GROUND IMPROVEMENT CASE STUDIES
CHEMICAL LIME PILES AND DYNAMIC REPLACEMENT

Patrick K. Wong
Coffey Geosciences Pty Ltd

ABSTRACT

This paper presents two case studies of soft ground improvement, one involving the use of Chemical Lime Piles for a project in Penang, Malaysia and the other involving the use of Dynamic Replacement in Alexandria in Egypt.

The background and the design approach of each of these ground improvement techniques are presented together with a description of their application in two recent projects. In both cases, field performances are compared with the original designs, and are found to exceed design predictions with respect to strength increase.

1 CHEMICAL LIME PILES

1.1 BACKGROUND

The method of ground improvement using chemical lime piles consists of placing columns of specially prepared quick lime into the soft soils without mixing. The procedure involves screwing a hollow casing to the desired depth in the soil. The direction of rotation is then reversed and the casing is withdrawn progressively as the quick lime is injected, by compressed air, through the opening located at the bottom end of the casing. This method has been used in Japan for stabilisation of soft soils up to 45 m deep.

Figure 1 illustrates the typical installation process of lime piles.

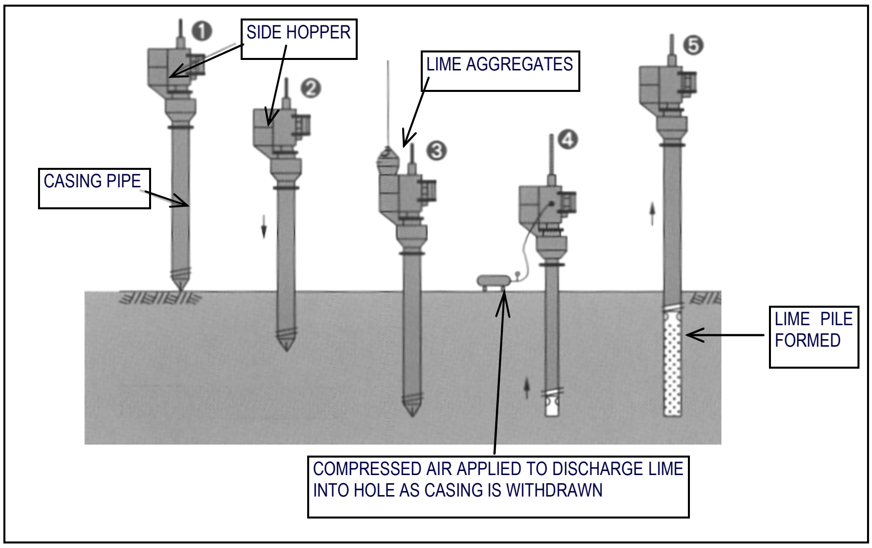


Figure 1: Chemical Lime Pile Installation Process.

This method produces both a consolidation and strength gain effect on the treated soil, without additional loading, via lateral expansion of the lime columns as they absorb water from the soft soil. These lime columns have the following effects on the adjacent soil:

a. Consolidation / dewatering effect

Quick lime, CaO, absorbs water from the surrounding ground, causing the lime to swell and form slaked lime ($Ca(OH)_2$) as per the following chemical reaction:

	CaO	+	H_2O $\rightarrow$	$Ca(OH)_2$ + 15.6 Kcal/mol	
molecular weight	56		18	74	[280 Kcal/kg]
specific gravity	3.3		1	2.2	
weight ratio	1		0.32	1.32	(absorption ratio 1.3 times)
apparent volume	1		1.06	2.0	(swelling ratio 2 times)

b. Ion exchange effect

As the surface of fine particles of clay is negatively charged, calcium ions (Ca^{++}) from the slaked lime are absorbed by the surface of clay particles. As a result, clay particles are bonded with each other and the weak clay is improved with a resultant increase in shear strength.

c. Pozzolanic effect

Calcium ions continue to react with SiO_2 and Al_2O_3 in the clay for a long time thereafter, forming compounds that cause the clay strength to be improved. This reaction is termed a pozzolanic reaction. The lime piles themselves have considerable strength and therefore act to reinforce the soil as well as alter its properties.

1.2 ASSESSMENT OF STRENGTH AND STIFFNESS GAIN

In assessing the strength and stiffness increase in the soil mass treated by lime piles, the longer term ion exchange and pozzolanic effects are generally ignored. The dewatering/consolidation effect is the main process by which the strength of the soil mass is improved in the shorter term. The dewatering/consolidation effect of the lime piles causes a reduction in void ratio of the surrounding soil which in turn causes an equivalent increase in the effective preconsolidation pressure as illustrated in Figure 2:

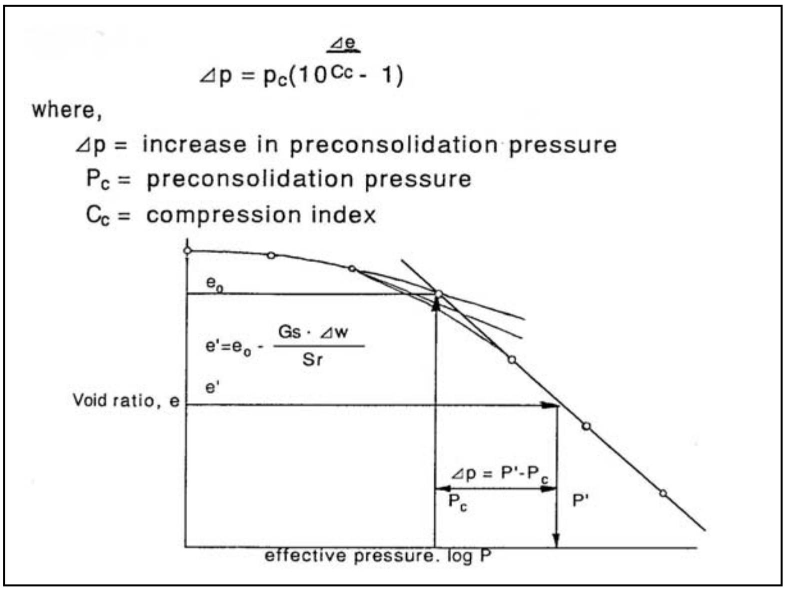


Figure 2: Equivalent Effective Pressure Increase Due to Void Ratio Reduction.

Onoda Chemicals Co. Ltd. has produced the following semi-empirical method for estimating the increase in shear strength of the treated ground by lime piles:

i) The general area ratio is computed as:

$$A_p = \pi . d_0^2 / 4s^2 \qquad \text{Eq.(1)}$$

where
- A_p = area ratio of lime piles
- d_0 = diameter of lime pile (often 0.4 m)
- s = spacing of piles (assuming a square grid pattern of lime piles)

ii) The reduction in water content of treated soil is estimated as:

$$\Delta w = [(100 + w_0) / \gamma_t].A_p .\{h.\gamma_c + [n'(1+\varepsilon_v).(S_r'/100).\gamma_w]\} \qquad \text{Eq. (2)}$$

where
- Δw = reduction in water content of treated soil
- w_0 = original water content of soil (%)
- γ_t = unit weight of untreated soil (t/m^2)
- h = absorption value of water by lime column; this depends on the additives used in preparing the unslaked lime aggregates (a value of 0.3 is often used)
- γ_c = unit weight of chemical lime (taken as 1.2 t/m^2)
- n' = porosity of lime column after chemical reaction (a value of 0.55 is often used)

ε_v = expansion ratio of lime column (a value of 0.75 is often used)
S_r' = degree of saturation of lime pile after treatment (a value of 80% is often used)
γ_w = unit weight of water

iii) The equivalent change in void ratio of the improved ground is given by:

$$\Delta e = G_s \, \Delta w / S_r \qquad \text{Eq. (3)}$$

where
Δe = reduction in void ratio
G_s = specific gravity of original soil
Δw = reduction in water content
S_r = degree of saturation of original soil (100% for most soft clays)

iv) The new void ratio is then:

$$e' = e_0 - \Delta e \qquad \text{Eq. (4)}$$

v) The increase in confining pressure due to the improved soil is calculated (for a soil which is normally consolidated or very lightly over consolidated) as follows:

$$\Delta p = p_c \, (10^{\Delta e/Cc} - 1) \qquad \text{Eq. (5)}$$

where
Δp = increase in preconsolidation pressure
p_c = original preconsolidation pressure
C_c = compression index

vi) The increased shear strength, s_t, of the treated soil is estimated as:

$$s_t = s_0 + s_u/p' \, . \, \Delta p \qquad \text{Eq. (6)}$$

where
s_0 = shear strength of untreated soil
s_u/p' = shear strength ratio

The strength of the composite ground (soil plus piles), s_t', can be estimated as:

$$s_t' = A_p s_p + (1 - A_p) \, s_t \qquad \text{Eq. (7)}$$

where
s_p = shear strength of lime pile (200 kPa is often reasonable)

vii) The reduction in settlement can be considered using the following equation:

$$\Delta S = H_c \, . \, \Delta e/(1+e_o) \qquad \text{Eq. (8)}$$

where
ΔS = preconsolidation settlement due to lime piles (or settlement reduction)
Δe = change in void ratio due to lime piles as defined by Eq. (3)
H_c = thickness of soft clay treated by the lime piles

viii) The drained modulus of the treated soil, E_t, may be estimated as:

$$E_t = [(n-1)A_p + 1) \, E_s \qquad \text{Eq. (9)}$$

where
E_s = drained modulus of untreated soil
A_p = area ratio of lime columns, as defined by Eq. (1)
n = stress distribution ratio (a value of 10 is suggested by Onoda Chemico)

Obviously, the value "n" would depend on the in-situ and applied stress levels as well as the relative stiffness of the lime piles and the soft clay. A better approach of assessing the drained modulus of the treated soil would be to back calculate the equivalent value from the residual settlement following the lime pile treatment via Equation (8) and the calculated settlement without ground treatment. The reader should also refer to Section 2.3 for discussions on alternative methods for assessing stiffness increase achieved by granular columns in soft ground.

The timing for the ground improvement effect to develop depends on the spacing of the lime piles and the permeability characteristic of the soft soils, and can be assessed using methods such as that developed by Barron (1948) and Hansbo et al. (1981) for drain wells or vertical wick drains.

1.3 CASE STUDY

Chemical lime pile ground treatment was applied to the Runnymeade Commercial Project in Penang (Malaysia), constructed during 2000 and 2001. For this project, 400 mm diameter lime piles were installed at 1.7 m spacing to increase the strength and modulus of an 8 m deep soft clay layer. The purposes were as follows:

- To increase the passive resistance and reduce deformation for the shoring wall for the purpose of basement construction.
- To improve trafficability for construction machinery at the base of the excavation.
- To increase the stiffness of the remaining soft clay layer for a piled raft foundation system that was proposed for the 23-storey tower structure.

1.4 SITE GEOLOGY AND SOIL PROPERTIES OF UPPER SOFT CLAY LAYER

The site is located on Lot 131, Jalan Sultan Ahmad Shah, Georgetown in Penang, Malaysia. It covers an area of approximately 0.785 hectares, and is on the northern coastline of Georgetown, adjacent to the Strait of Malacca. It is bounded by the sea (North Channel) to the north and Jalan Sultan Ahmad Shah to the south.

A review of the 1:500,000 Geological Map of the Peninsular of Malaysia reveals that the regional geology of the city of Georgetown consists of unconsolidated Quaternary deposits. These deposits constitute marine and continental deposits of clays, silts, sands and gravels. Typically, such coastal deposits are soft clays and silts of up to 10 m depth.

The subsurface profile is summarised in Table 1 below.

Table 1: Subsurface profile.

Stratigraphic Unit	**Typical Depth Interval (m)**
Soft Clay / Silt	0 to 8
Firm Clay / Silt	8 to 13.5
Stiff Clay / Silt	13.5 to 24.5
Dense Sandy Gravel and Gravelly Sand	24.5 to 29.0
Stiff Sandy clay and Silt	29.0 to 54
Very Stiff to Hard Clay and Silt	54 to > 120

The upper soft clay layer is of particular interest with regard to the chemical lime pile ground treatment carried out. The basic soil properties of this unit as derived from laboratory testing are summarised in Table 2.

Table 2: Soil Properties of Upper Soft Clay Layer.

Borehole	**Depth (m)**	**N.M.C. (%)**	**W_L**	**W_P**	**PI**	**Silt %**	**Clay %**
BH1	6.0 to 6.5	97.1	65	35	30	55	42
BH2	2.5 to 3.5	100.7	54	29	25	57	38
BH2	5.5 to 6.5	110.3	55	30	25	56	42
BH2	5.5 to 6.5	110.3	55	30	25	56	42
BH4	3.5 to 4.5	104.7	58	29	29	52	44
BH5	6.0 to 6.45	94.1	54	27	27	53	43
Minimum		**94.1**	**54**	**27**	**25**	**52**	**38**
Maximum		**110.3**	**65**	**35**	**30**	**57**	**44**
Average		**102.9**	**56.8**	**30.0**	**26.8**	**54.8**	**41.8**

For this site, the original soil shear strength was about 15 kPa within the upper 5 m of soft soil, then increasing to about 30 kPa at about 8 m depth as indicated by field vane shear testing as shown in Figure 3.

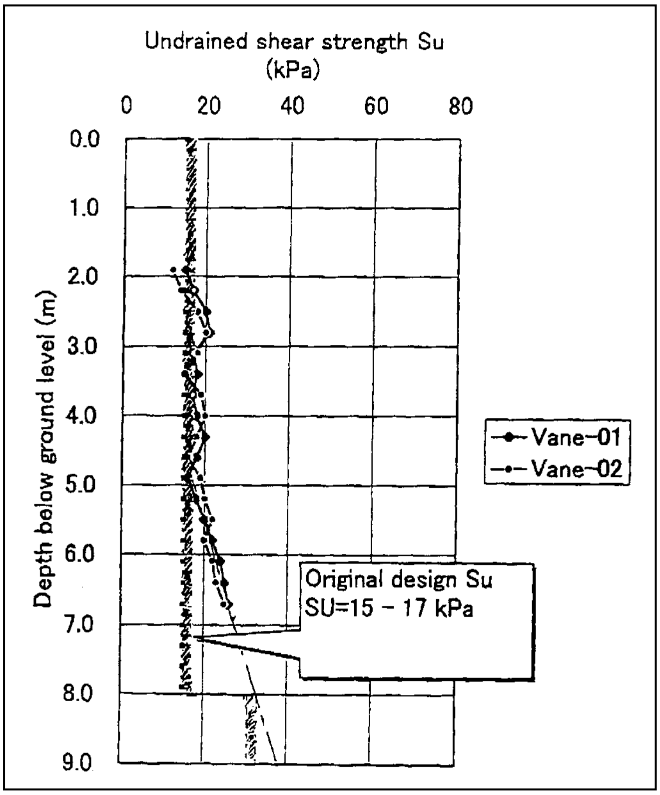


Figure 3: Vane Shear Test Results for Untreated Ground.

1.5 GROUND IMPROVEMENT AND SHORING DESIGN ANALYSES

The proposed 4 m deep excavation (actual depth increased to 5 m during construction) for the basement car park had a plan area of about 90 m by 76 m, and was about 3 m below the sea level. After considering various options for the temporary excavation retention, in-situ soil-cement mixed walls were adopted as the retention system for the basement excavation.

To improve the strength and stiffness of the soft soils, lime piles were installed to a depth of 10 m at the following spacing within the site prior to excavation:

- Zone of 13 m width adjacent to retaining walls - 1.7 m square grid spacing, waiting time of 6 to 8 weeks after lime pile installation before basement excavation.
- Remaining area of site – 2 m square grid spacing because longer time is available for strength gain to occur for support of building loads.

A lime pile strength of 150 kPa was adopted for design. The composite shear strength of the treated soil was estimated to be 25 kPa, using the procedure described in Section 1.2, with an estimated modulus increase of 40%.

The commercially available program WALLAP was used for the shoring wall analyses. Initial design analyses showed that with the anticipated soil strength increase from the lime piles, it was feasible to achieve stability using cantilevered walls. However, the assessed wall deflection would have been in the order of 100 mm which was not considered to be acceptable particularly on the western site boundary where there was an existing two storey building within 2 m of the boundary. To reduce the wall deflection to a design value of 25 mm, temporary soil berms were adopted in the final shoring design. The temporary berms were to be 2.2 m high and 3 m wide at the crest with a temporary batter slope of 4H:1V. They were removed after the diagonal steel struts were installed.

1.6 FIELD PERFORMANCE

Approximately 5 weeks following the installation of the lime piles, field vane testing indicated that the shear strength of the soil at the quarter point between the lime columns had increased to between 27 kPa and 36 kPa in the upper 5 m of soil. The shear strength of the lime column was estimated to be at least 175 kPa from cone penetrometer testing. The composite strength of the treated soil, as constructed, was estimated to be at least 33 kPa, which was comfortably greater than the design composite strength of 25 kPa.

Figure 4 shows typical CPT cone resistance and Figure 5 shows typical vane shear strength measurements taken at the quarter and centre points between the lime piles.

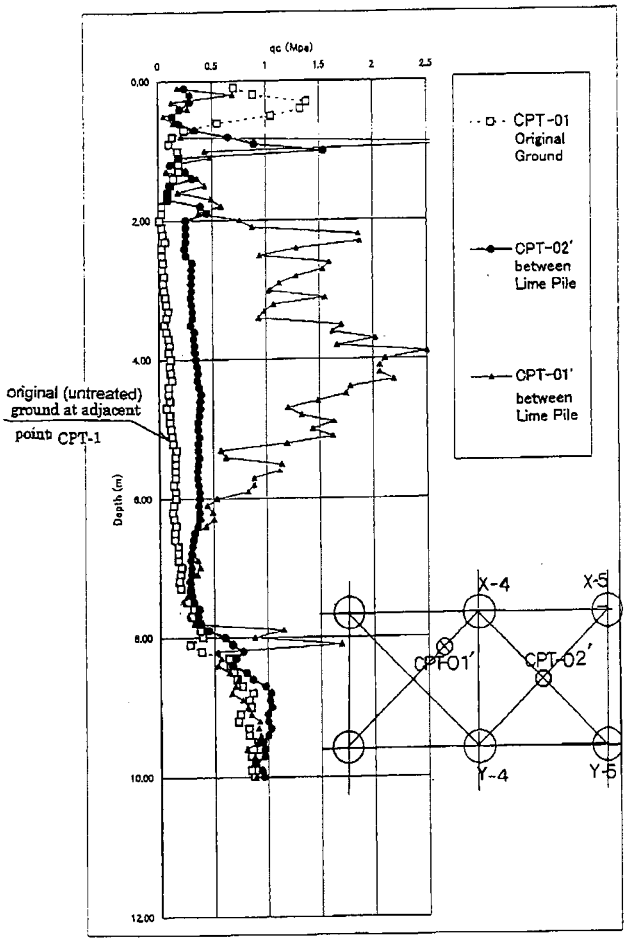
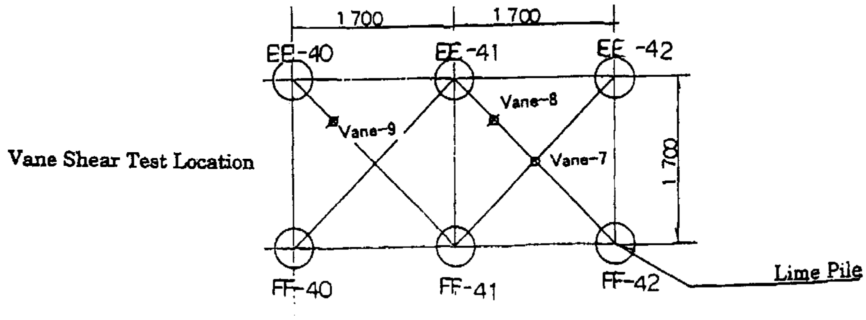


Figure 4: Comparison of CPT Results Before and After Lime Piling.

It should be noted that the lime piles were terminated about 2 m below the ground level and therefore no improvement is expected above this level. It can be seen from Figure 4 that significant improvement in cone resistance was achieved, particularly close to the lime piles at the quarter point (i.e. CPT-01'). The CPT test results also indicate that other than a thin band at about 8 m depth, the strength increase was relatively minor or insignificant below a depth of about 7 m, at least at the time of the CPT which was done about 3 to 4 weeks following installation of the lime piles. The reason for this lack of significant increase is not known, although the soil below about 7m already had shear strengths greater than the design strength profile, and we expected that some strength gain would occur with longer periods of time.

Figure 5 shows the vane shear testing conducted during excavation of the site, and confirmed that significantly higher strength than the design strength of 25 kPa was obtained below the design excavation depth of 4 m. Again, the quarter points closer to the lime piles achieved higher strength gain compared to the centre points between lime piles.

No direct measurement of soil modulus was made. However, shoring movements based on inclinometer monitoring were generally less than those predicted at the design phase, despite the actual excavation depth being increased to 5 m during excavation compared to the design depth of 4 m and the berm batter increased to 1H:1V compared to the design batter of 4H:1V. On the three landward excavation boundaries, maximum lateral movements were measured to range from 5 mm to 30 mm towards the excavation, with a wall tilt of 0.03^o to 0.18^o. On the seawards boundary, however, the lateral movement at the top of the wall reached up to about 90 mm with a corresponding tilt of 0.53^o towards the excavation. The unexpected high lateral movement observed on this boundary was thought to be caused by the deeper excavation than designed coupled with the fill levee bank placed on this side of the site. Despite the fact that the excavation was 25% deeper than the design depth, the relatively small wall movements on the other three sides indicate that the desired improved ground stiffness and strength had been achieved.



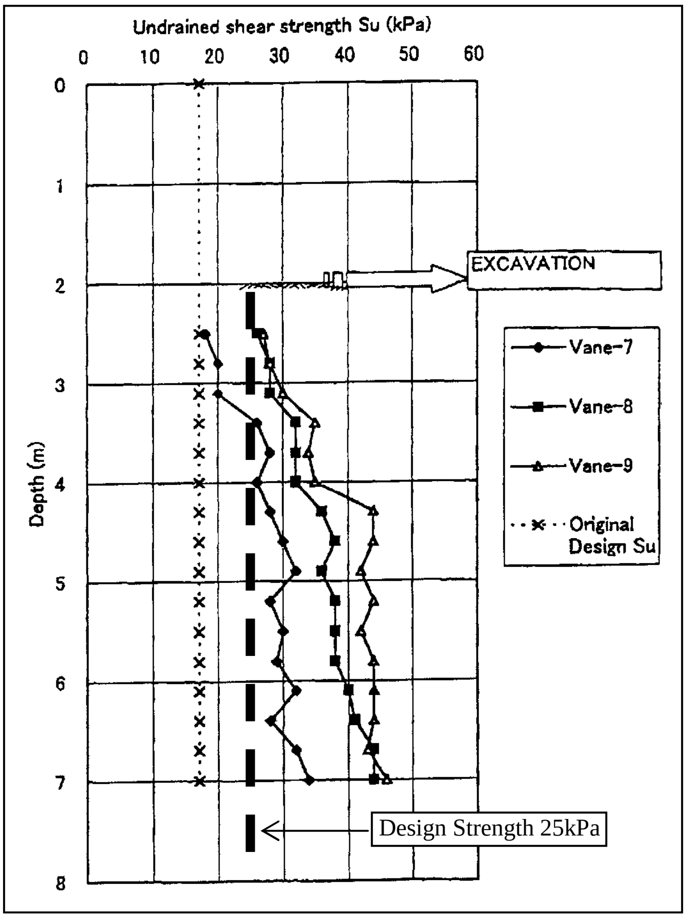


Figure 5: Comparison of Shear Vane Strengths before and after Lime Piling.

Overall, the lime piling achieved the desired ground improvement objectives for this project. It would appear that higher shear strength was actually achieved compared to the theoretical estimate.

2 DYNAMIC REPLACEMENT

2.1 BACKGROUND

The **Dynamic Replacement (DR)** technique, pioneered by Menard, is a "marriage" of **Dynamic Compaction** (dropping of a heavy weight from a substantial height to cause deep compaction of the ground) and **Stone Columns** (gravel columns placed in the ground using a vibrating probe to increase the stiffness and strength of the ground). In Dynamic Replacement, however, stone columns are introduced into the ground by a heavy weight dropped repeatedly onto a gravel layer while the craters created by the impact of the heavy weight are backfilled with gravel during the process as shown in Figure 6. The resulting stone columns are significantly larger in diameter, have higher load carrying capacity, are more rapid to install, and hence more economical compared with the conventional Stone Column ground treatment method. The disadvantage of dynamic replacement, however, is that there is a limiting depth to which the DR stone columns can be installed, and at which the gravel near the top of the columns will tend to heave rather than being pushed downwards by the falling weight. Some previous usage of dynamic replacement have been reported by Juillie and Sherwood (1983), Lee and Lo (1985), and Varaksin et al. (1994).

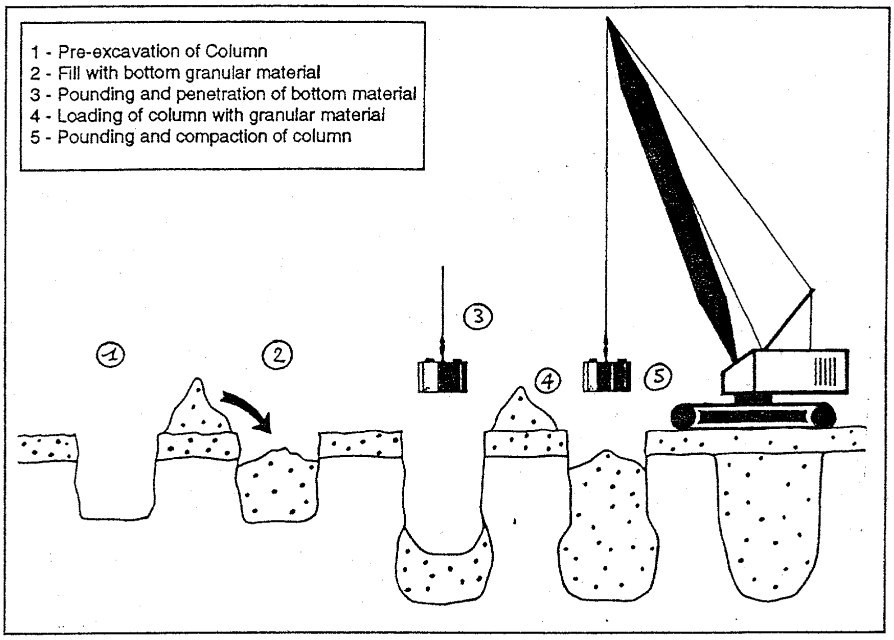


Figure 6 : Dynamic Replacement Installation.

2.2 ASSESSMENT OF EQUIVALENT STRENGTH

The assessment of the equivalent strength of the soil mass improved by dynamic replacement may be carried out in the same manner as that for stone columns. Madhav and Nagpure (1996) have developed the following expressions for the equivalent strength properties of the clay treated with stone columns:

$$N_{eq} = a_r N_1 + (1 - a_r)\, N_2 \qquad \text{Eq. (10)}$$

$$c_{eq} = \frac{c_1\, a_r \sqrt{N_1} + c_2\,(1 - a_r)\sqrt{N_2}}{\left(a_r N_1 + (1 - a_r) N_2\right)^{0.5}} \qquad \text{Eq. (11)}$$

$$\gamma_{eq} = a_r\, \gamma_1 + (1 - a_r)\, \gamma_2 \qquad \text{Eq. (12)}$$

where

N_{eq}	$= \tan^2(45 + \phi_{eq}/2)$
ϕ_{eq}	= equivalent angle of friction
a_r	= area ratio
	$= f_1\,(d/s)^2$
f_1	= 1.13 for a triangular column pattern or 1.27 for a square pattern
d	= stone column diameter
s	= stone column spacing
N_1	$= \tan^2(45 + \phi_1/2)$
N_2	$= \tan^2(45 + \phi_2/2)$
c_{eq}	= equivalent cohesion
γ_{eq}	= equivalent unit weight
c_1, ϕ_1, γ_1	= cohesion, friction angle, and unit weight for stone column
c_2, ϕ_2, γ_2	= cohesion friction angle, and unit weight for soil.

2.3 ASSESSMENT OF EQUIVALENT STIFFNESS

Over the past 20 years, numerous theories have been developed for assessing the equivalent stiffness of soft ground reinforced by granular columns such as stone columns. Such theories are also applicable for dynamic replacement ground improvement. Balaam et al. (1976) have published design charts for fully and partially penetrating stone columns based on finite element studies. Shahu et al. (1998) have extended the one dimensional (mechanistic) model based on the unit cell concept of Alamgir et al. (1993), and conducted a parametric study of soft ground reinforced with granular piles and with a granular mat on top. Shahu et al. (1998) also compared the results of their parametric study with field measurements reported by Bergado et al. (1987). In this paper, the results from Shahu et al. (1998) and Bergado et al. (1987) are compared with the equivalent stiffness relationship proposed by Watts et al. (2000) and Poulos (2002) to assess if a simplified analytical procedure could be used for settlement assessment.

Watts et al. (2000) proposed the following relationship for assessing the equivalent modulus of the soil mass reinforced by stone columns supporting a strip footing:

$$E_{eq} = \frac{E_c.A_c + E_s.A_s}{A_c + A_s} \qquad \text{Eq. (13)}$$

where E_c = Young's modulus of the stone column
E_s = Young's modulus of the untreated soil
A_c = Sum of the area of the stone columns
A_s = Sum of the area of the soil mass treated by, but excluding the stone columns

Equation 13 may be rewritten as follows:

$$E_{eq} = E_c\left[a_r + \frac{E_s}{E_c}(1 - a_r)\right] \qquad \text{Eq. (14)}$$

where E_c , E_s as defined above for Eq. 13, and
a_r = area ratio of stone column $A_c/(A_c + A_s)$

For stone columns supporting general fill embankments, Poulos (2002) has proposed a modified form of Equation 14 for the assessment of equivalent modulus as follows:

$$E_{eq} = E_c\left[{a_r}^2 + \frac{E_s}{E_c}(1 - {a_r}^2)\right] \qquad \text{Eq.(15)}$$

A comparison of the effect of these two equations on settlement assessment may be made with the parametric studies by Shahu et al (1998) and field measurements by Bergado et al. (1987), by defining the settlement ratio SR as follows:

$$SR = S/S_o \qquad \text{Eq. (16)}$$

where S = settlement of the treated ground
S_o = settlement if the ground was not treated

And as settlement is proportional to the Young's modulus, the settlement ratio, SR, may be rewritten from Equations 15 and 16 as follows:

from Equation 15
$$SR = \frac{E_s}{E_{eq}} = \frac{E_s}{E_c}\left[a_r + \frac{E_s}{E_c}(1 - a_r)\right]^{-1} \qquad \text{Eq. (17)}$$

from Equation 16
$$SR = \frac{E_s}{E_{eq}} = \frac{E_s}{E_c}\left[{a_r}^2 + \frac{E_s}{E_c}(1 - {a_r}^2)\right]^{-1} \qquad \text{Eq. (18)}$$

Typical results from these equations are plotted against the area ratio a_r , for E_c/E_s values of 20, 50 and 100 on Figure 7(a), 7(b) and 7(c) respectively, together with the results reported in Shahu et al. (1998) and Bergado et al. (1987) for comparison.

The E_c/E_s ratio was not quoted in the field trial reported by Bergado et al. (1987), but the ratio is expected to be in the range of 20 to 50 for the stone columns used, and their results are therefore more appropriate for comparison with other results in Figures 7(a) and 7(b), and less appropriate in Figure 7(c)

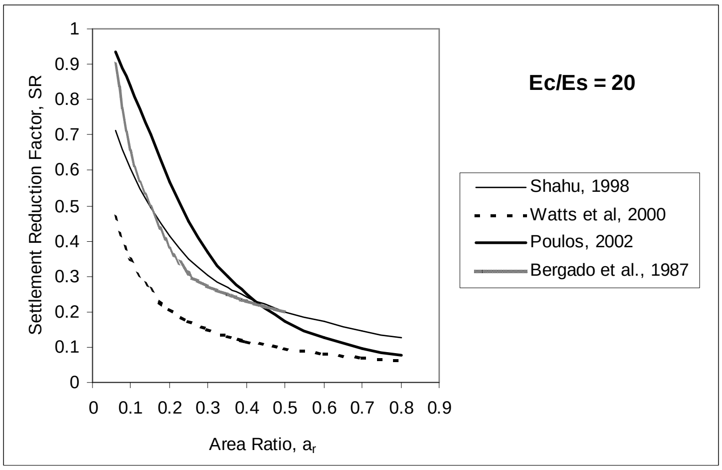


Figure 7(a): Settlement Reduction Ratio for $E_c/E_s = 20$.

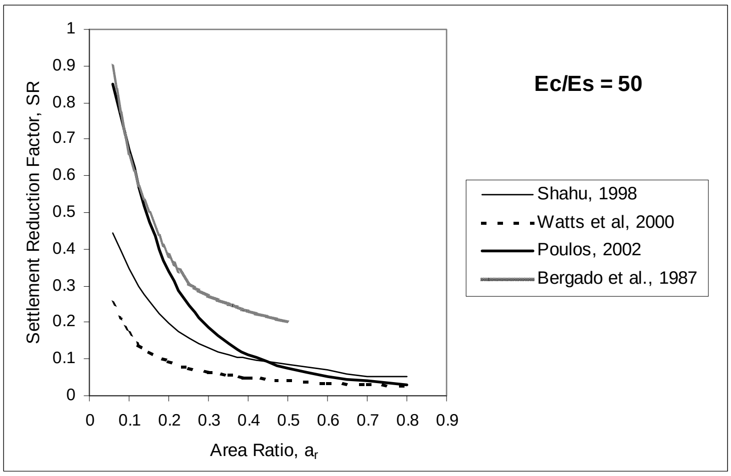


Figure 7(b): Settlement Reduction Ratio for $E_c/E_s = 50$

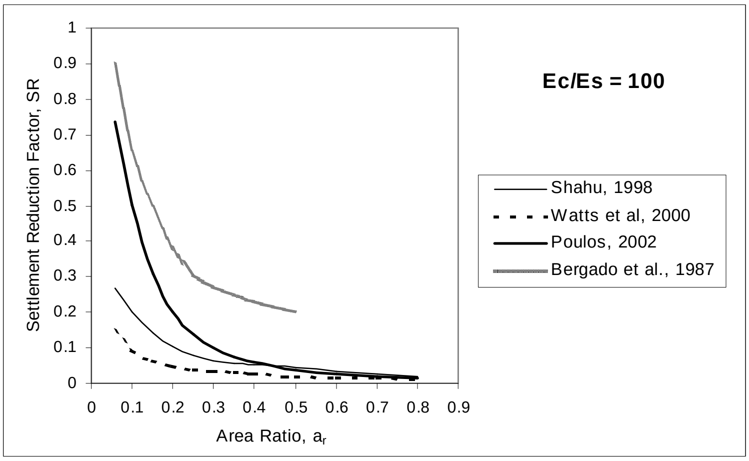


Figure 7(c): Settlement Reduction Ratio for $E_c/E_s = 100$.

It should be pointed out that the equation proposed by Watts et al. (2000) is based on equal strains in the soil and stone columns, and this would only apply if a rigid raft exists over the stone columns, or if the area ratio is sufficiently large that the applied load will be taken mostly by the stone columns via arching action. Bulging of the stone column, particularly near the top of the soft soil profile where confining pressures are low, also causes non-uniform strain conditions. In most practical applications, therefore, the use of the equal strain approach could under-estimate settlement and differential settlements. For the purpose of initial assessment, the modified analytical method proposed by Poulos (2002) appears to provide a relatively conservative approach, which could then be refined by finite element analysis methods for detailed designs. It should be recognised that in certain favourable situations where the column bulging is limited and full arching develops, the equal strain method may provide reasonable results.

2.4 CASE STUDY

Dynamic replacement was adopted as the ground treatment solution for the Alexandria City Centre project in Egypt during 2001 and 2002. The project involved the construction of a very large shopping centre on a 220,000 m^2 site in Alexandria, Egypt. The initial earthworks contract required reclamation of part of a lake. Very soft, compressible organic clay deposits existed up to 9 m deep in places beneath the lakebed. The specification required the site to be raised by 2 m above the lake water level. The design criteria was for post-construction settlement under the specified loads to not cause the site to drop below the design level, and for differential settlements to be within design tolerance. In particular, proposed tiled floors required stringent differential settlement limits of 1 in 1000.

The design column load was 700 kN, and columns were to be supported on shallow footings founded at 1.5 m depth below bulk earthworks level. However, as the layout of the buildings was not finalised at the time of the earthworks design, the challenge was to come up with an economical earthworks/ground treatment strategy to enable shallow footings to be adopted at the site, irrespective of the building column locations.

2.5 SITE CONDITIONS

The site is situated east of Alexandria on the Cairo Desert Road, on the edge of Lake Maryout, in the Western Nile deltaic zone of Egypt. A significant part of the site is below the existing lake level, with an average water depth of 1.5 m.

The subsurface profile at the site is characterised by three main units as summarised below:

Unit 1	Very soft clay with organic matter	4 m to 9 m thick (typically 7 m)
Unit 2	Stiff silty clay and clayey silt	5 m to 9 m thick
Unit 3	Very dense silty sand	not penetrated

A typical piezocone test result is shown in Figure 8.

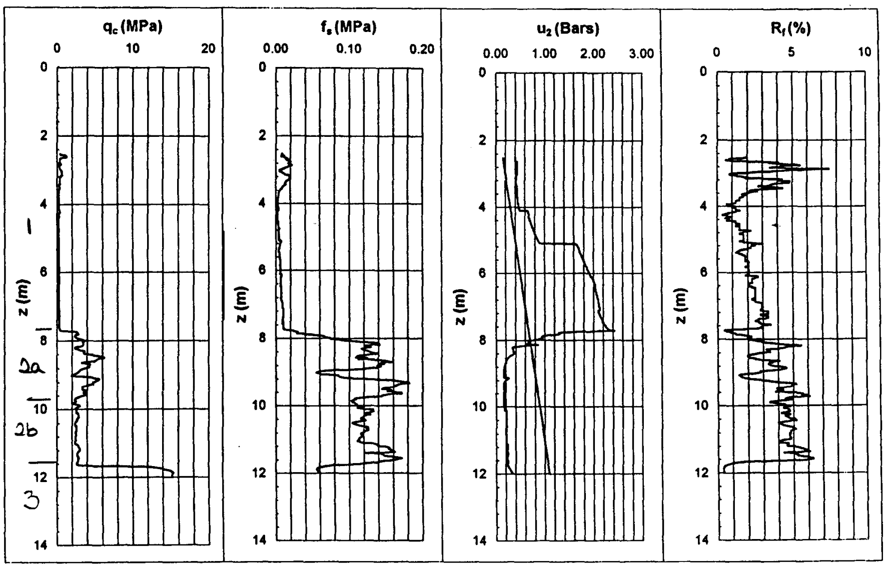


Figure 8: Typical Piezocone Test Result.

It was obvious that Unit 1 would control site settlement and would govern the design of ground improvement works. Based on the laboratory testing results, the following soil properties were adopted for Unit 1:

Moisture content, W_n	76% to 130%
Liquid Limit, W_L	102% to 146%
Plastic Limit, W_P	35% to 43%
Plastic Index, I_P	67% to 106%
Bulk Unit Weight, γ_b	14.5 kN/m^3
Vertical Coefficient of Consolidation, c_v	2.0 m^2/year
Horizontal Coefficient of Consolidation, c_h	10.0 m^2/year
Modified Compression Index, $C_c/(1 + e_0)$	0.3
Modified Recompression Index, $C_r/(1 + e_0)$	0.03
Modified Creep Coefficient, $C_\alpha/(1+ e_0)$	0.015

2.6 DESIGN OF DYNAMIC REPLACEMENT

Following preliminary assessment of a number of ground improvement options which included vacuum consolidation and rigid inclusion methods, it was decided to adopt dynamic replacement as the ground improvement solution for Phases 1 and 2 of the project (72,000 m^2 of building area and 50,000 m^2 of on-grade car parking area) due to its relative speed of construction and economy.

To meet the stringent post-construction settlement criteria, it was also necessary to preload the site. And to meet the limited time programme, prefabricated wick drains were installed to increase the rate of consolidation even though the DR columns would already facilitate radial drainage to occur in the soft clay.

The preload and wick drain spacing were designed using conventional one-dimensional and radial drainage theory (Schmertmann,1955; Barron,1948 and as described in Fell, Wong & Stone, 1987). The design of wick drain spacing took into account soil disturbance and discharge capacity of the drains using the procedures described by Hansbo et al. (1981). The design solutions are summarised in Table 3:

Table 3: Summary of Design Solutions.

Proposed Development	**Phase 1** Buildings	**Phase 2** Car park
Approximate Area	70,012 m^2	41,370 m^2
Settlement Limit	13.5 mm under uniform live load of 20 kPa; 20 mm under 700kN column load; 34 mm creep over 50 years; Differential settlement 1:1000	100 mm over 50 years
DR Spacing	5.5 m	7m
DR Diameter	2.5 m at surface	2.5 m at surface
Wick Drain Spacing (square grid)	1.1 m	1.25 m

An important aspect of the design was that the DR columns would not be fully penetrating. After placement of a 1.7 m thick working platform to provide access, the maximum depth of penetration of the DR columns was assessed to be 6.5 m, thereby leaving about 2.2 m thickness of the soft clay layer (for a design soft clay thickness of 7 m) untreated by DR but would be improved by preloading.

Details of the design approach are published in Wong and Lacazedieu (2004). In summary, it involved initial one-dimensional settlement analysis using conventional consolidation theory of the untreated soil, preliminary assessment of settlement reduction that could be achieved by the DR using Equation 18, followed by a three-dimensional finite element analysis using FLAC 3D (Fast Lagrangian Analysis of Continua, ITASCA, 1999). The design assumed a Young's modulus of 50 MPa for the crushed limestone used for construction of the DR columns, giving a modulus ratio, E_c/E_s of about 100 in this case. The replacement area ratios were about 0.16 and 0.1 for Phase 1 and Phase 2 of the project respectively. For Phase 1, the calculated settlement reduction ratio for the DR treated layer (upper 4.8 m) was about 0.3 using Equation 18, and this was found to be consistent with the FLAC 3D analysis subsequently carried out.

2.7 FIELD PERFORMANCE

Extensive instrumentation and monitoring were carried out for the Dynamic Replacement ground treatment carried out for this project. In addition to settlement plates, down-hole extensionmeters, and piezometers, static cone penetration testing (CPT) and field vane shear testing were also carried out before and after the ground treatment to validate the design. The results from Phase 1 were back-analysed and enabled refinement of the design to be made for subsequent stages of the project.

The settlement plate results from Phase 1 of the project are presented in Figure 9 together with the design prediction:

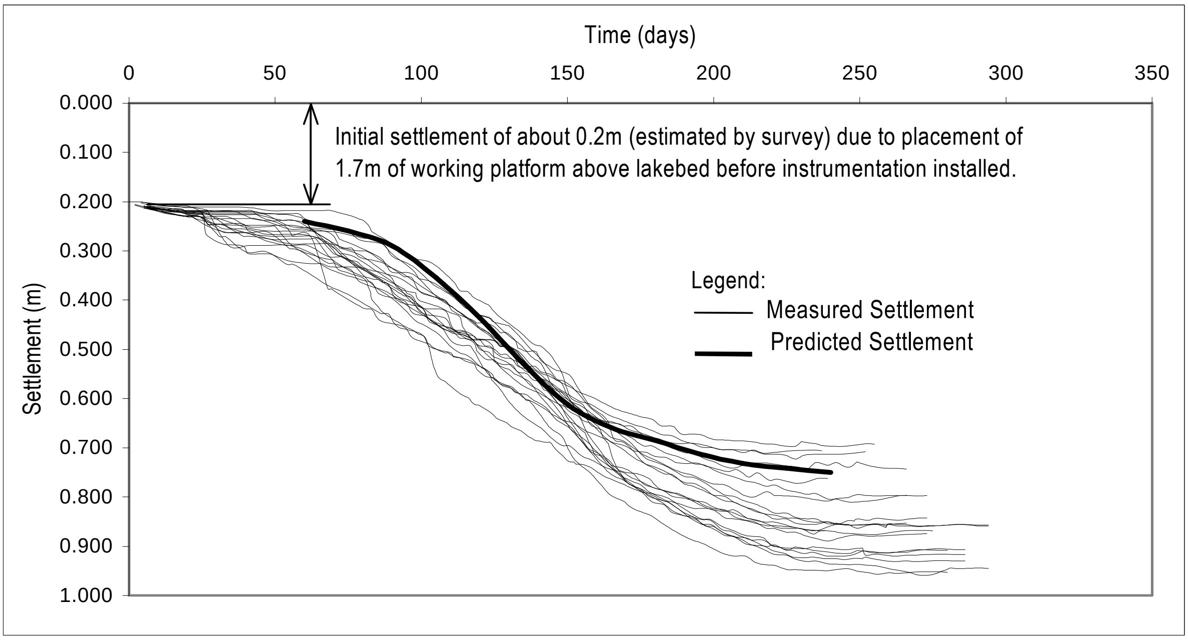


Figure 9: Settlement Monitoring Results (Phase 1).

The following observations were made from the monitoring results:

- On average, the settlement was under-estimated by 150 mm to 200 mm. This was assessed to be caused by a lower actual modulus of the DR columns based on the CPT results carried out through the columns after installation. Cone resistances were found to be only 8 MPa to 13 MPa, probably indicating a significant degree of particle breakdown of the crushed limestone due to heavy pounding in the DR process. Back-analysis of the results indicated a Young's modulus of 25 MPa for the DR columns would provide a better match with the observed settlements and this was used for designing the ground improvement for the remainder of the project.
- The estimated time-rate of settlement agreed closely with field performance, with 90% consolidation occurring in 3.5 months on average. However, due to soil variability and other factors such as uncertainties associated with smearing of the wick drains, there was a variation of ±0.5 months required to reach the required consolidation. While the observed variation was small, it could be critical for "fast-track" projects. To allow for soil variability and other uncertainties, the design c_v and c_h were reduced to 1.5 m^2/yr and 7.5 m^2/yr for subsequent stages of the project.
- Higher strength gain was observed compared to the design estimate. From the 100 kPa effective vertical stress increase resulting from the preload, an increase in undrained shear strength of 30 kPa was initially expected. Field vane shear testing, however, indicated that an increase of 70 kPa to 85 kPa was achieved in the upper part of the soft clay profile and an increase of about 40 kPa was achieved at depth. We suspect that this unexpected increase in undrained shear strength was the result of high lateral stresses induced in the soil due to the large displacement associated with the DR process. Such lateral stress increase causes an apparent increase in the over-consolidation ratio of the soil. This aspect is considered worthy of further research due to the obvious benefit that could be derived from the strength increase.

2 SUMMARY

Design methods for Chemical Lime Piles and Dynamic Replacement for the treatment of soft grounds have been described in this paper together with two case studies which were successfully implemented and achieved the desired outcomes. Both ground improvement methods are able to provide increases in strength and stiffness of the soil mass.

From the limited results of these two case studies, it would appear that both techniques have something in common in that the resulting increase in undrained shear strength gain after the ground treatment was higher than the theoretical prediction. While there may be a secondary chemical effect associated with the Chemical Lime Piles technique, the lateral displacement from swelling of the lime piles during hydration has a similar impact as the Dynamic Replacement technique. The lateral displacement obviously causes an increase in the lateral stress field and this is thought to produce an over-consolidation effect. This effect is considered worthy of further research due to the obvious benefit that could be derived from the strength increase.

3 REFERENCES

Alamigir, M., Miura N. and Madhav, M. R. (1993) “Analysis of Granular Column Reinforced Ground – I: Estimation of Interaction Shear Stresses” Reports of the Faculty of Science and Engineering, Saga University, Saga, Japan, Vol. 22, No. 1, 111-118.

Balaam, N.P., Booker, J.R. and Poulos, H.G. (1976) “Analysis Of Granular Pile Behaviour Using Finite Elements” *Proc. Int. Conf. Finite Elements in Eng*., 1-13, Adelaide, Australia

Bergado, D.T. and Lam, F.L. (1987) “Full Scale Load Test of Granular Piles with Different Densities and Different Proportions of Gravel and Sand in the Soft Bangkok Clay, *Soils and Foundations Jnl.,* Vol 27, No.1, 86-93.

Barron, R.A. (1948) “Consolidation Of Fine-Grained Soils By Drain Wells” *Trans. ASCE*, vol 113, 718-754.

Fell, R., Wong, P.K., & Stone, P. (1987) “Slope Instability In Soft Ground” Proceedings of an extension course on soil slope instability and stabilisation. (Ed.), Walker, B., and Fell, R. (Pub.), Balkema 231-278.

Hansbo, S., Jamiolkowski, M. and Kok, L. (1981) “Consolidation By Vertical Drains” Institution of Civil Engineers, Symp. On Vertical Drains. *Geotechnique* 31, 45-66.

ITASCA, (1999) 3D FLAC – “Fast Lagrangian Analysis of Continua” User Manual, ITASCA Group, USA.

Juillie, I. and Sherwood, D.E. (1983) “Improvement of Sabkhas soil of the Arabian Gulf coast.” *Proc. of the Eighth European Conference on Soil Mechanics and Foundation Engineering*, Helsinki, (Pub.), A.A. Balkema 781 - 785.

Lee, S.L. and Lo, K.W. (1985) “Ground Improvement By Dynamic Replacement And Mixing” *Third International Geotechnical Seminar on Soil Improvement Methods*, Singapore 27-29 November 1985, 19 - 30.

Madhav, M.R. and Nagpure,,D.D. (1996) “Design Of Granular Piles For Embankments On Soft Ground” *Proc. 12th SE Asian Geot. Conf.*, Kuala Lumpur, 1: 285-290.

Oneda Chemical Co. Ltd., (undated) “Deep Soil Stabilisation Method – Chemico-pile Method” Technical Manual.

Poulos, H. G. (2002) “Foundation Design For Very Soft Clays” Coffey Geosciences Pty Ltd Internal Research Development Report RD004/1-AB April 2002. (unpublished)

Schmertmann, J.M. (1955) “The Undisturbed Consolidation Of Clay” *Trans. ASCE*, Vol. 120, 1201.

Shahu, J. T., Madhav, M.R., Hayashi, S. and Miura, N. (1998) “Parametric Study of Soft Ground Reinforced with Granular Pile-mat System.” *Thirteenth Southeast Asian Geotechnical Conference*, 16-20 November, 1998, Taipei, Taiwan, ROC, pp 415 – 420.

Varaksin, S., Liausu, P., Berger, P., and Spaulding, C. (1994) “Optimisation Of Dynamic Consolidation And Dynamic Replacement Pillars To Limit Surface Deformations Of Man Made Fills Overlaying Heterogeneous Soft Subsoil” Proceedings of seminar organised by the Geotechnical Division of the Hong Kong Institution of Engineers on “Ground Improvement Methods” 103 - 116.

Watts, K. S., Johnson, D., Wood, L.A. and Saadi, A., (2000) “An Instrumented Trial of Vibro Ground Treatment Supporting Strip Foundations in a Variable Fill”, *Geotechnique* 50, No. 6, 699-708.

Wong, P. K. and Lacazedieu, M. (2004) “Dynamic Replacement Ground Improvement – Field Performance Versus Design Predictions for the Alexandria City Centre Project in Egypt” to be published in The A. W. Skempton Memorial Conference, 29 to 31 March 2004, London.