Vibroflotation of Calcareous Sands

D. C. ANDREWS
Partner, Dames & Moore, United Kingdom
D. B. McINNES
Ground Engineer, Public Works Department, Perth, Western Australia.

SUMMARY

This paper describes a vibroflotation field trial conducted in loose, calcareous sands. The results of the trial indicated that little ground improvement occurred in material which was expected to show substantial density increase following treatment. Probable reasons for the ineffectiveness of the vibroflotation technique are discussed. The effect of the ground vibrations induced on nearby structures and detailed operational data are also presented. The work reported is an example of a trial, conducted during the planning phase of a structure, intended to assess the feasibility and possible advantages of one form of ground treatment compared with other foundation systems.

1. INTRODUCTION

In late 1976 field trials were carried out at the site of proposed major additions to Fremantle Hospital in Western Australia to evaluate the effectiveness of vibroflotation for compacting loose, calcareous sands.

This paper describes the work undertaken and presents the results obtained and conclusions drawn from the study.

The 0.54 hectare site of the proposed additions to Fremantle Hospital is located in an area bounded by South Terrace, Alma Street and existing hospital facilities and is shown on Figure 1. The additions will comprise four adjoining structures, three eleven-storey buildings together with a five-storey building. The layout of these buildings is also shown on Figure 1.

The three high-level buildings will be constructed using post-tensioned primary beams and reinforced concrete columns and walls. The five-storey building will be constructed using reinforced concrete flat plate slabs supported on reinforced concrete columns.

Underside of basement floor level is RL-0.07 AHD (Australian Height Datum). This is approximately seven metres below the level of South Terrace with the lower two storeys of each building forming a double basement to the whole complex.

Foundation design studies by engineers of the Public Works Department of Western Australia, Architectural Division based on site investigation (Ref. 1) indicated that a piled foundation system would be the most satisfactory both technically and economically. However, since early 1973 when these studies were completed, construction costs have

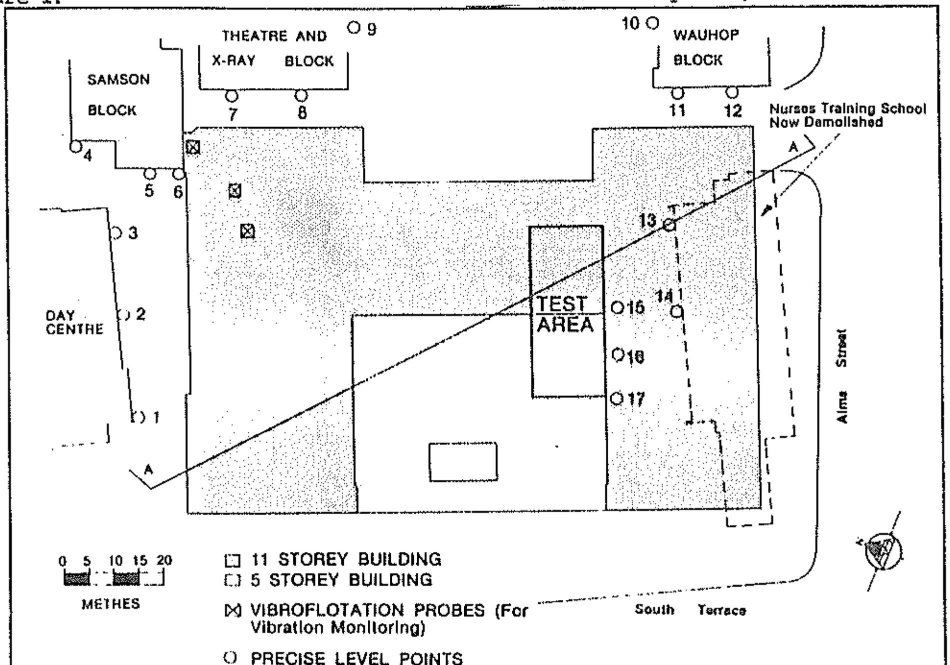

Figure 1 - Plan of Fremantle Hospital Site and Vibroflotation Test Area

risen significantly and differentially to the extent that, if technically sound, a shallow foundation system supported on ground compacted using vibroflotation could yield substantial time and cost savings.

The latter development prompted additional investigation into the use of vibroflotation and shallow foundations which culminated in the field trials described in this paper.

2. SITE CONDITIONS

Subsurface soil and rock conditions were initially examined by investigations conducted during the periods 1971 to 1973 (Ref. 1). Further investigation was conducted at the time of the vibroflotation trial (Ref. 2) and additional boreholes were drilled during the subsequent piling contract primarily to assess pile founding conditions within the sandstone beneath the site (Ref. 3).

Generalised subsurface profiles across the site are shown on Figure 2.

Material underlying the site to a depth of approximately 20 metres is believed to belong to the Safety Bay Sands of Recent age comprising aeolian and sedimentary deposits overlying calcareous sandstone. It is probable that material below this depth belongs to the Tamala Limestone Series (otherwise known as "Coastal Limestone") of Quaternary Age. (Ref. 4).

A generalised description of the various strata penetrated is given in Table 1.

The uppermost unit shown in Table 1 is dune sand and exists in a medium dense state with SPT values generally ranging between 12 and 60, the higher values probably indicating cemented zones.

The central unit contains a wide variety of material types, many of which are in a very loose condition. Typically, SPT values obtained were less than ten, many less than five and several times the SPT sampler fell up to two metres after one blow of the drop hammer. A gradation envelope constructed from gradings of a number of samples in this unit is presented in Figure 3 and illustrates the size range of material encountered.

TABLE 1

SUMMARY OF SUBSURFACE CONDITIONS

Reduced Level (metres)		Description	Density or Strength
Top	Bottom		
Up to +6.9	-1.0 to -5.0	SAND, calcareous, white, pale grey or pale brown, fine to coarse grained. Weakly cemented in part.	Medium dense to very dense.
-1.0 to -5.0	-6.1 to -16.8	SILTY SAND and SAND with discontinuous calcareous SANDSTONE bands. Very shelly. Contains decomposed organic matter.	Loose to very loose.
-6.1 to -16.8	-30.3*	Calcareous SANDSTONE, white to yellow, fine to coarse grained. Mainly moderately cemented, some calcreted surfaces. Cavernous in part.	Very weak to medium strong.

*Only one borehole penetrated the calcareous sandstone. This borehole intersected stiff black clayey silt of the Osborne Formation at RL-30.3.

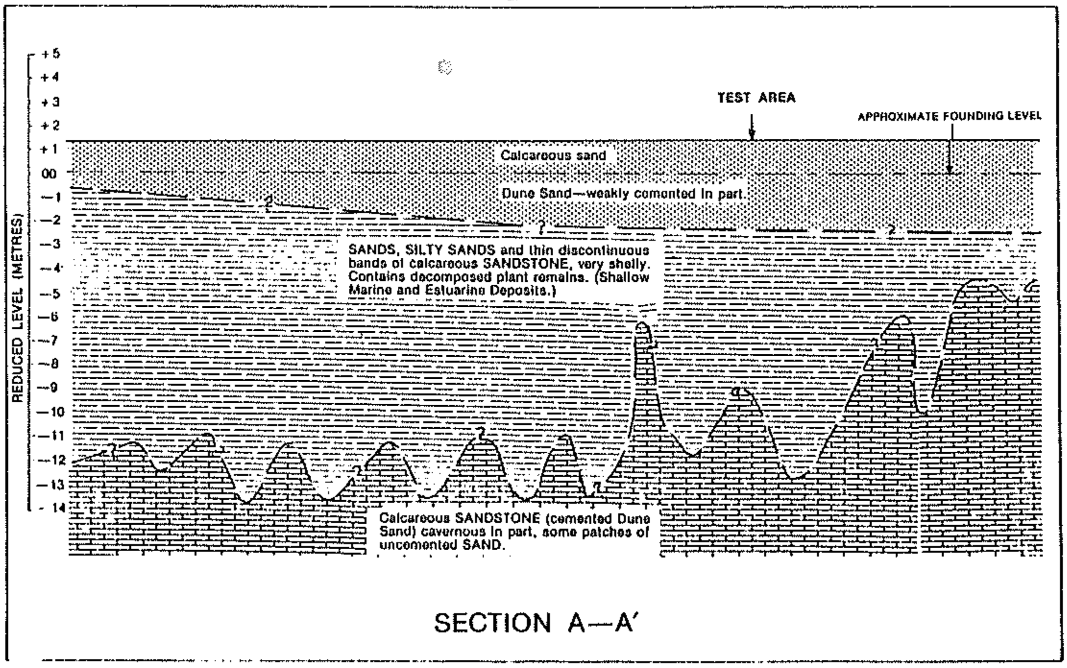

Figure 2 - Generalised Subsurface Profiles Across Site (Refer Figure 1 for Section A-A

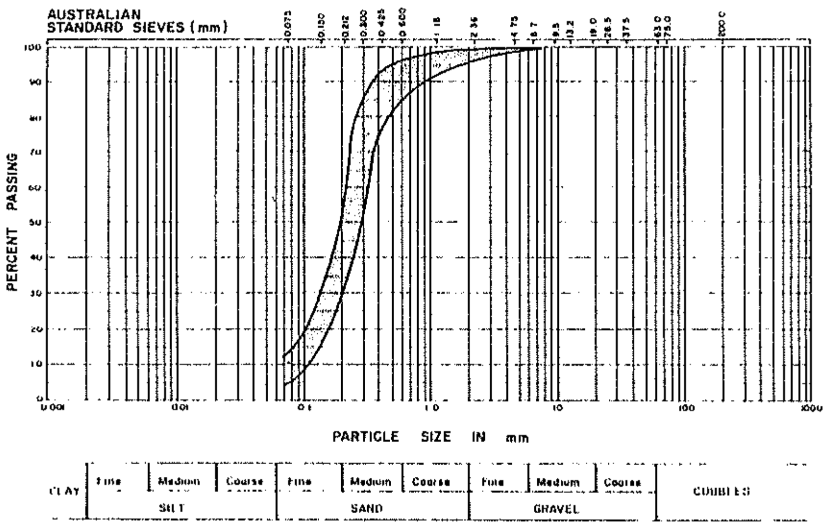

Figure 3 - Envelope of Particle Size Distribution Curves for Material from Central Unit, Table 1.

The lower unit is a calcarenite or calcareous sandstone formed of cemented dune sand. The upper surface is irregular. Closely spaced vibroflotation probes and boreholes indicated variations in levels of several metres, suggesting that the "surface" is in the form of a series of steep-sided pinnacles. There is evidence of cavities in this material and the degree of cementation varies widely.

Groundwater level was observed in a number of boreholes and occurs at approximately RL + 0.6.

3. TRIAL PROGRAMME

The vibroflotation trial had two main purposes. One was to evaluate the effectiveness of vibroflotation as a means of densifying the underlying loose silty sands and the other was to examine the impact of vibrations upon adjacent buildings during the operation of the vibroflot.

An area of the site was selected for the trial and is shown on Figure 1. The main basis for selection of this area was that it had been excavated to approximate founding level and was located clear of other work progressing on site.

Prior to the trial three boreholes were drilled in the trial area to investigate subsurface conditions in the zone above the calcareous sandstone. In addition five Dutch cone probes were performed within or close to the trial area. The results of this work are shown on Figures 4 and 5 respectively.

A suitable pattern of vibroflotation probes was then designed and agreed to by those involved in the trial. This pattern is shown on Figure 6. Further probes using the in situ sand and slag respectively as backfill, in place of the crushed rock used in the main trial, were installed after the main trial was completed. The locations of these probes are also shown on Figure 6.

The 75 kw vibroflot used in the trial is owned by the Vibroflotation Foundation Company and operated locally by Grouting and Foundations (W.A.) Pty Ltd.

Since some concern over the effect of the vibroflot operation on adjacent buildings was expressed, it was decided to monitor both vibrations and levels at a number of points around the site perimeter. To implement this a number of stainless steel precise level studs were installed in adjacent buildings and in concrete paving slabs on the floor of the excavation; these were then surveyed using precise levelling techniques. The locations of these precise level points are shown on Figure 1.

Before vibroflotation work commenced a photographic survey of all adjacent buildings was undertaken; this was augmented with notes concerning existing structural damage, cracks etc.

During the vibroflotation trial, ground vibrations were measured at several locations around the site using a Sprengnether seismograph. In addition noise levels were monitored during operation of the vibroflot.

Approximately 48 hours after the completion of the vibroflotation trial, three exploratory boreholes were drilled in the trial area at the locations shown on Figure 6. The borehole locations were selected to examine the soil density in the centre of four vibroflotation probes, in the centre of three probes and close (0.6 metres) to a probe centre. Dutch cone probing was also undertaken. The results of this works are shown on Figures 4 and 5 respectively.

At this stage a survey of all the precise level points was carried out.

The vibroflot was then moved to the northeast of the site and three probes were installed at locations progressively closer to the Samson Building (See Figure 1). The purpose of this work was solely to measure ground vibrations; again these were monitored using a Sprengnether seismograph. On completion of this work another survey of the precise level points was undertaken.

Following completion of the main vibroflotation trial the vibrflot was used to compact an area to the west of the trial area using in situ sand as backfill. The following day one borehole was drilled in the centre of three probes to measure the densification achieved. Later, an additional eight vibroflot probes were installed at a closer spacing than used in the main trial, utilising slag as backfill. The success of this operation was checked by drilling, testing and sampling at the centre of these probes. The locations of these probes and boreholes are shown on Figure 6 and the results of drilling and testing are shown on Figure 7.

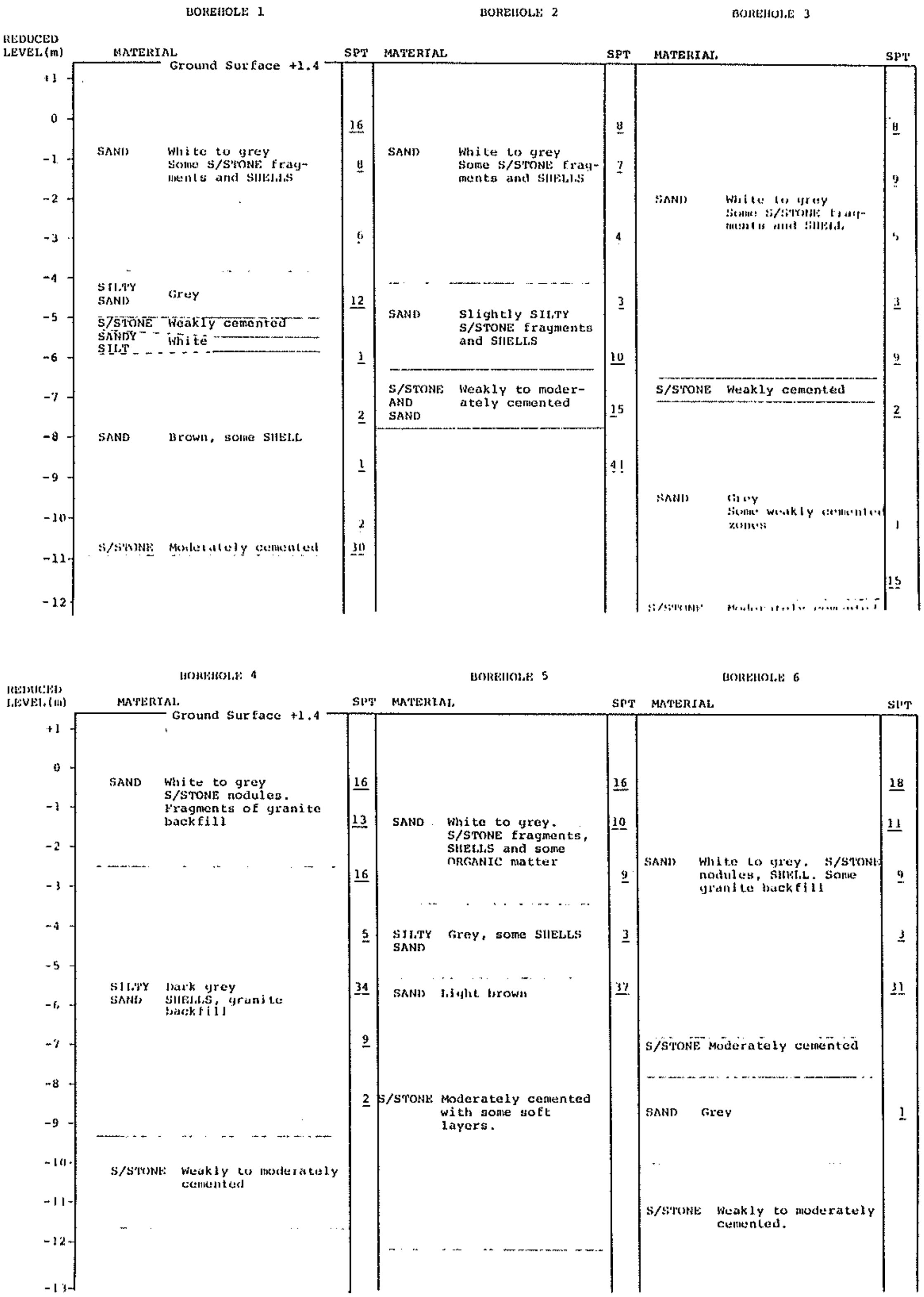

Figure 4 - Borehole Logs and SPT Results Prior to and Following Vibroflotation in Test Using Crushed Rock Backfill. (Refer Figure 6).

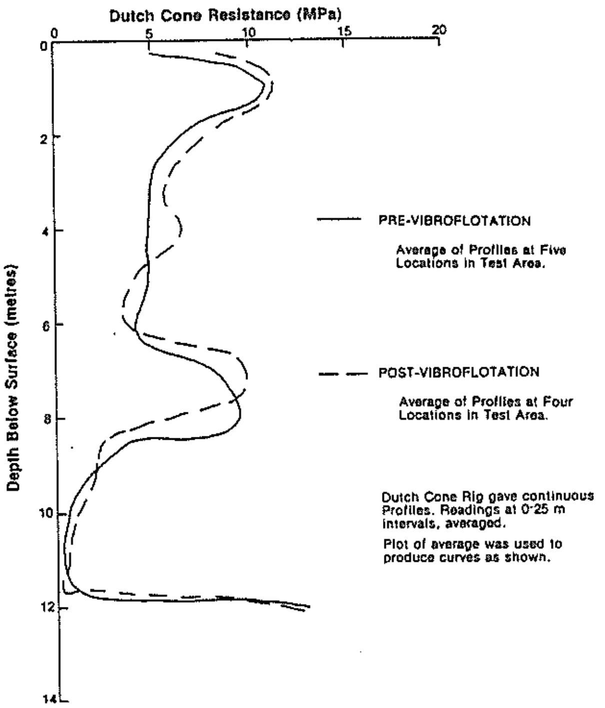

Figure 5 - Dutch Cone Resistance Prior to and Following Vibroflotation.

4. RESULTS OF TRIAL

4.1 Vibroflot Operation

During the vibroflotation trial the time taken to complete each probe, the depth of penetration of the vibroflot and the approximate volume of backfill used to form each stone column were recorded. These data are summarised in Table 2. These volumes were used to estimate the approximate diameter of the stone columns and this information is also presented in Table 2.

4.2 Evaluation of Compaction

Evaluation of the degree of densification achieved was based on a series of precompaction and post compaction Standard Penetration Tests and Dutch Cone probes. The locations of these tests are shown on Figure 6 and the results of the tests are shown on Figures 4, 5 and 7 respectively. Dutch Cone probing was only carried out in the area where crushed rock was used as backfill.

Examination of Figures 4 and 5 indicates that the vibroflotation achieved only slight increases in soil density. Figure 7 presents both results of SPT conducted in an area compacted using in situ sand as the backfill and in an area compacted using slag as backfill with stone columns more closely spaced than in the main trial. Again, perusal of these results also suggests that only slight soil density increases were achieved.

4.3 Precise Level Survey

Overall, the precise level data indicated that no measurable settlement of the buildings adjacent to the excavation occurred during the vibroflotation trial.

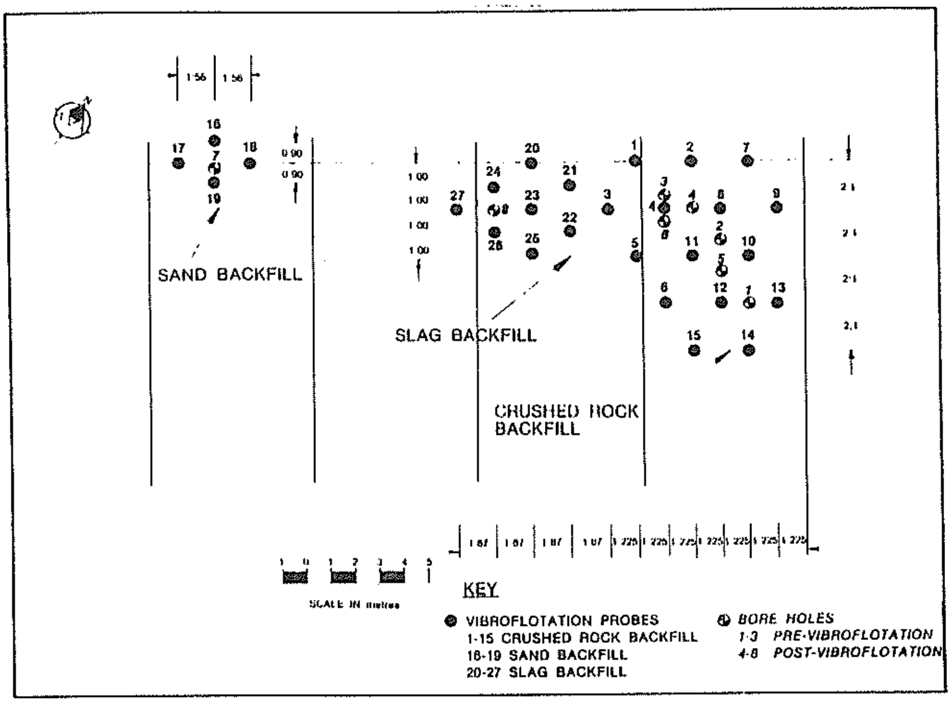

Figure 6 - Pattern of Vibroflotation Probing and Location of Boreholes in Test Area.

TABLE 2 DETAILS OF VIBROFLOTATION PROBES

Number*	Depth (metres)	Time Taken To Probe (minutes)	Time Taken To Backfill (minutes)	Backfill Used (cubic metres)	Estimated Diameter of Probe (metres)
a)	Crushed Rock Backfill Probes	(In Order of Completion)			
1	11.89	3	16	10.8	1.07
2	11.89	2	16	10.8	1.07
3	7.93	2	13	7.7	1.11
4	8.24	2	16	9.2	1.19
5	14.34	2	14	12.3	1.05
6	13.27	3	17	15.4	1.22
7	10.37	3	14	12.3	1.23
8	8.85	4	11	10.0	1.02
9	10.37	2	13	10.8	1.15
10	10.06	2	12	7.7	0.99
11	13.57	4	12	13.8	1.14
12	9.61	4	12	9.2	1.10
13	7.02	3	-	-	-
14	7.02	-	-	6.2	1.06
15	9.46	3	11	7.7	1.02
Average Depth	10.26 m	Average Backfill Per Metre of Probe 1.0cu.m		Average Diameter 1.11 m	
b)	In-situ Sand Backfill Probes				
16	9.07	4	34	7.3	1.01
17	11.29	4	11	5.4	0.78
18	9.53	3	11	6.2	0.91
19	12.20	3	11	7.7	0.90
Average Depth	10.52 m	Average Backfill Per Metre of Probe 0.6cu.m		Average Diameter 0.90 m	
c)	Crushed Slag Backfill Probes				
20	7.63	2	20	13.8	1.52
21	7.63	2	17	13.8	1.52
22	7.63	2	12	9.2	1.24
23	7.63	3	14	10.0	1.29
24	7.63	3	11	6.9	1.07
25	7.63	2	14	9.2	1.24
26	7.63	3	10	8.5	1.19
27	7.63	3	11	10.0	1.29
Average Depth	7.63 m	Average Backfill Per Metre of Probe 1.3cu.m		Average Diameter 1.30 m	

* Refer to Figure 6 for probe location

4.4 Ground Vibration Monitoring

The vibroflot specification indicates that when in operation it has a free standing horizontal amplitude of approximately 10 mm at a frequency of 30Hz.

During installation of a vibroflotation probe two points of maximum vibration were observed. These were at the initial penetration of the vibroflot into the ground and at the start of backfilling to form the stone column. At most other points during vibroflotation the vibrations were approximately constant with the backfilling operation producing a slightly larger vibration than penetration.

Measurements taken approximately three metres from the vibroflot indicated predominantly horizontal vibrations, although the vertical component was quite large on initial penetration of the vibroflot. The maximum vibration amplitude was 0.13 mm at the start of backfilling and the average amplitude thereafter was approximately 0.08 mm. The major frequency recorded was 60Hz (although 30 Hz did occur in some cases) giving maximum peak accelerations of the order of 1.1 g both vertically and horizontally.

At a distance of 15 metres from the vibroflot, the average vibration amplitude was of the order of 0.02 mm with peaks of up to 0.04 mm. The influence of the vertical vibration was more evident in this case than at three metres from the vibroflot. The major frequency recorded was 30Hz in all cases - this being the frequency of vibration of the vibroflot - giving maximum peak accelerations of 0.04 g both vertically and horizontally.

4.5 Noise Levels

In general, the noise produced by the vibroflot could not be heard above the noise of the generator and the front end loader moving the stone backfill. The noise from the generator was measured at 86dB(A) at a distance of seven metres although this was located near a large earth embankment which would constitute a reflective surface. The noise level recorded could hence be reasonably reduced by approx-

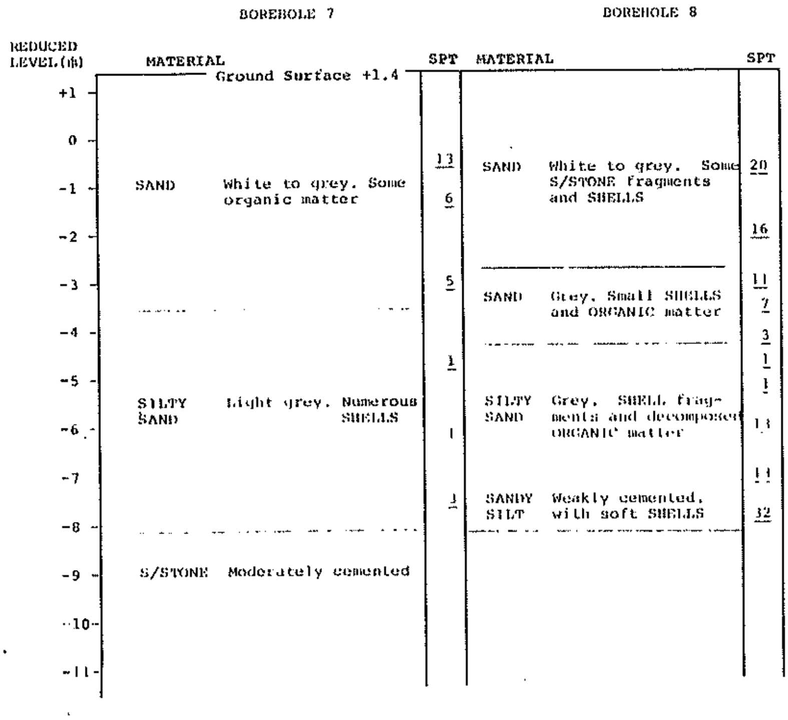

Figure 7 - Borehole Logs and SPT Results Following Vibroflotation in Test Using Sand and Slag Backfill (Refer Figure 6)

imately 3 to 4 dB (A) to a resultant level of 82 to 83 dB (A).

5. DISCUSSION AND EVALUATION OF RESULTS

Numerous accounts of successful vibroflotation projects, many at sites underlain by fine sands and silty sands, have been documented (e.g. References 5, 6 and 7) and it was generally assumed that vibroflotation would achieve compaction of the loose, saturated sands beneath this site.

Imported aggregate was used as fill material in preference to in situ sand because the latter was considered too fine to sink rapidly through the water surrounding the vibroflot. This was seen as an added advantage in the attempt to achieve good compaction.

Despite the apparent near ideal conditions for vibroflotation, evaluation of the results of testi. indicates only marginal improvement. The question remaining to be answered is why the apparent lack of success at this site? Two possible explanations for the poor response of the fine calcareous sands to compaction by vibroflotation are offered. One relates to the formation of a zone of liquefied soil around the vibroflot inhibiting transmission of energy waves, while the other suggests weak cementation between sand grains could be the reason for lack of compaction.

5.1 Liquefaction of Soil around Vibroflot

Geological events related to the formation of the calcareous sands may have created a situation in which the present effective vertical stress between discrete grains is uniformly lower than theoretical overburden pressure calcualtions would suggest. Circumstances which could have resulted in low vertical stress conditions include large scale arching between cemented zones, leaching of calcareous material leaving only a skeletal structure and water table movements.

It is possible that liquefaction (wherein pore pressures under cyclic shear stress application build up to exceed effective stresses) in the zones close to the vibroflot reduced the shear modulus of the sands to almost zero. The development of such a liquefied state would be assisted by the low effective vertical stress situation. Transmission of energy from the vibroflot into the surrounding sand could therefore be greatly impaired or even negated through loss of effective shear strength in the sand.

A parallel to this loss of shear strain transmission capability has been reported in recent work relating to seismic induced ground movements of sand overlying rock. The estimation of ground surface accelerations under seismic influences, in terms of effective stress concepts, has been developed by Finn et al. (Ref. 8). In their approach the development of a zone of liquefaction in sand above the bedrock is deemed responsible for a reduction of surface accelerations to approximately zero shortly (6 to 10 seconds) after the onset of seismic induced shear stresses from the bedrock. As is pointed out by Finn et al the case histories of seismic activities for saturated sands on bedrock indicate the actual surface accelerations fall to relatively low values a certain time after seismic influences initiate ground movements.

5.2 Weak Cementation of Soil

Sands in the coastal fringe area of Perth generally have a high proportion of calcium carbonate; previous work (Ref. 9) has indicated that these sands often have a lime content of between 60 and 80 percent. Solution and redeposition of the lime as calcium bicarbonate has caused various degrees of leaching and cementation of the sands with depth. Evidence of this cementation was observed during the site investigations when a range of material from weakly cemented sand that could easily be broken in the hand to strongly cemented calcarenite core that could only be broken by a sharp hammer blow were intersected.

With a low density and weak cementation of the particles it is very probable that in a saturated condition the soil structure would break down under mechancial pressure (SPT sampler, Dutch Cone probe or vibroflot). This hypothesis is supported by the low penetration values recorded during the investigation and by the large volume of backfill consumed during the vibroflot operations.

It is believed the rapid attenuation in energy with increasing distance from the vibroflot that was measured during the trial resulted in insufficient disturbance to break the weakly cemented bonds between soil particles. Consequently, in areas relatively close to stone columns the loose sand remained unaffected. This hypothesis has been supported by engineers with considerable experience of vibroflotation who suggest that in the ground conditions occurring at the Fremantle Hospital site, more time should have been spent in forming the stone columns. It is considered that this additional time during the compaction phase would have broken the relatively weak inter-particle bonds in the sand outside the immediate area of the vibroflot, thereby increasing the amount of compaction achieved by the operation.

6. CONCLUSIONS

The information presented indicates that caution should be applied to the use of vibroflotation for improving the density of loose, calcareous sands outside the immediate area of the vibroflot. The accepted overall improvement does not appear to be achieved in these sands despite the consumption of large volumes of backfill. The reason for the poor performance of vibroflotation may be attributable to the weakly cemented bonds between soil particles and/or the development of liquefied sand zones adjacent to the probe significantly reducing transmission of energy from the vibroflot to the surrounding ground.

It seems the apparent lack of compaction achieved in this trial may well indicate that potential liquefaction of the sands beneath the site would not occur under seismic induced accelerations, which would reasonably be assumed to be lower than those produced by the vibroflot. If this were the case then it is feasible that the use of stone columns installed by vibroflotation deriving their lateral support from the weakly cemented sands could provide a foundation system with adequate bearing capacity and acceptable reductions in potential settlement. However, evaluation of such a foundation support system could only be achieved through large scale load tests.

6. ACKNOWLEDGEMENTS

The authors wish to acknowledge the permission granted by the Principal Architect and Director of Engineering, Public Works Department of Western Australia, to present the above information.

7. REFERENCES

1. SOILMECH PTY LIMITED, (January 1972) Job No. NW3C and (February 1973) Job No. PW33. Reports on Site Investigations to Proposed Major Additions at Fremantle Hospital, Western Australia for Public Works Department.

2. DAMES & MOORE, (1976) Job No. 8076-010-71. Report Foundation Investigation - Proposed Major Additions Fremantle Hospital, Fremantle, Western Australia.

3. FRANKIPILE AUST PTY LIMITED. Diamond Drill Borehole Log Sheets - Fremantle Hospital.

4. GEOLOGICAL SURVEY OF WESTERN AUSTRALIA (1970) Perth and Environs Geological Map Sheet 3.

5. D'APPOLONIA, E., (1953) Loose Sands - Their Compaction by Vibroflotation, Symposium on Dynamic Testing of Soils Spec. Tech Publication No. 156, ASTM.

6. GRIMES A.S. & CANTLAY W.G. (1965). A Twenty-Storey Office Block in Nigeria Founded on Loose Sand. The Structural Engineer No. 2 Vol. 43 February, 1965.

7. FRANKIPILE AUSTRALIA PTY LIMITED (1974) C.B.H. Grain Terminal. Contracting & Construction Engineer, November 1974 pp 8-33

8. FINN, W.D.L., BYRNE, P.M. and MARTIN, G.R. (1976) Seismic Response and Liquefaction of Sands ASCE, Journal of the Geotech. Div. August 1976. p 841.

9. ANDREWS, D.C. (1970). Unpublished data on lime content of Perth sands.