The design and performance of soil improvement solutions in a deep collapsing residual soil

L'étude et le bilan des solutions techniques de consolidation de couches profondes de sols résiduels métastables

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ABSTRACT: The stratigraphy, geotechnical characteristics and behaviour of a deep mantle of highly cemented residual sandstone of Pliocene age on the southern Moçambique coastal plain are described. The performance of both soil improvement and deep foundation solutions at natural moisture content as well as "wetted-up" conditions founded within the collapsing soil fabric are outlined. The results of three soil improvement test areas as well as an extensive trial pile test programme for a large Aluminium Smelter Complex are evaluated and discussed. The key issues of foundation choice and performance where metastable soils are encountered are put forward. The changes in soil moisture regime beneath the Smelter site due to changes in the boundary conditions as well as extreme climatic events are tabulated and discussed. Recommendations on the design of foundations in the unique conditions encountered on the Smelter site are given and conclusions are reached on controlling critical aspects of foundation performance in this problematical soil.

RÉSUMÉ: Description de la stratigraphie, des caracteristiques geotechniques et du comportement d'un epais manteau de gres residuel datant du Pliocene et situe en plaine cotiere du Mozambique Sud. Revue de la performance de techniques d'amelioration du sol et de-fondations profondes autant dans l'ambiance humide naturelle qu'en conditions accentuees d'humidite du sol instable. Evaluation des esultats d'amelioration de sol obtenus sur trois zones d'essai et de tout un programme d'epreuve de pieux pour la construction d'un centre important de production d'aluminium. Elaboration des enjeux gouvernant le choix d'un type de fondation et de sa performance dans un sol metastable. Analyse des changements de taux d'humidite sous le site du complexe resultant de changements aux conditions peripheriques et de conditions climatiques extremes. Recommandations pour l'etude de fondations dans les conditions uniques de ce chantier et conclusions pour le controle des aspects critiques du comportement des fondations dans cet environnement problematique.

1 INTRODUCTION

The feasibility of constructing a large Aluminium Smelter Complex near Maputo, southern Moçambique was investigated in July 1996 and a 110 hectare site adjacent to the Rio Matola approximately 20km from the Maputo CBD was chosen. A preliminary geotechnical investigation in June 1997 indicated the presence of surficial soils exhibiting a collapsing grain structure and thus a shallow piled foundation solution was envisaged. Further detailed geotechnical investigations and trial pile testing carried out at the end of 1997 confirmed the adequacy of the shallow piled foundation solution as well as the efficacy of Dynamic Compaction as a ground improvement technique. Evidence of significant reduction in pile capacity and stiffness under wetted-up conditions became apparent. The contract for the construction of approximately 9500 driven cast-in-situ piles with an enlarged base, 300 large diameter bored piles and 85000m² of Dynamic Compaction commenced in June 1998. Further detailed investigation and trial pile testing was carried out during the Contract and the nature and depth of the metastable red soils underlying the southern Moçambican coastal plain was revealed. A critical issue in the design of foundations was the extent with depth of the collapsing horizon across the site for the piled foundations as well as the depth below NGL of the highly cemented collapsible red soils for the effectiveness of the Dynamic Compaction Solution.

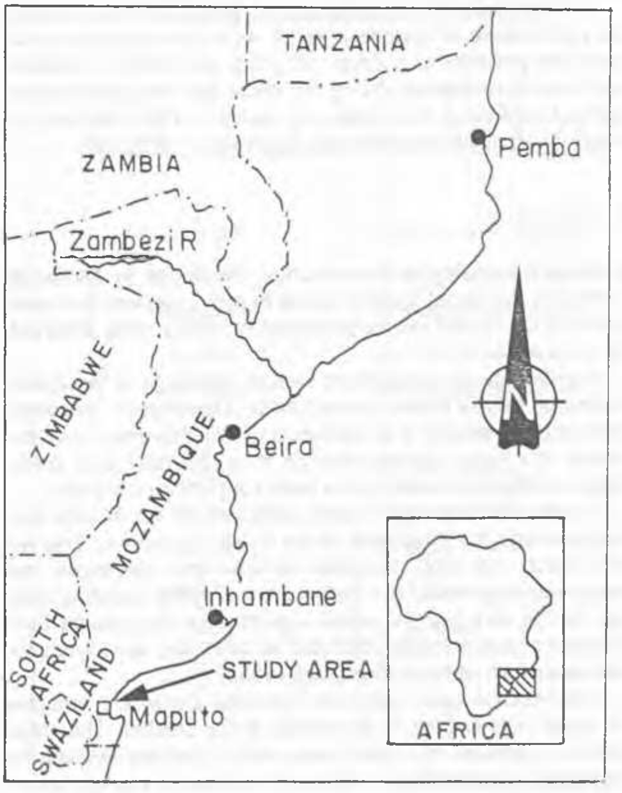

Figure 1

2 GEOTECHNICAL INVESTIGATION

The Smelter site adjacent to the Matola river shown in Figure 1 was chosen on the basis of a preliminary investigation of the site comprising rotary cored boreholes with S.P.T's at 1,5m intervals carried out in June 1996.

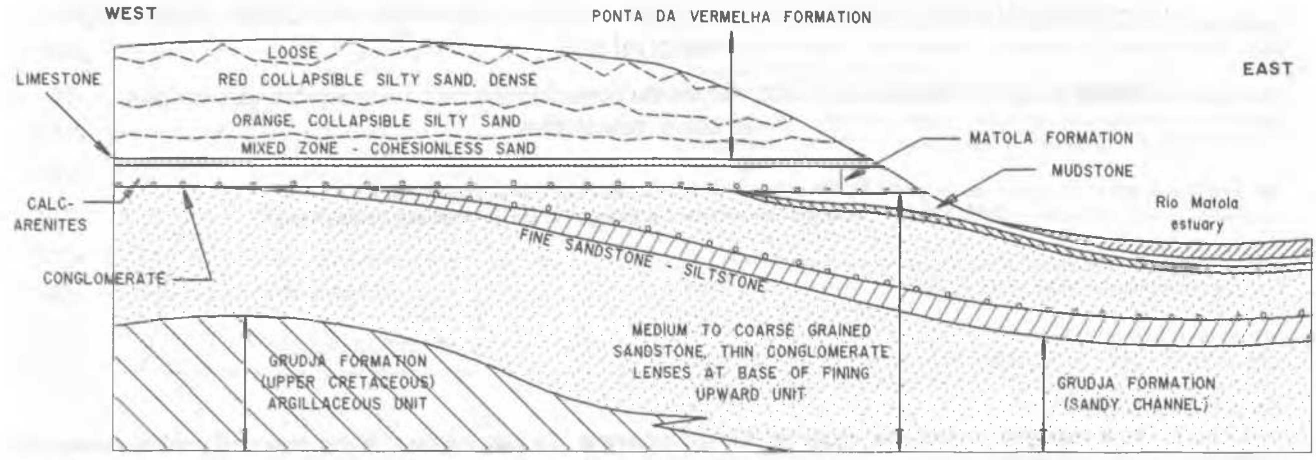

Figure 2

A more detailed intermediate investigation was carried out in July 1997 and comprised a combination of rotary-cored boreholes with SPT's, dynamic cone penetration tests, testpits and in-situ plate load tests. All these tests were carried out on the soils at their natural moisture content.

A detailed geotechnical investigation was carried out at the end of 1997 prior to the construction phase with the objective of optimizing the foundation solutions for the Smelter. The investigation comprised the augering of large diameter trial holes, the drilling of rotary cored boreholes into Cretaceous Bedrock, and the execution of vertical plate load tests at both natural moisture content and wetted up conditions. These tests were carried out up to depths of 12m below N.G.L. to ascertain the characteristic of the soil at anticipated founding depths of piles for the Smelter structures. The investigation also comprised a trial pile and soil improvement test programme at two locations on the 100 hectare site.

The tactile inspection of the undisturbed soil fabric as well as the performance of plate tests under wetted up conditions indicated the presence of a deep collapsing soil profile. Extensive additional investigation during the piling and soil improvement contract confirmed the extent and severity of the problem red Ponta Da Vermelha residual soils encountered on the site.

3 GEOLOGY

A schematic geological cross-section, developed by McKnight (1998) for the Mozal Smelter site in Figure 2 outlines the subdivision of the coastal marine sediments into three units, from eldest to youngest.

Argillaceous and arenaceous marine sediments of the Grudja Formation overlie Karoo volcanic rocks. These upper Cretaceous sediments are present at an elevation of 3m above sea level and consist of a lower siltstone overlain by a 15m thick fine to medium sandstone sequence with a basal conglomerate horizon.

Younger Tertiary aged littoral sediments of the Matola Formation overly the sandstones of the Grudja Formation. This approximately 6m thick succession of limestone, calcarenite and conglomerate provided the shallowest acceptable founding horizon for the very heavily loaded structures on the Smelter Site. Diagnostic macro-fossils identified in this zone are Pecten sp. and Amussium umfulozianum King (1953).

The Pliocene aged Ponta da Vermelha Formation occupies the upper 10m to 20m of the profile at the Smelter. This thick horizon is typified by a basal weak rock calcarenite overlain by completely weathered very weak rock medium to coarse-grained sandstone. The upper zone is typically iron-rich, orange and red/brown in colour, dense, cemented, silty, medium and fine sand. It is likely that these residual soils of the Ponta da Vermelha Formation in southern Moçambique represent shallow marine sediments that formed due to the reworking of extensive aeolian sand dunes during the major marine transgression of the early Pliocene King (1972). The weathering of these soils has resulted in leaching of the soil fabric, evidenced by the open structure of the soil skeleton. The cementing of the soil skeleton is due to the deposition of iron-oxide compounds. Repeated wetting and dessication of the soil has resulted in apparent overconsolidation. The metastable characteristics of the residual soil is apparent over the full soil profile and significant collapse potential is evident up to 16m below natural ground level. A typical cross section through the residual soil horizon gives a characteristic distribution of the potential for collapse settlement as defined by Jennings & Knight (1975) and is shown diagrammatically in Figure 3.

The uppermost horizons of the cemented red sands have undergone podsolic weathering during the Quaternary and consist of an irregular zone of loose, compressible, slightly clayey fine and medium sand. This uncemented horizon is generally 2m to 3m thick across the site with localised zones up to 10m in thickness.

4 SOIL CHARACTERISTICS

The red soils of the Ponta da Vermelha Formation represent reworked residual aeolian sands and are typically silty fine and medium sands with a low clay content in the upper 6m. The particle size distribution falls within a normal aeolian sand grading envelope but there is a significant medium and coarse sand fraction which distinguishes the red soils of the region from typical aeolian deposits.

Extensive laboratory testing carried out over the full depth of the red soil profile on the Mozal Smelter site showed remarkable uniformity of some of the physical parameters of the residual soils. Dry density remained relatively uniform between 1600 - 1750kg/m³ up to a depth of 16m and void ratio similarly remained between 0,45 and 0,55. Desiccation over the full profile was noted with a gradual soil moisture increase from an average 3% near the surface to 6% at a depth of 16m. The variation with depth of the principal characteristics is shown on Figure 4a-c.

The most significant physical parameter of the soil skeleton is the potential for collapse settlement to occur on load application when "wetting-up" of the naturally dessicated soil fabric drastically reduces the strength of the iron cementing agents. The variation of collapse potential with depth in the profile is shown graphically in Figure 4d where it is clearly evident that the distribution and magnitude of collapse potential is highly variable. The in-situ moisture content is directly related to the magnitude of collapse potential and this is clearly evident where the trend of decrease in collapse potential with depth is reflected by a corresponding gradual increase of in-situ moisture content with depth.

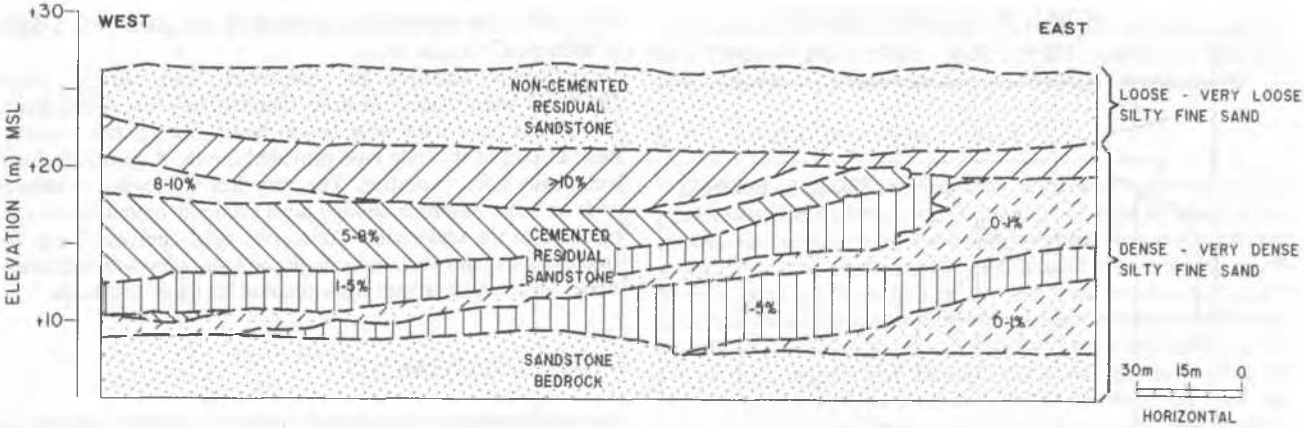

Figure 3

Both moisture content and collapse potential are however erratically distributed.

It is widely accepted that full collapse of the soil fabric occurs at a critical moisture content when dissolution and softening of the cementing agents occurs. Using the relationship of % passing the 0,075mm sieve versus degree of saturation developed by Schwartz (1984), the critical moisture content for the red Ponta da Vermelha soils is approximately 6-7%. The potential for large collapse settlement to occur over the full profile is therefore the most significant problem associated with these red soils, which are drier than the critical moisture content to depths in excess of 18m.

The magnitude of collapse potential as well as the marked reduction in the shear strength of the wetted up soils is clearly illustrated by Figure 5 showing alarming reduction of cone point resistance after wetting. Similar drastic reduction was noted in post "wetting-up" standard penetration test (SPT) values. The upper zones of the Ponta da Vermelha Formation (upper 12m) can be classified as potentially highly collapsing with collapse potentials in the range 4-13%. The remainder of the profile is potentially moderately collapsing with collapse potential in the range 0-4%. From Figure 5d it is clearly evident that the risk of collapse settlement occurring is present over the full soil profile down to a depth of 18m. This risk is further enhanced by the dessicated nature of the residual soils with recorded natural moisture contents well below the critical value of 6,5% down to bedrock.

The uppermost horizon of loose uncemented sand is characterised by a compressible rather than collapsible soil fabric. Standard penetration test (SPT) values in the range 3-10 indicate low shear strength and highly compressible soils.

5 FOUNDATION OPTIONS

The Smelter Complex required foundation solutions for a wide range of structures from light administration buildings to very heavily loaded silo storage facilities. Many of the structures are extremely settlement sensitive and both soil improvement as well as deep foundations were considered for the project.

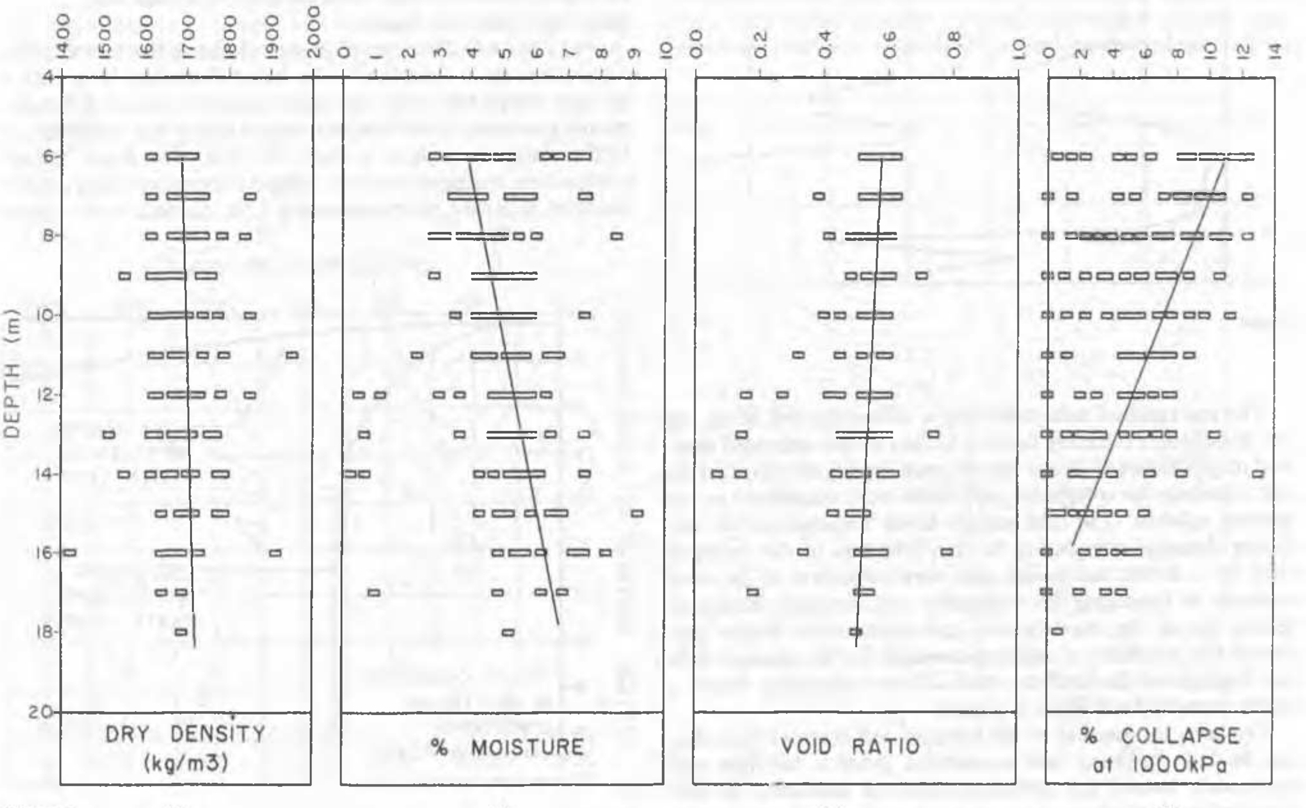

Figure 4 (a) (b) (c) (d)

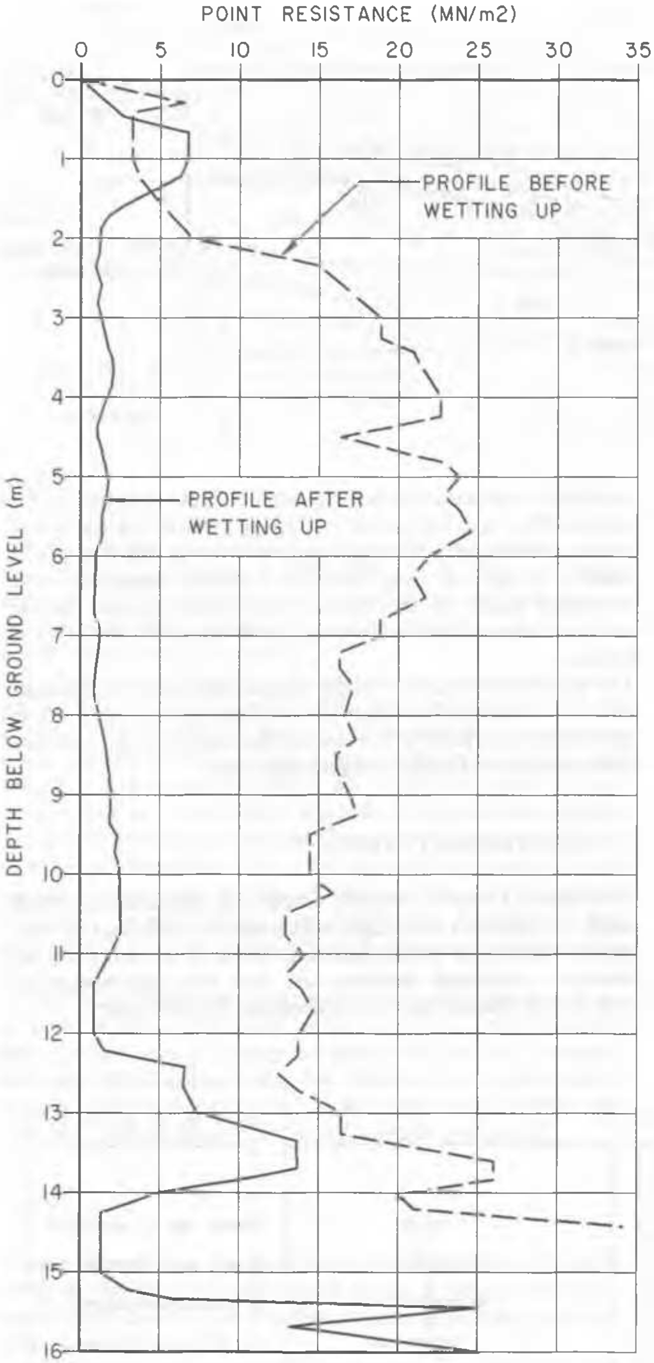

Figure 5

The red residual soils exhibiting a collapsing soil fabric are not suitable as a founding horizon in their in-situ untreated state. Soil improvement solutions which could densify the in-situ soils and eliminate the collapsible soil fabric were considered as the optimal solution. The high energy levels imparted to the soil during dynamic compaction or the formation of the enlarged base for a driven cast-in-situ pile were considered to be ideal methods of improving the metastable soil structure. Extensive testing during the investigation and construction stages confirmed the suitability of soil improvement for the residual soils but highlighted the ineffectiveness of the techniques where a highly cemented soil fabric is present.

Dynamic compaction of the surficial soil horizons was chosen on both technical and economical grounds for light and moderately loaded non settlement-sensitive structures. It also provided optimal solutions for on-grade surface beds and loaded floor slabs. The solution was adopted for structures with a depth of influence less than 5m.

Settlement sensitive and moderately and heavily loaded structures were placed on deep foundations comprising driven cast-in-situ piles with an enlarged base. The ground improvement effects of this pile type provided the most economical and technically robust solution. The very heavily loaded structures such as large Alumina storage silos required foundations to be placed into the underlying Cretaceous aged bedrock. Large diameter bored piles socketed into the soft and very soft rock sandstones and conglomerates were adopted for these structures.

6 SOIL IMPROVEMENT

Soil improvement solutions have been successfully adopted on both residual and transported soils exhibiting a collapsing grain structure in the southern African region. Dynamic Compaction in the form of Impact Rolling, Rapid Impact Compaction and Heavy Tamping provide an effective and economical method of improving a wide range of potentially collapsible soils. These methods were therefore chosen for evaluation during the investigation phase on the red sands of the Ponta De Vermelha Formation at the Smelter site.

6.1 *Compaction trials*

Two areas representing typical soil profiles beneath The Smelter site were selected for the evaluation process.

At each test site the area was divided into compaction zones for the three compaction techniques representing ranges of applied energy levels associated with equipment available in the region. Soil strength and stiffness at natural and increased moisture content were measured beneath each area pre and post compaction. The increase in soil strength and stiffness was measured for each of the compaction processes and the depth of influence of each technique at optimal energy levels was assessed.

Significant densification within the upper uncemented horizons was achieved with each method of compaction. The significant improvement of soil modulus in wetted-up conditions is graphically shown in Figure 6.

The 25kJoule three sided Impact Roller achieved effective compaction up to a depth of 1,5m below formation level with a 30 pass compaction while the higher energy levels of 8,5 tonne metres generated by the Rapid Impact Compaction achieved effective compaction up to a depth of 3,5m. The Rapid Impact Compaction technique involves surface compaction using a track mounted hydraulic plate compactor, 1.5m diameter with a mass

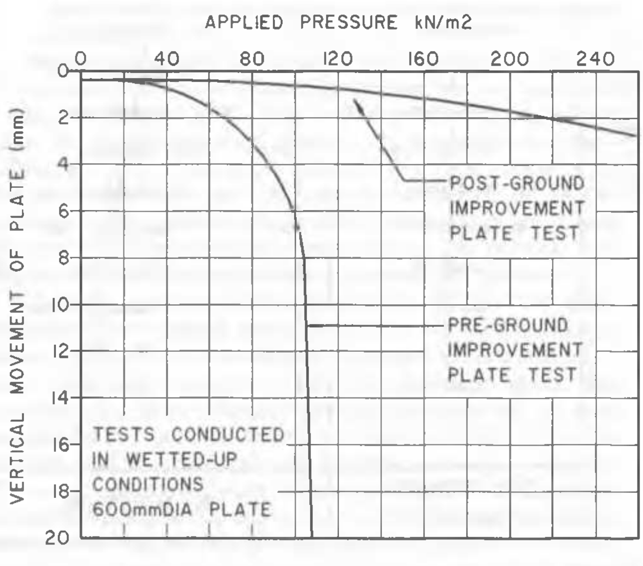

Figure 6

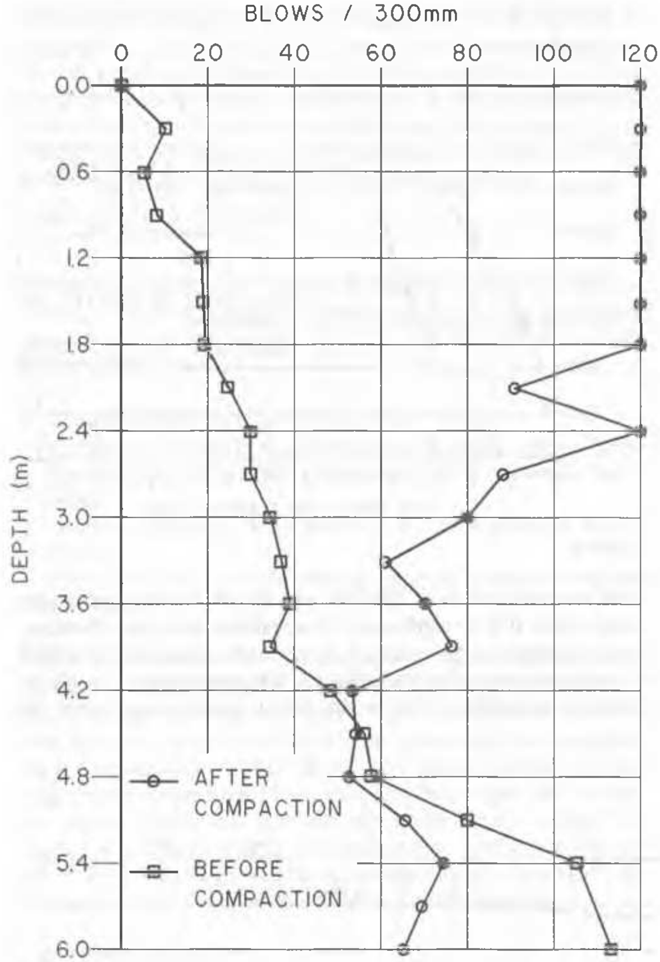

Figure 7

of 7 tonne and stroke height of 1,2m. The high energy levels of 240 tonne metres associated with the heavy tamping used on the project resulted in compaction depths up to 6,5m being achieved.

The typical plot shown in Figure 7 of the pre and post compaction Dynamic Probe Super Heavy (DPSH) test results illustrate the improvement with depth achieved by heavy tamping.

6.2 *Design of ground improvement*

The results of the compaction trials were used to determine the performance and compaction depth requirements for the various structural elements where soil improvement was envisaged. In general, the uncemented sands were treated over their full depth with a practical limitation of 6m imposed by the equipment available. A soil modulus of 50MPa determined from the results of soaked plate load tests was used for the performance criteria of the soil improvement. Impact rolling was generally used for the improvement of the subgrade horizons beneath the roads on the site. Rapid Impact Compaction and low-energy heavy tamping was used beneath on-grade paved structures and heavy tamping was specified for heavily loaded on-grade slabs and for light and moderately loaded structures where shallow spread foundations were to be incorporated.

6.3 *Cemented soils*

A feature of the metastable soils on the Smelter site is the presence of moderately to highly cemented and dessicated soils. The behaviour of these soils is dramatically affected by in-situ soil moisture contents as outlined above. At the low in-situ moisture contents exhibited by the red soils, the densification of the soil fabric could not be effected at the high energy levels of heavy tamping. The upper surface of the cemented soils therefore clearly delineated the depth to which soil improvement could be achieved. A closely spaced grid (25m) of Dynamic Probe Super Heavy (DPSH) tests was put down to define the contours of the cemented horizon beneath each structure requiring soil treatment. A typical contour profile depicting conditions is shown in Figure 8. The rapid and marked variability of the presence of the cementing of the soil skeleton posed the greatest difficulty in assessing the soil improvement requirements.

Soaked plate load tests carried out on cemented potentially collapsible sands showed that in the low range of applied stress above overburden pressure, pressure increments ranging typically from 30kN/m² to 180kN/m² could be applied without the

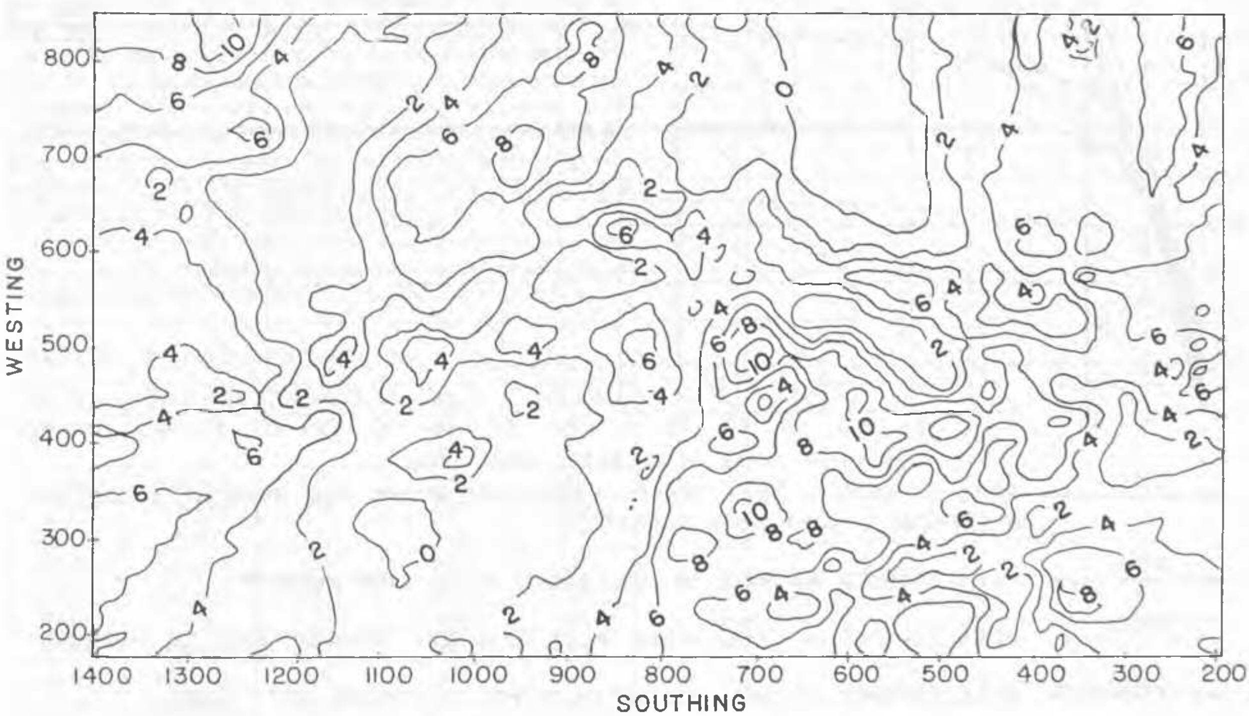

Figure 8

onset of collapse settlement with wetting up of the soil fabric. This is clearly indicated by the pre-ground improvement plate test results shown in Figure 6.

6.4 *Pre-wetting*

The effects of pre-wetting of the soil profile within the cemented horizons was investigated in a trial compaction area during the execution of the soil improvement contract. The marked reduction in the bonding of the soil fabric with increase in soil moisture presented a possible method of improving the cemented red sand in-situ using heavy tamping. Significant measured surface settlements (up to 800mm) and improved soaked plate load modulus test results in the pre-wetted trial compaction area were clear evidence of the suitability of this technique for improving the cemented red sands. The technique could not be used on the Smelter site for logistical reasons. The success achieved in the preliminary testing of the pre-wetting technique is a clear indication that further research and verification of the technique is warranted.

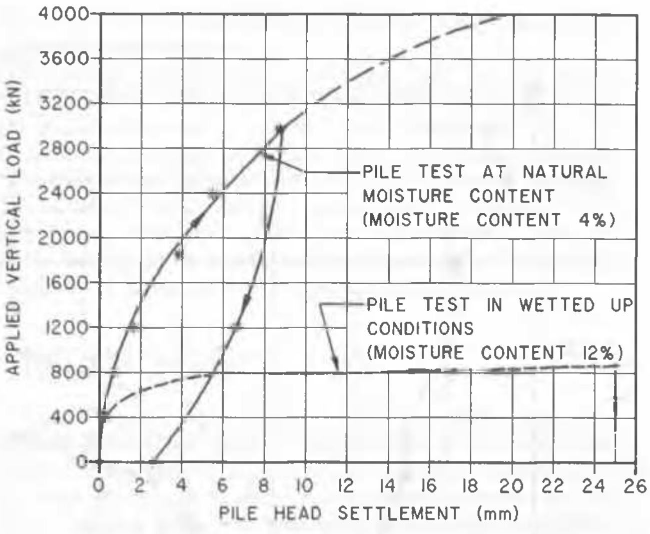

Figure 9

7 DEEP FOUNDATIONS

Early geotechnical investigation work carried out on the Smelter site indicated dense and very dense sands at shallow depth. Preliminary comparative trial pile testing of driven cast-in-situ piles with an enlarged base (DCIS) and bored Continuous Flight Auger piles (CFA) confirmed the suitability and cost effectiveness of the driven pile solution. In the early stages of the project it was anticipated that satisfactory pile performance could be achieved at depths of 6m to 8m below ground level using the

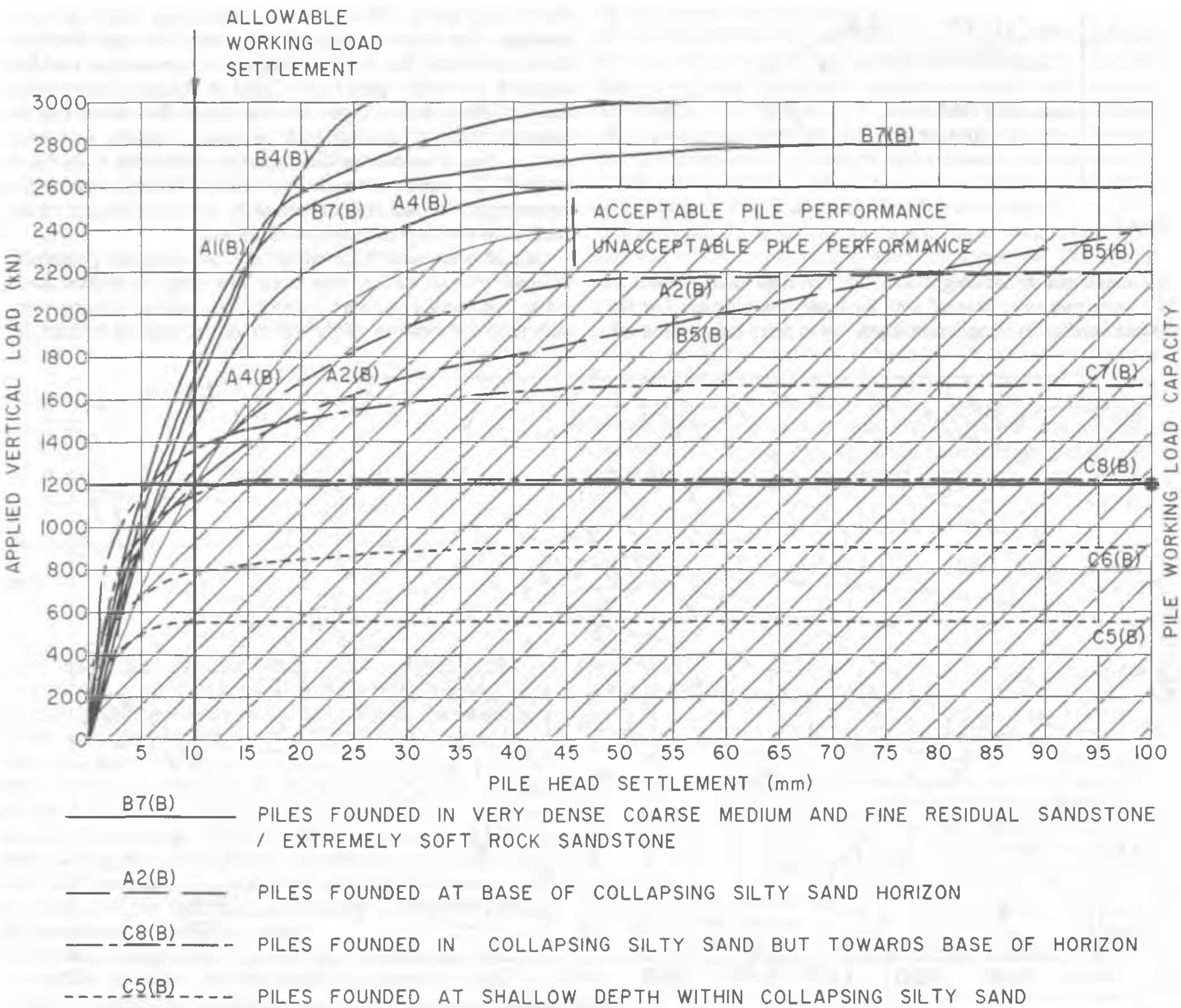

Figure 10

basing energy method for determining pile base resistance developed by Nordlund (1965). Subsequent investigation confirmed significant reduction in both pile capacity and stiffness for piles founded within the potentially collapsing soils under "wetted-up" conditions. An extensive trial pile programme to ascertain both pile base and shaft resistance in wetted-up conditions over the full metastable horizon was undertaken.

7.1 *Pile performance*

The heavily loaded and settlement-sensitive structures requiring a deep foundation solution necessitated piles having the following pile performance criteria at both in-situ and wetted up moisture regimes:

- maximum settlement at nominal load = 10mm
- maximum settlement at 150 percent of nominal load = 18mm
- residual settlement after removal of 100% nominal load = 6mm
- residual settlement after removal of 150% nominal load = 12mm
- Factor of safety on pile bearing capacity failure at nominal load of 2,5

The entirely satisfactory performance of piles at natural in-situ moisture content founded at depths of 6,0m to 8,0m below N.G.L. is clearly shown in Figure 9 and the dramatic effect of wetting up of the soil profile with the pile under load is graphically demonstrated in this figure. The extensive trial pile test programme demonstrated that piles founded within the metastable red soils could not meet the pile performance criteria. The options available to ensure pile performance over the full life of the Smelter structures was to either significantly down-rate the piles or to found the piles below the collapsible horizon.

7.2 *Base resistance*

A series of ten enlarged bases were formed by expelling between 0,29m³ and 0,41m³ of dry mix concrete from the base of a 520mm diameter pile tube. A 5,2 tonne hammer with a 1,2m drop height was used to expel the dry mix concrete from the base of the tube. The bases were formed at depths of between 8,0m and 18,5m below N.G.L. and approximately 2,0m of the pile shaft was formed above the base. A 400mm diameter steel casing was inserted into the pile tube to facilitate load transfer from the surface to the enlarged base without contribution to the base resistance. Pre-boring was used to facilitate penetration of the highly cemented zones within the soil profile and four enlarged bases were formed at shallow and intermediate depth within the metastable horizon. The remaining six tests were carried out at the base of and below the collapsible horizon.

The full soil profile was pre-wetted to ensure moisture contents of the soil surrounding the base were in excess of 10% before the loading was commenced. The bases were loaded to failure or to a maximum load of 2,5 times the nominal load of 1200kN for the 520mm diameter DCIS pile bases. The results of base tests are shown in Figure 10 and the results of typical Cone Penetration tests carried out pre and post "wetting up" are shown in Figure 11. The measured ultimate base resistance values derived from estimated base areas using material volumes expelled from the pile toe are tabulated below.

Table 1. Measured ultimate base resistance

Trial Pile Number	Founding Horizon	Base Area (m^2)	Failure Load (kN)	Base Resist kN/m^2
A1 (B)	Non-coll	0.70	3000	4285
A2 (B)	Non-coll	0.70	2200	3150
A4 (B)	Rock	0.525	2600	4950
B4 (B)	Non-coll	0.70	3000	4285
B5 (B)	Non-coll	0.525	2400	4570
B7 (B)	Rock	0.525	2800	5335
C5 (B)	Collapsing	0.525	550	1050
C6 (B)	Collapsing	0.70	900	1285
C7 (B)	Collapsing	0.525	1450	2760
C8 (B)	Collapsing	0.70	1250	1785

Increase of base resistance within the collapsible soil horizon was attempted by forming an enlarged base after pre-wetting the soil at the toe of the piling tube. The slightly increased resistance measured on pile C6B compared to the base for pile C5B formed in the dry indicated only slightly improved performance. Due to the marginal advantage gained using the technique of pre-wetting the concept was abandoned.

7.3 *Pile shaft resistance*

At the commencement of the project, pile shaft resistance was not relevant in contributing to overall pile resistance and performance. The significant increases in required pile founding depth necessitated a change to this approach and pile shaft resistance under wetted-up soil conditions required careful assessment. A series of eight "shaft only" trial piles where the base re-

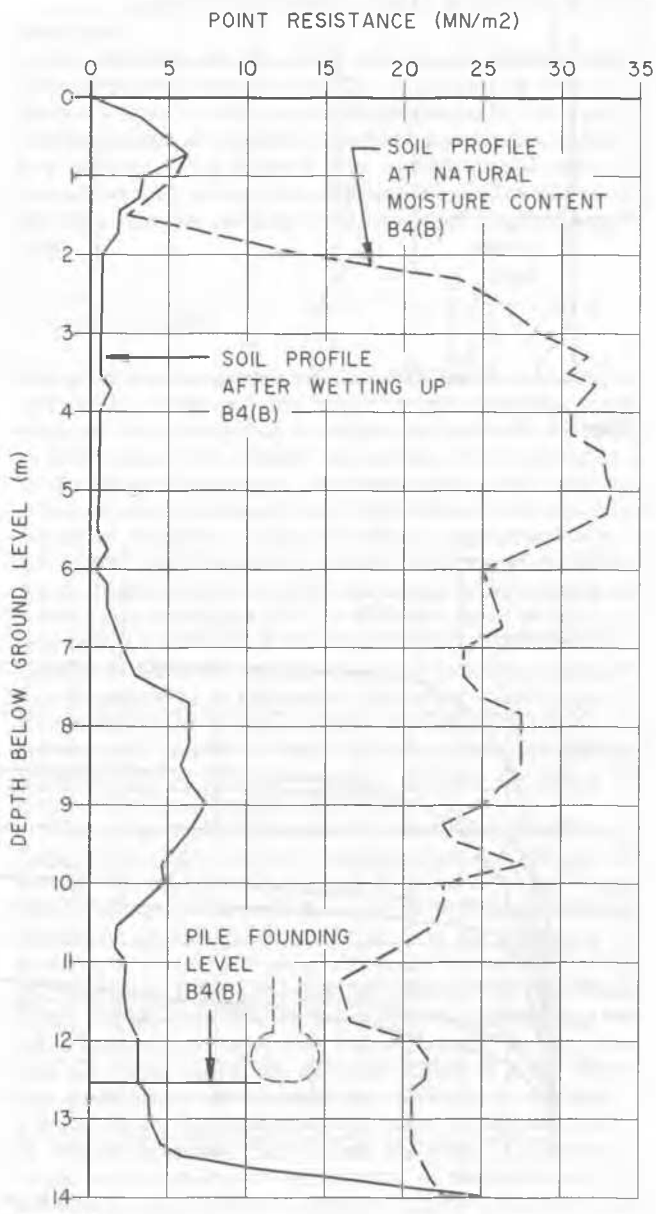

Figure 11

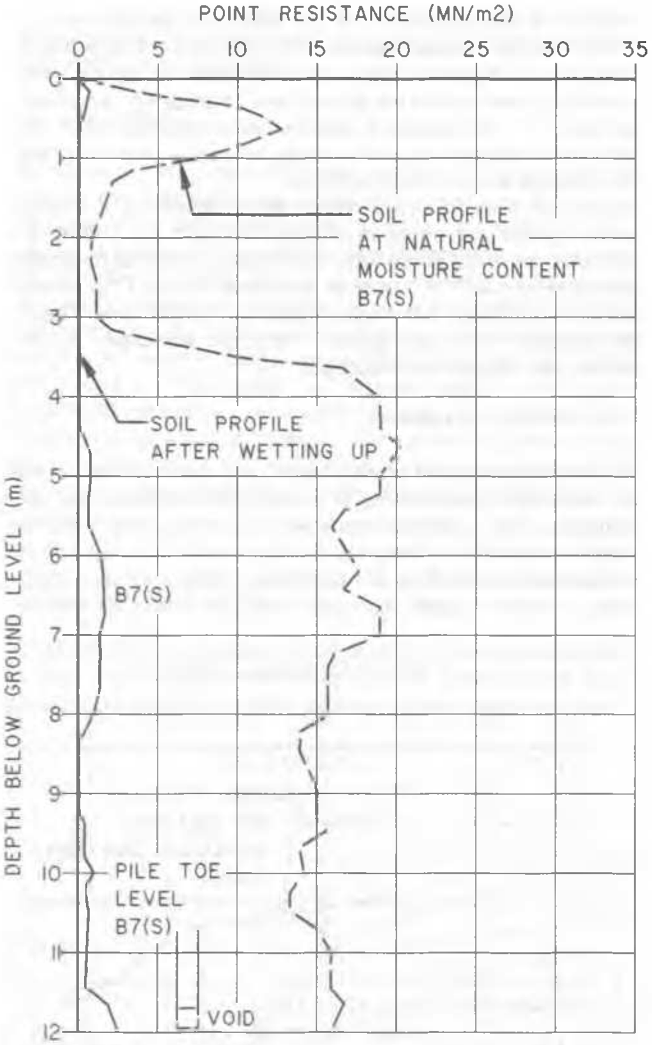

Figure 12

Table 2. Measured ultimate shaft resistance

Trial Pile Number	Pile Shaft Length (m)	Measured Average Shaft Diameter (mm)	Failure Load (kN)	Average Shaft Resistance (kN/m²)
A35	12.5	590	900	39
A45	12.1	600	1200	53
B65	12.5	590	950	41
B75	12.0	600	600	27
C15	6.0	560	300	28
C25	8.0	560	350	25
C35	13.5	570	800	33
C45	12.0	600	650	29

sistance was eliminated were carried out simultaneously with the base tests noted in 7.2 above. Piles with a nominal shaft diameter of 520mm were installed to depths of between 6,0m and 13,5m below terrace level. The piles were pre-bored to ensure penetration of highly cemented horizons and the pile shafts were formed by filling the drive tube with high slump concrete and extracting the tube. Base resistance under compressive loading was eliminated using a void former placed at the pile toe.

The soil surrounding the pile shaft was pre-wetted by increasing the soil moisture content to in excess of 10% and the piles were loaded in compression until failure or pile head movements exceeded 50mm. All tests were carried out with the metastable red soils occupying the full length of pile shaft tested. The results of the six "shaft only" tests are shown in Figure 13. The results of the Cone Penetration Tests immediately adjacent to the pile shaft pre and post wetting up are shown in Figure 12 The pile shafts were exhumed on completion of the test to measure the actual pile shaft geometry since minor collapse of the sidewalls during pre-boring was experienced on a significant percentage of the piles installed. The measured average ultimate pile shaft resistance values in a wetted up soil condition are given in Table 2.

7.4 *Pile design and performance*

The site-wide design of the pre-bored driven cast-in-situ piles with an enlarged base was undertaken using the ultimate resis-

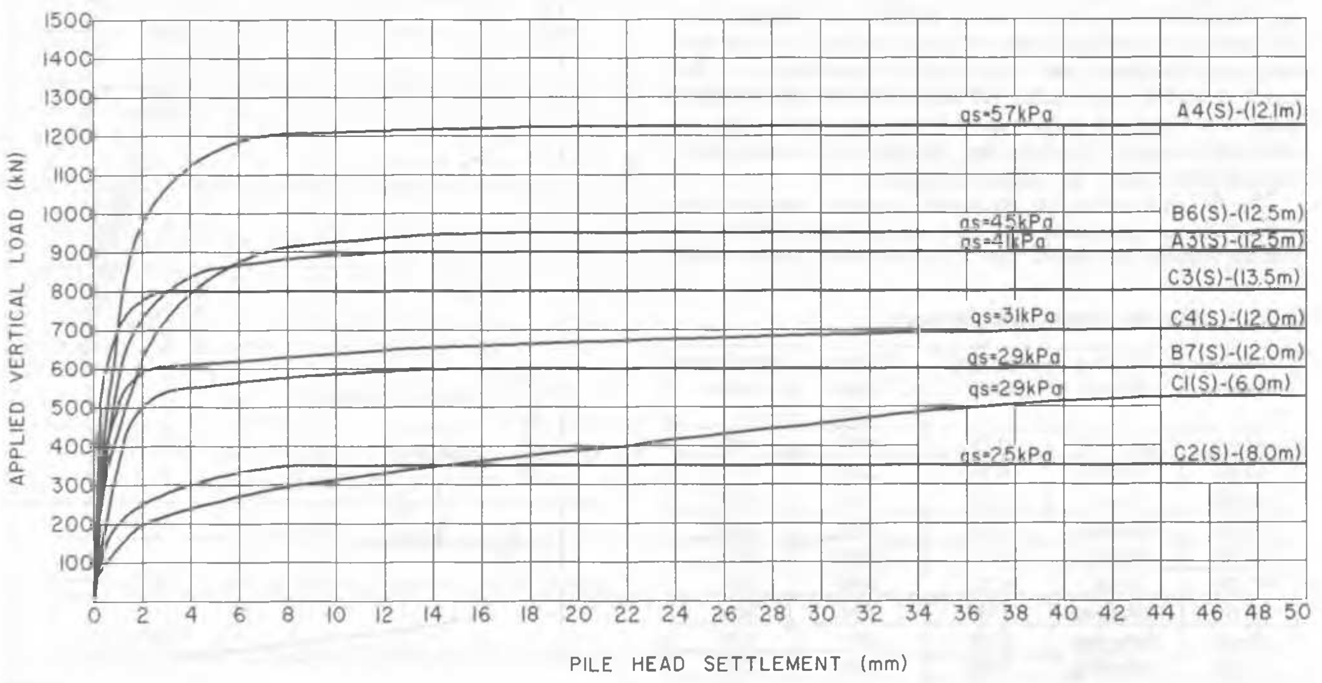

Figure 13

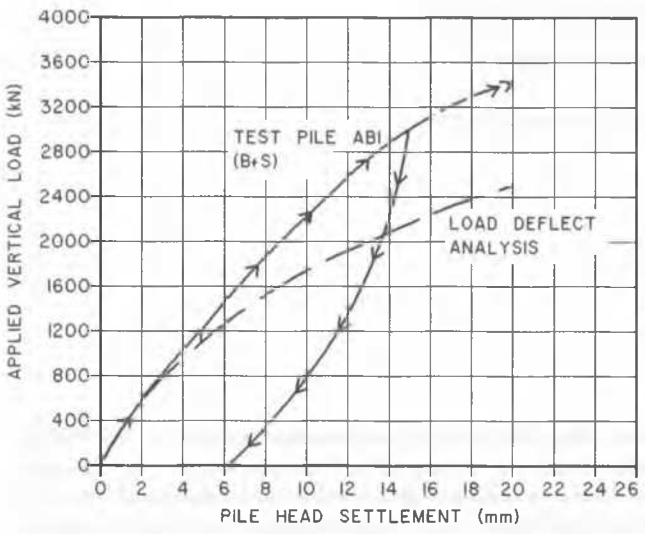

Figure 14

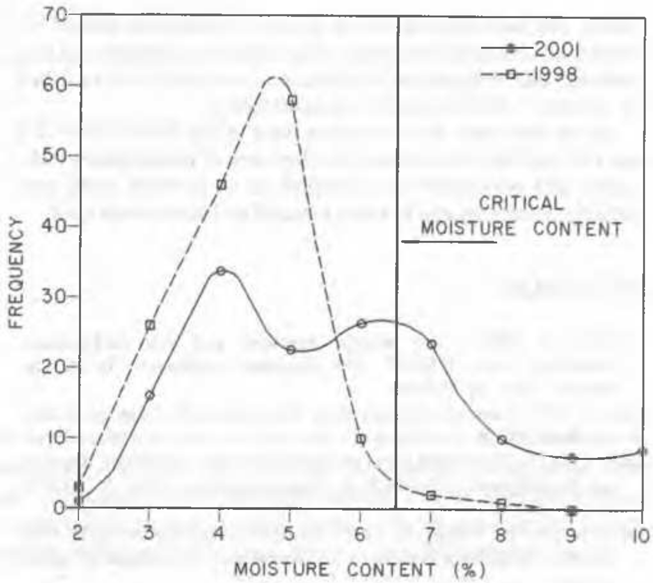

Figure 15

tance values derived from the trial pile results noted above. The load transfer function methodology developed by Everett (1991) was used to predict pile load vs. pile head deflection characteristics over the full range of loading up to the failure load. Comparison of the method with measured trial pile data is given in Figure 14.

The difficulties in predicting the geotechnical characteristics and behaviour of the collapsing soil fabric in a wetted-up soil moisture regime using available site investigation techniques posed unique difficulties for the design and installation procedures of the piled foundations. Due to these difficulties and the risks of unacceptable post construction settlement on settlement-sensitive structures, the piles on these structures were founded below the mantle of collapsing residual soil. The most critical design parameter for these structures became the clear definitions of the collapsing soil profile and the assurance that each pile was founded below this horizon which exhibited rapid variation over short distances. Normal installation procedures for driven piles such as driving resistance or basing energy are ineffective for these collapsing soils since the measurements are carried out at natural moisture contents during installation and a changed soil moisture regime will have a deleterious affect on long term pile performance which could have serious consequences on the serviceability of the structures on the project.

8 SOIL MOISTURE REGIME

There is little data or methodology available for the prediction of the soil moisture regime beneath structures due to a change in the post construction boundary conditions.

The moisture regime beneath the Smelter site in its "green fields" state demonstrated a dessicated soil profile over the upper 15m to 20m of the potentially collapsing soil profile. Data on the moisture balance of the region indicated a nett water balance deficit with evapo-transpiration in the range 1000 - 1200mm per annum and average annual rainfall of 850mm per annum. This was supported by the presence of tree root activity at depths up to 19m below N.G.L.

A major factor in the choice and design of the structures was the risk of soil moisture increase due to removal of the vegetation and the covering of the ground surface over most of the 110 hectare Smelter site. Leakage of services during the life of the structure is a factor adding to the risk of localised wetting up and consequent unacceptable foundation differential settlement.

Additional investigation for extensions to the Smelter carried out approximately 2 years after the commencement of construction indicated little change to the average soil moisture profile as demonstrated in Figure 15 but there is a trend of wetting up as shown by a marked change in the moisture distribution shown in this figure.

The surprising stability of the average soil moisture regime after significantly changed boundary conditions as well as an extreme cyclonic rainfall event at the beginning of 2000 is difficult to explain. The recently completed investigation did however indicate several zones of local wetting up well above the critical moisture content and confirmed the wisdom of the conservative approach adopted in the design of the plant foundations.

9 CONCLUSIONS

The problems encountered with the deep mantle of metastable soils on the Smelter site highlighted several limitations in conventional site investigation techniques and methods of analysis in the dessicated and cemented soil profiles often encountered in the Southern African region. The measurement of soil properties at natural moisture contents are of little value for the choice and design of foundation solutions without comparative data in "wetted-up" conditions being available. Soil properties in a wetted up conditions are best measured in situ. Widely used in-situ investigation techniques (CPT & SPT) can easily be used after pre-wetting the collapsible soil mass or existing methods can be adopted to allow pre-wetting during the execution of the test. The Pressuremeter or Dilatometer are possibly ideally suited to these techniques. It is also imperative that the soil skeleton is observed in an undisturbed state and this simple observational technique can be effectively used in defining the extent and magnitude of the collapsing soil profile.

The design and performance of foundations placed on or within a potentially collapsing soil should proceed with extreme caution. The methods of predicting the potential for and magnitude of collapse settlement developed by Knight and Jennings (1954) can provide guidelines for defining the severity of the problem of collapse settlement but should be used with caution to predict long term foundation performance with confidence. There is little doubt that the metastable soils exhibit long term creep behaviour as was clearly demonstrated by the long duration pile testing carried out. Full scale testing of actual foundation performance under wetted up conditions is advisable if foundations are located in metastable soils in an untreated state.

Ground improvement in the form of Dynamic Compaction is effective in enhancing the performance of metastable soils but the effectiveness of these techniques is highly susceptible to the soil moisture regime and bonding of the soil fabric. Further research and development on the effects of moisture content on dessicated cemented collapsing soils could be of benefit and it is essential that compaction methods are evaluated and analysed for a project with site specific characteristics.

In the drier and less developed parts of the World there is a need for further research and development of investigation techniques and performance of foundations in problem soils, particularly collapsing and heaving residual and transported soils.

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