TRANSCANADA HIGHWAY – 200TH STREET LANGLEY INTERCHANGE BRITISH COLUMBIA, CANADA – PART 1: TEST FILL OBSERVATIONS

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ABSTRACT
Embankment settlement was the principal geotechnical concern of the 200th Street Langley interchange structure located on the Trans-Canada highway about 50 km from Vancouver, British Columbia. The site subsurface conditions consist of soft, sensitive, and highly compressible clay of about 20 m thickness overlying dense glacial till. The site has a history of high settlements, up to 2 m during the last 40 years, which required several adjustments by jacking of the existing highway overpass abutments. The geometry of the new proposed single point interchange required large embankments with a maximum crest width of about 60 m and a maximum height of 6.5 m. This paper describes the monitoring of a pre-construction test fill carried out in order to characterize the pore water pressure and settlement response to gradually increasing embankment loads. Subsequent papers will consider the stability and long-term settlement prediction aspects and will describe the adopted solution for the design and construction of the final embankment.

RÉSUMÉ
Le tassement de remblais était la difficulté principale de la structure pour l'échangeur de la 200eme Rue à Langley situé sur l'autoroute TransCanadienne à environ 50 km de Vancouver, Colombie Britannique. Les conditions souterraines du site consistent en une argile, molle, sensible, et très compressible d'environ 20 m d'épaisseur reposant sur un till glacial dense. Le site a subi de larges tassements, jusqu'à 2 m durant les dernières 40 années, qui ont nécessité plusieurs ajustements de l'échangeur existant sur l'autoroute. La géométrie du nouvel échangeur unique à quatre voies requiert la construction de remblais d'une largeur maximale au sommet de 60 m et d'une hauteur maximale de 6.5 m. Ce document décrit les observations d'un remblai initial d'essai construit pour caractériser les pressions interstitielles et les tassements sous des charges croissantes. Des documents ultérieurs considéreront la stabilité et décriront la solution adoptée pour le dimensionnement et la construction du remblai final.

1. INTRODUCTION

The behaviour of clay foundations supporting embankments is one of the most documented problems in the geotechnical engineering literature (Bjerrum, 1972; La Rochelle et al, 1974, Leroueil et al, 1977a, 1977b, Tavenas & Leroueil, 1980). The test fill built at the 200th Street interchange on the Trans-Canada highway adds to the many published case histories in which the construction pore water pressures and settlements have been observed in detail. It is the purpose of this paper to present the observations under the 200th Street Langley Interchange test fill and to invite others to compare them with pore water pressure and settlement prediction methods presently available.

2. SITE AND SUBSURFACE CONDITIONS

The test fill is located on the new 200th Street alignment, immediately south of the Trans-Canada highway. The test site was selected for the similarity of its subsurface conditions with the area where the old overpass bridge abutment had experienced large settlement. Since construction of the old 200th Street Langley overpass in 1964, repeated jacking of the bridge deck - with the last one completed in 1999 - indicated a cumulative settlement of about 800 mm at the bridge north abutment and 1700 mm at the south abutment.

The geological history and deposition process of the clay underlying the site area are related to the withdrawal of the Vashon ice from the Fraser Lowland in British Columbia between 13000 and 11000 years ago. The clay at the 200th Street Langley interchange site appears to have been deposited in a marine environment following the ice retreat (Armstrong, 1981).

A fairly detailed subsurface site investigation was carried out, before construction of the test fill, and consisted of in situ Nilcon vane test profiles, electric Piezo-cone penetration tests, and auger holes with soil sampling by means of 75 mm diameter thin-walled steel tubes. The resulting information is summarized on Fig. 1a for the test fill location.

The typical soil profile at the test fill consists of 0.2 m of topsoil, 1.0 m of weathered clay crust, 22 m of soft silty clay of marine origin with occasional thin sand lenses below 20 m depth, and a deep layer of dense till-like soil extending to more than30 m depth.

Routine laboratory tests indicated that the silty clay is of medium to low plasticity with a plasticity index ranging from 20 to 50%. Its natural water content ranges from 52 to 88% and its liquidity index ranges from 0.56 to 1.42. The measured peak undrained strength at the test fill is essentially constant at 24 kPa, from 3 to 10 m depth, and increases to 35 kPa from 10 to 23 m depth. At other test holes near the test fill, the measured undrained shear strength decreased from higher than 50 kPa near the

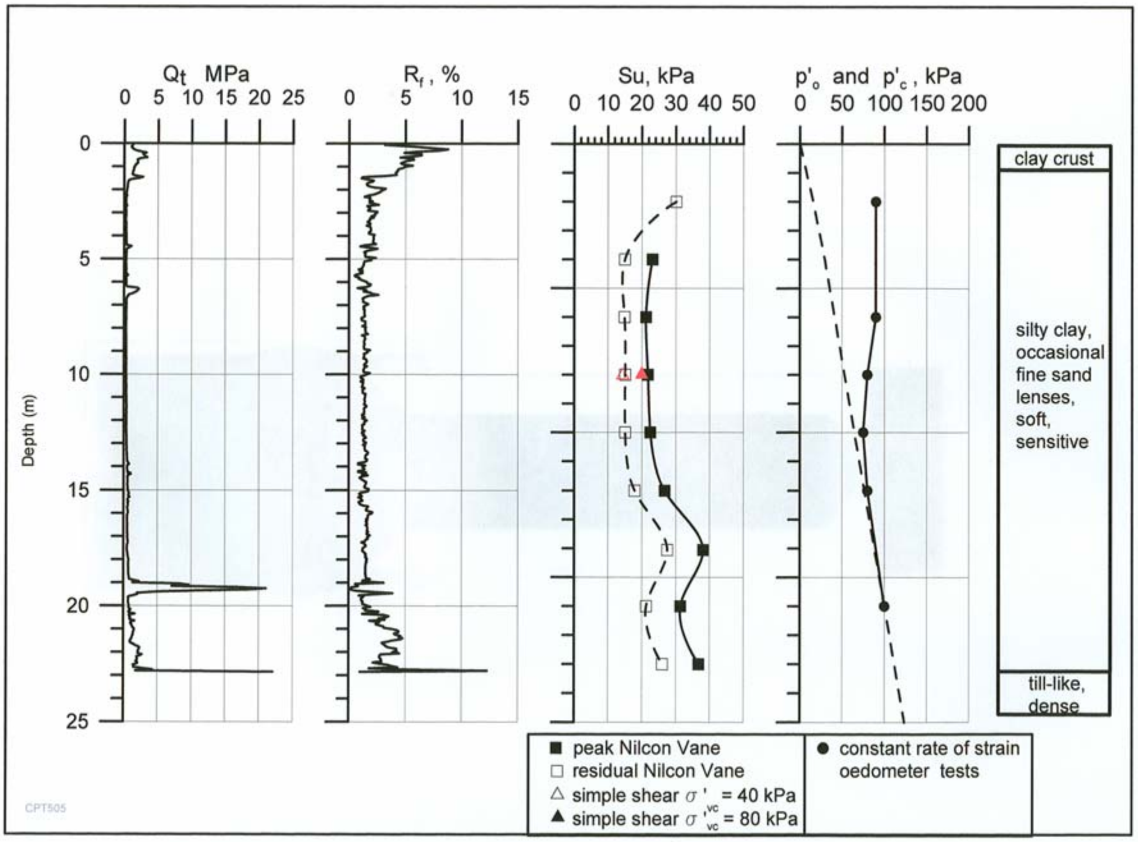

Fig. 1a Soil profile at 200th street interchange on the TransCanada highway, Langley, B.C.

surface to a minimum of about 14 kPa at about 5 m depth and then increased to a maximum value of 50 kPa at a depth of 20 m. The vane test results indicated a sensitivity of the clay, defined as the ratio of peak to residual undrained strength, ranging from 2.1 to 9.2. However, the majority of values ranged from 2.6 to 5.8 with an average of 4.1 from 14 tests. The limited scatter of the vane test results indicates that the clay is fairly homogenous. The in-situ undrained strength measured by vane tests compared reasonably well with the results of constant volume simple shear laboratory tests on 75 mm diameter thin-walled tube samples.

The results of 7 constant rate of strain oedometer tests on 75 mm diameter thin-walled tube samples show an interesting feature of this deposit: the estimated preconsolidation pressure values are essentially constant at about 95 kPa from 2 to 6 m depth, then gradually decrease with depth and become about equal to the effective overburden pressure below a depth of 12 m. Fig. 1b shows typical volumetric strain versus log vertical stress results from the constant rate of strain tests. These results show that the clay is highly compressible and that the preconsolidation pressure and the entire compression curve is strain rate dependent. From these data, it was not possible to distinguish geological preconsolidation from the pseudo-preconsolidation that has developed due to secondary compression of the deposit (Bjerrum, 1967).

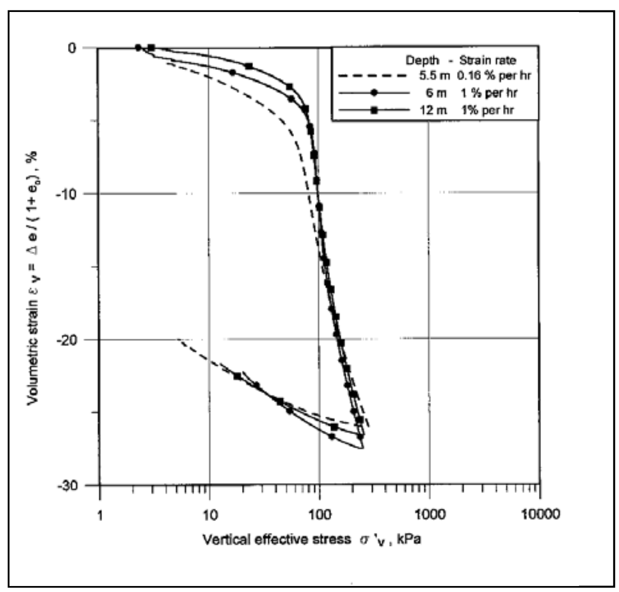

Fig. 1b Typical constant rate of strain oedometer test results.

3. TEST FILL AND INSTRUMENTATION

The test fill construction started in December 1999 for the purpose of measuring the pore water pressure response and settlement performance of an embankment with increasing height. A typical cross-section and the location of the instrumentation are shown in Fig. 2. The test fill was 32 m in length, 16 m in width at the crest and its height was successively increased from 1 to 3 m in 1 m steps with typical side slopes of 1 vertical to 2.5 horizontal.

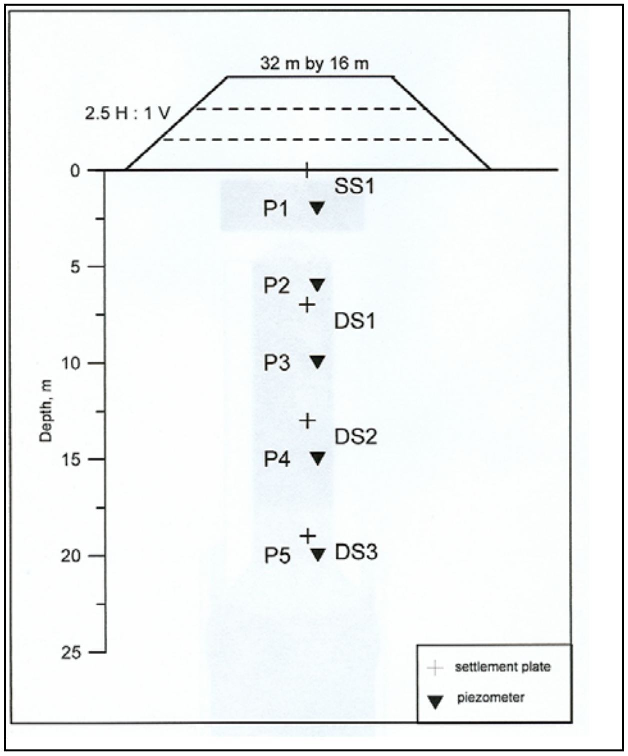

Fig. 2 Test section with piezometers and settlement plates location.

In the present paper the attention will be limited to the observation of pore water pressure and settlement generated during construction at selected depths below the centre of the test fill. The pore water pressure, settlement, and lateral displacement data collected from instrumentation located way from the centre of the test fill would require an understanding of the effect of stress axis rotation, which is difficult to account for adequately at this time. Stability of the test fill, long-term settlement analysis, and final embankment design considerations will be discussed in future publications.

It should first be noted that the piezometric levels obtained from all pore pressure points prior to the beginning of construction indicated an artesian condition near the deep till-like soils equal to approximately 2 m head of water and a resulting upward gradient, which increased from 0.02 at 6 m depth to 0.10 at 19 m depth. This relatively small artesian component was included in all measured pore ater pressures in excess of hydrostatic levels.

3.1 Pore Water Pressure Observations

The development with time of the applied embankment loads and the pore water pressures generated at selected points below the centre of the test section are shown on Fig. 3. It may be seen that the pore water pressure response is relatively fast upon the application of each increment of fill load. The time for each step of embankment construction was about 1 to 2 days, which resulted in an estimated average construction rate of about 15 kPa per day. This rate is markedly higher than the average construction rates ranging from 0.5 to 8.4 kPa per day for various other embankments reviewed by Leroueil et al (1977b).

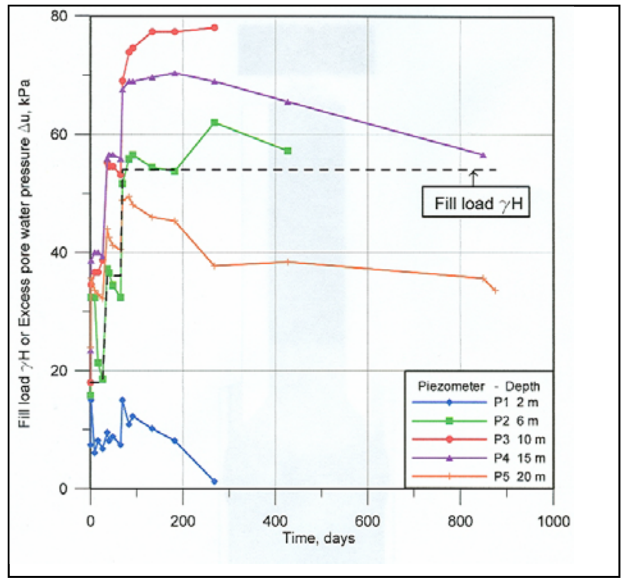

Fig. 3 Applied embankment load and induced pore water pressure as a function of time.

The dissipation of pore water pressure starts occurring shortly upon completion of the fill load application but at different rates with depth. The maximum observation time was limited to 900 days due to project constraints and some piezometers became defective after about 300 to 400 days.

The pore water pressures generated at different depths below the centre of the test fill are plotted as a function of the applied embankment load on Fig. 4. Upon fill load application, the pore water pressures are the highest at 10 and 15 m depth, slightly below the middle of the clay layer. With the exception of the point at 2 m depth, all generated excess pore water pressures were equal or higher than the applied fill loads γH.

Fig. 5 shows the measured pore water pressures with depth as the ratio $r_u = \Delta u / \gamma H$ under increasing fill loads. The measurements show that the coefficient r_u is higher than 1.0 below 5 m depth. The r_u coefficient is the highest at about 2.1 under 18 kPa fill load between 10 and 15 m depth. Increases of fill loads to 36 and 54 kPa cause the r_u coefficient to decrease to about 1.5

between 10 and 15 m depth. At 2 m depth, the r_u coefficient is about 0.8 under 18 kPa fill load and

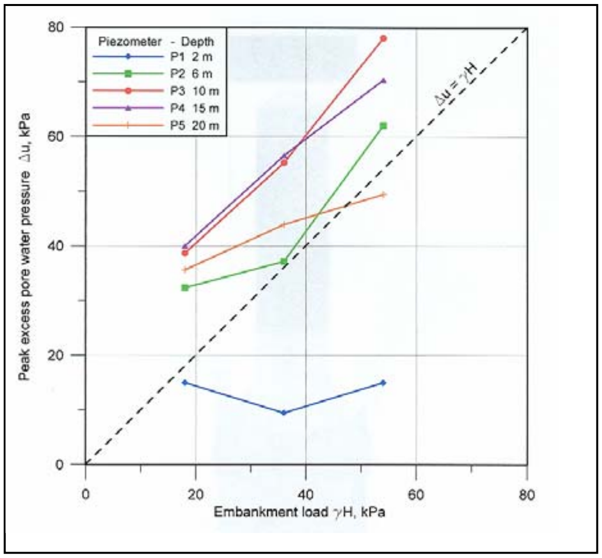

Fig. 4 Pore pressure observed below the centre of the test section versus embankment load.

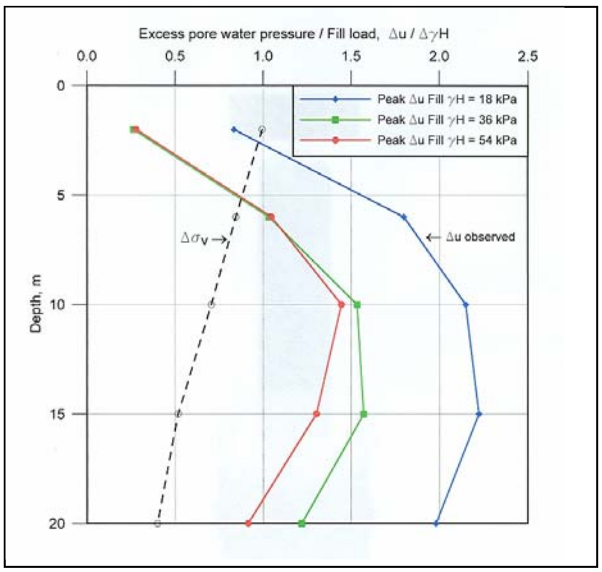

Fig. 5 Observed pore water pressure under the centre of the test section with increasing embankment load.

decreases to about 0.3 under 36 and 54 kPa fill load. For comparison purposes, Fig. 5 also shows the total vertical stress $\Delta\sigma v$ with depth. It may be seen that, below 5 m depth, the measured excess pore water pressures are markedly higher than $\Delta\sigma_v$.

Fig. 6 shows the variation of final vertical stress due to the applied fill loads based on Boussinesq's equations and a comparison with the variation of preconsolidation pressure with depth from the oedometer tests. It may be seen that the final vertical stresses become higher than the preconsolidation pressure at depths of 10, 7, and 6 m for fill loads of 18, 36, and 54 kPa, respectively.

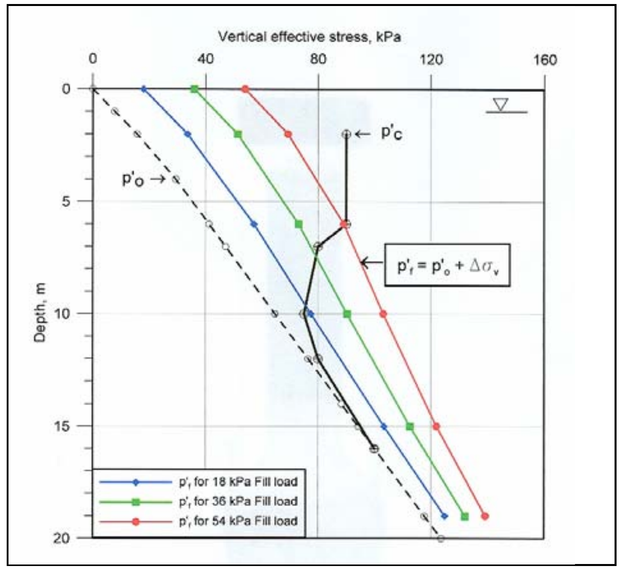

Fig. 6 Preconsolidation pressures and vertical effective stresses with depth.

This observation coincides with the higher r_u coefficients below 6 m depth shown on Fig. 5.

The application of the empirical methods suggested by Leroueil et al (1977b) and Leroueil and Tavenas (1980) to predict pore water pressure and critical embankment height was found difficult to apply to the above observations. As the embankment construction for this test fill is relatively high and the excess pore water pressure ratio $\Delta u/\Delta\sigma_1$ is higher than the theoretical limit of 1.0, it is likely that local failure occurred during filling and strain softening started to develop. Therefore, the attention is now directed towards the measurement of settlement at selected depths.

3.2 Settlement Observations

The development with time of the applied embankment loads and the settlement generated at selected points below the centre of the test section are shown on Fig. 7. It may be seen that the early settlement at the surface for fill loads less than 36 kPa is small - less than 75 mm in one month - compared to the settlement under 54 kPa fill load, which is about 200 mm in one month. The application of the third fill load of 54 kPa resulted in a markedly disproportionate settlement increase, such that the time rate of settlement was multiplied by a factor of 10. The final lift was left in place for just over two years with little change to the average time rate of settlement.

The distribution of settlement with depth shown on Fig. 8 indicates that, in the early stages following fill load application, the largest and fastest settlement occurs in the top third (0 to 7 m depth) of the soil profile. The magnitude and time rate of settlement decreases markedly below 10 to 15 m depth. The higher magnitude and faster settlement rate in the top part of the soil profile correspond with the higher degree of overconsolidation as indicated by the stress profiles shown on Fig. 6.

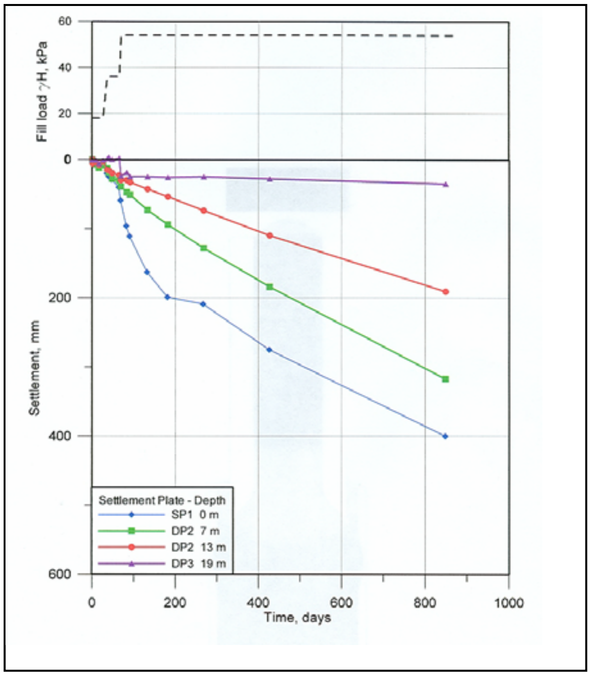

Fig. 7 Applied embankment load and settlement as a function of time.

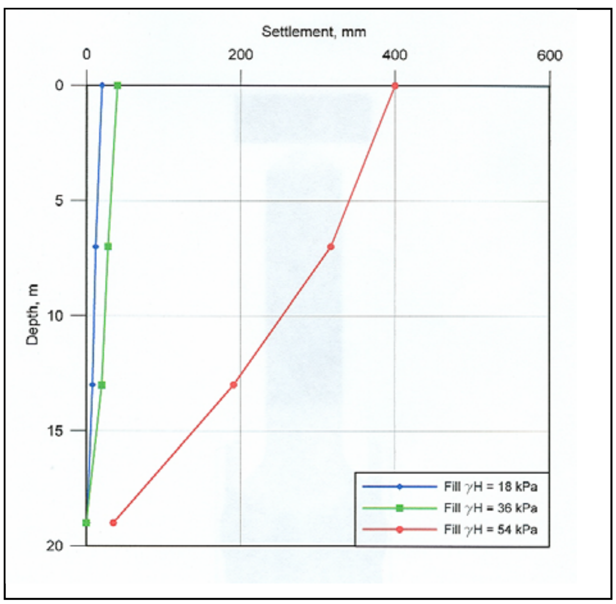

Fig. 8 Measured settlement under the centre of the test section with increasing embankment load.

Tavenas and Leroueil (1980) have pointed out that overconsolidated clay is characterized by a relatively high stiffness, represented by a small recompression index C_s in the volumetric strain versus log vertical effective stress plot and a resulting high coefficient of consolidation since $C_v = k\sigma'_v(1 + e_o)/0.434\gamma_w C_s$.

4. CONCLUSION

This paper described the monitoring of a pre-construction test fill carried out to characterize the pore water pressure and settlement response of the soft foundation clay to gradually increasing embankment loads for the 200th Street Langley interchange structure located on the Trans-Canada highway, British Columbia. We invite geotechnical engineers with expertise in embankment analysis to compare these observations with the results obtained using available prediction methods. Subsequent publications will consider the stability and long-term settlement prediction aspects and will describe the adopted construction solution for the design embankment.

ACKNOWLEDGEMENT

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