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Ground Improvement by Vibroflotation in Urban Residential Development

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Summary Ground improvement by deep compaction is frequently applied for commercial and industrial applications but is less frequently applicable for residential developments due to the lower intensity of loading and the cost. This paper describes one such application where vibroflotation has been used to improve the density of weak zones within superficial layers over limestone for a residential development in a southern suburb of Perth. Vibroflotation treatment was used for approximately 25 lots within the development and proved to be a cost effective and technically successful solution on a site with very extensive variations in ground conditions.

1. INTRODUCTION

Following the identification of localised areas of compressible peat and extensive areas of very loose sand beneath a site in the southern suburbs of Perth designated for residential development, a programme of ground improvement by excavation, replacement and vibroflotation was designed to render affected areas suitable for construction without caveats in relation to foundation requirements.

The development consisted of 96 lots, approximately 25% of which were located in areas where a high risk of excessive settlement was perceived due to the presence of very loose sand. Areas underlain by peat deposits were generally restricted to road reserves.

This paper presents an overview of site conditions and a description of the methodology adopted in the design of treatment by vibroflotation and in analysis of the effectiveness of the treatment.

2. THE SITE

The overall development comprised 9.5 ha of open land formerly used for market gardening, bounded to the north and east by existing residential properties and to the south by an access road. An area of environmental significance and dry lake formed the western boundary. The ground profile generally sloped from 8 m AHD on the eastern boundary to 1 m AHD along the western boundary at lake level. The area requiring ground improvement comprised the western half of the site with an overall area of approximately 5 ha.

3. GEOLOGICAL SETTING

Published information (Geological Survey of Western Australia) indicates the site area to be underlain by residual sands derived from the underlying Tamala Limestone of Pleistocene age. Bores in the vicinity typically indicate a depth to rock of less than 5 m. The conservation area is a perennial marsh comprising recent silts of lacustrine origin. The site location and regional geology are illustrated in Figure 1.

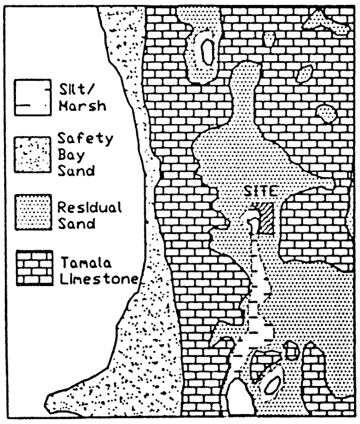


Figure 1. Geological setting.

The surface of the Tamala Limestone commonly exhibits solution cavities and surface calcrete and may be variably lithified. The limestone is comprised mainly of medium to coarse quartz sand with carbonate cement (calcareous eolianite). The exposed and variably leached upper surface is coincident with the eolian Spearwood Dune system described by McArthur and Bettenay (1960).

4. GROUND CONDITIONS

Review of the results of the initial site investigation, comprising 47 Electric Friction Cone Penetrometer tests (CPT) and 25 test pits, highlighted the presence of very loose sands in some areas, typically up to 10 m thick but occasionally in excess of 15 m. In other areas however, refusal on rock was attained at depths less than 5 m. The upper 4 m of the very loose sand generally exhibited cone resistance (q_c) in the region of 2.5-5.0 MPa but this decreased markedly to approximately 0.3 MPa (extremely loose) in the lower zone beneath which rock was generally identified (Figure 2).

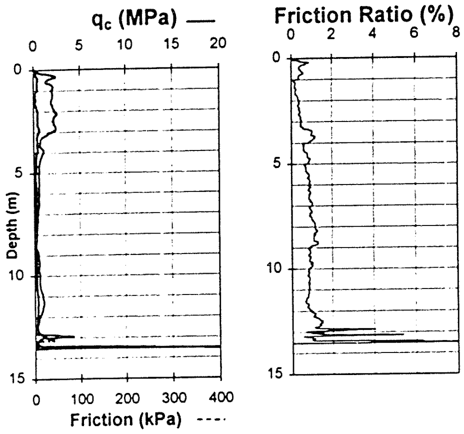


Figure 2. Typical CPT plot.

An additional 16 CPTs were carried out to further assess the distribution of the very loose materials and those areas most affected were identified by plotting contours on rock head. Where rock was found to be deepest this was generally associated with the thickest zones of extremely loose material. Distribution of deeper zones was shown to be somewhat irregular but the northern part of the site appeared to be affected more than the southern area.

In addition, extensive deposits of peat were identified fringing the lake area along the western site boundary. These deposits were up to 3 m thick on the boundary, thinning out eastwards over approximately 10-20 m. Groundwater was encountered at shallow depth in this low lying area.

It is considered likely that the combination of high groundwater and acidic peat deposits overlying the residual sands may have led to accelerated local dissolution of carbonate within the residual sand and upper surface of the limestone leading to a subsurface karstic morphology.

Values of q_c in the region of 2 MPa are not uncommon in these residual sands but values as low as 0.3 MPa are unusual. Particle size distribution plots indicate the lower deposits of extremely loose material to be coarser than the upper deposits. A potential explanation for the decrease in strength is that the finer grained upper sands comprised relatively tightly packed quartz grains on deposition, with a relatively low initial voids ratio and a soil mix deficient in carbonate content compared to the underlying limestone. The upper material would therefore be more resilient to dissolution than the surface of the limestone where high void ratios developed with leaching of carbonate due to percolation of acidic groundwater.

A schematic section indicating the conjectured subsurface conditions is given on Figure 3. Plots in which CPT probes had identified extremely loose sands were selected as being potentially problematical.

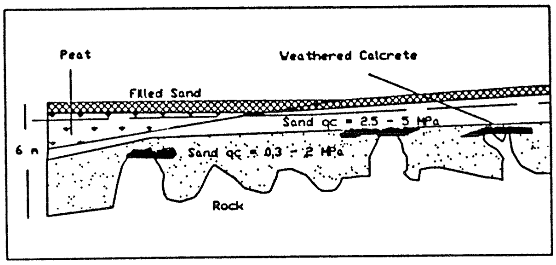


Figure 3. Schematic section.

5. POTENTIAL PROBLEMS

Although the exact details of the type of construction were not available, it was likely that single or two storey houses with slab on ground foundations would be favoured. Such foundations, whilst reducing the gross contact pressure on the subsoil, also stress the soil to a greater depth than strip foundations.

For a typical development lot of 600 m^2 with a plot development ratio of 55%, slab areas of 20 m × 15 m were assumed for general settlement analysis. Adopting typical two storey loads of 50 kN per metre run on external walls plus an allowance of 5 kPa for floor loading, gives an estimated contact pressure (assuming UDL) of 17 kPa.

In addition to the housing loads, regrading of the site would require placement of up to 2 m of controlled fill at some locations, resulting in an additional surcharge of approximately 36 kPa throughout the superficial deposits.

For centralised slabs within typical lots, a potential spacing of 7 m between adjacent structures was estimated. Assuming 1:2 shedding of load, stress interference could occur below 7 m depth, i.e. within the extremely loose material.

At this depth, equivalent flexible foundation dimensions up to a maximum of 88 m × 54 m were estimated for the largest blocks of lots. An average net stress increase at the interference surface of 53% q_n was estimated where q_n = net applied foundation load. Consequently, a significant amount of the total imposed load would be transmitted throughout the deep areas of extremely loose sands.

For the surcharge fill, settlements of approximately 140 mm were estimated. In reality, settlement of the fill would occur prior to house construction (a five month delay was envisaged). Estimated maximum settlements of slabs subsequently constructed on the new fill were in the region of 70 mm (Figure 4). Up to 70% of this settlement occurred in the deep, very loose sand deposits.

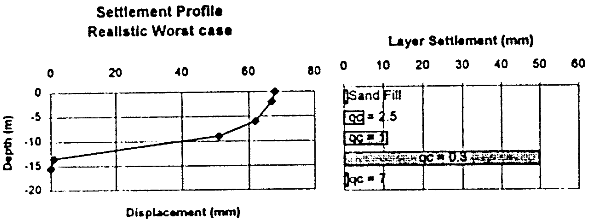


Figure 4. Estimated settlement profile.

Further settlement due to creep was also a concern. It has been suggested that creep settlement in low density calcareous sand can be significant due to structural alterations and void development and may be greater than the construction settlement (Smith, 1990).

In addition, correlations between the low cone resistance, typical grading curves and earthquake acceleration for the Perth metropolitan area indicated that the very loose saturated sands could be susceptible to liquefaction under relatively minor earthquake events (Robertson and Campanella, 1985).

6. GROUND IMPROVEMENT OPTIONS

Various options were considered to achieve a required level of ground improvement so as to permit unrestricted standard residential construction throughout the sub-division. These options included:

(a) Dynamic compaction; and
(b) Bulk Excavation and conventional compaction.

The former was discarded due to the proximity of built up areas whilst the latter proved to be more expensive than the vibroflotation/stone columns option adopted.

7. VIBROFLOTATION DESIGN

7.1 Column Depth and Spacing

In those areas identified as being affected by significant thickness of very loose sand, a programme of deep compaction by vibroflotation was devised. Due to the extremely variable profile of the very loose sand and the variation in final proposed ground level on the site, it was considered impractical and unnecessary to treat the material to its full depth, i.e. rock head. As the upper 4-5 m was relatively consistent laterally, it was decided to concentrate the treatment within this material to effectively form a stiffened raft and also generally reduce the thickness of the very loose material to less than 5 m, allowing a less severe earthquake classification in accordance with AS1170.4.

By inspection of the friction ratios measured in the CPT tests which were generally 0.5-1%, the upper materials were considered suitable for densification since it has been found that soils with a friction ratio less than 1% almost always densify (Osborne *et al.*, 1994). Grading curves from the site investigation data show the material to fall within the grading envelopes commonly accepted for suitable material (Figure 5) (Glover, 1982).

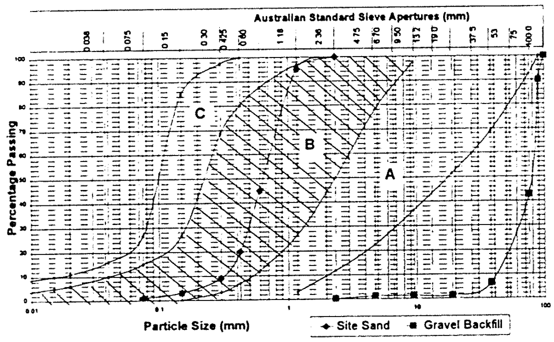


Figure 5. Grading envelope for vibrocompaction.

For design, a target depth for treatment of 6 m below finished ground level was selected, resulting in treatment depths from existing ground level of between 4 m and 6 m.

The creation of a stiffened raft would reduce settlement in the upper deposits, but perhaps more importantly reduce stress levels in the deeper deposits. The Boussinesq distribution of stress tends to overestimate stresses in material overlain by a stiff layer. To overcome this, the stiff upper layer is increased in thickness for the purpose of calculating stresses in the compressible stratum by a factor equal to the ratio of its stiffness relative to that of the compressible stratum (Figure 6). A maximum increase in the region of 15% has however been suggested for stiffness ratios greater than 10 (Winterkorn and Fang, 1975).

Adopting a relationship between cone resistance and modulus for loose sand of:

$$E_s = 4 \times qc \tag{1}$$

where: E_s = modulus of elasticity of sand (MPa)
q_c = cone resistance from CPT (MPa)

an initial stiffness ratio of between 5 and 8 was generally estimated for the upper and lower sand layers. If by introduction of stiff stone columns and an improvement in relative density of the soil around the columns, this stiffness ratio could be increased to say 25, the estimated increase in vertical stress at the zone of interference would reduce to between 5% and 20% q_n.

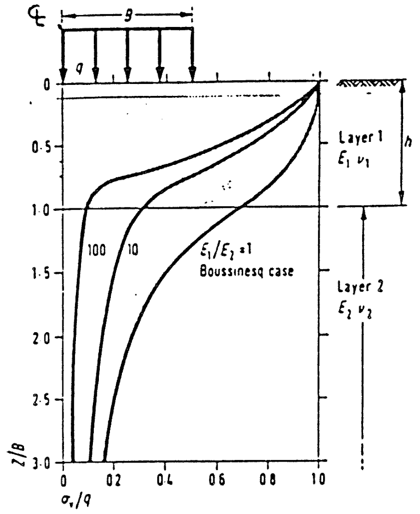


Figure 6. Reduction in vertical stress due to stiff overlying layer (circular foundation radius = B). (Modified from Burminster, 1958)

A reduction in stress levels from 53% q_n to 20% q_n was estimated to reduce settlement in the deep, very loose deposits to approximately 10 mm.

To assess the equivalent modulus of elasticity of ground treated by vibrocompaction, the following simple relationship was adopted derived from a consideration of relative areas:

$$E_{eq} = E_s\{A_c/(FS)^2[E_c/E_s - 1] + 1\} \qquad (2)$$

where:

E_q = Equivalent average modulus of elasticity after ground improvement (MPa)
E_s = Modulus of elasticity - in-situ soil (MPa)
E_c = Modulus of elasticity - stone column (MPa)
F = Pattern factor
= 2 for square pattern
= 0.866 for triangular pattern
S = Spacing of vibroflot probes (m)
A_c = Area of stone column (m^2)

Typical vibrocompaction treatment methods involve treatment points in a square or triangular pattern at regular spacings. As the lateral effects of treatment vary inversely with distance from the treatment centre for a given spacing, a triangular layout gives an improved aerial coverage with the point of least improvement being the centre of the triangle and potential overlapping effects occurring within the triangle itself. Suitable treatment point spacings in the loose relatively clean sands commonly encountered in the Perth area have generally been found to be between 2.5 m and 3.5 m to achieve acceptable levels of improvement in relative density of the soil surrounding the columns.

7.2 Column Backfill

Backfill materials to the hole created by the cylindrical vibrator are commonly natural sand or imported stone. The use of stone provides benefits in the form of higher rates of production and the creation of a stiffer central zone within the treatment pattern leading to an increased equivalent soil stiffness for settlement analysis. From back analysis of settlement records and plate load testing, modulus of elasticity of a stone column installed in sand have been estimated to be as high as 250 MPa (Osborne *et al.*, 1994). However for this project a lower bound value of 150 MPa was adopted.

Local experience within Perth sands indicates that typical column diameters of 1.1 m are obtained using similar vibroflotation equipment to that used on this site.

For this project, stone was specified to be used in the zone below the water table (approximately 0.5 m below ground level on the western boundary) with the provision for sand fill to be used above this zone. In six lots close to the western boundary up to 2 m of peat was removed and replaced with loosely placed sand prior to vibrocompaction. Since there was a possibility of thin layers of peat remaining, stone fill was specified throughout in these areas.

The specification required the stone to be a coarse graded material with 100% passing the 100 mm sieve and not more than 5% passing the 75 µm sieve. The particle size distribution for the backfill utilised is shown on Figure 6. Using the rating system presented by Brown (1977), this material was shown to rate as "excellent" with a suitability number (S_N) <0.2, from

$$S_N = 1.7\sqrt{\{3/(D_{50})^2 + 1/(D_{20})^2 \times 1/(D_{10})^2\}} \qquad (3)$$

where D_{50}, D_{20}, D_{10} = grain size in millimetres at 50%, 20% and 10% passing on a grain size analysis.

S_N	0-10	10-20	20-30	30-40	>50
Rating	Excellent	Good	Fair	Poor	U/S

The stone backfill was fresh limestone, however the potential for dissolution of the material forming the columns was not considered significant. The solution of intact limestone is a very slow process, for instance, Kennard and Knill (1968) quoted mean rates of surface lowering of limestone areas by solution in the British Isles which ranged from 0.041-0.099 mm annually. Although this rate of dissolution may be accelerated by acid groundwater conditions, it was considered unlikely that loss of column strength would occur during the design life of the structures.

8. TREATMENT ECONOMICS

For a given treatment area, significant cost savings can be made by increasing the spacing of probes even by 0.25 m. Figure 7 shows the relative savings which may be expected for a nominal treatment area of 500 m^2 with triangular spacings varying from 2.5 m to 3.5 m and an assumed column length of 5 m.

In initial site trials a triangular spacing of 3 m was selected for this project giving an area/probe ratio of 7.79 m^2 ($= 0.866 \times 3^2$).

Under these conditions, even if no improvement in the soil surrounding the columns were achieved such that $E_s = 4 \times 2.5 = 10$ MPa, equation 2 indicates that an equivalent modulus in the region of 33 MPa would be obtained. This gives a stiffness ratio of $33/4 \times 0.3 = 27.5$ which was considered to be sufficient to reduce predicted settlement to acceptable levels (<25 mm).

As a result of field testing (See Section 9) stone column spacing was increased to 3.25 m.

On average the cost of the vibroflotation treatment was $19,000 per lot. This represented a marginal development cost increase of $4,500 per lot for the overall development. This additional cost was considered to be more than offset by being able to market the development free of caveats on required building foundations.

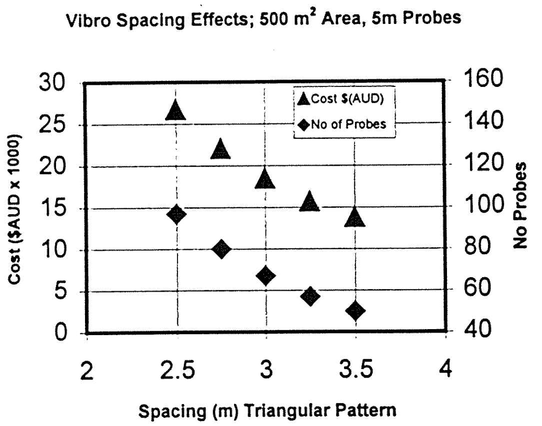


Figure 7. Influence of probe spacing on cost.

9. FIELD OPERATIONS

The equipment used for the vibrocompaction comprised both electrical and hydraulic powered poker vibrators suspended from crawler mounted cranes. Both types of poker required a significant supply of water sufficient for jetting during penetration and maintain hole stability during compaction.

The excavation and replacement of peat deposits along the western boundary indicated that significant water ingress occurred into excavations with water levels rising to 0.5 m below ground level overnight. Inflows were particularly strong where limestone pinnacles were encountered in the base of the excavation. The presence of these pinnacles also highlighted the variable nature of the subsurface morphology. Whilst this was problematical for the excavation and replacement process, it allowed the creation of water supply ponds for the vibro process. The water supply ponds also doubled as silt traps for arisings from the vibroflotation probes with return water being channelled back to the ponds.

Electric flots were initially used, however due to a series of breakdowns they were subsequently replaced with lower power hydraulic flots giving the opportunity to compare the effectiveness of different machines (see Section 9).

The jetting of water and the vibration of the poker created a hole which, once the target depth or refusal had been reached, was progressively backfilled with stone by a front end loader and compacted by the flot during staged withdrawal. A measure of relative compaction was obtained by either power consumption (amps) or hydraulic pressure measurement in the cab of the crane.

In order to obtain an estimate of approximate column diameter (relevant to estimation of equivalent soil modulus), the volume of stone introduced at each location was recorded. This generally took the form of number of front end loader buckets per column. Adopting a compaction factor of 85%, typical column diameter of 1.2 m was estimated for probes on this site. This exceeded design requirements.

The length of a particular column was measured by use of 1 metre markers on the poker body. Some care was required in depth measurement as the ground surface tended to mound at the top of the hole during poker penetration giving a rising datum as penetration proceeded.

Once a block of lots had been treated, the ground surface was graded and rolled. Allowance had been made for the removal and replacement of up to 500 mm of silty arisings, however as the material was predominantly medium to coarse sand, removal was not considered necessary.

10. TESTING

In order to assess the effectiveness of the method, CPT testing was carried out in selected lots following treatment.

Generally, test locations were selected where previous CPT's had been performed during the site investigation.

Each test consisted of a group of 3 CPT tests between a particular column and the centre of the triangle formed with adjacent columns. For 3 m spacings, the CPT test locations were:

(a) at the edge of the column (typically 600 mm from column centre);
(b) 950 mm from column centre;
(c) 1300 mm from column centre.

In general, two of the probes were taken to a depth of 1 m below the treated zone, while the third was taken to the anticipated depth of loose material indicated by previous CPT's or to refusal.

Figure 8 shows a typical set of results from an area treated on a 3 m spacing using the electric flot. The depth of treatment was 5.3 m. P48 represents the pre-treatment conditions.

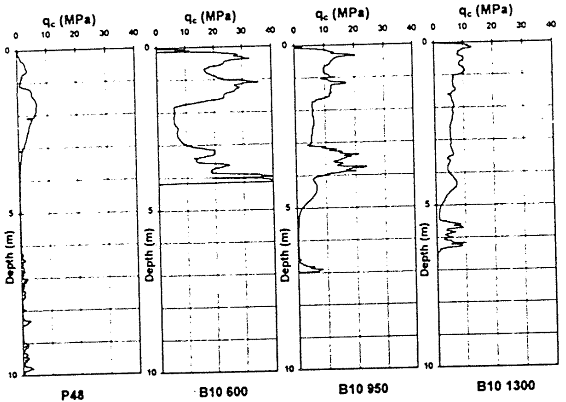


Figure 8. CPT results, electric flot.

The results indicate a marked increase in soil density, particularly within the upper 2 m and between 3 m and 5 m where the lower sand was reached. As expected, the effects of treatment decrease rapidly away from the column although some improvement is still evident at 1300 mm from the stone column.

Figure 9 shows a similar set of results from an area treated to 4.1 m using the hydraulic flot. The results again indicate that improvement has occurred within the upper 2 m in particular, however below 2 m the improvement is less apparent apart from at the column edge. The general level of improvement is also much less than that previously recorded with the electric flot, reflecting the influence of differing power output with this machine.

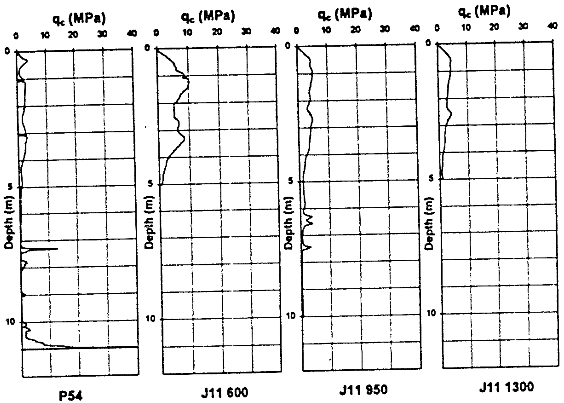


Figure 9. CPT results, hydraulic flot.

Settlement analysis indicated that the treatment was more than adequate irrespective of probe type, and probe spacings were subsequently increased to 3.25 m.

Figure 10 illustrates the variability of the subsurface profile even over a very short distance. The original CPT profile (P59) suggested that rock would be encountered at around 3.5 m. The first two test CPT's at this location (650 mm and 1100 mm) seemed to support this whilst also demonstrating the improvement in density due to vibrocompaction. However in the third test (C5 1500), material with cone resistance decreasing from 5 MPa to 0.3 MPa was identified to at least 12 m depth. This may be indicative of the presence of a steep sided pinnacle or a localised sand filled solution cavity and highlights the potential pitfalls associated with site investigation in such areas.

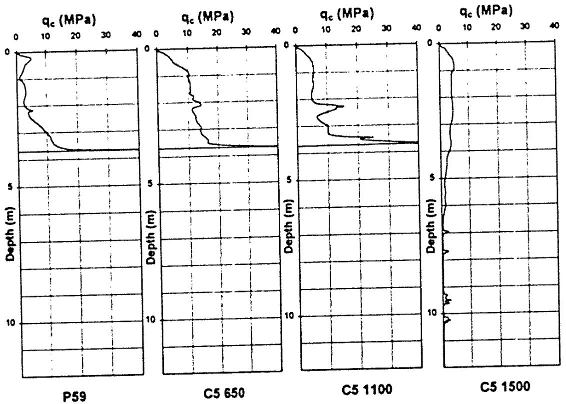
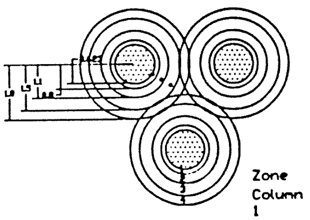


Figure 10. Lateral variation in stratigraphy.

In some of the test CPTs, the presence of thin stiff layers (possibly relict calcrete horizons) was identified typically around 5 m depth, below which extremely loose conditions continued. This situation had not been readily apparent from the site investigation probes, some of which may have terminated in such 'hard' layers, failing to reveal the low density material below. However, as the presence of extensive deep areas of extremely loose material had been allowed for in design and it was likely that the vibro pokers would also refuse on the hard layers, it was not considered necessary to alter the method or increase the scope of works.

The results of the CPT testing were used to estimate the improved soil modulus using the following simple procedure which takes into account the lateral variation in improvement with distance from the column.

The area of influence of the test column was divided into concentric rings and a weighted value of improved cone resistance assigned to each ring on the basis of relative areas (Figure 11).



Zone	Radius	Area	Rel Weight
Column	0.623	1.31	
1	0.80	0.70	0.08
2	1.10	1.79	0.20
3	1.50	3.27	0.37
4	1.80	3.11	0.35

• CPT Test Location 3.25m Spacing

Figure 11. CPT test pattern and soil q_c factors.

Table 1. Derivation of equivalent treated soil modulus.

J11 Test Area										Soil Mod	Vib	Equiv
Depth	P29	646	996	1348	Zone 1	Zone 2	Zone 3	Zone 4	qc	qc x 4	Y/N	Mod
					0 08	0 2	0 37	0 36				
00-05	3	2 5	2 5	2 5	0 2	0 5	0 925	1 05	2 68	10 7	Y	30 60
05-10	2 5	7	5	5	0 56	1	1 85	0 875	4 29	17 14	Y	36 12
10-15	3	10	5	4	0 8	1	1 48	1 05	4 33	17 32	Y	36 28
15-20	3	7 5	4	3	0 6	0 8	1 11	1 05	3 56	14 24	Y	33 64
20-25	4	5	3	3	0 4	0 6	1 11	1 4	3 51	14 04	Y	33 47
25-30	3 5	7	5	3 5	0 56	1	1 295	1 225	4 08	16 32	Y	35 42
30-35	3 5	8	4	2 5	0 64	0 8	0 925	1 225	3 59	14 36	Y	33 74
35-40	2	5	3	2	0 4	0 6	0 74	0 7	2 44	9 76	Y	29 80
40-45	1	2	2	1	0 16	0 4	0 37	0 35	1 28	5 12	Y	25 82
45-50	2	1 5	1	1	0 12	0 2	0 37	0 7	1 39	5 56	N	5 56
50-55	1		1						1	4	N	4 00
55-60	10		1						1	4	N	4 00
60-65	1		2						2	8	N	8 00
65-70	1		1						1	4	N	4 00
70-75	2		0 8						0 8	3 2	N	3 20
75-80	10		0 4						0 4	1 6	N	1 60
80-85			0 4						0 4	1 6	N	1 60
85-90			0 4						0 4	1 6	N	1 60
90-95			0 4						0 4	1 6	N	1 60
95-100			0 4						0 4	1 6	N	1 60

The sum of the weighted q_c values was then obtained for each depth increment of 0.5 m. Below the depth of treatment, lower bound q_c values from the deepest of the three tests and/or the original site investigation CPT were taken for the soil mass. The results of such a procedure and the subsequent derivation of equivalent soil modulus are shown in Table 1. The values of equivalent soil modulus so obtained were used to estimate settlement for individual slabs or blocks of slabs. The results indicated that estimated settlements within the treated area reduced by up to 85% and that in all cases, settlement of less than 20 mm were predicted beneath slabs.

11. CONCLUSIONS

Vibroflotation involving the installation of stone columns has been utilised to provide ground improvement to a very loose stratum overlying limestone for a residential sub-division. The process proved to be cheaper than alternatives and meant that no caveats needed to be applied to the developed lots in relation to required foundation construction methods. Testing during the initial stages of the project enabled a wider spacing to be adopted for subsequent work. Ongoing testing demonstrated the adequacy of treatment.

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