Cumbalum Instrumented Embankment – Ballina Bypass

M A Allman
BE, PhD
Principal, RCA Australia Pty Ltd, Australia

Summary: Data from an instrumented 4.5m high embankment constructed in 1998/1999 on the alignment of the proposed Pacific Highway Ballina Bypass is presented. Features of the embankment include a subsurface profile comprising in excess of 20m of soft and firm, normally consolidated clay and provision of vertical wick drains at 1.35m centres.

Results of monitoring indicate vertical settlements in excess of 2.5m with maximum horizontal movements of approximately 0.4m at 3m depth. Periodic measurement of soil strength by in situ vane during consolidation has allowed a comparison to be made between soil strength gain and effective stress.

Notable results include:

- The degree of consolidation after 4.5 years near the centre of the upper soft clay layer is approximately 75%.
- Settlement rate during year 2002 was approximately 1mm/day.
- The ratio c_u/σ_{vo}' is approximately 0.45-0.55 for the normally consolidated clay.
- The strength increase over time (ratio $\Delta c_u/\Delta\sigma_v'$) for the clay is approximately 0.21.

INTRODUCTION

Data from an instrumented 4.5m high embankment constructed in 1999 on the alignment of the proposed Pacific Highway Ballina Bypass is presented. Features of the embankment include a subsurface profile comprising in excess of 20m of soft and firm normally consolidated clay and provision of vertical wick drains at 1.35m centres.

BACKGROUND

Approximately 7km of the proposed route of the Ballina Bypass traverses the coastal floodplain of the Richmond River and Emigrant Creek. Surface elevation for most of this area is less than 1m AHD, drainage is tidal and groundwater is generally within 0.5 to 1m of the surface. During Quaternary and Recent periods fluctuating sea levels have resulted in the deposition of estuarine, deltaic and alluvial sediments along the coastal plain. The sediments predominantly comprise shelly clays and some sand interbeds. The clays are deep, near normally consolidated and soft to firm to depths up to 20m. These conditions are typical of other Australian eastern seaboard coastal river plains.

Concept design for the bypass requires embankments on the lowlands typically 1.5-3.0m high but up to 5-8m at the location of bridges and grade separated interchanges. The geotechnical conditions will result in significant constraints on construction through a combination of large and long-term primary and secondary settlements and the requirement for staged construction of higher embankments as a result of stability considerations.

Prior to the investigation for the Ballina Bypass project, settlement of embankments on the soft clay profile in the Ballina area had been previously noted to be large and long term. Two examples of available data are illustrated in Figure 1 and 2. Figure 1 shows settlement data from a monitored embankment on the Bruxner Highway near the intersection with the Pacific Highway near Emigrant Creek south of Ballina. The soil conditions at that site comprise approximately 20m of compressible soil. The embankment was 2.5m high and survey data for a number of locations was available over a 7.5 year period. Settlement of up to 1m was measured with no apparent development of a classical "S" shaped settlement curve. After approximately 8 years of monitoring the embankment was settling at an approximate rate of 0.2mm per day. This is characteristic of this type of soil profile and is considered to be the result of slow primary consolidation and a significant component of secondary consolidation.

Figure 2 shows an interpreted cross section through the existing embankment of the Pacific Highway between Emigrant and Duck Creeks to the north of Ballina in similar soil conditions to the monitored embankment on the Bruxner Highway. The highway works were apparently completed in the 1960's and the cross section indicates that the embankment is 4.5-5.0m above surrounding ground level while the depth of fill indicates that up to approximately 3m of settlement has taken place.

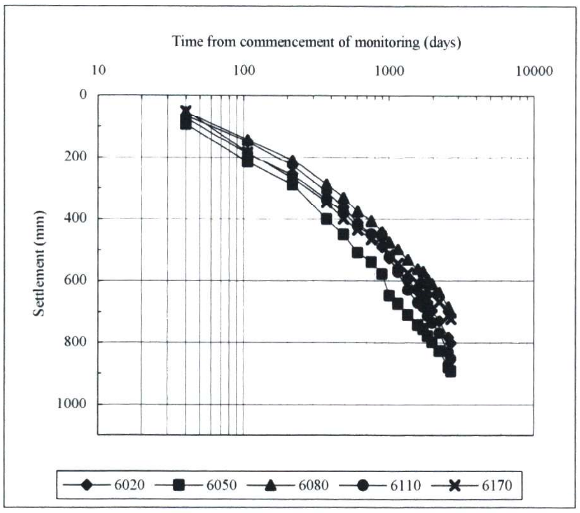

Figure 1. Settlement Data at Bruxner Highway

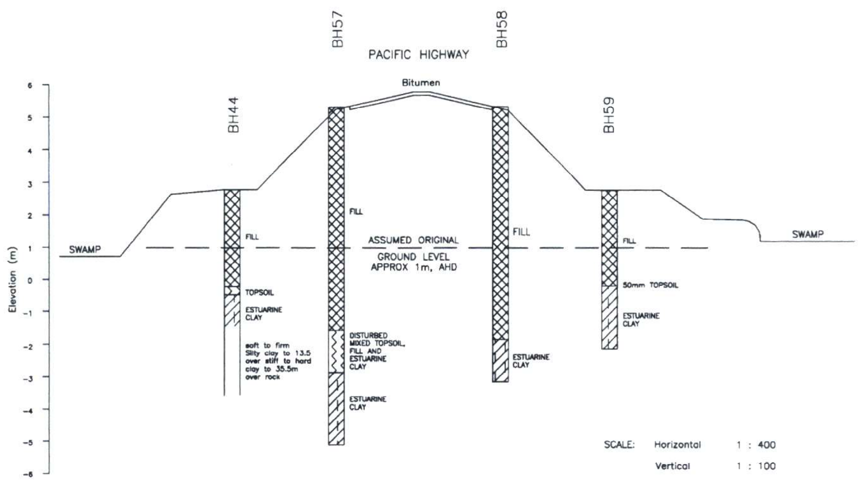

Figure 2. Inferred Cross Section Through Pacific Highway Embankment

The risks associated with design, construction and contractual arrangements on the Ballina Bypass, along with problems encountered under similar geotechnical conditions elsewhere, saw the NSW Roads and Traffic Authority construct two instrumented trial embankments on the lowlands of the proposed route during 1998/1999. The embankments were constructed at Teven Rd and Cumbalum. This paper presents details and results from the Cumbalum instrumented trial embankment.

SOIL PROFILE

The subsurface profile at the site of the Cumbalum trial embankment comprises approximately 20m of soft to firm, normally consolidated estuarine clay overlying stiff to hard clay. A discontinuous sand lens at a depth between approximately 10 and 15m is evident beneath some of the embankment area. The geotechnical profile of the soft clay found at Cumbalum is shown in Figure 3. Features to note are:

- Moisture content increasing from 40% near the crust to up to 130% at 7.5m depth then reducing with depth.
- Plasticity Index up to 110%
- Low bulk unit weights of between 13 and 17kN/m^3 as a result of high moisture content and voids ratio.
- Some over consolidation within the upper 2-3m and near normal consolidation beneath that.
- Field vane indicates undrained strength of approximately 25kPa at 1.5m depth reducing to as low as 12kPa at between 3 and 4m depth and then increasing with depth to near 50kPa at 20m depth.
- Vertical coefficient of consolidation (c_v) of approximately 2-4m^2/year.

Other features of note for the soils at Ballina include:

- A significant coefficient of secondary consolidation (c_α) of up to 0.06 (Ewers and Allman, 2000).
- A clay content between 32% and 62%.
- High acid sulphate potential.
- Organic content of 2.5%-5.5%.

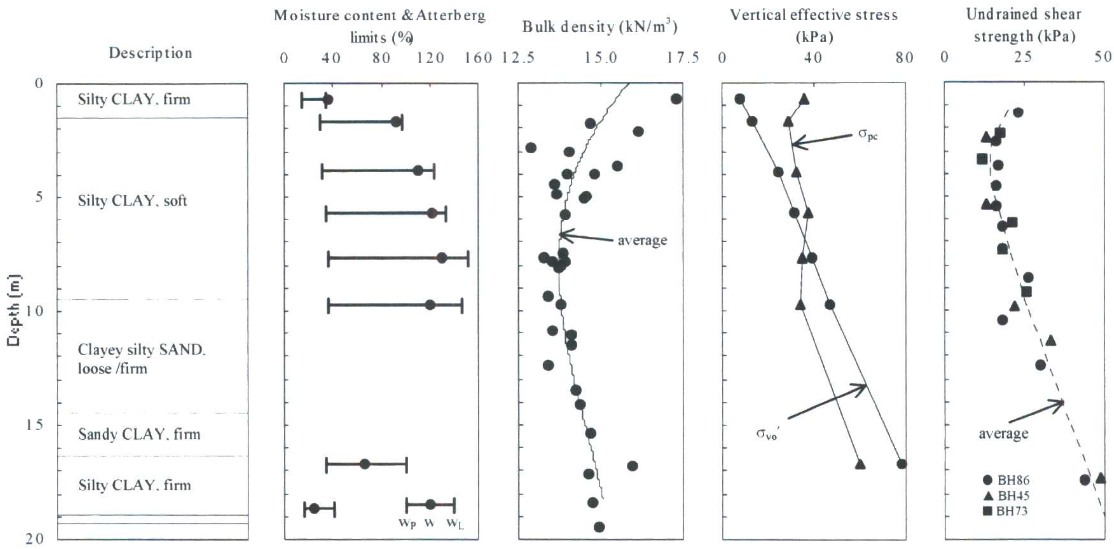

Figure 3. Geotechnical profile at Cumbalum

TRIAL EMBANKMENT

The Cumbalum trial embankment was constructed over an eleven month period. Construction involved clearing of vegetation including trees and grass, placement of a layer of geotextile, a 750mm layer of crusher dust to act as a bridging and drainage layer, and a 250mm layer of 7mm aggregate for drainage. Vertical wick drains were then installed at 1.35m triangular centres over the entire pad as indicated in Figure 4. A total of 2772 wicks with an average length of 22.4m were installed. It is noted that pre-boring through the drainage blanket was required as a result of resistance to penetration of the wick mandrel. A layer of class C1 geotextile was placed over the drainage layer and all instrumentation installed. The embankment was then constructed using a shale based quarry spoil.

Progress with embankment placement was severely hampered by rainfall and deviated significantly from what was planned but resulted in a 4.5m thick embankment being constructed over the period October 1998 to September 1999. The approximate sequence of construction is shown in Figure 5. The embankment was provided with 20m wide by 1.5m high stabilising berms on both sides as well as a drainage bund. A plan and cross section of the completed embankment is shown in Figure 4. Figure 4 shows instrumentation locations as well as bore and CPT locations.

RESULTS

Vertical Settlement

Settlement has been monitored by means of a survey of a series of settlement plates with all levels taken relative to a fixed datum. Figure 5 shows the variation of settlement for seven plates on Line 'C' across the embankment (SP6 to SP12). The depth of fill is also identified on Figure 5 for reference. The wick drains were installed at a fill height of 1.0m between days 25 and 110. It is noted that throughout the paper Day 1 has been considered to be the of 28th September 1998.

The following features are noted:

- Maximum vertical settlement at SP9 is close to 2.5m, and slightly in excess of 2.5m at SP4.
- The settlement rate at SP9 in April 2003 after 1600 days was 1.0 mm/day.

- Unlike the Bruxner Highway embankment settlement data presented in Figure 1, the settlement data for Cumbalum has developed a classical 'S' shaped profile. This is considered to be the result of accelerated primary consolidation through provision of closely spaced vertical wick drains.

Maximum lateral movement is approximately 400mm at a depth of 3m below the edge of the embankment. The ratio of maximum lateral to maximum vertical movement is approximately 0.16.

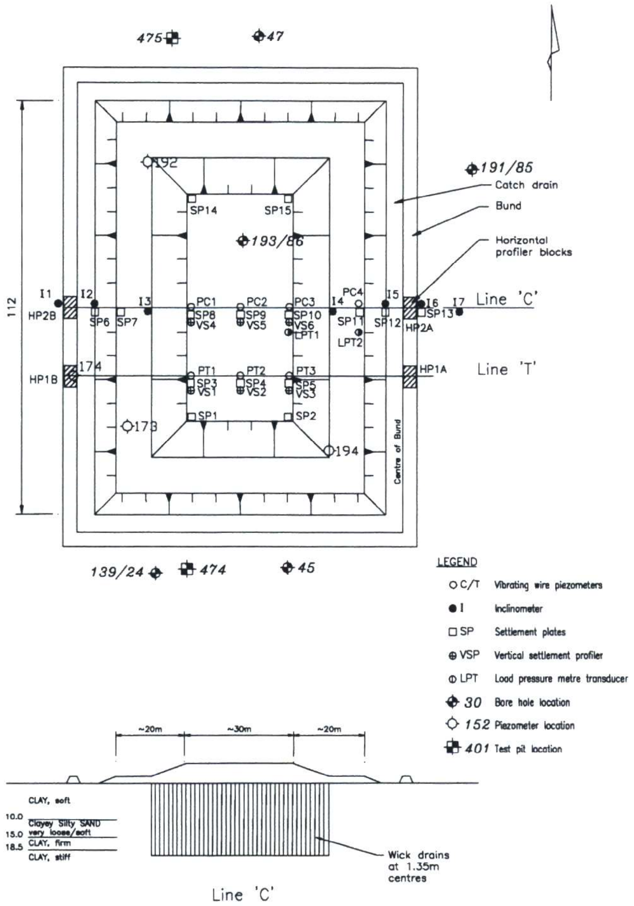

Figure 4. Cumbalum Trial Embankment Plan and Cross Section

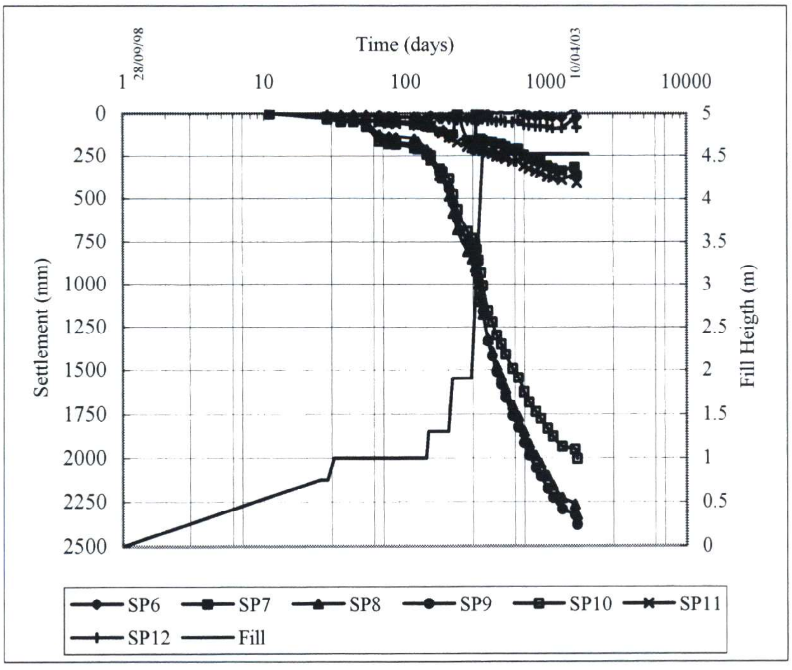

Figure 5. Settlement with time at various surface settlement plates

Pore Pressure

Pore pressures are being monitored at a variety of locations typically with 6 to 7 piezometers covering a range of depths up to 25m. Monitoring locations are identified on Figure 4. The results from PC2 at the centre of the embankment are summarised in Figure 6 through to the 11th April 2002. Expected hydrostatic pressures for each piezometer are plotted in the Figure as is depth of fill. It should be noted that the hydrostatic pressure for the shallower piezometers has been adjusted to allow for ongoing settlement relative to the stable groundwater level at the surface, which has been assumed to be 0.4m.
Features to note are:

- The depth of fill placed corresponds to an increase in total vertical stress of approximately 100kPa. The observed maximum excess pore water pressures at for each piezometer as a percentage ($\Delta u/\Delta\sigma_v$) were 50% for PC2/3, 77.5% for PC2/5.8, 66% for PC2/9, 10% for PC2/12, 61% for PC2/17 and 10% for PC2/20.
- Dissipation of excess pore pressure continues, particularly near the centre of the upper and lower clay layers (ie. PC2/5.8 and PC2/17).

Profiles of effective vertical effective stress at various times are shown plotted in Figure 7. It is noted that all results are corrected for the settlement of the piezometers relative to the groundwater level as discussed above. If this correction (up to -16kPa at PC2/3) is not made the degree of dissipation will be underestimated, particularly at shallow depths. Features to note are:

- The influence on pore pressures of the sand lens between 10m and 15m may be clearly seen.
- The inferred degree of pore pressure dissipation at April 2002 was approximately 77% at 5.8m depth and 84% at 17m depth.

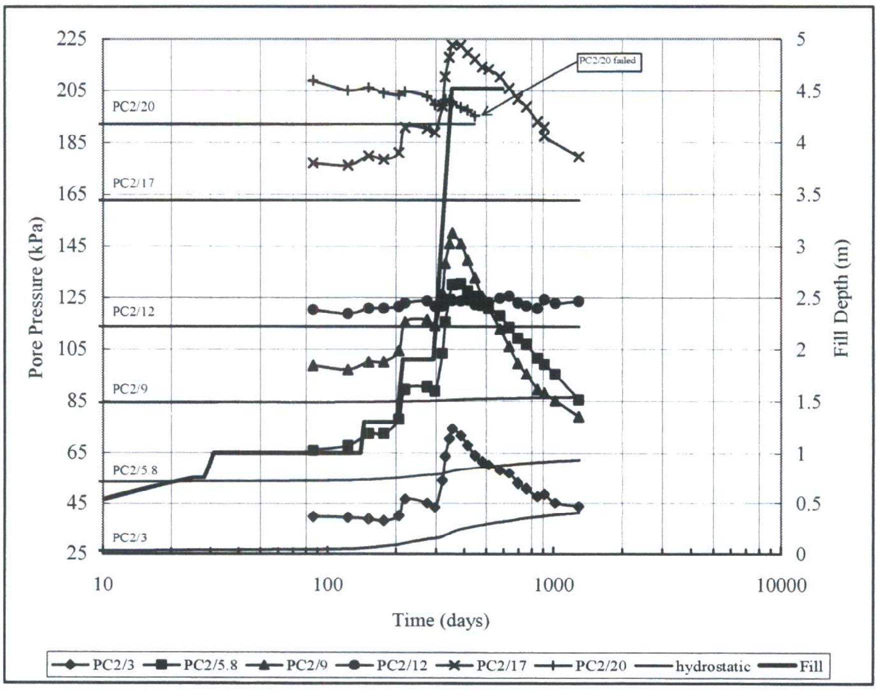

Figure 6. Piezometer Readings at PC2

Undrained Shear Strength

One of the key elements of the Cumbalum trial was the provision of strength gain data to provide greater certainty with staging of embankments. Undrained shear strength of the soil profile prior to the construction of the embankment was obtained by a variety of means including in situ vane shear, CPT and laboratory triaxial tests. Figure 3 shows the profile of in situ vane strength at a number of locations at Cumbalum prior to the placement of the embankment. An approximate average vane strength trend line has been provided. Figure 8 shows the same data normalized by initial vertical effective stress (ie. c_u/σ_{vo}'). The ratio c_u/σ_{vo}' can be seen to decrease from about 1.0 at 2.5m to in the range 0.45-0.55 over the normally consolidated depth range of 5m to 20m. It is noted that all vanes strengths are uncorrected. The relationship of Skempton for normally consolidated clays ($c_u/\sigma_v'=0.11 + 0.0037I_P$) would suggest a ratio of approximately 0.40 to 0.52 for I_P between 80% and 110% for the Ballina clay.

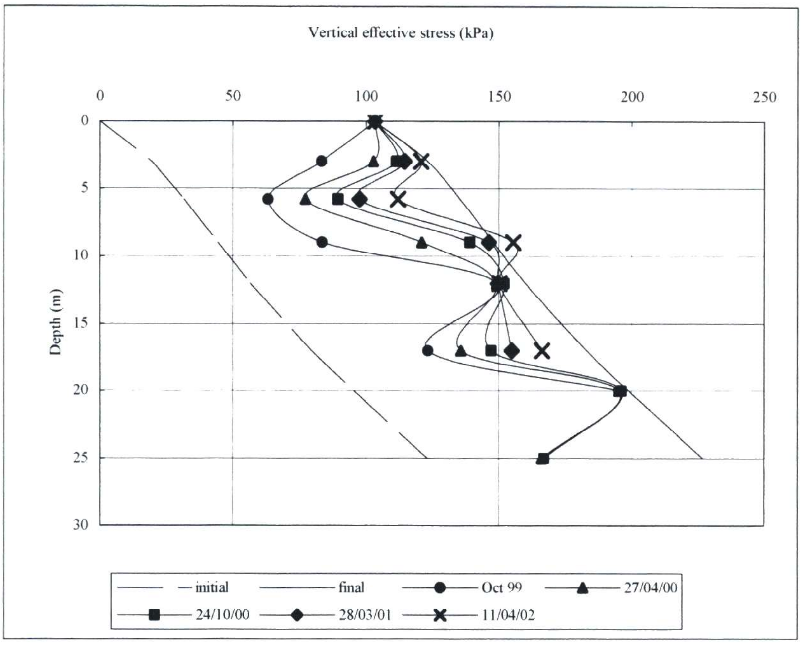

Figure 7. Profiles of Vertical Effective Stress at PC2

Following construction of the embankment the current strength profile in the upper soft clay has been periodically measured in the vicinity of BH86 by means of in situ vane. The results of those tests are shown in Figure 9 and indicate a variable increase in undrained strength with depth over time. The increase in undrained strength is up to as much as 30kPa at shallow depths and near the sand layer at 10m depth. Strength gains at intermediate depths have a degree of time dependence and this increase is investigated in Figure 10 where the relationship between increase in effective vertical effective stress and undrained strength is plotted. As the depth of strength measurement and the depth of piezometers do not match exactly, some interpolation of data has been required in this analysis. Features of Figure 10 to note are:

- The ratio $\Delta c_u/\Delta\sigma_v'$ is in the range 0.1 to 0.3.
- The results at 9.0m indicate a higher $\Delta c_u/\Delta\sigma_v'$ ratio and this is likely to indicate the presence of some sand at that depth.
- Over the period from construction to April 2002 the average ratio $\Delta c_u/\Delta\sigma_v'$ at depths 3m and 5.8m is approximately 0.21.
- The early readings at depths 3m and 5.8m indicate lower ratios of $\Delta c_u/\Delta\sigma_v'$, however, the sensitivity of these ratios to test accuracy are higher, and are subsequently considered to be less reliable.

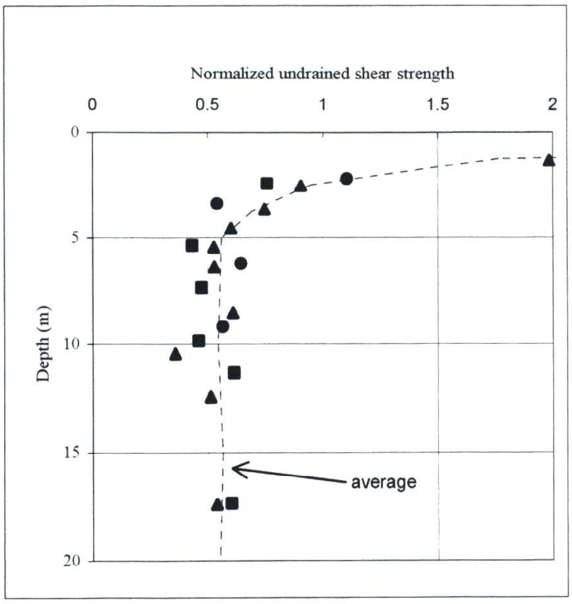

Figure 8. Variation of Normalized Undrained Shear Strength With Depth

CONCLUSIONS

Monitoring data from an instrumented 4.5m embankment on the alignment of the proposed Ballina Bypass has been presented. The data confirms that the geotechnical conditions impose significant risks and constraints upon highway construction. In particular, large and long term settlement together with low soil strength make construction timetables lengthy through the necessity of allowing substantial completion of settlements and staging of embankment lifts. These constraints, even with the use of stabilising berms, reinforcing geotextiles and vertical wick drains, make the use of alternative construction methods such as lightweight fill and piled embankments potentially attractive.

The instrumentation of the embankment has provided a detailed picture of the effective stress state within the soil and the development of vertical and horizontal settlements. Ongoing measurement of current shear strength as pore pressures have dissipated has provided a valuable guide to prediction of strength gain for construction timing.

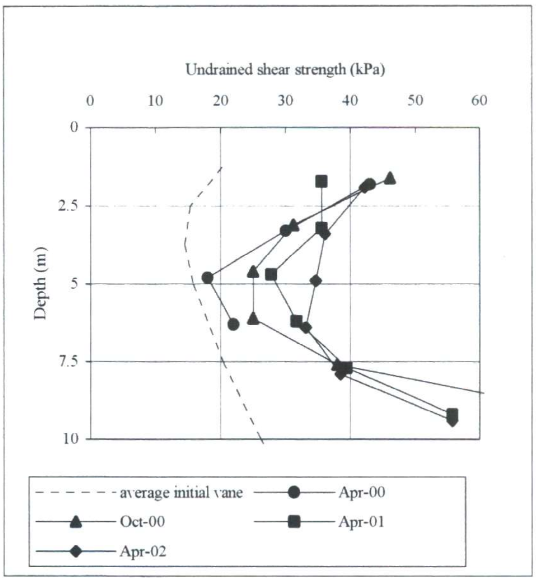

Figure 9. Profiles of Vane Shear Strength

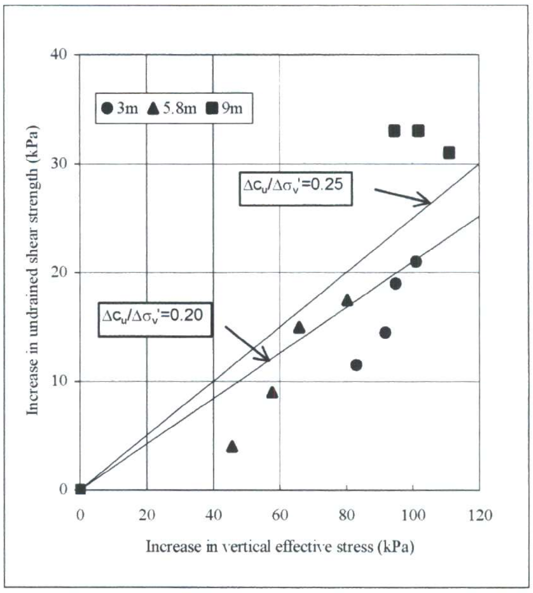

Figure 10. Variation of Vane Shear Strength and Vertical Effective Stress

ACKNOWLEDGEMENTS

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REFERENCES

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