Interpretation of Stress History in Soft Plastic Soils: Some Case Histories from the Lower Fraser Valley

by

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Abstract

Design of structures on sites underlain by soft plastic soils is governed by consideration of allowable bearing capacity and settlement, though ultimately settlement becomes the limiting criterion for moderate to heavy loads. If the applied stress in the soil is permitted to exceed the maximum past stress there is potential for large and damaging settlements. Thus, accurate determination of stress history is a critical part of understanding the constraints on design.

The near surface soils of the Fraser Valley region are varied and complex, deposited during many periods of glaciation and subsequently during the post Vashon period of glacial melt and continental rebound. The major post Vashon depositional environments within the lower Fraser Valley include glaciofluvial, glaciomarine and more recently fluvial. Glaciomarine deposits in the region include clays, silts, sands and gravels, though the clays and clayey silts provide the most significant challenge to the design practitioner.

The Nicomekl and Serpentine River valleys and the adjacent sloping valley walls contain glaciomarine deposits of clayey silts which vary dramatically in terms of shear strength, sensitivity, plasticity, thickness and degree of overconsolidation. This paper presents a methodology for estimating overconsolidation ratio from electric cone penetration test (CPT) data.

Introduction

Overconsolidation of soil deposits occurs as a result of three main physical mechanisms: unloading, desiccation, cementation or a combination of the above. Casagrande (1936), suggested a graphical procedure which allowed for the interpretation of pre-consolidation pressure from a laboratory consolidation test. Somewhat modified but generally similar procedures have been presented by Schmertmann (1953) and others. Figure 1 presents the results of a consolidation test for a sample of marine silt, plotted as void ratio against log effective stress. Superimposed on the consolidation test results is a construct for determining pre-consolidation pressure using the procedures recommended by Casagrande.

Procurement of laboratory samples of clayey soils is typically done from open cuts (block samples) or from open or cased borehole using special tube samplers (piston samples). Care must be taken during sampling transport and sample set up to minimize disturbance. In cases of extremely sensitive soils conventional sample procurement cannot be done without remolding the sample.

The consolidation test, presented on Figure 1, was performed on a sample of sensitive marine clay obtained using a piston sampler from 6 metres below grade from a site in south Surrey. Interpretation of the consolidation test results predict a pre-consolidation stress of about 70 kPa. The curve appears to lack the sharp "bump" between the re-compression and virgin compression portions of the consolidation curve (shown with the dotted line) typically seen with sensitive marine clays, indicating sample disturbance may have occurred prior to testing. Sample disturbance can often lead to under estimation of pre-consolidation stress.

The foregoing presents an alternate methodology for determination of stress history which is based on insitu testing. The methodology requires a detailed understanding of soil and groundwater conditions and is therefore particularly well suited to use with the CPT which provides near continuous data. Careful interpretation of tip resistance recorded during the CPT sounding is used to infer maximum past pressure at three sites in south Surrey.

Quaternary Geology of the Lower Fraser Valley

The lower Fraser Valley is underlain by a thick sequence of Quaternary sediments described in considerable detail in several publications by the Geological Survey of Canada, the most relevant of which to geotechnical engineers is *Environmental and Engineering Applications of the Surficial Geology of the Fraser Lowland, British Columbia* (Armstrong, 1984). Readers are referred to this publication for detailed discussion on the origins and properties of the various sediments.

The study area, shown on Figure 2, is limited to the municipality of Surrey, however the approach to stress history determination presented can be extended to most jurisdictions in the Lower Mainland. Quaternary geology of the study area can be summarized in terms of three major historical events, **Glaciation**, during which period the Vashon Drift is believed to have been deposited by an advancing glacier, **Glacial Recession,** where inter glacial through post glacial sediments were deposited during and after glacial recession and **Recent Fluvial** where current fluvial and marine processes dominated deposition of sediments.

Iso-static uplift of the continental land mass accompanied the period of Glacial Recession. Glaciofluvial processes dominated near the receding ice mass while glaciomarine and marine processes dominated further away. Ablation of the continental ice sheet and deposition of sediments continued as the land mass rebounded. Thus the marine and glaciomarine clays and silts can be found over a large range of elevations above current sea level. The concurrent deposition and uplift resulted in the downward fining sequence of sediments typically seen in the major post glacial depositional group referred to as the Capilano Sediments. Glaciomarine clays and silts of the Capilano Sediments are widespread throughout the study area. Deposition in a brackish to marine environment and subsequent groundwater leaching has rendered many of these clays and silts sensitive with sensitivities of up to 29 reported (Greig, Campanella and Robertson, 1986). Clays and silts deposited at higher elevations tend to be older than those deposited closer to current sea level and more likely to be overconsolidated. Clays and silts deposited in areas of groundwater recharge or downward groundwater flow are more likely to be overconsolidated than those deposited in areas of groundwater discharge or upwards groundwater flow.

Figure 3 shows an inferred cross section along the Fraser Highway in Surrey, between 160th and 200th Streets. The figure shows the relative positions of the geologic units described. Note the deeply scoured post glacial Serpentine Valley which has been in filled with recent Salish Sediments.

The interpretation of stress history in clays and plastic silts of the Capilano Sediments is the subject of all of the case histories presented in this paper.

Methodology

The method of stress history determination is based on a comparison of measured undrained shear strength to vertical effective stress ratio, $(Su/\sigma'_{VO})_{meas}$, with the expected ratio for a normally consolidated soil, $(Su/\sigma'_{VO})_{NC}$, as proposed by Schmertmann (1975). It is generally accepted that undrained shear strength increases with degree of overconsolidation for clays and plastic silts. It is also generally accepted that in most cases the undrained shear strength of normally consolidated soils increases with plasticity index (Ip). Considerable laboratory and insitu testing research has been has been focused on determining reliable means of predicting increases in undrained strength as a function of increased effective stress (σ'_{VO}) due to consolidation (Ladd, 1991). Back analyses of failed cuts, natural slopes and embankments have demonstrated the importance of case specific factors which include direction of loading, plasticity index, fabric, cementation as well as many other factors. A stress history interpretation based on undrained shear strength requires careful consideration of all of the aforementioned parameters. Laboratory consolidation testing of high quality tube or block samples is generally the preferred method of determining stress history, however this approach can be relatively expensive (in terms of sample procurement and testing costs) and in some cases high quality samples are difficult to obtain due to the high degree of sensitivity of the stratum of interest. In situ tests are becoming increasingly relied upon to supplement and if necessary replace laboratory tests.

Undrained shear strength can be measured directly in the field by way of the field vane shear test or indirectly from the cone penetration test (CPT). In the writers experience the latter method is more convenient since the data is recorded in more regular intervals (usually every 5 cm or 2 inches) and provides a means of inferring subsurface soil and groundwater conditions. The digitized data from the CPT can be easily read into interpretive programs or customized spreadsheets to allow for rapid processing. Detailed information on the use and interpretation of CPT data can be obtained in *Guidelines For Geotechnical Design Using CPT and CPTU* (Robertson et al, 1989) and *Cone Penetration Testing in Geotechnical Practice* (Lunne et al, 1997). The acronyms CPT and CPTU are used interchangeably herein as CPT with pore pressure measurement has become standard.

A review of proposed methods of OCR determination using measured CPT parameters, by Lunne et al (1997) shows 16 "significant" empirical and theoretical correlations for OCR determination. These correlations can be generally divided into ones that are based on measured cone resistance (q_c) and ones that are based on measured pore pressure (U). As noted the method advanced herein is empirical, using CPT data (converted to Su) and therefore belongs to the empirical-q_c group of correlations. The range of methods available is potentially overwhelming, especially for persons inexperienced with the CPT.

The writers own experience in the lower mainland suggests that undrained shear strength can be inferred most reliably from corrected cone tip resistance (q_t) using the following generally accepted correlation:

$$Su = (q_T - p_O)/N_{KT}$$

where Su = undrained shear strength
q_T = tip resistance, corrected for unequal end area effects
p_O = total overburden stress
N_{KT} = empirical cone factor

The empirical cone factor varies with overconsolidation ratio (OCR), sensitivity and other factors. A more compete discussion of factors effecting N_{KT} can be found in Greig et al (1988) and in Lunne et al (1997). In general N_{KT} appears to vary between about 10 and 20, though in the writers experience 11 through 15 appears to a more appropriate range for clayey soils in the lower mainland. An N_{KT} of 15 was found to be provide reasonable good agreement with Nilcon vane results below the desiccated crust (soil described as firm to soft, OCR less than 8) in the study area. This value is close to the 14 reported in Greig et al (1988) and 14 to 15 in Crawford and Campanella (1990) for marine clays and silts in Cloverdale. Both Greig et al and Crawford and Campanella report a high degree of variability in N_{KT} within the desiccated crust. This is consistent with the writers own experience. It is interesting to note that data from Greig et al shows that the CPT and the Screw Plate Test (both of which induce vertical, bearing capacity type, failure) consistently measure a higher apparent shear strength than the Field Vane and the Dilatometer (both of which induce failure in the horizontal plane) in the crust. This is believed to be due to strength anisotropy, rather than differences in N_{KT}. The anisotropy is believed to be caused by a number of factors including soil fabric and the occurrence of fine sand partings in the upper portion of the soil profile. Regardless, errors in undrained strength interpretation of up to 50% can be made in the upper crust with very little impact on the interpretation, since the OCR is often in the range of 20 to 30 through the stiff portion of the crust. Field Vane Tests are generally recommended to confirm or fine tune the selection of an appropriate empirical cone factor. For the reasons discussed above, the tests should be done below the desiccated crust.

For normally consolidated clays, the undrained shear strength to vertical effective stress ratio $(Su/\sigma'_{VO})_{NC}$ is related to plasticity index (I_P), with $(Su/\sigma'_{VO})_{NC}$ generally increasing with I_P for most clays and silts. A review of available laboratory and field data suggests that the relationship between $(Su/\sigma'_{VO})_{NC}$ and I_P varies depending on factors such as depositional environment, sensitivity and method of shear strength determination. Based on data presented by Ladd (1991), the observed dependence of $(Su/\sigma'_{VO})_{NC}$ on I_P is greater for field data than for laboratory data. Ladd's data also suggests that organic clays and silts plotting below the A line on the Unified System plasticity chart have little change in strength ratio with I_P. A summary of four of the recommended relationships proposed by others is shown on Figure 4. The writer's own experience has been with that relationship proposed by Schmertmann (1978), based on data from Skempton (1957) and Ladd & Foott (1974), though in general 0.20 was used as a lower limit for $(Su/\sigma'_{VO})_{NC}$. The relationships presented on Figure 4 appear to be quite different, though all but the Schmertmann relationship are quite similar over the low plastic range of 10 to 30. From the data reviewed and the writer's own experience in the study area an $(Su/\sigma'_{VO})_{NC} = 0.21$ appears to give good results where plasticity index is between 15 and 30.

Caution is recommended when extending the approach to Holocene soils especially those with low plasticity (I_P less than 15).

As noted above, the measured strength ratio $(Su/\sigma'_{VO})_{meas}$ is strongly influenced by overconsolidation ratio (OCR). Schmertmann (1978), proposed a relationship between the ratio of the measured strength ratio to the normally consolidated strength ratio and OCR. The Schmertmann recommended average relationship takes the form of the following equation for OCR's up to 10:

$$OCR = (SR^{1.67} + 0.82) / 1.82$$

where SR is the strength ratio, $(Su/\sigma'_{VO})_{meas} / (Su/\sigma'_{VO})_{NC}$

As discussed above, even large errors in OCR interpretation are normally of little consequence when OCR's exceeds 10. It is important to be aware of the sensitivity of the OCR analysis to the input parameters which include, I_P, q_T, N_{KT}, soil unit weight and water level. Tip resistance (q_C and q_T) can be measured with a high degree of accuracy using modern electric cones, particularly low range cones (2.5 to 5 ton). In the writer's experience temperature drift is the single largest cause of measurement errors. The following example illustrates the potential importance of measurement or rounding errors. For the case of a corrected (for unequal end area) tip resistance measurement of 4.5 bar (0.45 MPa) at 5 metres below grade, with the water table at 2 metres below grade, an error of 1 bar (0.1 MPa) results in an error of up to 7 kPa in Su and up to 0.11 in $(Su/\sigma'_{VO})_{meas}$. The resulting OCR interpretation would vary from 1.4 to 2.4 in the case of negative and positive errors respectively compared with the "real" value of 1.9. Based on the same simple example, an error of 1 metre in water table location would results in a similar 0.099 error in $(Su/\sigma'_{VO})_{meas}$. The effect of erroneous measurements decreases with depth. Varying the parameters within some range of reasonable limits will greatly assist in determining ones confidence level with the analysis. In situ testing contractors should be asked to provide uncorrected CPT data with base line measurements to allow the interpreter to make any necessary corrections. Site specific field vane and laboratory testing should be carried out when working in new areas. Field observations and geology can often be used to provide useful corroborative support for the interpretation.

The remainder of this paper deals with case histories presented to illustrate the application of the methodology and provide some validation where available.

Site 1: South Surrey, Bridge

The site is located in South Surrey, south of the Nicomekl River flood plain. The development of a large multi-phase residential sub-division required the construction of a creek crossing. The creek was identified as fish bearing and maintaining it as such was critical to both the B.C. Ministry of Environment (M.O.E.) and the developer. To permit a near 15 metre set back from the creek required a span of 30 metres.

Design loads for the bridge were as follows:

Self Weight of Abutment	190 kN/m
Superstructure Dead Load	150 kN/m
Total Live Loads	60 kN/m

Two CPT's were advanced at the site, one at the location of each abutment, using a 10 tonne piezocone. The subsurface conditions were essentially the same at both locations. The CPT sounding completed at the north abutment, shown on Figure 5 below, showed up to 1.5 metres of fill over natural soils which consist of 1.5 metres of silty sand, grading in to sandy silt with depth, over 19 to 20 metres of soft very sensitive marine clayey silt over very dense well graded silty, gravelly sand (glacial till). The CPT sounding was terminated 10 cm into the glacial till where effective refusal for the equipment was reached.

The groundwater level observed in an adjacent test holes was inferred to be about 1.5 metres below existing grade. Dynamic pore pressures were allowed to dissipate in the glacial till. The equilibrium pressure head observed in the till was consistent with a water level of 1.5 metres below existing grade, identical to that observed in the test holes, indicating a very small vertical pressure gradient (if any). A full understanding of any vertical pore pressure gradient is important since it affects the interpretation of σ'_{VO} and consequently $(Su/\sigma'_{VO})_{meas}$. Fine grained soils can sustain significant equilibrium pressure gradients, especially towards the contact with underlying tills or sands and gravels. Assuming hydrostatic conditions in areas of significant upwards or downward gradients can lead to incorrect estimates of $(Su/\sigma'_{VO})_{meas}$.

The interpreted undrained shear strength and overconsolidation ratio (OCR) profile is presented on Figure 6a. Figure 6a shows a desiccated crust up to 4 metres thick below which the clayey silt becomes moderately overconsolidated to about 12 metres, with OCR's of 5 down to 2, and undrained shear strengths which range from a low of 30 kPa at 4.5 to 5 metres below existing grade, increasing to about 60 kPa at the contact with the underlying glacial till. The I_P of the silt was measured to be 15. From 12 to 23 metres below grade the silt is interpreted as being lightly over consolidated. The ripples on the OCR curve are believed to be due to variations in plasticity with depth.

The top of deck elevation for the proposed bridge varied between 13.81 and 13.96 metres (geodetic) at the south and north abutments respectively compared with existing grades of about 10.2 and 8.5 metres. Construction of the approach ramps would require placement of up to 5.5 metres of permanent fill behind the abutments.

The two alternative foundation options considered for founding the bridge were spread footings and piles (pre-loading was not considered due to the thickness and sensitivity of the deposit). The piled option was considerably more expensive and did not reduce the major stresses which were from the approach ramp fills. Conventional light weight fill (hogfuel) was rejected by the M.O.E because of the proximity to a fish bearing creek and alternative light weight fills were expected to be comparatively expensive. As shown on Figure 6a, our analysis showed some overconsolidation in the silt. An analysis was carried out to determine the suitability of spread footings for supporting the bridge abutments and span with conventional mineral fill (sand or sand and gravel) used for the approach ramps.

The analysis of insitu stresses carried out for the heaviest loading condition is shown on Figure 6b . The inferred maximum pre-consolidation pressure determined is shown alongside the cumulative stresses imposed by the dead plus live load from the bridge and the area load from the approach ramps. Stresses (from dead plus live loads) were distributed using an isotropic elastic (Boussinesq) stress distribution. Figure 6b shows the combined (imposed) unfactored stresses to be very near or above the pre-consolidation envelope over a significant portion of the soil profile. An estimate of consolidation settlement was made based on a comparison between insitu and pre-consolidation stresses. The cumulative settlements are also shown on Figure 6b. Total settlements of up to 76 mm were predicted at the north abutment. Settlements at the south abutment was expected to be within the expected elastic range of less than 25 mm due to the lesser fill height.

Since there was some flexibility in construction schedule and the owner insisted that the work be completed the same year, it was suggested that the abutments and approach ramps, which were the heaviest components of the structure, could be constructed in late fall (before the span was to be placed) and monitored over the winter. The span would be added in spring, assuming that the settlement of the abutments was within the range predicted and the deck could be levelled. The design team and the developer agreed on this approach.

The results of the monitoring are presented on Figure 7. The figure shows that while normal elastic settlements were observed at the south abutment, significant settlements were observed at the north abutment. Total settlements of up to 80 mm where recorded behind the north abutment over a period of 215 days of monitoring. Differential settlements measured along the north abutment were up to 40 mm over 15 lineal metres of footing. The vertical line at 59 days approximately corresponds to the date when preparation work for placement of the bridge deck began. The maximum total and differential settlements observed since the deck was added are 24mm and 15 mm over 15 metres respectively.

Table 1, presented below, summarizes the pre-construction and post construction (geodetic) elevations at the bridge abutments and the observed settlements.

Table 1: Summary of Fill Thicknesses and Settlements

Location	Original Grade	Finished Grade	Fill Thickness	60 Day Settlements	200 Day Settlements
North Abut. - East	8.5 m	13.96 m	5.46 m	40 mm	64 mm
Centre	9.7 m	13.96 m	4.26 m	20 mm	39 mm
West	10.2 m	13.96 m	3.76 m	10 mm	19 mm
South Abut -East	10.0 m	13.81 m	3.81 m	0 mm	10 mm
Centre	10.1 m	13.81 m	3.71 m	10 mm	15 mm
West	10.2 m	13.81 m	3.61 m	10 mm	15 mm

As shown on Table 1, the east end of the north abutment was the location of the thickest abutment and approach ramp fills. There is a clear relationship between fill height and observed settlement.

In the foregoing example, the foundation design was heavily influenced by a number of external factors including an available construction window, limited by the Ministry of Fisheries, permitting and time constraints imposed on the developer and as usual a tight budget. The potential for comparatively large settlements was made clear to the owner and the design team. The savings in schedule and costs derived from proceeding in the manner proposed was large and thus the approach was adopted. It is clear to the writer that the settlements were driven mainly by the weight of the approach ramp and abutment fills. A pile supported bridge foundation would have eliminated differential settlement between the two abutments and across the abutments themselves however it would have resulted in large differential settlements (up to 80 mm based on the settlement data collected) at the abutment/approach ramp fill contact. This would have caused significant problems for the services (including water, power and gas) which cross from the grade supported approach ramp over to the pile supported abutment. The total settlement measured at the north abutment appears to far exceed the limits which would be considered acceptable for most structures however the total and differential settlements of 24 mm and 15 mm over 15 metres (3/4 inches over 50 feet) since the deck was placed appears to be more reasonable. Good agreement was observed between the predicted and observed settlements at the north and south abutments (76 mm compared with 80 mm at 200 days and 10 to 15 mm compared with less than 25 mm). This case history shows how rational engineering decisions can be made at settlement sensitive sites.

Site 2: South Surrey, Warehouse

The site is located near 152nd Street and Highway No. 10 in Surrey. A large tilt up warehouse was to be constructed at the site. Finished floors elevations were set up to 1.5 metres above existing grades. Structural loads were as follows:

Maximum Floor Loads	25 kPa
Column Loads (dead plus live)	up to 1000 kN
Wall Loads (dead plus live)	58 kN/m

Published geologic maps predict the site to be underlain by pre-glacial sand to sand and gravel of the Quadra Formation. Previous work in the area had revealed a localized "pocket" of near surface soft marine clayey silt, up to 6.5 metres thick, covering a portion of the site. Visual examination of disturbed samples from auger test holes showed very little stiff crust at the site. A total of 6 CPT's were carried out at the site, using a 5 tonne piezocone, to assess the strength and compressibility of the soft silt pocket. Atterberg limit tests showed the soft silt to have an I_p of 20. Figure 8 shows the results of a typical CPT sounding for the site, with interpreted soil types along the right hand column. The figure shows 6.5 metres of clayey silt over very dense till like sand and gravel. The groundwater level is believed to vary between 1.5 and 2.0 metres below pre-development site grades.

Interpretation of undrained shear strength and overconsolidation ratio for the sounding is presented on Figure 9a. Figure 9a shows that the silt deposit has a stiff, desiccated crust underlain by 0.8 metres of soft silt over firm to stiff silt towards the contact with the underlying till like sand and gravel. Characteristic undrained shear strengths of about 20 kPa were interpreted within the soft silt layer. The soft silt was interpreted to have an OCR of about 3 while the stiffer silt above and below the soft zone appeared to have an OCR in excess of 10.

Foundation options considered included spread footings after pre-loading and pile supported foundations. Pre-loading was not favoured since consolidation was expected to proceed quite slowly due to the lack of drainage on either side of the soft silt and it was expected that it may uneconomical, due to the size of the building (60,000 square feet), when compared with the piled option. A wide variety of pile types would be suitable given the depth to good bearing and the expected pile length.

A stress history interpretation was performed and the proposed insitu stresses were compared with the interpreted pre-consolidation envelope. The results are presented on Figure 9b. For the purpose of the analysis the heaviest loading, with the maximum fill height, floor and column load was considered. In fact only one of 27 columns was carrying 1,000 kN and it was in an area where floor loading was expected to be less than 12 kPa and fill heights were in the order of 0.5 metres. The remaining columns loads carried about 500 kN. This somewhat conservative approach was considered appropriate given the type of structure and the uncertainties in the analysis.

Reference to Figure 9b shows that the superimposed fill plus floor plus column loads somewhat exceeded the interpreted pre-consolidation envelope over the thin soft silt layer (about 800 mm). The imposed stresses exceed the pre-consolidation stresses by an average of less than 10 kPa. A cumulative settlement calculation, also shown on Figure 9b, suggested that the increase in insitu stress (above the pre-consolidation stress) would result in only 5 mm of consolidation settlement within the zone of soft silt. The settlement was recalculated based on a 20 kPa error in pre-consolidation stress (overestimation). The resulting settlement was less than 15 mm. Columns spacings were 15 metres (50 feet) so that up to 25 mm differential between columns was considered acceptable. Thus final recommendations were for spread foundations without any pre-construction site improvement. The building was completed in late summer of last year (1997). Monitoring was not carried out. However it is unlikely that any of the design assumptions would have been tested since live loads, of which snow would be the most significant, are 60% of the total superstructure loads.

A recommendation in favour of normal spread foundations would not have been made without a rational methodology for estimating post construction settlements and a very clear understanding of the consequences of interpretation errors.

Site 3: South Surrey Residential Development

The site is located near the U.S. border in south east Surrey. The area is generally low lying and predominantly agricultural. A residential sub-division had been proposed for the site, several acres of which was below the designated "flood proofing" elevation. The owner wished to raise grade in this area by up to 3 metres and then construct roads, services and relatively light, wood framed structures on the fill. The area to be filled was adjacent to a salmon bearing river and placement of fill was only permitted within a narrow time window.

A previous preliminary geotechnical report prepared eight years before was available. The report indicated that the site was underlain by a considerable thickness of soft clayey silt. A laboratory consolidation test carried out on a sample from 6.7 metres depth suggested that the silt was normally consolidated. A supplementary program of 4 CPT soundings was recommended to the owner. A 5 tonne piezocone was used to complete the soundings

Figure 10 presents the results of a CPT sounding performed within the proposed fill area. The CPT sounding shows the site to be underlain by two metres of fill over a thick sequence of soft clayey silt to silty clay to the maximum depth of the sounding (about 22 metres). Groundwater conditions were assumed to be hydrostatic, with a water table 3.0 metres below grade, on the basis of the shape of the pore pressure response.

Figure 11a shows the interpreted undrained shear strength and OCR profiles for the CPT sounding presented on Figure 10. Two metres of fill was observed to overlie the natural soils at the time of the field investigation and a further 1.5 metres was to be added to reach final flood proofing grade. The upper 4.5 metres of the natural profile was interpreted as firm to stiff and heavily overconsolidated, beyond which the profile becomes lightly (but distinctly) overconsolidated. The characteristic undrained shear strength at 5 metres below grade is about 30 kPa. Figure 11b shows a comparison of interpreted pre-consolidation stress and insitu effective stress after completion of filling for the same profile. Note that the addition of the fill advances the insitu stress line towards the pre-consolidation envelope.

Since the developer did not intend to commence construction for several years, we recommended that the lot filling proceed with bi-weekly monitoring of settlement gauges. The low lying portions of the site were filled over a period of several weeks. Settlements recorded over a three month period were within the accuracy limits of the survey equipment indicating that the stress history interpretation carried out on the CPT data was correct.

In this case the decision to proceed with the filling was a relatively simple one since the low lying portions of the site had to be filled to flood proofing elevation regardless of the final foundation design. The low lying area was located within a wide and relatively flat U-shaped valley which contained the existing river channel. The valley was likely formed by channel erosion so that some overconsolidation was extremely likely.

Summary and Conclusions

The case histories presented illustrate a methodology for interpreting stress history in soft plastic soils. The interpretation requires the selection of number of empirical factors. The empirical factors to be chosen for the analysis can vary considerably and depend on soil properties, including OCR, sensitivity and plasticity index, variations in the water table, dry and wet unit weights and other unknown site specific factors that cannot be accounted for with any single parameter (such as I_P). Nevertheless soils which occur within a particular region (for example the lower Fraser Valley) are often depositionally related and therefore have a much narrower range of properties.

Values of N_{KT} below the desiccated crust were found to be quite consistent for the study area. Considerable variability in N_{KT} is found in the desiccated crust though in general the apparent N_{KT} is higher (20 compared with 14 to 15) in the crust. Since the overconsolidation in the crust is typically very high even large "errors" in N_{KT} don't appear to effect the outcome of the stress history interpretation significantly. The clay and silt soils at the three test sites were all stratified and sensitive to varying degrees yet the variation in sensitivity or stratification was not accounted for directly in the analysis. Factors related to soil fabric and structure such as sensitivity, stratification and strength anisotropy are accounted for in N_{KT}.

Plasticity index is a key a parameter. Figure 4 shows that, for soils of low plasticity, four independently derived $(Su/\sigma'_{VO})_{NC}$ vs I_P relationships fall within a vary narrow band of the chart. This suggest that $(Su/\sigma'_{VO})_{NC}$ for low plasticity clays and silts ($15 < I_P < .30$) can be estimated with a considerable degree of accuracy A review of the test data for high plasticity silts and clays indicates that $(Su/\sigma'_{VO})_{NC}$ determination for these soils can be subject to large errors. Without site specific testing, a conservative value of $(Su/\sigma'_{VO})_{NC}$ should be chosen for the stress history interpretation. Very little data is available for clays and silts where I_P is less than 15 and extrapolation into this portion of the chart should be done cautiously. Site specific correlations are recommended. It is unusual for Holocene age clays and silts to be overconsolidated and caution is recommended when interpreting any of theses soils as overconsolidated.

Published surficial geology maps, available from the Geological Survey of Canada, are excellent guides to soil type and age. With sufficient experience soil type, soil unit weight, and groundwater level can be readily inferred from CPT soundings. $N_{KT,}$ and I_P can be accurately determined with simple field and laboratory tests. As discussed above, the sensitivity of the analysis to variations in key parameters such as $N_{KT,}$ $(Su/\sigma'_{VO})_{NC}$ and water level decreases with depth. Where confidence in some parameters is low, they should be varied to provide a range of possible interpretations.

The case histories presented demonstrate that pre-consolidation pressure of plastic soils can be reliably determined from CPT data for a range of soil plasticities provided that select index properties can be obtained from other laboratory and field tests.

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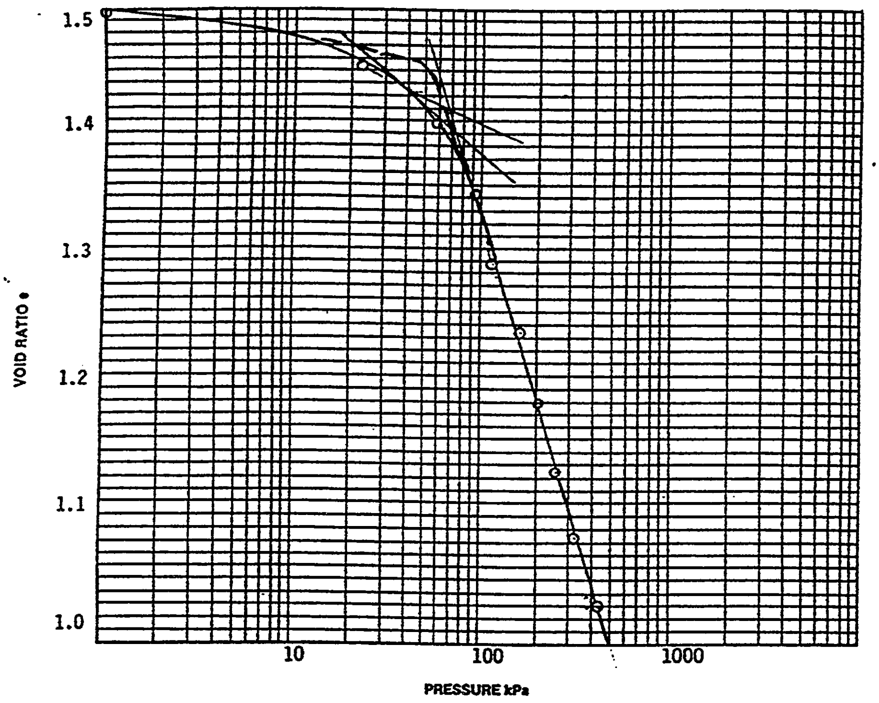


LIQUID LIMIT............58.2 %
PLASTIC LIMIT..........28.9 %
PLASTICITY INDEX....29.3
SAND.........................3 %
SILT..........................36 %
CLAY.........................62 %
INITIAL WATER CONTENT...............55.9 %
FINAL WATER CONTENT..................35.0 %
INITIAL VOID RATIO..........................1.491427
FINAL VOID RATIO.............................0.9345 %
PRE CONSOLIDATION PRESSURE..70........KPa
COMPRESSIVE INDEX..........................0.46

FIGURE 1: RESULTS OF CONSOLIDATION TEST ON SAMPLE OF SENSITIVE MARINE CLAY

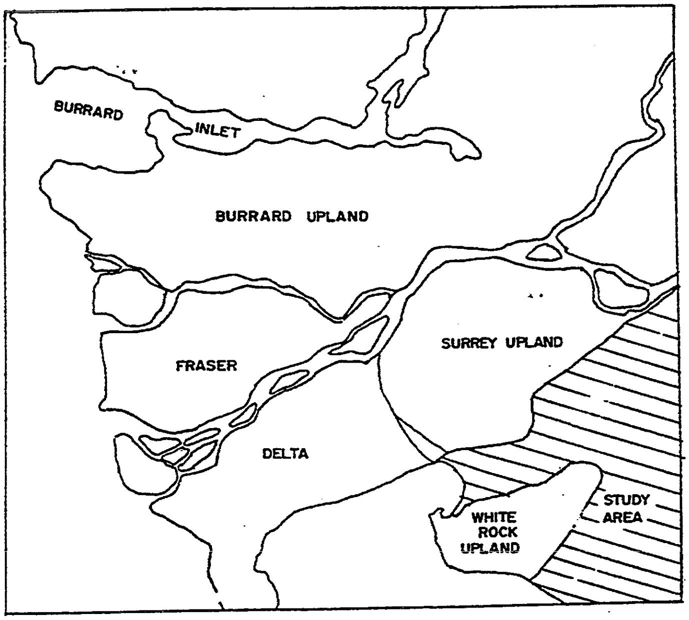


FIGURE 2: STUDY AREA

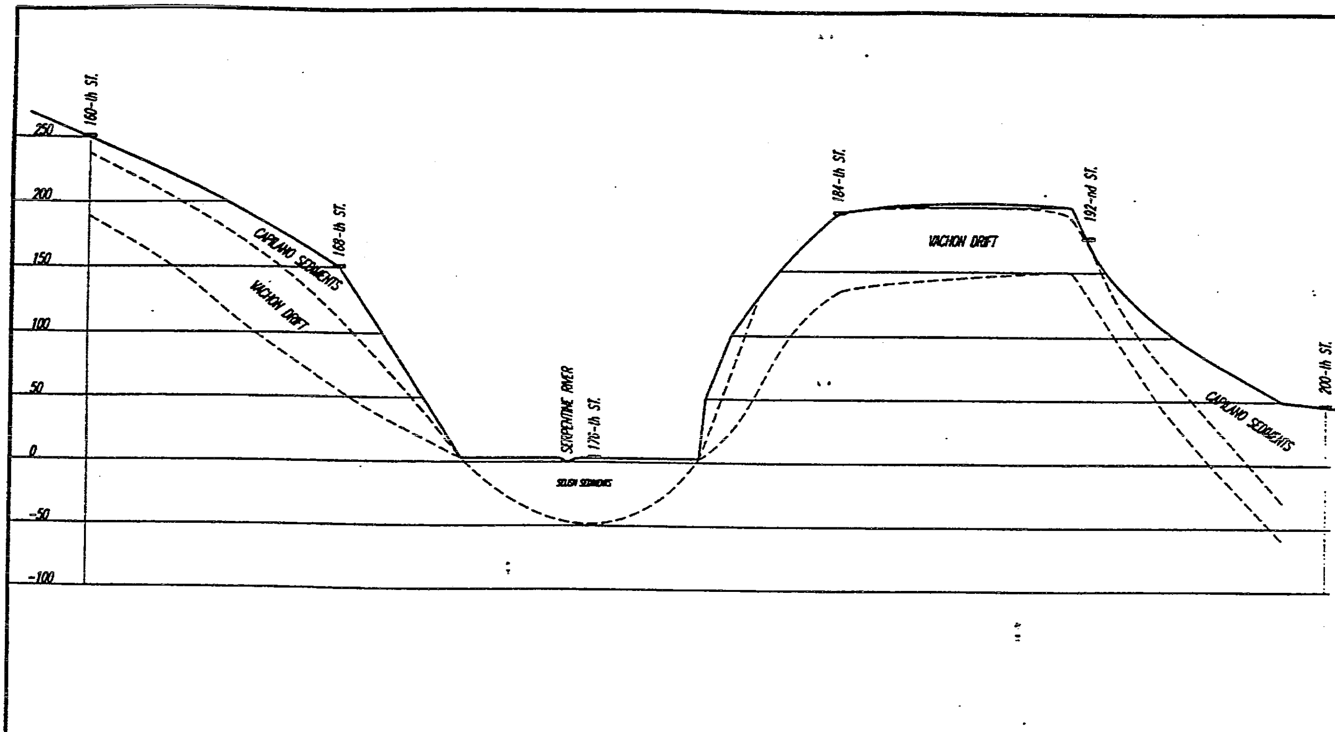
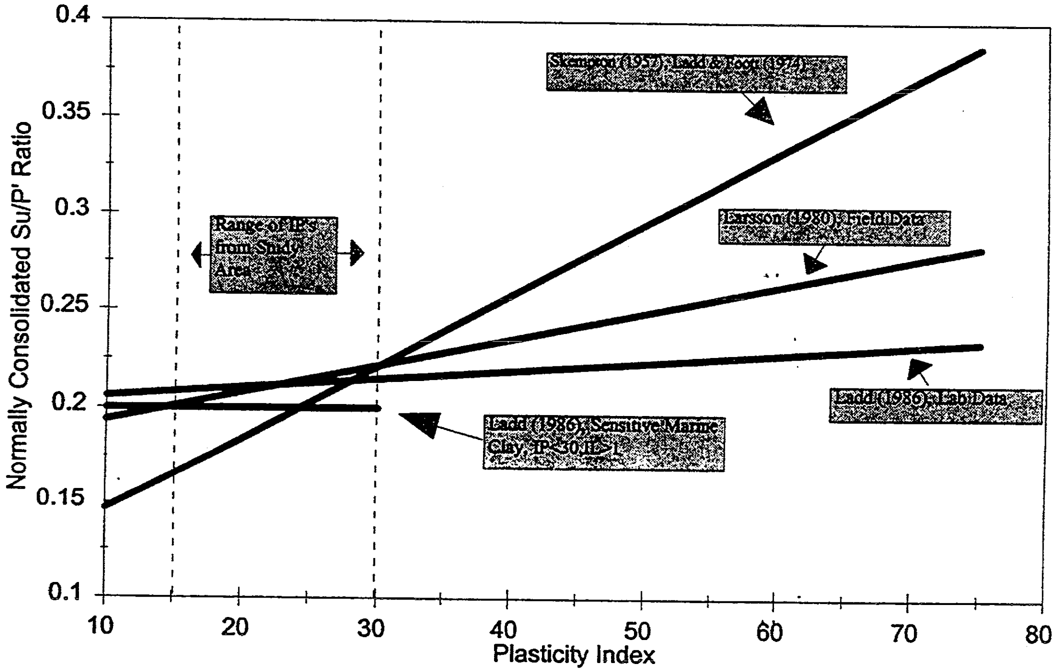


FIGURE 3: INFERRED CROSS-SECTION ALONG FRASER HIGHWAY, FROM 160-th STREET TO 200-th STREET

Figure 4: Proposed Relationships
Strength Ratio vs Plasticity Index



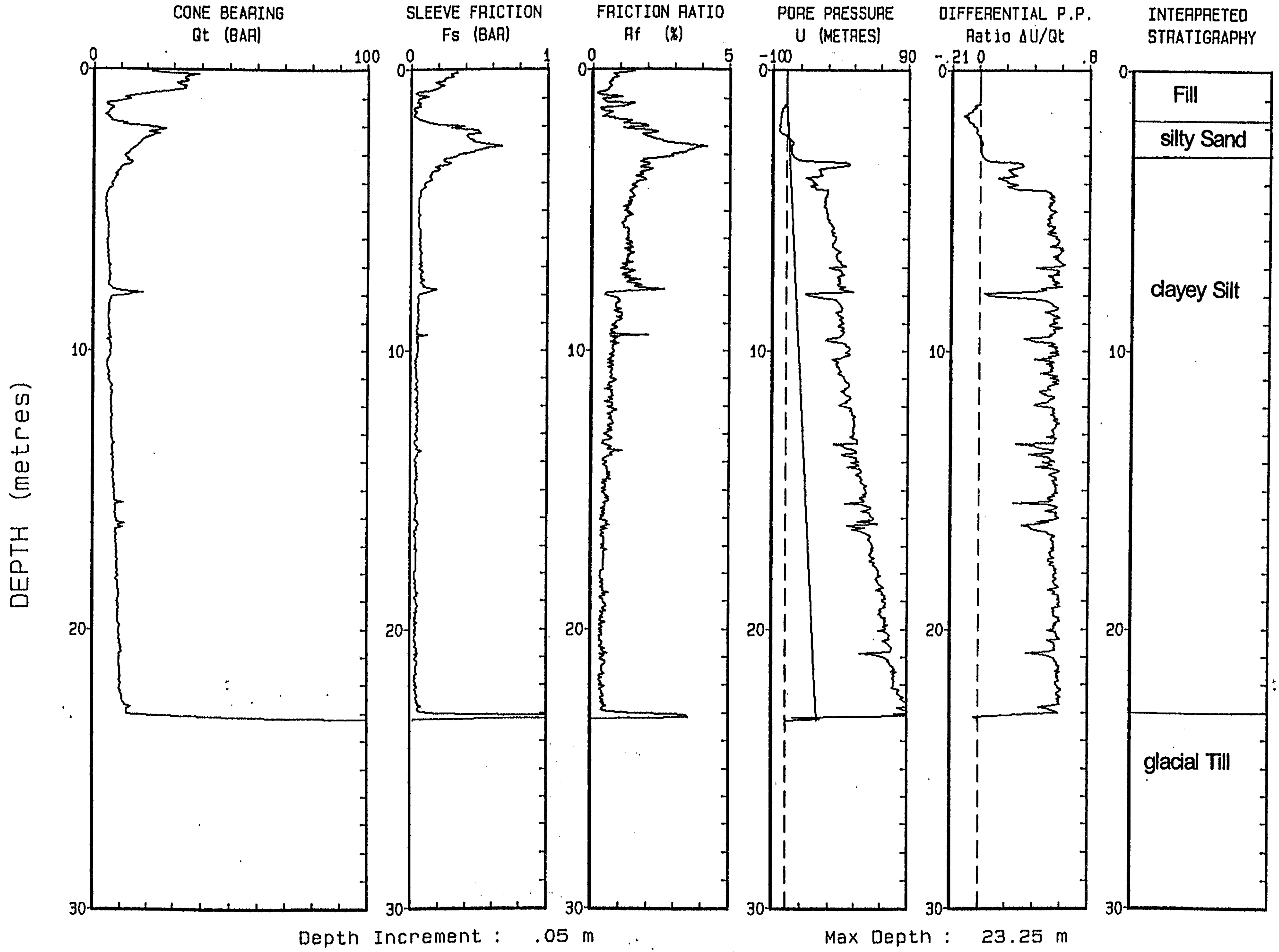


Figure 5: Results of CPT Sounding for South Surrey Bridge Site

Figure 6a: Bridge Crossing
Undrained Shear Strength and OCR

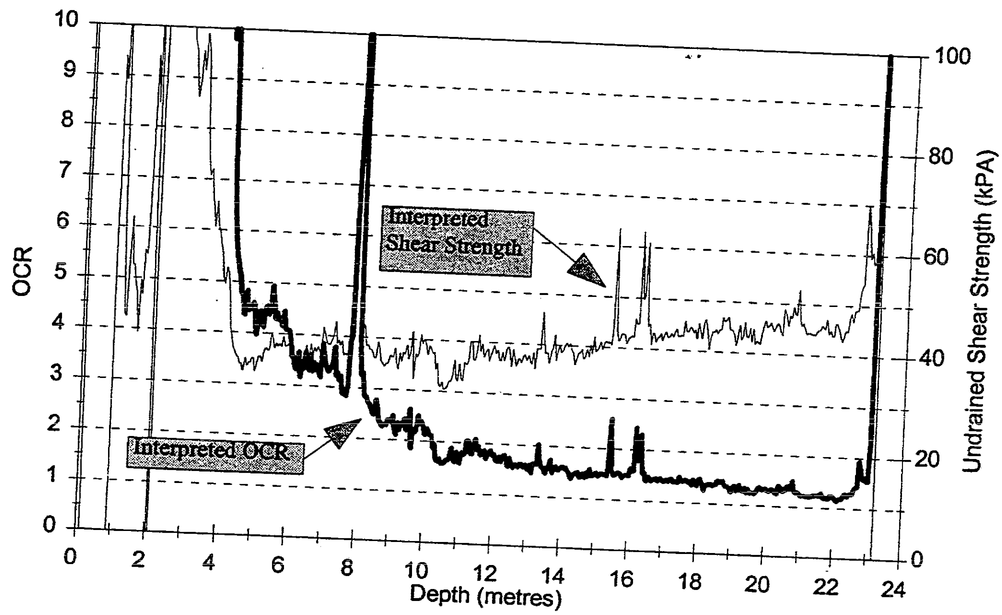


Figure 6b: Bridge Crossing
Vertical Effective Stress vs Depth

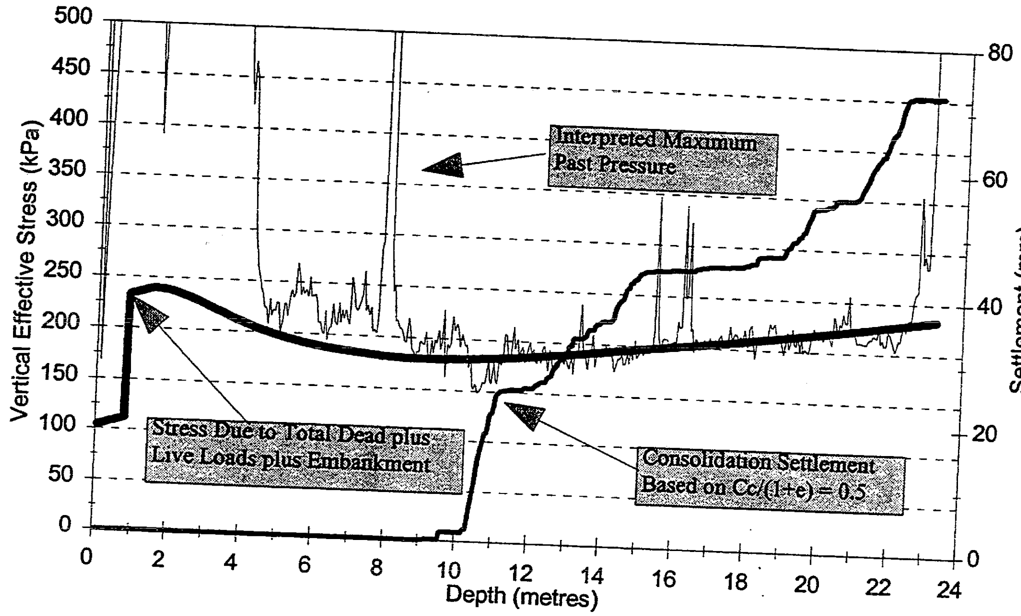
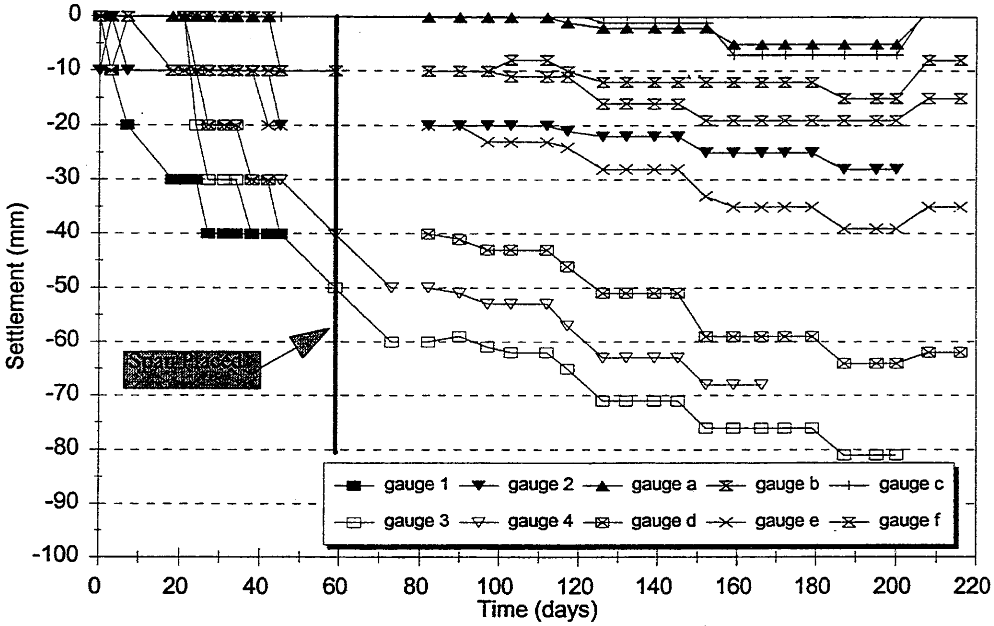
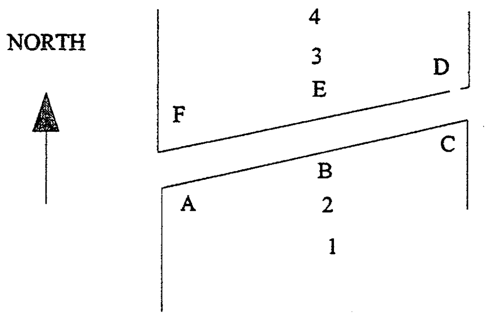


Figure 7: Bridge Crossing
Settlement Monitoring



Schematic Plan View of Monitoring Points at Bridge Abutments



Gauges A through F on Abutment (fixed)
Gauges 1 through 4 in Approach Ramp Fill

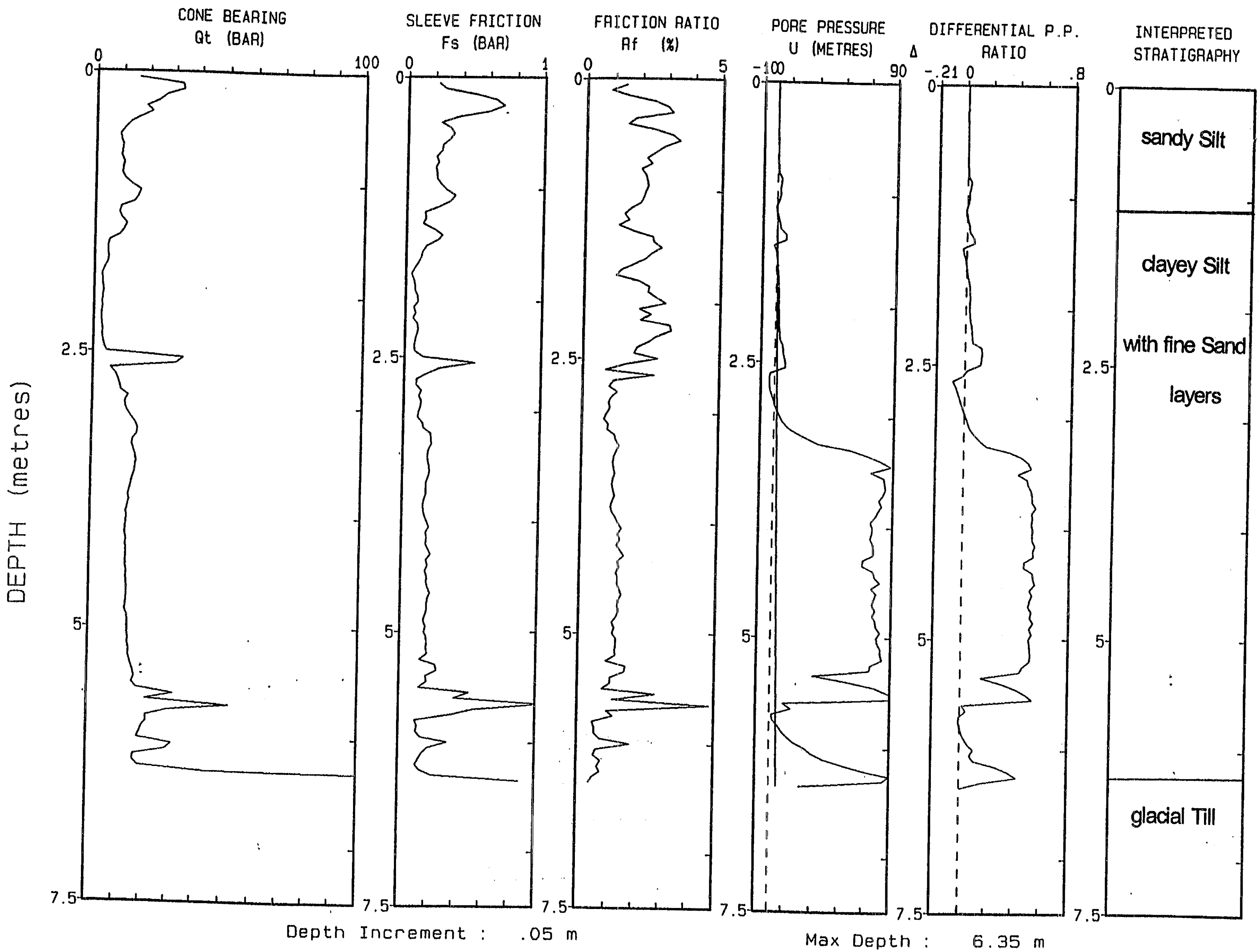


Figure 8: Results of CPT Sounding for South Surrey Warehouse

Figure 9a: South Surrey Warehouse
Undrained Shear Strength and OCR

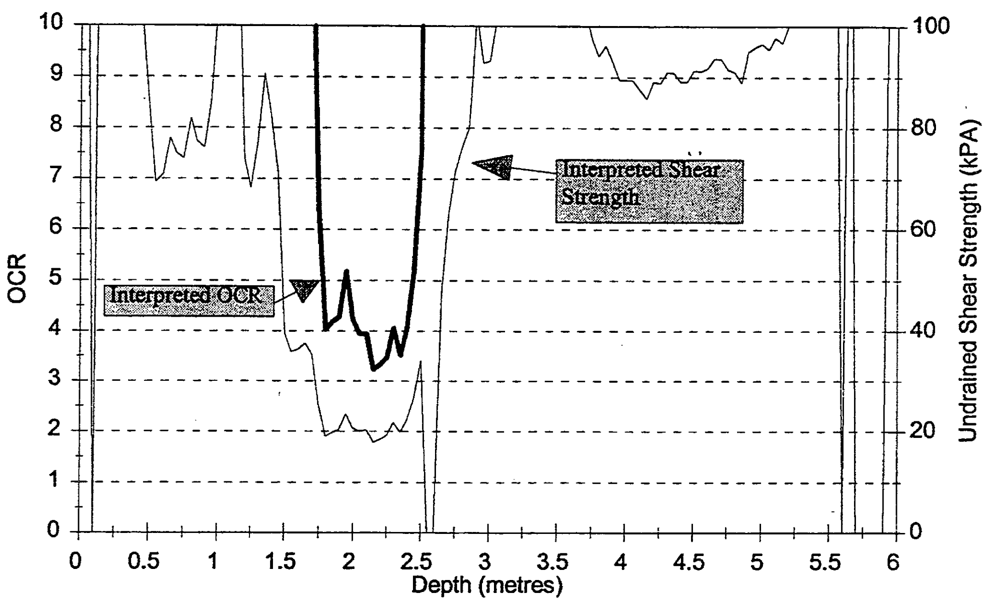
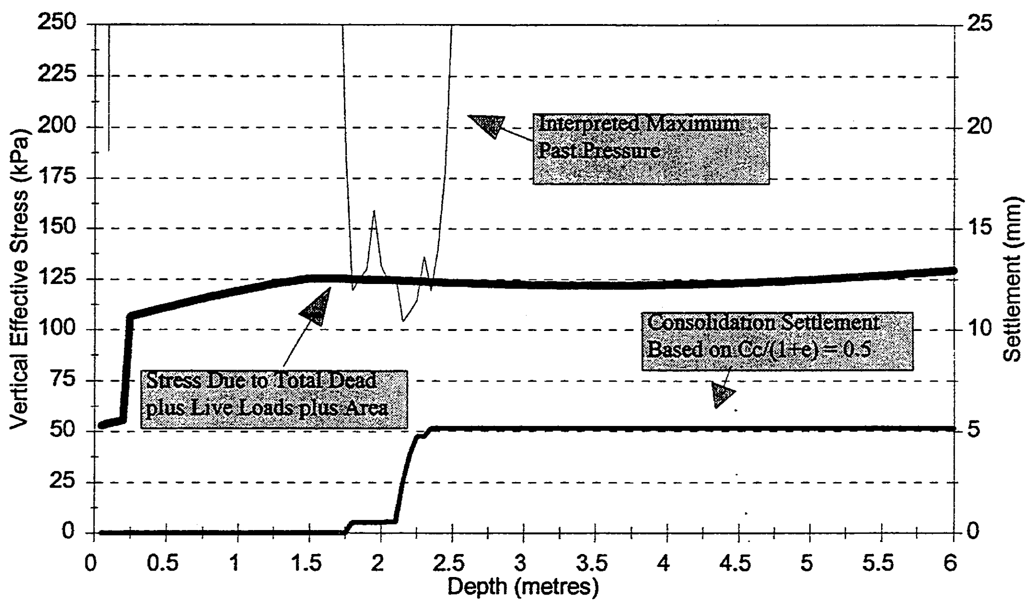


Figure 9b: South Surrey Warehouse
Vertical Effective Stress vs Depth



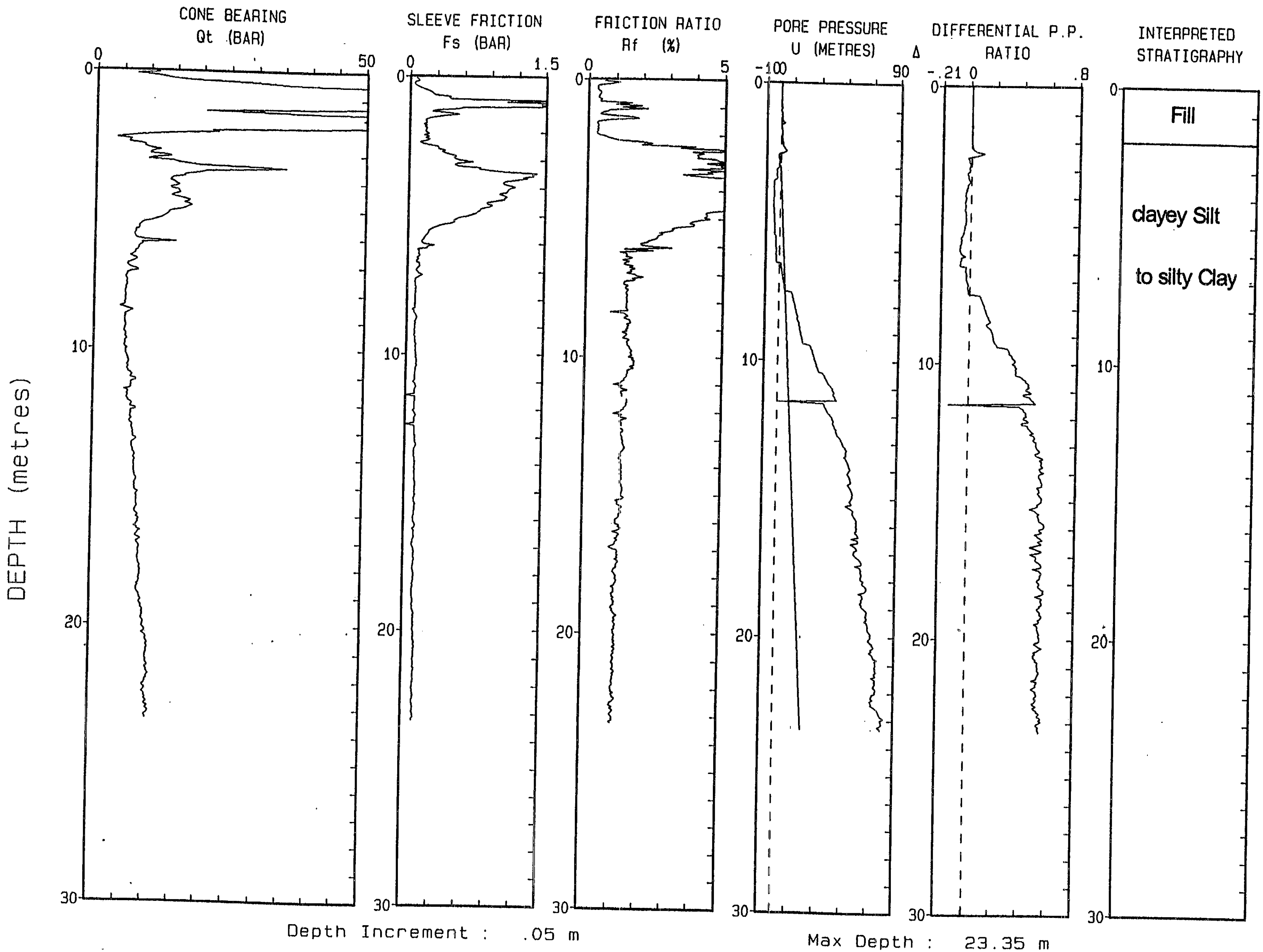

CONE BEARING
Qt (BAR)
SLEEVE FRICTION
Fs (BAR)
FRICTION RATIO
Rf (%)
PORE PRESSURE
U (METRES)
Δ DIFFERENTIAL P.P.
RATIO
INTERPRETED
STRATIGRAPHY
Fill
clayey Silt
to silty Clay
DEPTH (metres)
Depth Increment : .05 m
Max Depth : 23.35 m

Figure 11a: Residential Development
Undrained Shear Strength and OCR

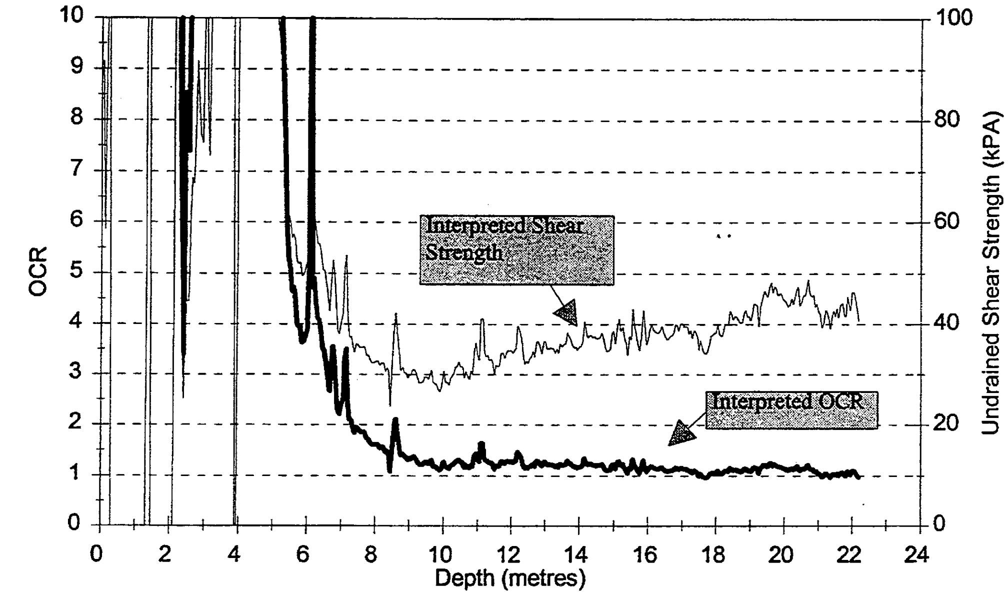


Figure 11b: Residential Development
Vertical Effective Stress vs Depth