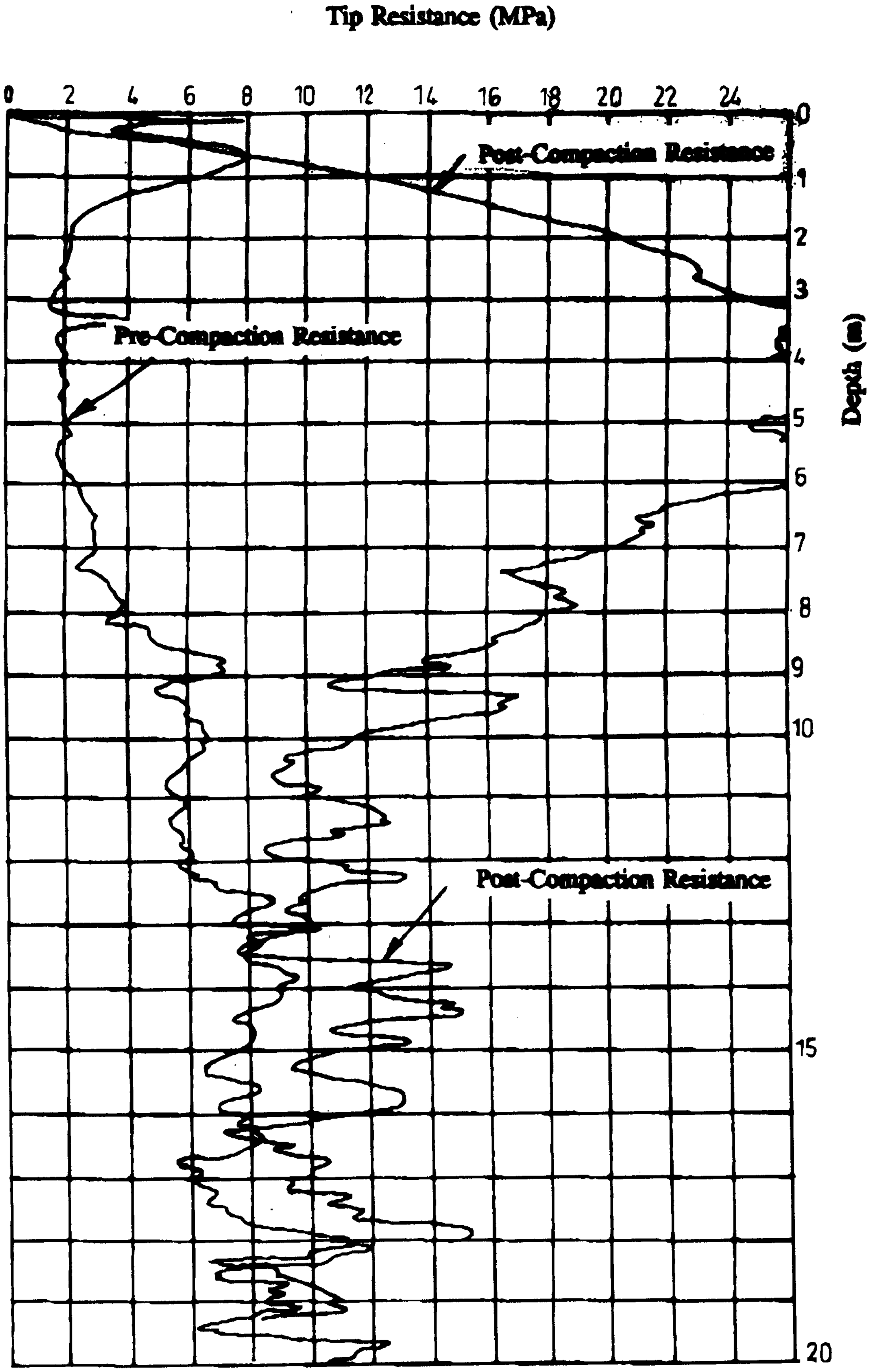
Courtesy of B+B Ground Treatment Ltd



Piezo Cone Penetration Test

Key: ----- Pre-Compaction
____ Post-Compaction

Figure 5



Comparison of Pre- and Post-Dynamic Compaction CPT Tip Resistances

Figure 6