International Conference on Case Histories in Geotechnical Engineering

(1984) - First International Conference on Case Histories in Geotechnical Engineering

11 May 1984, 8:00 am - 10:30 am

Pretreatment of a Soft Soil by Surcharging - A Case History

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SYNOPSIS A normally consolidated silty clay deposit, varying in depth from 0 to 15 m, was pretreated by surcharging combined with P.V.C. vertical drains.

Representative field values of the vertical coefficients of primary (0.0015 m^2/kN) and secondary consolidation (0.01) were calculated from the settlements caused by a preliminary fill placed some six years prior to the detailed geotechnical studies. A conservative representative field value of the horizontal coefficient of primary consolidation was established to be 8 m^2/year. Asaoka's method was used to predict future settlements due to both primary and secondary consolidation.

The vertical drains were installed at a spacing of 1.8 m using a modified crawler crane. The average daily output was 158 drains over 128 working days with an average cycle time per pair of drains of eight minutes. The total cost was \$(A) 0.40 per cubic metre, or \$(A) 3.23 per square metre of the area treated.

Surcharging decreased the settlements due to primary and secondary consolidation. Calculated values of settlement over a 10 year period were 80 mm for a 15 m deposit with 50 mm being contributed by secondary consolidation.

INTRODUCTION

Sydney is located on Australia's eastern coast on shales and sandstones of Triassic age. Uplift in the early Tertiary period resulted in rapid erosion to produce the steep sided river valleys typical of the region. Housing development within a section of the valley of the Woronora river some 15 km south of Sydney was hindered by the potential settlement of a thick fluvial deposit of silty clays. The area consisted of two sections of tidal flats divided by a sandstone spur and situated between the river and an escarpment of Hawkesbury sandstone. Reclamation had been effected by placement of fill consisting of quartz sand with shells and shell fragments pumped from the river in 1969. When levelled the fill was up to 2.5 m in thickness, and the surface was about 1.5 m above high water level.

Several possible pretreatment techniques were considered by the land development manager and the geotechnical consultants including surcharging, lime piles, and electro-osmosis. Feasibility studies clearly established that surcharging combined with some type of vertical drain was the most cost effective and expedient solution. Preliminary compressibility data for the silty clay in the normally and over-consolidated states suggested that surcharging would reduce the total final settlements by a factor of at least five, thus restricting the settlement to less than 100 mm for the design pressure of 20 kPa.

Experiences with silty-clay soils of similar strength and compressibility characteristics (Lee et al, 1970) suggested that the vertical permeability was too low to rely on vertical drainage without some method of increasing pore pressure dissipation when the layer thickness exceeded a few metres. Thus the need for vertical drains became evident. At the time of the geotechnical studies it was common practice to use sand drains or sand wicks but in view of the successful experiences with paper and P.V.C. drains in Europe and Japan a detailed evaluation of P.V.C. drain technology was completed. The decision was made to use P.V.C. drains and an installation rig was constructed to place a pair of drains simultaneously to a maximum depth of 15 metres.

Some limited settlement data was available for the period 1969 - 1976. This data was found to be sufficient to establish representative values of the coefficient of volume decrease (0.0015 m^2/kN) and the vertical coefficients of primary consolidation (5 m^2/year) and secondary consolidation (0.01). A trial surcharge was subsequently completed which provided a conservative representative value of the horizontal coefficient of primary consolidation (8 m^2/year) based on Asaoka's (1978) observational method involving settlement measurements at regular time intervals.

Prior to the installation of the P.V.C. drains, piezometers were installed. The primary objectives of such instrumentation was to provide pore pressure data over the surcharge period and thus give the opportunity to check the predicted degree of consolidation. However, experience showed that large pore pressures were generated in the immediate vicinity of the drain installation. This fact was not initially appreciated and there were difficulties in differentiating between the pore pressures induced by installation and by the surcharge unless surcharge placement was delayed by 4 - 6 weeks after drain installation. For practical reasons this delay was rare and, consequently, the settlement measurements* were considered to be a more reliable performance measurement.

Details of the methods adopted to establish representative properties of the soil deposit are included in the present paper together with the techniques, costs, problems, and performance of the surcharge-vertical drain system.

SITE CONDITIONS

Soil Profile

During 1969 quartz sand had been pumped from the Woronora River forming a fill to a depth of 1.5 to 2.5 m. There had been a heavy growth of mangroves within the in-situ surficial layer of sand so that there were roots and other detritus remaining in the top few metres of the natural deposit. The clay and silt content progressively increased with depth and at a typical depth of 4 to 6 m there was a clearly defined deposit of soft silty clay varying in thickness from 13.5 m at the river frontage to zero over a distance of about 100 m. There was a further underlying layer of a stiff sandy clay above the bedrock, varying from 0 to 4 m over the site. Ground water was controlled by the river level at a depth of about 2 m below fill surface.

Preliminary geotechnical studies were completed in 1975 which established the potential settlement problem. Contours of the underlying sandstone surface were established expeditiously by a flight auger. Undisturbed samples were obtained in 80 mm diameter thin walled sample using a Mobile B80 rig with a 200 mm diameter auger. Piezometers were installed at selected sites together with surface and sub-surface settlement points. Water levels in the piezometers were recorded to within an accuracy of 3mm.

Typical CPT values within the top 4 m of the natural deposit varied from 1000 to 1500 kPa but there were local values of 3000 to 4000 kPa associated with sand lenses. CPT values of the soft silty clay deposit varied from 500 kPa at a typical depth of 6 m increasing to 1000 kPa at 12 m (Fig. 1). Adopting a factor for the cone of 0.06, based on some calibration data for the site, the corresponding undrained shear strength values were 30 and 60 kPa, respectively. Within the silty clay the friction ratio was consistently 3 - 4 per cent which would be anticipated on the basis of existing data for this type of soil.

In-situ void ratios of the silty clay were as high as 300% with water contents of the order of 100%.

*Non-linear effects need only be considered when using pore pressure data (Lee, 1983).

Typical properties of the sand fill, silty clay, and the underlying stiff clay layers are summarized in Table I. These values are based on information obtained from laboratory studies, the trial surcharging, and measurements over the surcharge period.

Values of the compressibility are high (m_v^2 = 0.0015 m²/kN) even for a normally consolidated deposit. It will be noted that the primary compressibility is decreased by a factor of 7.5 by overconsolidation. Properties marked by an asterisk in Table I were obtained by field and laboratory testing except the horizontal coefficient of consolidation which is based on field settlement measurements.

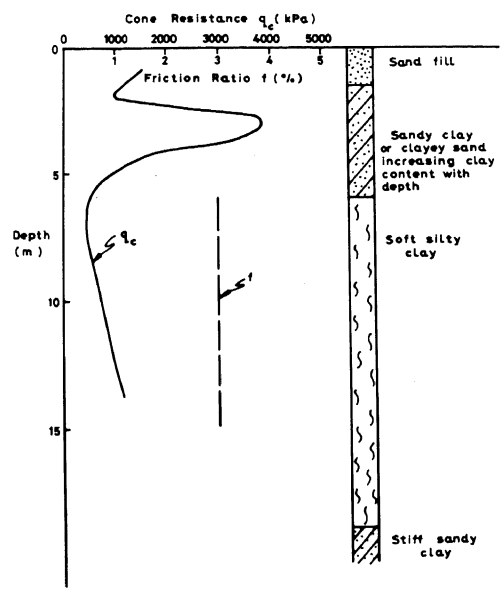

Fig. 1 Typical Profile and CPT Data

It is also noted that overconsolidation decreased the coefficient of secondary consolidation, so that the effect of surcharging is to greatly reduce the secondary settlements as well as diminishing the primary settlements.

Rate Parameters

Some limited settlement data was available for the period 1969 to 1978 (TableII) as settlement points had been established when the fill was placed in 1969. It was felt that the surface settlement readings of the three remaining settlement points were probably reliable and back-calculations were made to give values of the vertical primary and secondary rate parameters.

Table I

Summary of Typical Properties

Sand Fill	
γ	16.7 kN/m³
Silty-Clay	
w_{LL}	100%
e	290%
c_u (surface)	20-30 kPa
w	110%
γ	13 kN/m³
γ_d	6.2 kN/m³
m_v (NC)	0.0015m²/kN
m_v (OC)	0.0002m²/kN
C_c	1.0 to 1.3
c_v	5 m²/year
c_r	8 m²/year
c_α (NC); (OC)	0.01; 0.001-0.005
Stiff Clays	
w	20%
e	50%
γ	20 kN/m³
γ_d	17 kN/m³
m_v (NC)	0.0002 m²/kN
m_v (OC)	0.00005m²/kN
C_c	0.18
C_{cR}	0.015
c_v	10 m²/year

The total settlement in 1973, 1976 and 1978 can be expressed as, respectively

$$S_{73} = U_{73}\, S_{FP} + c_\alpha H \log \frac{t_{73}}{t_{69}} \qquad 1(a)$$

$$S_{76} = U_{76}\, S_{FP} + c_\alpha H \log \frac{t_{76}}{t_{69}} \qquad 1(b)$$

$$S_{78} = U_{78}\, S_{FP} + c_\alpha H \log \frac{t_{78}}{t_{69}} \qquad 1(c)$$

Absolute values of S_{73}, S_{76} and S_{78} are unknown since the levels were not referred to 1969 when the fill was placed. However, the increases in settlements from 1973 to 1976 are given in Table II,thus we base the analysis on these differences. (Lee et al, 1983). S_{FP} is the final primary settlement, t is time.

Table II

"Reliable" Field Settlement Data

Site	11/1973	1/11/1976	24/2/1978
D16	3.048	2.676	2.6545
D17	2.896	2.551	2.4774
D18	2.835	2.493	2.3995

The results of the analysis giving c_v and c_α are shown in Fig. 2 for a range of layer thicknesses and total final primary settlement values. Settlement data from sites D17 and D18 were substituted into equation (3) and found to give solutions to the equation. However, the data from D16 did not lead to a convergent solution.

There was some lack of precision concerning the depth of sand fill at the measuring sites. A sensitivity analysis showed the calculated values of the coefficients were not significantly affected as reflected in Fig. 2 where values for fill depths of 1.3 m and 1.5 m are plotted.

Rather surprisingly these values compared extremely well with laboratory recorded values of 6.0 to 8 m²/year for an effective stress range of 47 to 715 kPa. The coefficient of secondary consolidation in long term laboratory measurements under constant temperature conditions was close to 0.01, again agreeing with the field value.

It was not anticipated that the values calculated from the (fortuitous) field data and laboratory data would agree because past experience suggests that such a deterministic approach is likely to be subject to data errors. A preferred approach is to obtain a series of settlement measurements at a specified time interval, δt, and this procedure was adopted for the period after placement of 1.5 m of test fill. Asaoka (1978) showed that the primary settlement, S_{ci}, at time i δt can be closely approximated by the expression

$$S_{c(i)} = \beta_0 + \beta_1 S_{c(i-1)} \qquad (2)$$

where β_0 and β_1 are constants and $S_{c(i-1)}$ is the primary consolidation settlement at time $t = (i - 1)\,\delta t$. If $S_{c(i)}$ is plotted against $S_{c(i-1)}$ the initial ordinate is β_0 and the slope of the line is β_1. This line is extrapolated to a 45 degree line passing through the origin to give the final consolidation settlement.

From settlement data following surcharging the Asaoka type analysis was completed for radial primary consolidation to the vertical drains. Table III quotes the calculated values of the representative radial coefficient of consolidation, c_r, and the predicted total final primary settlement.

Table III

Settlement Data Based on Primary Settlement Only. Constant Surcharge

Site	Coefficients β_0	β_1	Coefficient Consol. (c_r)	Final Consolidation Settlement (mm)
P102	.352	.869	8.9	250
P104	.306	.903	6.5	110
P108	.348	.896	7.0	115
D17	.461	.806	13.8	35
D18	.194	.905	6.4	140
D21	.254	.903	6.5	120

The calculated values are seen to be in excess of the conservative 'design' value of c_r of 5 m^2/year, that is, calculations based on the design value predicted that settlements occurred more slowly than in reality. The variability is consistent with expectations and known behaviour of such soils (Lumb, 1974).

Values of c_r cannot be directly obtained from the numerical analysis when the secondary consolidation contribution is taken into account. However, values of the coefficients are quoted in Table IV. The secondary component of settlement was considered by use of the settlement difference expression in the form

$$S_{c(i)} = \beta_0 + \beta_1 S_{c(i-1)} + \beta_2 S_{c(i-2)} \quad (3)$$

and the plot of settlement-time is included as Fig. 3. It is seen that the total settlement values recorded in the field exceeded the predicted values by up to 20 mm. This may be due to the limitations of the mathematical modelling and the small changes in surface level as the earth moving machines operated.

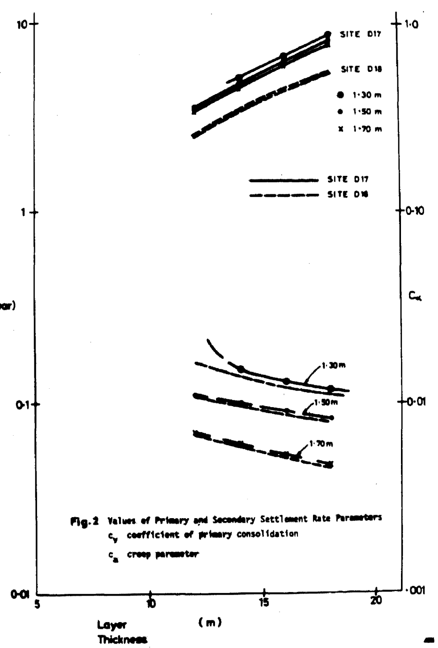

Fig. 2: Calculated Values of Vertical Settlement Parameters.

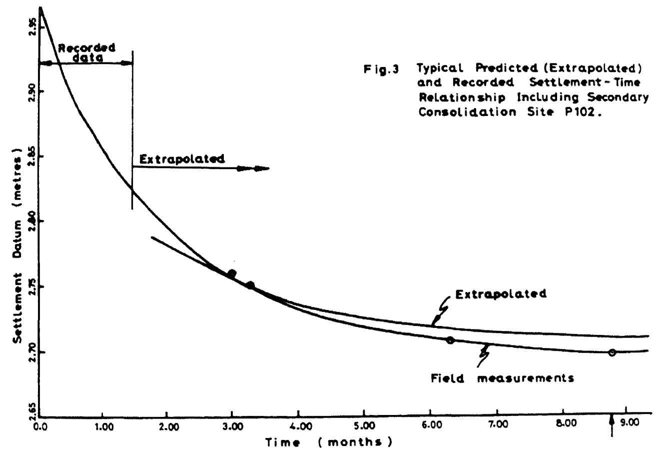

Fig. 3 Comparison of Predicted and Recorded Total (Primary plus Secondary) Settlements.

Table IV

Values of the Coefficients. Primary and Secondary Consolidation Settlements are Considered. Constant Surcharge.

Site	β_0	β_1	β_2
P102	0.305	1.008	+0.121
P104	0.226	0.928	-0.001
P108	0.378	0.558	-0.327
D17	0.369	0.858	-0.014
D18	0.041	1.456	-0.480
D21	0.230	0.875	+0.040

Referring to Table III it is seen that the representative radial coefficient of consolidation varied from 6.4 to 13.8 m^2/year with a mean of 8.2 m^2/year. This value can be compared with the representative vertical coefficient of consolidation of 5 m^2/years giving an anisotropic ratio of 1.6. This is lower than would have been anticipated from the observed laminated structure of the silty clay. It is partly due to the fact that there is a reduction in the effective horizontal permeability due to smear calculated to be of the order of 30% (Lee, 1983). Hence, it is concluded that the in-situ anisotropy, with respect to the rate of consolidation, was about 2, and in the design of the P.V.C. drains the effect of smear must be considered.

The coefficient of secondary consolidation is also reduced by surcharging. This fact was demonstrated by an analysis of the values of c_α for normally consolidated and overconsolidated samples loaded to simulate the surcharging history. This was a particularly important practical feature since the predominant component of settlement in the post-construction was due to creep.

Creep measurements were made on normally consolidated laboratory samples over a period of months. Samples were subjected to the field loading history. Although conclusions regarding the magnitude of the long term (years) prediction need to be carefully assessed due to the limitations of the data and the Buismann model, it was considered that such tests provided a valid basis for establishing the effect of over-consolidation. As anticipated (Lee, et al 1983) it was found that the value of c_α was considerably reduced. Recorded values varied between 10% and 50% of the soil in the normally consolidated state. Subsequent studies on a similar normally consolidated estuarine deposit showed the same characteristic. Furthermore, the laboratory data in both studies showed there was no statistically signifcant trend between c_α and the consolidation stress. This is a consequence of two compensating effects. An increase in effective stress increases secondary settlement, but this increase is partially compensated by the greater stiffness of the soil structure developed during primary consolidation. Fig. 4 shows some typical data from long term laboratory tests on undisturbed samples from a similar site (Lee,1983). This plot shows a relatively large variation in the data but the trend supports the view that the value of c_α is, for practical purposes, independent of the effective vertical stress.

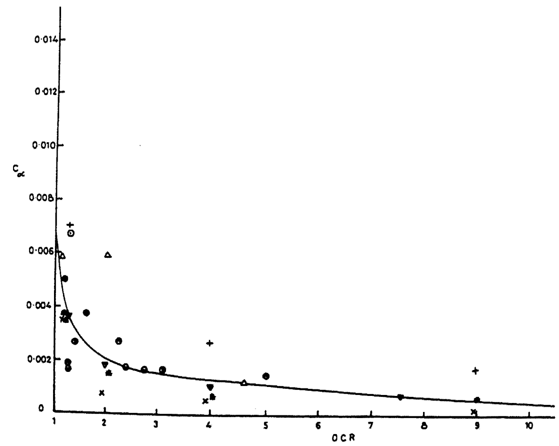

Fig. 4: Influence of Effective Vertical Stress Secondary Consolidation

SITE CONSTRUCTION

Vertical Drains

After consideration of the possible types of vertical drains it was decided to use P.V.C. drains, 100 m wide by 1.6 mm thick. Typical values of the longitudinal permeability and transverse permeability were 5 cm/sec and $1 - 2 \times 10^{-3}$ cm/sec. It is noted that the high permeability along the drain reduces the well resistance effect compared with conventional sand drains (Lee, 1982) but, as noted earlier, the effect of smear is likely to be significantly higher for P.V.C. drains. For predictive purposes the equivalent diameter of a 100 mm wide drain is taken as 50 mm.

Installation Equipment and Technique

Drains were installed in pairs using a modified Kockum Model KL250 crawler crane fitted with a conventional pile driving rig. The extra attachments which had to be fabricated consisted of

(a) two steel tubes 20 m long located at the drain spacing of 1.8 m

(b) a steel frame at the top of the tubes to locate the tubes and to provide housing for the Nippei Type 80, 165 kVA vibrator

(c) A Pethrow type TE152V 60T, diesel generator producing 415 volts at 1500 rpm

(d) a guide frame for locating the tubes at the base of the rig

(e) two reels of P.V.C. strips fitted at the base of the rig.

The P.V.C. drain strips were led to the top of the mast through lightweight protective tubes and passed down the vertical tubes. Plastic head cones were stapled to the strips with an air operated gun. Each cone was located by a mandrel located at the base of the corresponding steel tube. The combined effect of the vibrator and the mass of the tubes, frames, and vibrator led to rapid penetration of the pair of strips once the top layer of fill had been pierced. It was usually necessary for the operator to slow down the speed of vertical insertion by cutting out the vibrator and applying the braking system. Uplift of each plastic drain, during the period the steel tubes were being raised to the surface, was prevented by the anchorage developed at the cone head.

Japanese experience had established that the cycle time for drain installation was essentially independent of the depth of insertion of the drains. Acceptance of this fact by the contractor led to a very successful *incentive* type of contract in which a unit rate per drain was arranged and accepted by both parties. Payment to the contractor varied from $8 per drain for 0 to 130 drains per day progressively diminishing to $6.60 per drain for 200 drains per day.

It is of interest to note that pore pressures were generated by the vibrator. This was evident by the fact that water was discharged from drains during the installation of adjacent drains.

Field Performance

The team inserting the drains consisted of a crane operator plus two men as well as the engineer from the land development company responsible for the project. The average daily output was 158 drains over 128 working days, with a peak output over a 10 hour period of 300 drains, that is, the average cycle time for the insertion of a pair of drains was about 6 minutes, but, under favourable conditions this was reduced to 4 minutes. It is considered that a 5 minute cycle per drain pair is a reasonable basis for planning.

Total of 20178 drains were inserted over an area of 55340m². Taking an average thickness of 8 m the volume of soil treated was 442700 m³, 185330m of drain material was used costing \$(A)55,600. Altogether, 21000 hard cones were used totalling \$(A)3800. The total cost including labour was \$(A)179,000, that is, \$3.23 per m² of treated area, \$(A) 0.40 per m³ of treated soil, \$(A)8.87 per drain, \$(A)0.97 per m of drain (material \$(A)0.32 per m, labour \$(A) 0.65 per m). The latter figures can be compared with Japanese experience during 1977 - 1978 of \$(A) 0.86 per m of drain (material \$(A) 0.41 per m, labour \$(A) 0.45 per m).

The total cost of preconsolidation was \$(A) 515,000. This amount includes the P.V.C. drain operation, consulting fees, site investigations, material, labour and machine costs. It does not not include survey fees. The major difficulty leading to significant costs were the presence of the buried sandstone boulders. It was necessary to excavate such areas, remove the boulders, and replace the fill before drains were inserted.

CONCLUSIONS

Pretreatment of an extensive deposit of normally consolidated silty clays by surcharging reduced the post construction settlements by a factor of about seven. P.V.C. drains were installed over an area of 55,340 m² on a spacing of 1.8 m, and to a maximum depth of 13 m thus reducing the minimum surcharge period to 4 months. The average cost of treatment was \$(A) 0.40 per cubic metre and is similar to the cost of surcharging similar soft soils in Japan.

Consistent representative rate parameters for vertical and horizontal drainage were established from field data. It was clear that the surcharging greatly reduced both the primary settlement and the secondary settlement. In the post-construction period the major settlement will be due to secondary consolidation. Over a period of 10 years the total settlements were calculated to be 35, 55 and 80 mm for a single storey construction and layer thicknesses of 5, 10 and 15 m respectively. The contributions by secondary consolidation to these settlements were calculated to be 20, 30 and 50 mm respectively.

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