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Experience in Ground Improvement by Dynamic Compaction and Preloading at Half Moon Bay – Saudi Arabia

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Experience in Ground Improvement by Dynamic Compaction and Preloading at Half Moon Bay—Saudi Arabia

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SYNOPSIS: Ground improvement techniques were used to meet the design requirements for a beach resort complex at Half Moon Bay on the Arabian Gulf. Extensive site exploration revealed upper layers consisting of loose to medium fine sand, having variable silt content. The original specification called for filling and ground improvement using Dynamic Compaction to achieve $100 kN/m^2$ surface bearing capacity and 50% relative density at a depth of 10m. Dynamic Compaction (DC) and subsequent field testing, using full scale loading tests as well as CPT and SPT tests, proved that it was possible to achieve these requirements over most areas using dynamic compaction, however it was not suitable in a few other areas where ground response to DC was poor. Those areas contained silty layers and showed signs of rapid pore pressure build up under dynamic compaction, while dissipation was slow and penetration resistance remained poor. Therefore, it was decided to use pre-loading in those areas. Further testing and instrumentation showed that pre-loading was achieving the required ground improvement.

INTRODUCTION

This paper describes the case history of ground improvement at a site in the middle of Half Moon Bay some 50 km south of Al Khobar on the Arabian Gulf (Fig 1). The site covers approximately 3.7 km^2 bounded to the East by Half Moon Bay and to the South by a smaller inner bay.
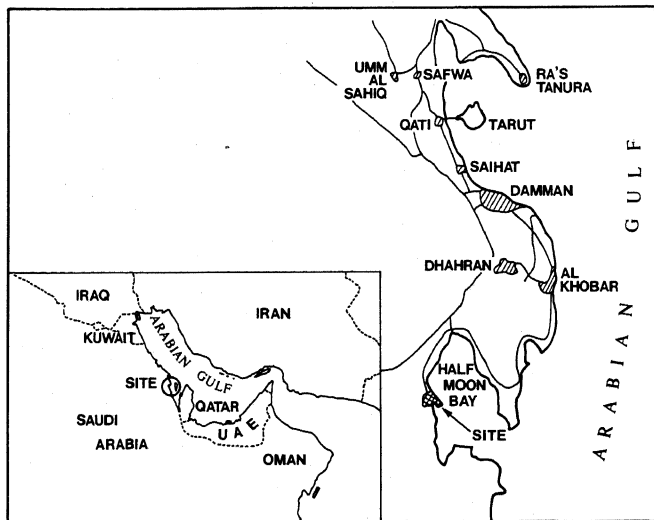

Fig 1   Site Location Map - Half Moon Bay.

The development scheme undertaken in Phase I of the Half Moon Bay Beach Resort Project is to consist of two hundred and fifty one chalets surrounding the inner bay together with infrastructure, services and a recreational area in the south eastern peninsula. Various areas of investigation referred to in the following text are shown on Fig 2.
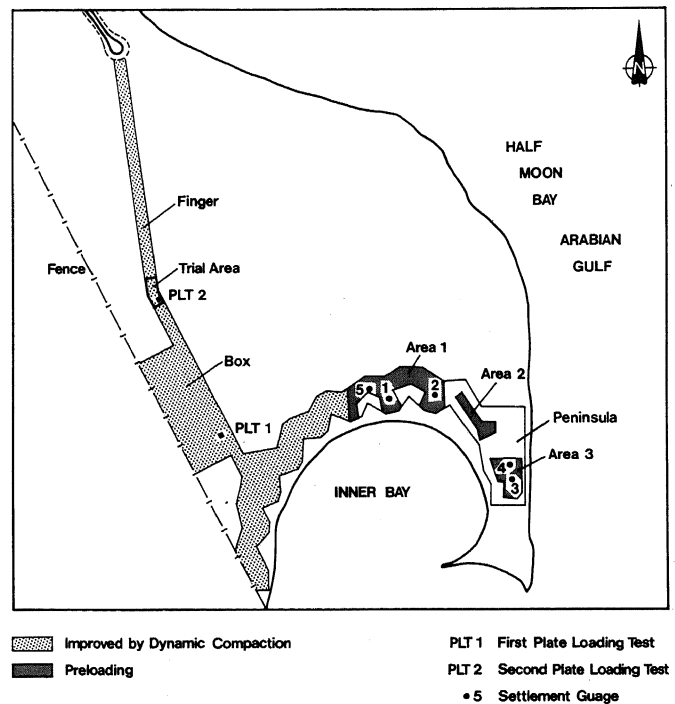

Improved by Dynamic Compaction
Preloading

PLT 1   First Plate Loading Test
PLT 2   Second Plate Loading Test
• 5   Settlement Guage

Fig 2   Layout of Ground Improvement Work.

Regionally, there are Sabkha deposits which cover most of the site, with intermittent caprock consisting of cemented shelly sandstone. These deposits are underlain by marine deposits of Quaternary Age of variable thickness containing non indurated but sometime cemented deposits of silty sand with silty clay bands. The sedimentary bedrock of the Dammam formation of Eocene Age lies at depths greater than 20m. This bedrock typically consists of

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dolomite and limestone interbedded with marl (calcisiltite) and shale (calcilutite) (Kent, 1976).

A preliminary site exploration was carried out using dynamic probing followed by a full scale investigation which included boreholes, trial pits, standard penetration tests (SPT) and static cone penetration tests (CPT). The in-situ soil conditions confirmed that buildings proposed in the main development area of the site would experience excessive settlement without treatment of the underlying, highly compressible soils. Following this, a ground improvement programme was undertaken using dynamic compaction on a surface reclaimed with dune sand up to 2m thick.

A part of the site, where the in-situ conditions prevented any improvement by dynamic compaction, was preloaded.

SITE CONDITIONS PRIOR TO GROUND TREATMENT

The ground surface originally consisted of Sabkha except in the central part where inter-mittent caprock was encountered. Underlying the Sabkha, fine to medium sand with traces of silt and a varying amount of shell fragments generally covers the site to a depth of 4m (Fig 3) in the peninsula and this increases to some 9m depth in the western part and in the box (Fig 2) where only traces of shell fragments were found (Fig 4). This stratum is referred to as the upper sand. Below this, extensive deposits of silt and clayey sandy silt some 6 to 7m thick are present in the peninsula (Fig 3) and some 5m were found in the western part (Fig 4). This is referred to as the intermediate silt layer. The SPT and the CPT results indicated that the upper
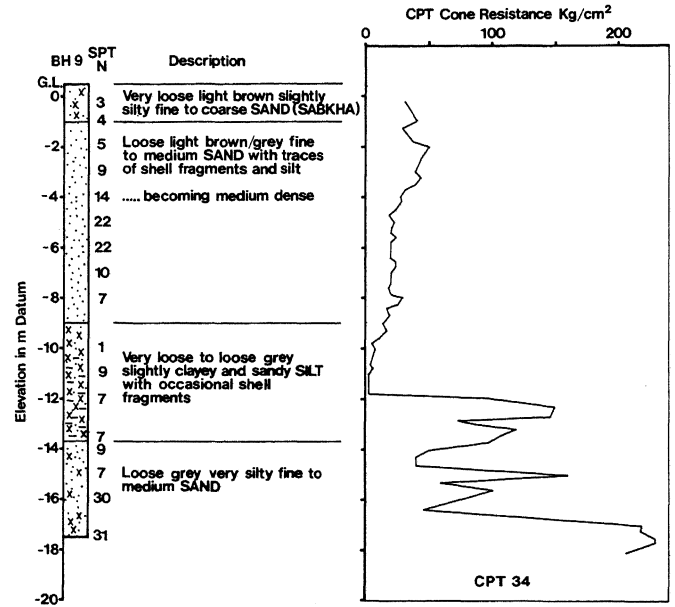

Fig 4   Typical borehole and CPT profile in the Western parts of the site.

sand and the intermediate silt strata were in a very loose to loose state except in the northern part of the finger and in some locations within the box area (Fig 2) where the upper sand layer was found to be medium dense with increasing depth. Underlying the silt stratum, sand deposits prevail across the site with a tran-sition zone of silty sand generally some 2m thick, and this stratum was normally found to be loose to very dense. This is referred to as the lower sand layer which overlies a hard silty clay generally encountered at 19 to 20m depth.

Groundwater was found to be within a metre of the ground surface at the northernmost part of the site, but it was within 250mm in the eastern and western parts of the site.

GROUND IMPROVEMENT

The proposed construction site over 44 ha, was initially cleared by removing the Sabkha and reclaimed with a maximum of 2m of dune sand, compacted in layers by vibro compactors, to provide a surface 2m above Datum. The dynamic compaction was applied on this surface with the aim of achieving a safe bearing capacity of 100 kN/m^2 at the surface and densification of the subsoil from the surface downward at least to 10m where the relative density of the soil should not be less than 50 per cent. The CPTs were used to verify the improvement and also to estimate the relative density at 10m depth using Schmertmann (1978), correlation with the cone resistance (q_c) and the effective over-burden pressure (p_o).

Dynamic Compaction Trial

An area of the site having relatively poor soil conditions was selected for the dynamic com-paction trials (Fig 2). The initial four (Trials 1 to 4) of six trials performed were
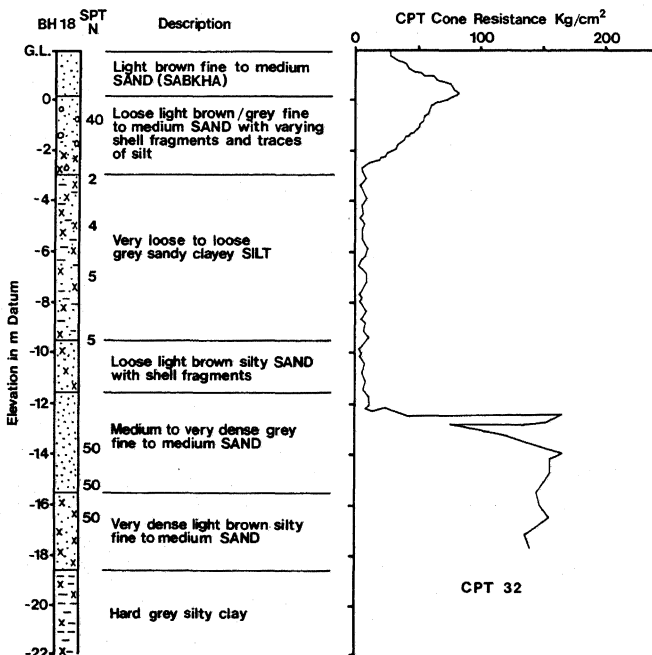

Fig 3   Typical borehole and CPT profile in the Eastern peninsula.

carried out using a 2 x 2m, 16 tonne pounder dropping freely from a height of 20m using a 120 tonne Manitowac-crawler mounted crane; the last two trials (Trials 5 and 6) were performed with a 25m drop. Normally, 10 blows were applied to each print, except in Trial 5 where 20 blows were applied. In each trial, quality control was performed using piezometers and CPTs. Trial 1 to 4 inclusive were carried out covering grid systems from 5 to 12m either by a single pass or up to 3 passes, whereas in trials 5 and 6 only a 10m grid was used with 2 passes. However, the timing for the passes was not always controlled by the dissipation of pore pressure generated. Three hollow stem auger holes were sunk in the trial area, prior to dynamic compaction. The intermediate silt layer in a very loose state was found at 3m below Datum. The results of the trials confirmed that only the initial 5m, including the compacted fill, improved satisfactorily while responses to dynamic compaction below this level were insignificant.

During the performance of some trials (2,4,5 and 6) it was noticed that the ground water (or in some cases a dark dense odoriferous fluid) overflowed from the standpipes (some 1.5m above the surface), but on some occasions immediate falls of piezometric heads were noticed during the course of tamping. This phenomenon might have been due to local liquefaction of the soil in the intermediate silt stratum. Subsequent monitoring showed that there was little change of these modified piezometric heads in the next six months.
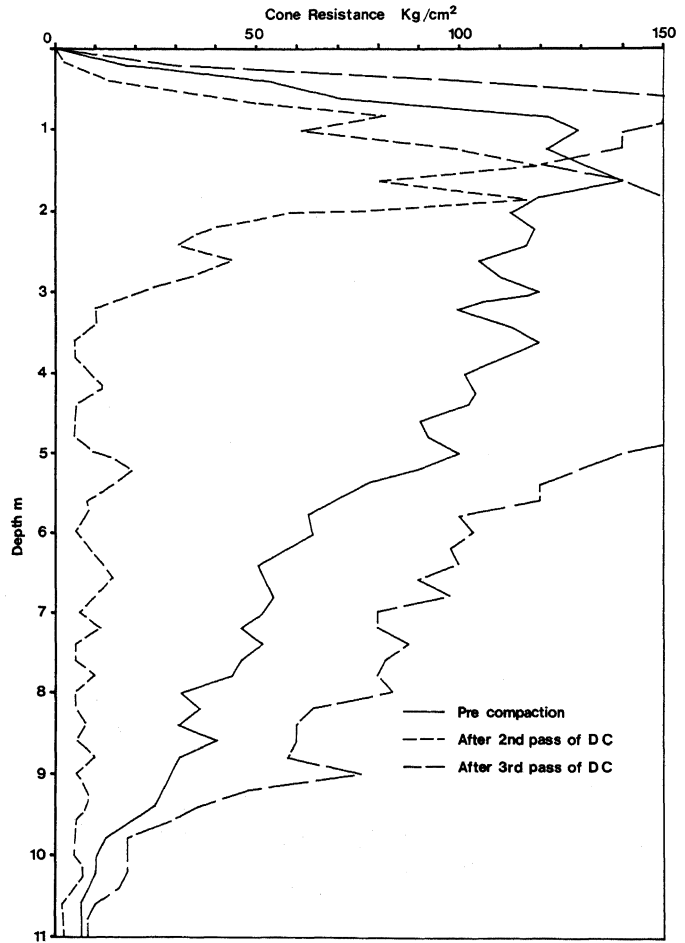
Dynamic Compaction

Following the disappointing results from the trials, the Contractor embarked on the full scale dynamic compaction programme using the same weight, 10 blows per print, 25m drop and 10m centres starting from the north of the finger. Piezometers were installed at 9m below Datum on a 50m grid.

Considerable ground improvement was noticed in many parts of the finger and the box areas, even after the first (north of the trial area), or second passes. However, such improvement in many areas was still insufficient to meet the specified relative density of 50 per cent, to be achieved at 8m below Datum in accordance with the Schmertmann's correlation. A full scale load test was carried out to overcome this situation, and this is described in the following section. The average CPT profile obtained from this test location (Fig 5) was finally used to
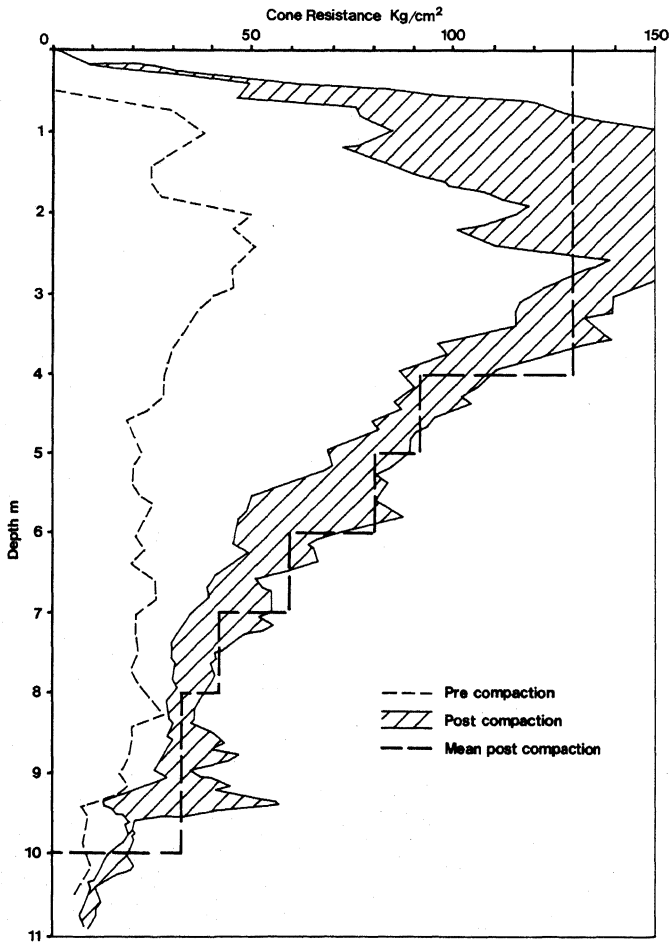
Fig 5 Pre-and-post dynamic compaction CPT results at Load Test 1 location.

Fig 6 Typical pre-and-post dynamic compaction CPT results from Western parts of the site.

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approve the dynamic compaction work at the site.

In areas satisfactorily compacted using two passes of dynamic compaction, pore pressures dissipated within one month, but in those parts treated with upto four passes, where pore pressure dissipated much more slowly. Fig 6 displays typical profiles of both the pre-and post-compaction CPT results. The settlements of the ground after each pass was in the order of 200mm. After the final pass, an "ironing pass" was performed to compact the near surface soils, especially around the craters.

Full scale plate load tests and settlement analysis

The full scale load test referred to above was carried out using a large concrete plate (4 x 4m x 0.4m thick) loaded to 150 kN/m² pressure. The location of the first such test (Load Test 1) was chosen in the box area (Fig 2), where two passes of dynamic compaction were given. The load test was carried out at 1.25m above Datum using concrete blocks as Kentledge. Fig 5 displays the results of a few post-compaction CPTs carried out prior to the load test, and a CPT profile from the original site investigation is also included. The load test commenced by placing the full load within the first 24 hour; the total settlement of 5mm took place within 48 hour. The load was maintained for 19 days when no significant movement was recorded.

The plate used acted as a 4 x 4m square footing (B = 4m) and the applied pressure must have influenced the ground at least to a depth of 2B (8m) which was some 7m below Datum.

By comparing the measured settlement with that calculated as suggested in Schmertmann (1978) a value of X = 7 was evaluated (E = Xq_c, where "E" is the soil deformation modulus). Schmertmann proposed that the values of X could be taken as 2.5 and 3.5 for square and long footings only with first loading case respectively. But, he quoted that if the sand has been strained by preloading or any other methods, the real settlement will probably be significantly less than predicted using values of X quoted above. Schmertmann, therefore, suggested that in such cases the values of X can be increased by a factor equal to or greater than that would indicate by the resulting increase in q_c. This apparently agrees with the results shown in Fig 5.

A second load test was performed at a location in the trial area (Fig 2) using identical arrangements except that the plate was placed at one metre above Datum. The results of this test was satisfactory, and was used to approve the work which did not meet the criteria of Load Test 1.

Relative Density

Following the encouraging result of Load Test 1, an investigation was made using SPT to assess relative density in areas where dynamic compaction work was approved on the basis of Load Test 1. Three boreholes were drilled using a hollow stem auger and SPTs were carried out at 1.5m intervals using a split spoon sampler. Relative

density for a corresponding blow count (N) was evaluated using the analytical expression on given in Giulani & Giulani Nicoll (1982) which incorporates effective overburden pres- sure, and these results are shown on Fig 7. Fig 7 shows that after dynamic compaction, 50 per cent relative density was generally achieved at a depth of 10m. It is interesting to note that some low N values under small overburden pressure are within the medium dense range in the relative density scale, even though this can be classified as loose when the general descriptive terms were used.
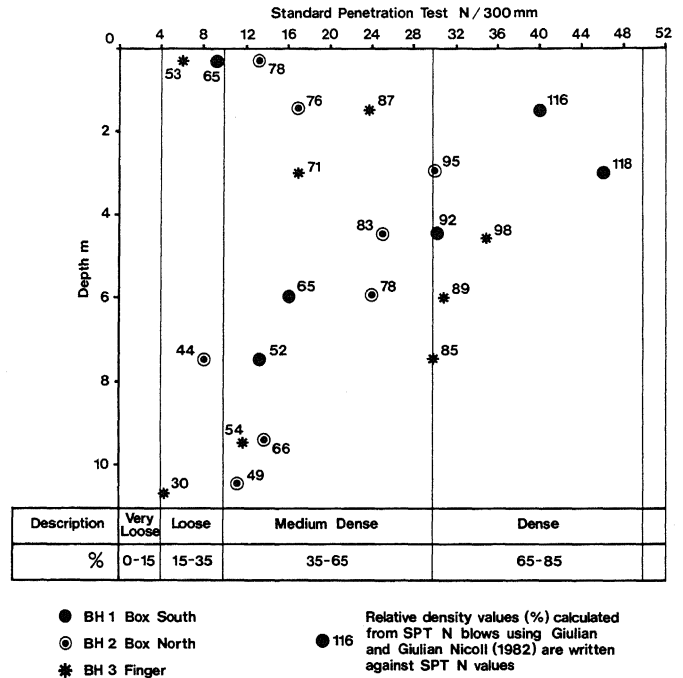
Fig 7   SPT (N) blows profiles with calculated relative density values.

Preloading

Despite the fact that dynamic compaction was successful over most of the site, quite a large part (122 ha) did not respond (Fig 2). The initial site investigation indicated that most of this area contains layer(s) 6 to 7m thick of loose to very loose silt and the groundwater, which is influenced by the tide, was found close to the original ground surface. An additional site investigation consisting of three 25m deep boreholes was carried out, with a view to predicting the time settlement under pre-loading. It was confirmed that no improvement of the intermediate silt layer was apparent after an initial dynamic compaction operation. Fig 8 displays an envelope of particle size distribution of this intermediate silt layer, which shows that the silt content varies between 40 and 80 per cent and the clay content is less than 10 per cent. Fig 8 also includes the grading envelopes of the upper and lower sand strata. The samples from the intermediate silt layer were mostly found to be of low to inter- mediate plasticity, and some were non-plastic.

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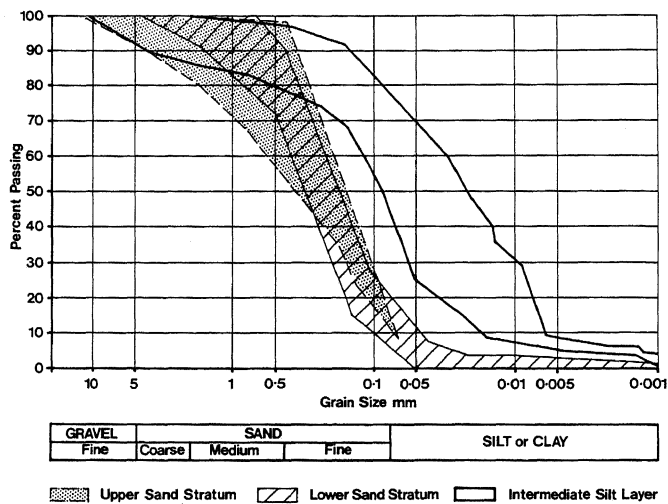
Fig 8   Particle size distribution of upper
         sand, intermediate silt and lower sand.

Thirteen one-dimensional consolidation tests
were performed on undisturbed samples taken from
the three boreholes. An attempt was made to
assess the pre-consolidation pressure (p_c')
and it was found to vary between 140 and 400
kN/m^2. However, the ratios p_c'/p_o' and
individual e-log p_o', plots (Halcrow, 1986b)
indicated that considerable sampling disturbance
may have taken place. The coefficient of
consolidation (C_v) values evaluated ranged
from 0.7 to $132m^2$/year for the corresponding
effective pressure range 100 to 200 kN/m^2.

Prior to placing the sand fill for preloading,
five settlement gauges were installed at 1.5m
above Datum (Fig 2); several piezometers were
installed at various depths in the upper three
strata. Fig 2 shows the part of the site
preloaded with 3.5m of dune sand fill which was
end tipped, dozed and compacted using construc-
tion traffic.

Settlements under preloading

The predicted settlements at five locations,
where the settlement gauges were installed, were
calculated using the soil profiles revealed by
the nearest boreholes. The settlements of the
sand strata (upper and lower sands) were
calulated using mean q_c values as appropriate
and the method described by Ghosh (1982). This
was because the applied load from the preloading
was equivalent to an elastic half space, and the
method suggested in Schmertmann tends to under-
estimate the deformation of the sand strata
under such load conditions. The settlements of
the intermediate silt layer were estimated using
m_v values corresponding to appropriate
effective pressures, evaluated from the
oedometer test results. The total settlement
assessed was of the order of 400mm.

The predicted periods for 90 per cent consoli-
dation of the intermediate silt layer were
estimated using the coefficient of consoli-
dation (C_v) value of 5, 15 and $55m^2$/year
were typically 1550, 515 and 140 days respec-
tively. The C_v values of non-plastic and
low-plastic silt deposits are of the order of
$100m^2$/year (Lamb and Whitman 1965), therefore,

140 days for 90 per cent consolidation of the
intermediate silt was predicted using a C_v
value $55m^2$/year and thought to be appro-
priate. Pore pressures built up in the upper
sand during the preloading operation, dissipated
within 40 days, and a similar period for the
dissipation of excess pressure in the lower sand
strata was assumed. Therefore it was considered
that the total settlement would be completed by
220 days. Fig 9 shows the total settlement
typically for Areas 1 and 3, which were
monitored till 25 September 1987.
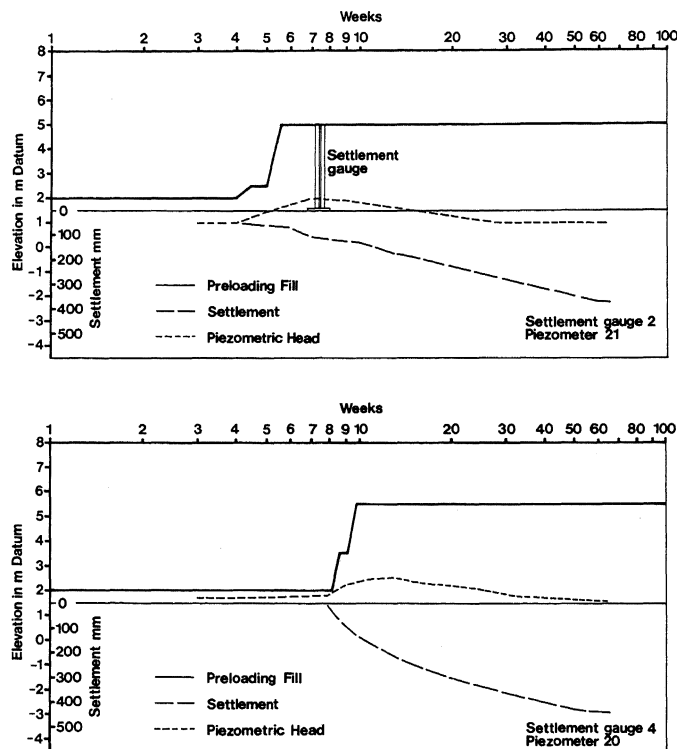
Fig 9   Two typical time settlement curves with
         piezometric heads under pre-loading.

DISCUSSION

The plots in Fig 9 would seem to indicate that
settlements are on the verge of completion,
although the excess pore pressure in the lower
sand stratum remains high. However, in the
intermediate silt stratum the excess pore
pressure is only marginally high compared with
the ground water level in this area. Table A
summarises piezometric heads monitored before,
immediately after and a year after the pre-
loading was placed, and the variations of the
pressure head within the three strata investi-
gated are noticeable. The excess pore water
pressure in the lower sand stratum, as mentioned
earlier, apparently remains unchanged in spite
of a moderate rise in pressure due to the
preload, which suggests, that this layer,
overlying the hard silty clay, probably does not
drain effectively in a horizontal direction.
Piezometers 19 and 20, installed in the upper
part of the intermediate silt layer in Area 3,
read 1.6m above Datum, whereas, the corre-
sponding pressure head at the centre of the same

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Stratum Description	Piezo-meters No.	Level of Installation (m Elv.)	Piezometric Head (M Elv.)		
			After Installation	Maximum After Preload	Recorded on 25/9/87
Upper Sand	22	- 3.0	+0.45	+1.30	+0.65
Upper Sand	*23	- 3.0	+0.65*	+0.95	+0.5
Intermediate Silt	6(GKN)	- 9.0	-	-	+0.76
" "	7(GKN)	- 9.0	+1.0	+1.5	+1.08
" "	18	- 7.5	+0.65	+1.4	+0.95
" "	19	- 5.0	+1.50	+2.0	+1.38
" "	20	- 5.0	+1.70	+2.5	+1.55
" "	21	- 4.0	+0.80	+2.0	+1.05
Lower Sand	16	- 18.0	+0.80	+1.40	+1.66
" "	17	- 14.0	+0.80	+1.60	+1.55

(a) *Fill material was placed near to piezometer 23 before the initial readings were taken.

(b) General groundwater level + 0.45m

TABLE A   Piezometer Monitoring Records

layer is about one metre above Datum. Reasons for this variation were not apparent from the site investigations.

The above evidence seems to indicate that the main factors causing failure of dynamic compaction in the eastern part of the site (Fig 4) were the shallow depth of the upper sand above a very loose, waterbound silt stratum together with the complex ground water regime.

CONCLUSIONS

1. It is clearly of prime importance to have a detailed study of the ground water regime before embarking on any ground improvement programme, and especially, when a site tains a thick layer of loose silt.

2. Dynamic compaction is a repid and economic method of ground improvement when it is applied to suitable ground conditions. Results are enhanced considerably by subsequent passes, provided that the excess pore water pressure generated due to the applied energy within the loose subsoil is allowed to dissipate. Ground conditions similar to those encountered in Area 3 are considered unsuitable for dynamic compaction.

3. Full scale load tests designed to meet foundation conditions are considered to be the most appropriate method of verifying ground improvement.

4. Relative density of in-situ silty sand may be assessed effectively using published analytical correlations established with SPT results.

5. The settlement of footings on sand may be assessed reliably using the method due to Schmertmann, but such methods tend to underestimate the deformation of the sand under extended fill loads.

ACKNOWLEDGEMENT

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