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Prediction of Embankment Performance at Vancouver International Airport Using In-Situ Tests

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SYNOPSIS

In-situ piezocone, flat dilatometer, and screw plate tests were performed adjacent to several large earth embankments founded on a deep deposit of compressible deltaic soils. Settlement records for 18 years since construction were available for two of the embankments.

Geotechnical parameters were interpreted from the in-situ tests and predictions made of the amount and rate of settlement for the two monitored embankments. Consolidation characteristics were interpreted from the measurement of dissipation of excess pore pressures using the piezocone and dilatometer. Both devices provided complementary results in terms of an appropriate coefficient of consolidation. The excellent stratigraphic profile provided by the piezocone (CPTU) tests proved to be a valuable feature in the analyses, since a continuous free draining sandy layer was identified within the thick compressible clayey silt deposit.

Predicted settlements with time were compared with the available measurements and a good agreement observed. A one-dimensional analysis formed the basis for the settlement predictions, and was found to be satisfactory.

INTRODUCTION

In order to accommodate increased traffic volume from the city of Vancouver to the airport, Transport Canada constructed a road system including a bridge and several high approach embankments on the eastern part of Sea Island. Due to the compressible nature of the soils much attention was given to the settlements which would occur as a result of the planned embankment construction. An extensive monitoring program was conducted by Transport Canada (Bertok, 1987) to monitor the performance of the embankments and bridge abutments.

The measurement of soil properties using in-situ tests is especially effective where field sampling in soft, sensitive sediments, such as the Fraser delta deposits, is known to be difficult. Therefore, a program of in-situ testing was set up with the goal of obtaining the necessary soil parameters to predict the rate and magnitude of settlement caused by the embankments constructed on Sea Island. A comparison was made between the results predicted by in-situ tests, the results predicted by the more traditional approach of borehole sampling and laboratory testing, and actual settlement measurements obtained by Transport Canada from 1971 through 1987. This paper presents a summary of this comparison.

EMBANKMENT SITE

The embankment site is situated at the eastern end of Sea Island, in the Fraser River delta at an elevation of about 1.5 m above sea level.

A description of the geology of the Fraser River delta is given in Blunden (1973). The region is founded on Pleistocene till sheets and Tertiary bedrock at depths of 225 m to 275 m below mean sea level. This base has experienced isostatic rebound which, combined with post-glacial sedimentation, has resulted in the emerging delta. The basic sedimentary sequence consists of about 2 m to 6 m of mixed clays, silts, and organics, underlain by up to 30 m of deltaic channel fill, predominantly sand, with interbedded silts, underlain by a stratified accumulation of marine clays and silts.

The area adjacent to the embankment site is level and lawn-covered, with a series of drainage ditches separating it from the surrounding roadways and embankments. General site stratigraphy consists of roughly 2 m of topsoil and sandy, silty clay, below which medium dense to very dense sand extends to a depth of about 20 m. Beneath this sand lies the compressible clayey silt marine delta deposit. At the site, the thickness of this normally consolidated clayey silt is 40 m to 45 m. Glacial till underlies the clay silt, from a depth of approximately 61 m.

The groundwater table was generally found to lie between 1 m and 1.5 m below ground surface, with fluctuations due to tidal influence.

EMBANKMENTS AND FIELD OBSERVATIONS

The case record of construction and instrumentation, and a comparison between predicted and observed settlements of the McConachie Way Overpass embankments and the Arthur Laing Bridge south approach embankment is given in Bertok (1987). A brief summary of the record follows.

McConachie Way Overpass Embankments

Two embankments, north and south, provide the foundation for the roadway over Grant McConachie Way. Embankments are some 73 m in length, with a base width of 39.6 m, and a height of 8.6 m at the highest point. A site plan showing the layout of the embankments is shown on Figure 1. The embankments were constructed between November, 1970 and February, 1971, from compacted sand fill. Uncompacted sand was placed as a surcharge 2 m thick at the high point of the embankment, decreasing in thickness to 0.9 m at the natural ground surface. The surcharge remained in place for 30 months, and was removed in August, 1973.

Abutments for the overpass were founded on 55 m^2 spread footings located in the compacted sand fill approximately 1 m above the original ground surface. Abutment construction took place over the six months between October, 1974 and March, 1975.

Field instrumentation included surface and deep settlement gauges and piezometers. Significant excess pore pressures were not recorded during fill construction, which was apparently indicative of piezometer malfunction.

Settlement gauges to measure surface or deep settlements consisted of wooden or steel plates attached to vertical pipes. Level surveys were taken on the tops of the pipes to record movement beneath the embankments. Field observations began in November, 1970, with embankment construction, and continued through 1973, when construction operations destroyed all but two of the gauges. Settlement of the abutments was monitored by periodic level surveys which began in November, 1974. These observations continue to the present time.

Settlement beneath the high portion of the south embankment had reached 67 cm at the commencement of abutment construction in October, 1974. Subsequent monitoring to September, 1987 indicates the north and south abutments have settled approximately 39 cm and 37 cm, respectively, for an average total settlement of 105 cm.

Arthur Laing Bridge South Approach Embankment

The south approach to the Arthur Laing Bridge is over an embankment 67 m in length, with a base width of 68 m and a height of 9.5 m at its highest point. Compacted sand fill was used for embankment construction, which took place from May to August, 1970. Uncompacted sand was placed as a surcharge 3 m thick at the highest point of the embankment, decreasing in thickness to 0.9 m at the natural ground surface. The surcharge remained in place for 34 months, and was removed in July, 1973.

Abutment construction began, in February, 1973, with the surcharge still in place, and was completed in December, 1973. The abutment was founded on a 91 m^2 spread footing located within the embankment, 4.8 m above the original ground surface.

Field observations were made on piezometers and settlement gauges distributed in a similar fashion to those in the McConachie Way Overpass embankments. The program of field observations began in May, 1970 and continued to the middle of 1973 when most settlement gauges were destroyed or damaged by abutment construction. One piezometer remained operational and was monitored through to early 1974. Monitoring of abutment settlement began in March, 1973 and is on-going to the present time.

By mid-1973, ground surface settlement had reached 100 cm beneath the high point of the embankment, due to the embankment and surcharge, however, the rate of settlement had slowed considerably. Surcharge removal and abutment loading produced little change in the settlement process. Field observations showed that high pore pressures developed in the underlying clayey silts, as a result of the placement of the embankment fill, and were slow to dissipate.

Comparison Between Predictions and Observations

Settlement predictions made in 1968-69 were based on the results of a laboratory testing program and standard geotechnical analysis. It was recognized that reliable prediction of the magnitude and rate of settlement would be difficult due to sampling and testing problems in the soft, sensitive soil. A comprehensive picture of the subsurface stratigraphy, including possible drainage effects due to sand layers, could not be fully developed from the drilling and sampling program conducted in the late 1960's.

Table I compares predicted with observed settlements to 1985. Settlement arising from the embankments were distinguished from those arising from the abutments, and, as settlement gauges were destroyed prior to the completion of abutment construction, no observed values were given in the case of the

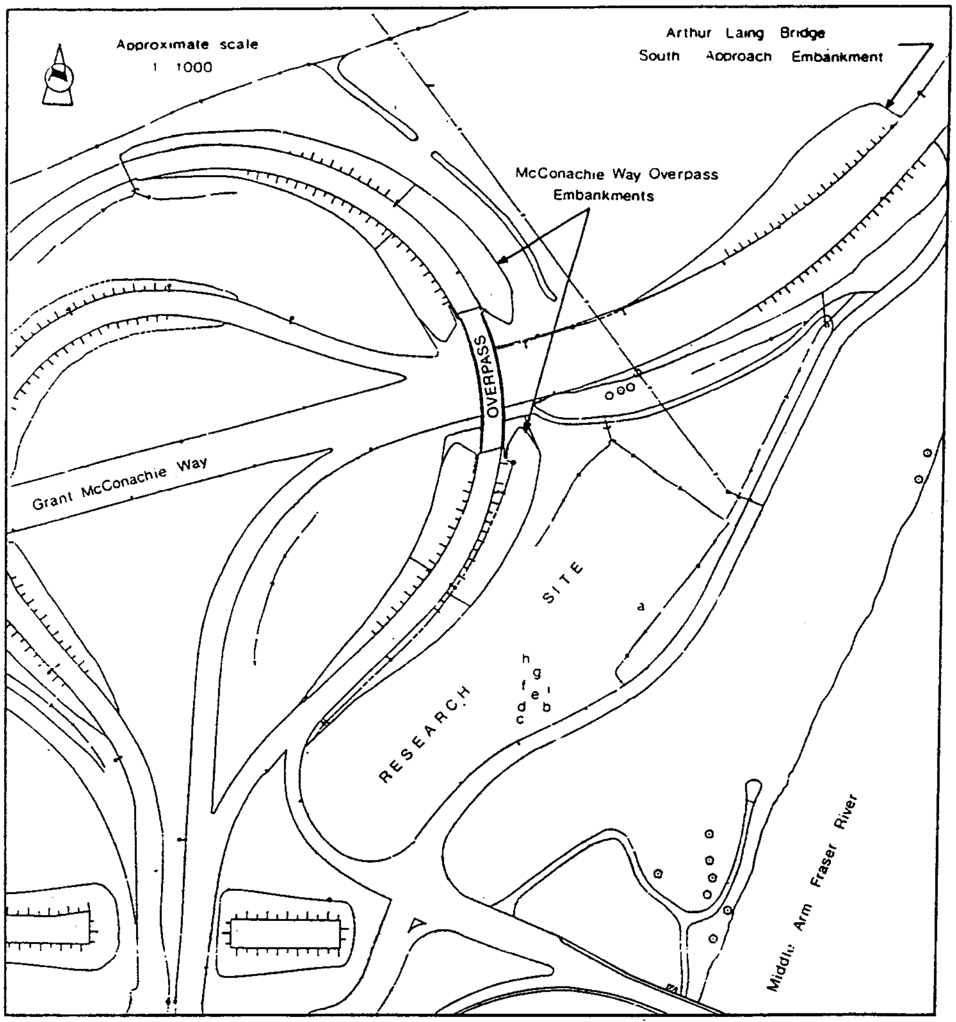

Figure 1. Embankment Site Plan (See Table II for index of test methods)

embankments. Additionally, no details were given on which to form a comparison of predicted and observed rate of settlement through the construction and post-construction period. As commented by Bertok (1987), reasonable agreement is shown between predicted and observed settlement in the case of the south abutment in the Arthur Laing Bridge approach embankment. However, appreciable difference is seen between predicted and observed settlement for the abutments of the McConachie Way Overpass. No explanation for the discrepancy was given.

Full details of the monitoring and predictions are given by Bertok (1987).

FIELD TESTING

All in-situ tests for this study were conducted from the University of British Columbia (UBC) Geotechnical Research Vehicle. Specifications and a description of the testing vehicle, is given by Campanella and Robertson (1981).

Figure 1 presents a plan of the embankment site and identifies the locations of the various in-situ tests performed as part of this study. Table II may be used in conjunction with Figure 1 for a description of the tests conducted at each location.

Piezocone Penetration Test (CPTU)

The piezocone penetration tests were performed using cones with a 10 cm^2 base area and 60° apex angle. Details on various cone designs; including the UBC and Hogentogler cones, are given in Robertson and Campanella (1986).

Pore pressure measurements were recorded at various locations along the cone both during penetration and pauses in penetration. Each cone was carefully prepared and calibrated prior to each test to ensure proper functioning in the field. Polypropylene porous filter elements, 5 mm wide, were saturated with glycerine under vacuum in the laboratory prior to piezocone penetration. In the field, cone data was automatically recorded using a computer based data acquisition system. Data was subsequently corrected for

Table I

Predicted and Observed Settlements to 1985
(modified from Bertok, 1987)

	Predicted settlement (cm)			
	Primary consolidation and elastic settlement			
Embankment or Abutment	With surcharge	Without surcharge	Secondary consolidation	Observed Settlement (cm)
South approach embankment of Arthur Laing Bridge*	165	119	10-12	100†
Approach embankments of McConachie Way Overpass**	84	67	6-8	70†
South abutment of Arthur Laing Bridge	23		2-3	20
Abutments of McConachie Way Overpass	20		5-6	33

* Embankment section 12.5 m high, including 3 m surcharge

** Embankment section 10.4 m high, including 2.1 m surcharge

† These are approximate values at the commencement of abutment construction, not ultimate settlement magnitudes.

Table II

In-Situ Testing Program

Map* Marker	Test Name	Test Type	Test Date (1987)	In-Situ Tool Used	Depth Penetrated (m)
a	CPTU-1	Piezocone penetration test	12 Aug.	UBC Cone #8	61.08
b	CPTU-5	Piezocone penetration test	11 Sept.	Hogentogler	64.38
c	CPTU-2	Piezocone penetration test	24 Sept.	Hogentogler	29.68
d	CPTU-3	Piezocone penetration test	24 Sept.	UBC Cone #8	29.7
e	DMT-1	Flat Dilatometer test	28 Sept.	Blade #89	21.0
f	DMT-3	Flat Dilatometer test	1 Oct.	Blade #89	29.8
g	DMT-2	Flat Dilatometer test	10 Oct.	Blade #89	35.6
h	SPLT-1	Screw Plate test	29 Oct.	250 cm^2 Plate	24.5
i	CPTU-6	Seismic Piezocone penetration test	22 Oct.	UBC Cone #7	54.78

* refers to location on Figure 1

pore pressure and temperature effects. A complete discussion of these procedures and their effects on geotechnical interpretation is given in Robertson and Campanella (1986).

Flat Dilatometer Test (DMT)

For the present study, a standard Marchetti dilatometer was used. A, B, and C readings were obtained for all tests, and dissipation data, C readings with time, were collected at selected depths. Full details of the DMT dissipation test are given in Robertson et al. (1988).

Screw Plate Test (SPLT)

The screw plate used in this study consisted of a single flight, helical auger having a cross-sectional area of 250 cm^2. A description of the instrument, installation and data acquisition system is given by Berzins (1983). In order to obtain soil deformation characteristics, the plate was screwed down to the desired test depth, an increasing load applied from the surface, and plate settlement recorded using a linear variable differential transducer (LVDT). The plate was then advanced to the next test depth. Tests were performed at 1 m intervals to a depth of 24 m. To obtain consolidation characteristics, plate settlement was monitored with time, under constant load conditions. These static load tests were performed at 1 m intervals from 22.5 m to 24.5 m.

GEOTECHNICAL PARAMETERS INTERPRETED FROM IN-SITU TESTS

The calculation of settlement requires the deformation and consolidation characteristics of the soil and a knowledge of site stratigraphy and groundwater conditions. For this study, these parameters were defined using only in-situ test techniques and the most recent interpretation methods available. As consolidation of the normally consolidated clayey silt was expected to be a major factor in the settlement of the embankments, particular attention was paid to the accurate definition of parameters within this stratum. Details of the analyses and interpreted parameters are given in LeClair (1988).

Soil Profile

An important feature of the cone penetration test is its detail and accuracy in stratigraphic logging. With the addition of pore pressure measurements, soil type identification becomes more precise. For the five CPTU profiles performed at the site, consistent, repeatable results were obtained and valuable details, critical to the prediction of settlement rate, were identified.

Sounding CPTU-1 was conducted approximately 35 m north of sounding CPTU-6. Corrected cone bearing values (q_t)for these two tests are shown on Figure 2, within a general geologic profile of the area. Only minor variability was evident among CPTU locations, giving confidence to the definition of site stratigraphy in the vicinity of the embankments prior to their construction.

Using the soil behaviour type classification chart developed by Robertson et al. (1986) a soil profile was interpreted for each CPTU. Good agreement was obtained between the CPTU interpreted soil profile and the boreholes conducted in the original 1968 site investigation. Both profiles identify the same basic sequence, that is, a few metres of predominantly mixed, silty soil, underlain by clean sand increasing in density with depth, underlain by sensitive clayey silt. Considerably more stratigraphic detail was obtained from the piezocone sounding (see Figure 2), as compared with the borehole. Of particular importance to this study was the identification of numerous layers, averaging 0.5 m to 0.7 m in thickness, of silty sand between the depths of approximately 26 m and 30 m. At the location of CPTU-1, penetration was refused below the 61 m depth, and this was assumed to be the top of the till sheet.

Consolidation Characteristics

Laboratory consolidation tests on field samples have traditionally been performed in order to measure properties for use in geotechnical settlement analyses. The ability to evaluate flow and consolidation characteristics from the time rate of pore pressure dissipation using the piezocone and, recently, the dilatometer, has provided an alternative approach to discerning the consolidation characteristics of fine grained soil.

A piezocone dissipation test can be conducted during a pause in penetration at any depth where consolidation characteristics are required. The decay of excess pore pressure is monitored with time and the results interpreted to give the coefficient of consolidation, c_h. Robertson et al (1988), have shown that the closing pressure (C reading) from the flat dilatometer can also provide information on the pore pressure in soft clays. Hence, dissipation tests can be performed using a standard Marchetti dilatometer by recording the closing pressure with time during a stop in penetration. When dissipation data are normalized, as suggested in Baligh and Levadoux (1986) and Robertson et al (1988), it is possible to compare CPTU and DMT data. Figure 3 shows that, at a depth of 23 m, it takes approximately eight minutes to reach 50% consolidation (t_{50} = 8 minutes) in a CPTU dissipation test using a 10 cm^2 cone, and approximately twice as long, (t_{50} = 16 minutes) in a DMT dissipation test.

Values of c_h, were determined by curve-fitting about t_{50}, using the method recommended by Torstensson (1977) and the dissipation test data from both the piezocone and dilatometer. Local research (Gillespie, 1981) has shown that consolidation around penetrometers in the clayey silt underlying the Fraser Delta is controlled by horizontal drainage. Figure 4 presents a summary of the c_h values determined from CPTU and DMT alongside laboratory-determined values. Points representing the average values obtained from the 1968 site investigation are also shown. Laboratory-determined c_v values and in-situ-determined c_h values show close agreement. The majority of values fall within a narrow band averaging 4 x 10^{-3} cm^2/s. Potential drainage layers of sandy silt, where c_h is considerably greater, are clearly identified by the piezocone dissipation tests.

Deformation Characteristics

The one-dimensional constrained modulus (M) and Young's modulus (E) were evaluated from the CPTU, DMT and SPLT data. The dominant parameter in the calculation was M for the compressible clayey silt deposit. Based on a comparison of DMT derived values of M and CPT penetration resistance values (q_t), the following relationship was obtained for the marine clayey silt deposit,

$$M = \alpha q_t \qquad (1)$$

where: $\alpha = 2.37 \pm 1.08$

This compares well with the recommended range of α = 1 to 3 given by Mitchell and Gardner (1975) for silts of low plasticity where q_t is less than 2 MPa. Since the DMT was carried out only to a maximum depth of 35.6 m, this correlation was used to obtain a profile of M throughout the clayey silt deposit. Full details of the interpretation are given in LeClair (1988).

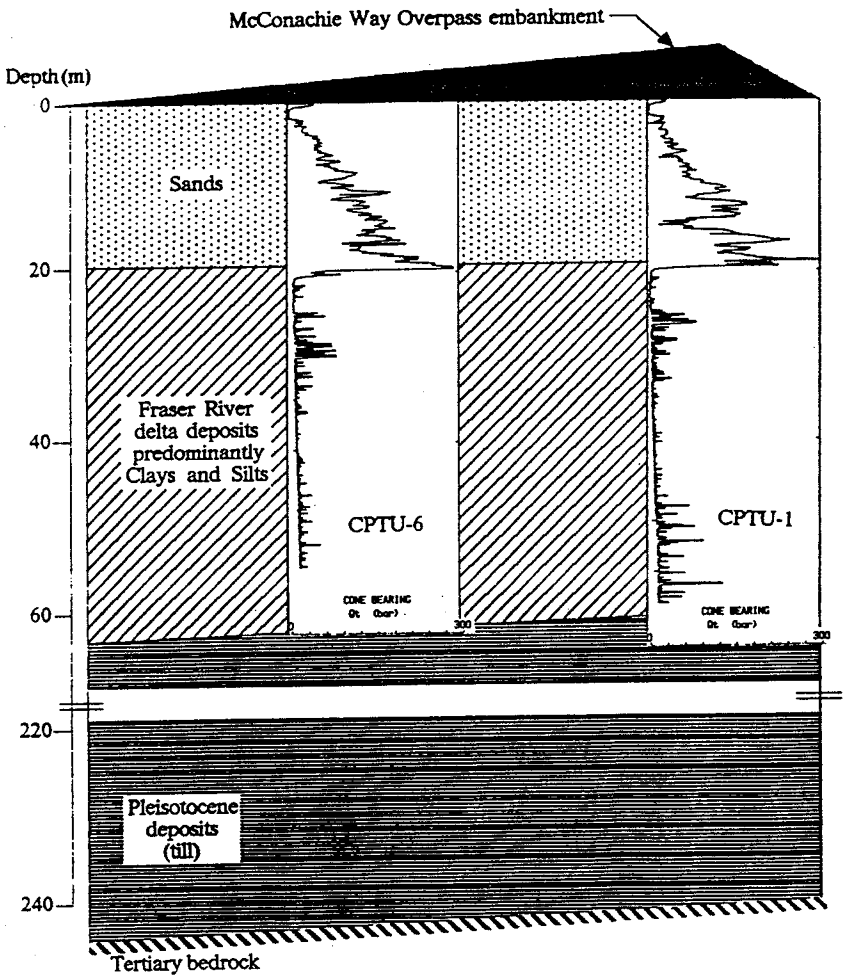

Figure 2. General Site Geology and Cone Bearing Profiles

PREDICTION OF PERFORMANCE

Embankment performance, in terms of rate and magnitude of settlement, was predicted using the results of CPTU, DMT and SPLT. For the predictions, no new methodology was introduced. Analyses were performed using current, accepted practice.

Simplified Approach: One-Dimensional Analysis

With, perhaps, the exception of high risk structures, most settlement calculations are based on one-dimensional analyses involving the estimation of vertical displacements induced by a design load. The total settlement is calculated as the sum of three components: elastic distortion, consolidation, and secondary compression. In order to evaluate each of these components, it is necessary to quantify the load applied, the resulting increase in stress, the distribution, with depth, of this stress increase, as well as the relevant soil properties. This process formed the basis of the simplified approach used in evaluating embankment settlements.

Stress Increase

To evaluate the distribution of stresses within a soil mass, the theory of elasticity is invariably used. Although soil is a non-linear material, and inherently anisotropic, the assumption is often made that the soil is isotropic elastic. Rigorous solutions for more complex non-linear constitutive relationships are only possible in very few cases, and for most applications, the use of elastic theory results in an acceptable degree of accuracy for the evaluation of changes in vertical stress distribution. The values of vertical stress increase at the ground surface, from each component of embankment loading, are identified in Table III. These values formed the basis for computing the distribution of stress increase with depth.

The solution for the distribution of stresses within a semi-infinite, homogeneous, isotropic mass, with a linear stress-strain relationship, due to a point load on the surface, is first credited to Boussinesq (1885). Because the solution is a linear function of applied load, the principle of superposition may be applied to account for variations in loading conditions. In an

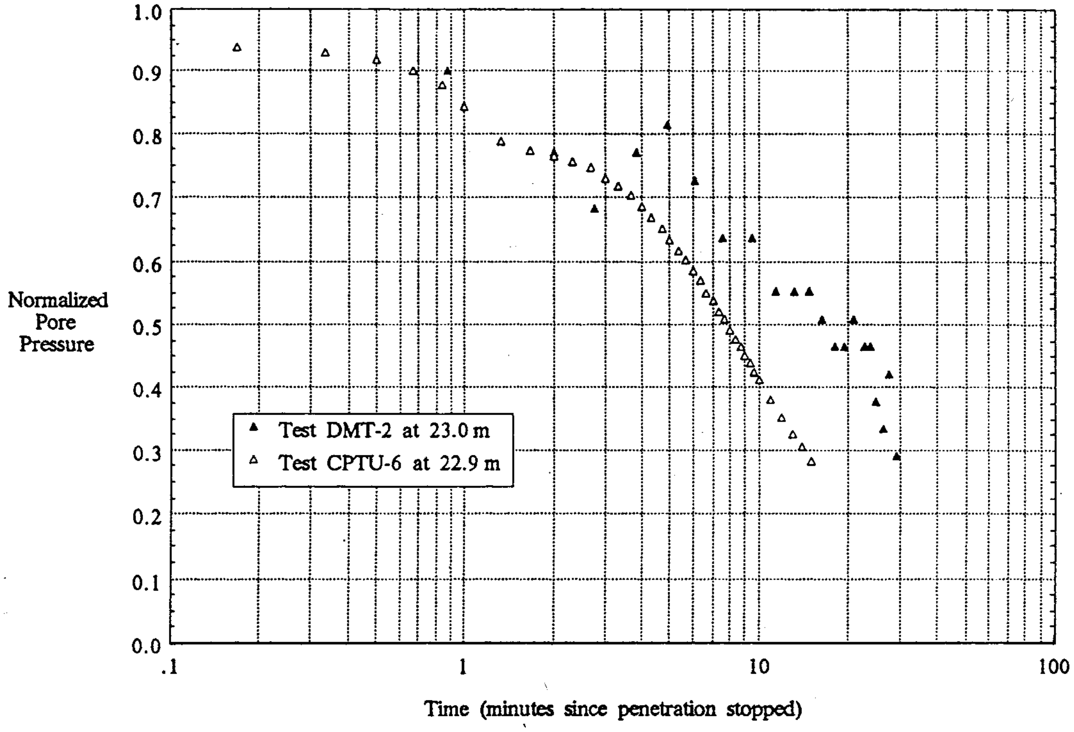

Figure 3. Example of Piezocone and Dilatometer Dissipation Record.

Table III

Stress Increase at Original Ground Surface Induced by Placement of Embankments, Surcharge and Abutments

Location	Stress Increase (kPa) due to:		
	Embankment	Surcharge	Abutment
Arthur Laing Bridge south approach	173	50	44
McConachie Way Overpass	151	35	106

elastic analysis, an embankment is considered to be an infinite strip area carrying a combination of uniform pressure and linearly increasing pressure. The calculated distribution of stresses using this assumption shall be referred to as the "normal loading approximation".

Perloff (1975) comments that the above approach neglects the shear stresses which develop between an embankment and its foundation, and proposed an alternate approach (Perloff et al., 1967), which considers the embankment and foundation as a single body loaded only by self weight. This is called the "elastic embankment" approach and may be more realistic because it considers the effect of the material itself on the distribution of stress, allows for shear distortions at the embankment-foundation interface, and produces a result found to be consistent with field measurements of pore pressures beneath an embankment (Bozozuk and Leonards, 1972).

Amount of Settlement

To determine the consolidation settlement due to embankment load and surcharge load for the embankments, the 61 m thick soil profile was divided into 56 layers. A vertical stress increase was determined at the centre of each layer using both the "normal loading approximation" and the "elastic embankment" methods.

In-situ tests for this study were performed at sufficient distance from the embankments to represent initial conditions prior to construction. The values of constrained modulus (M) determined from those tests are valid, therefore, for the initial loading conditions. However, once the foundation soils had been preloaded the soil would possess different properties, as the subsequent removal of preload would leave the soil with a stress history. The modulus values used to compute settlement due to embankment and surcharge, therefore, required some modification for the calculation of settlement due to abutment loading. The method for computing foundation settlement, from DMT results, which accounts for the variation in constrained modulus with varying stress level proposed by Schmertmann (1986) was used to account for the effects of preloading.

Table IV presents the results of the first step of this one-dimensional settlement analysis, showing the magnitude of distortion and ultimate consolidation settlement, beneath the embankment centreline, predicted due to embankment, surcharge, and abutment construction. Observed and predicted

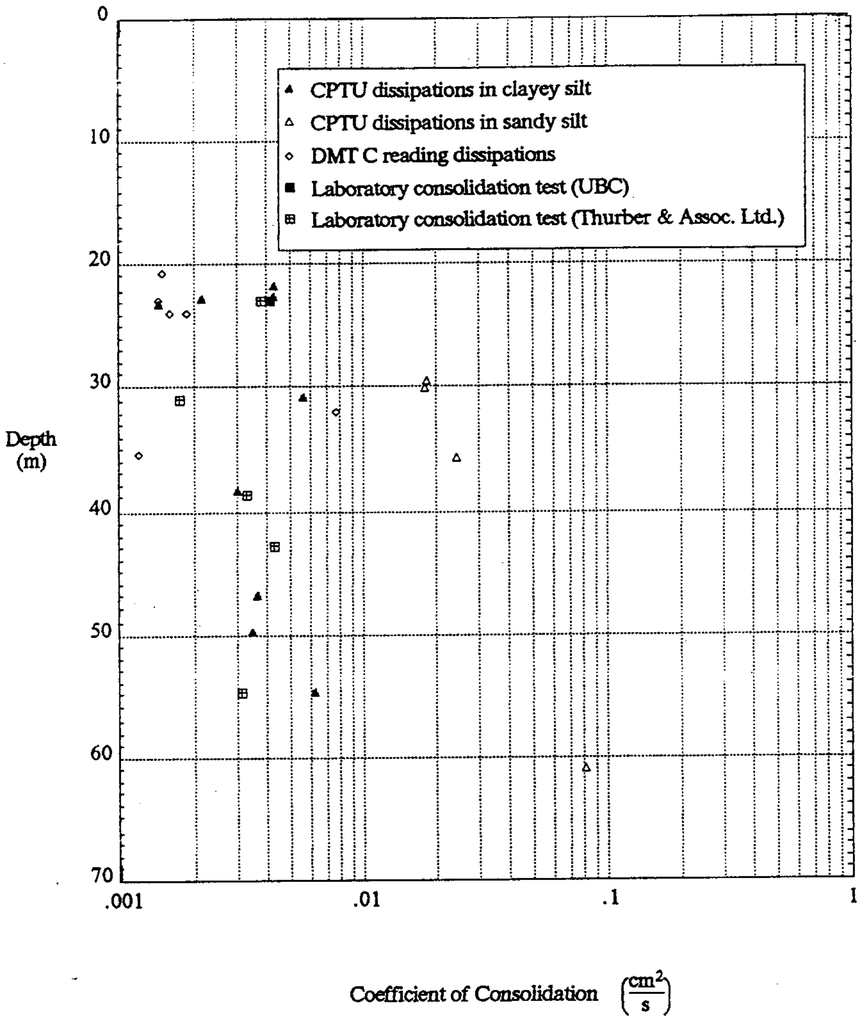

Figure 4. Profile of Coefficient of Consolidation with Depth

settlements given in Bertok (1987) are provided for comparison. Settlements predicted using in-situ test parameters appear somewhat closer to the observed settlements than those predicted from laboratory test results.

Rate of Settlement

A potentially large source of error in predicting time rate of settlement is the definition of drainage boundary conditions. For predicting rate of settlement at the embankment site, the compressible clayey silt deposit was divided into two strata, separated by a more freely draining sandy silt layer at a depth of about 28 m. A drainage layer at a depth of 28 m was assumed based on the stratigraphy defined from the CPTU data. Profiles of cone penetration resistance 30 to 40 m apart at the embankment site gave evidence of considerable sandy silt layering surrounding the 30 m depth, therefore the areal extent of this more freely draining layer was judged sufficient for it to act as an effective drainage path. Furthermore, horizontal coefficients of consolidation interpreted from CPTU dissipation tests at this depth were approximately one order of magnitude greater than those in the remainder of the clay silt, again indicating preferential drainage. Each of the two clayey silt strata was assumed to have double drainage. While dense glacial till would often be assumed to act as an impermeable boundary, rapid dissipation of pore pressures were observed in a CPTU dissipation test at a depth of 60.9 m, leading to the conclusion that drainage would take place at the interface of the clay silt and till. Local experience has shown that considerable weathering generally exists at the surface of the till, again supporting the idea of drainage at the base of the clay silt.

The drainage layer at the 28 m depth was assumed to be of insufficient thickness to contribute to consolidation settlement, merely acting as a drainage interface. It was assumed that consolidation settlement in the overlying sands took place almost immediately with each new construction loading phase.

Table IV

Comparison of Predicted and Observed Settlements

	Predicted primary consolidation and distortion settlement (cm)			
		Methods Using In-situ Test Parameters		
Embankment or Abutment	Reported by Bertok (1987)	Normal Loading	Elastic Embankment	Observed settlement (cm)
South approach embankment of Arthur Laing Bridge*	165	124	128	100†
Approach embankments of McConachie Way Overpass***	84	101	78	70†
South abutment of Arthur Laing Bridge	23	17	20	20
Abutments of McConachie Way Overpass	20	35	28	33

* Embankment section 12.5 m high, including 3 m surcharge

** Embankment section 10.4 m high, including 2.1 m surcharge

† These are approximate values at the commencement of abutment construction, not ultimate settlement magnitudes.

Correction for Construction Period

The empirical method proposed by Terzaghi (1943), whereby it is assumed the net foundation load is applied at a uniform rate during the construction period and that the degree of consolidation at the end of this time is the same as if the load had been acting for half that time was used to correct for settlements occurring during construction.

Secondary Compression

For the embankment site, the calculated time for completion of primary consolidation is approximately 35 years. Although it is generally assumed that secondary consolidation will occur after this time, it is likely that some secondary compression has occurred at the embankment site. In the present analysis, however, the component of settlement due to secondary compression has not been included.

Modified Approach

Where the thickness of a compressible stratum is large relative to the loaded area, the three-dimensional nature of the problem influences the magnitude and rate of settlement. Semi-empirical approaches are often used to modify settlement magnitude to account for these effects. Skempton and Bjerrum (1957) give an expression whereby consolidation settlement beneath the centreline, incorporating three-dimensional effects, can be expressed in terms of the settlement predicted from a one-dimensional test. For normally consolidated soils, however, no correction is required. It should be noted that some account was made for foundation size with respect to the thickness of the substrata by correction factors used in the one-dimensional approach for calculating distortion settlement.

Comparison of Predictions with Observed Settlement

Figure's 5 and 6 present the combined results of the one-dimensional settlement analyses, comparing the time rate of settlement predicted using in-situ test parameters with the observed settlements recorded by Transport Canada for the embankments.

McConachie Way Overpass Embankments

Both rate and magnitude of settlements were predicted with a high degree of accuracy by the "elastic embankment" method, when compared to the observed settlement, until the time of abutment loading in 1975. Following construction of the abutments, settlements predicted using in-situ test parameters were greater than those observed. The rate of settlement, however, continued to closely model the actual rate. The "normal loading approximation", overpredicts observed settlement from the outset of construction, however this result may not be unexpected, as Boussinesq solutions, while widely used, are generally found to be conservative.

At the time of the most recent Transport Canada survey (December, 1986), total settlement had reached 106 cm. Using in-situ test parameters, the settlement predicted by the "elastic embankment" method was 116 cm, an overprediction of 10 cm (9%), and the settlement predicted by the "normal loading approximation" was 143 cm, an overprediction of 37 cm (35%). This represents a significant improvement over the original predictions given by Bertok (1987), based on laboratory data, where the predicted settlement due to abutment construction alone was 20 cm compared to the 33 cm observed, a difference of 13 cm (40%).

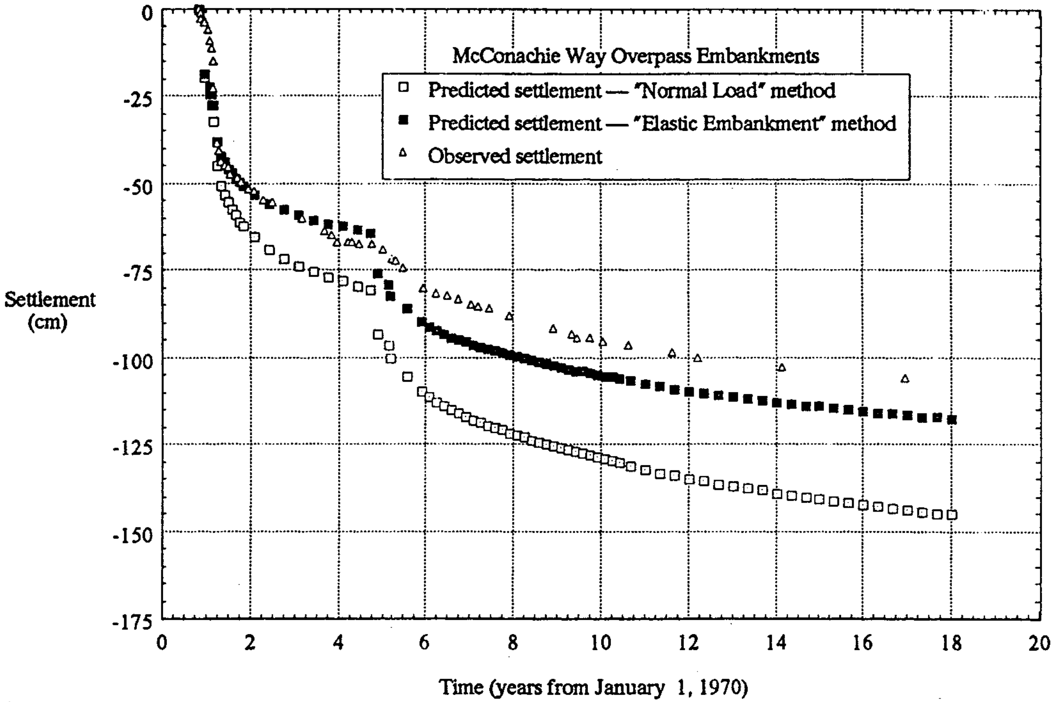

Figure 5. Predicted and Observed Settlements - McConachie Way Overpass Embankments

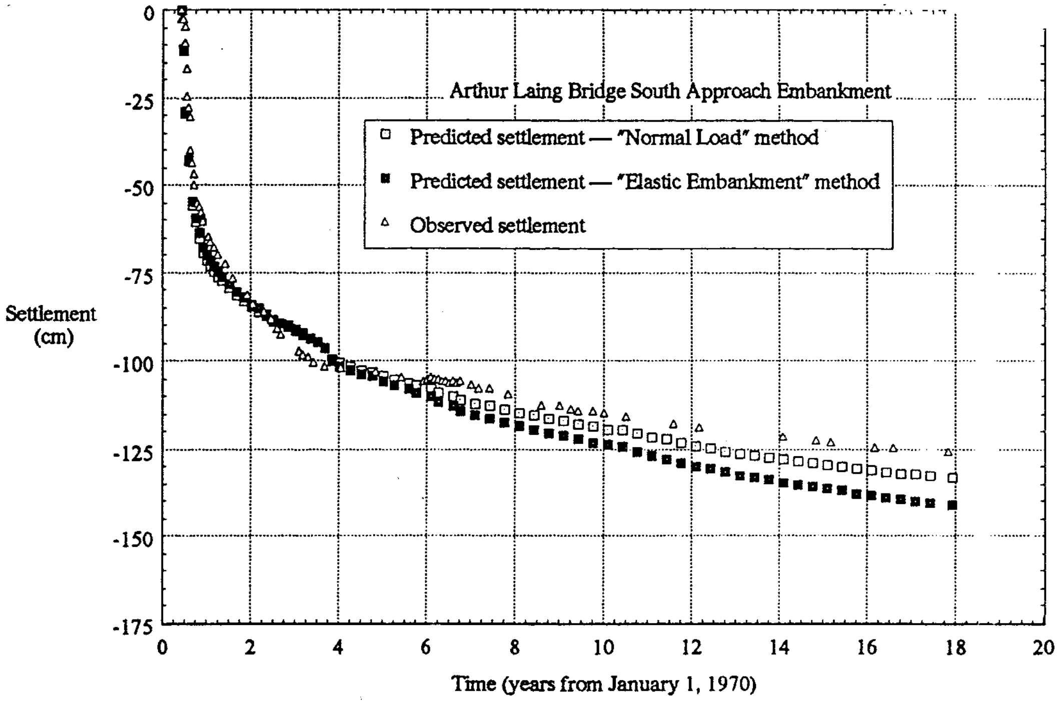

Figure 6. Predicted and Observed Settlements - Arthur Laing Bridge South Approach Embankment

Arthur Laing Bridge South Approach Embankment

Rate and magnitude of settlements as predicted using in-situ test parameters match observed settlements closely. Figure 6 indicates that throughout the construction, preloading, and abutment construction phases, predicted settlements are very similar to the observed settlements. Following abutment construction, settlement is slightly overpredicted by in-situ test derived parameters. The predicted rate of settlement, however, continued to closely model the actual rate through to 1987.

Differing from the McConachie Way Overpass embankments, the settlements predicted by the "normal loading approximation" matched the observed settlements more closely than that predicted by the "elastic embankment" method, although the difference is small. This may result from the assumption inherent in the Boussinesq analysis, that the loaded area is an infinite strip. The size and shape of the south approach embankment is closer to the assumed infinite strip than the embankments for the McConachie Way Overpass.

At the time of the most recent Transport Canada survey (November, 1987), total settlement had reached 126 cm. Using in-situ test parameters, the settlement predicted by the "elastic embankment" method was 143 cm, an overprediction of 17 cm (13%), and the settlement predicted by the "normal loading approximation" was 135 cm, an overprediction of 9 cm (7%). Again, this represents some improvement over the original predictions, where the predicted settlement due to abutment construction alone was 23 cm compared to the 20 cm observed, a difference of 3 cm (15%).

In general, the "elastic embankment" method proposed by Perloff et al. (1967) provided better predictions of settlement. However, this case record is not sufficient to state which is the better approach, since both the "elastic embankment" and the "normal loading approximation" methods generated reasonable results.

DISCUSSION

The present analysis cannot be considered as a Class A prediction, since actual settlement data have been previously published. However, it should be noted that the settlement records were not back analysed in an effort to refine estimated parameters. Lack of sufficient monitoring data, such as pore pressure and surface and deep settlement measurements prohibited the back-analysis of geotechnical parameters. The study would have been improved if more detailed monitoring data were available, and if it were possible to determine whether the good prediction of settlements using in-situ test data was due to counterbalancing errors.

Nevertheless, the results of this study have a twofold significance. Firstly, it has been shown that, for this embankment case history, settlement magnitudes can be predicted with reasonable confidence based on parameters interpreted from in-situ tests. Also, it has been shown that the detailed stratigraphic information gathered using in-situ profiling tests provides a solid basis for accurate prediction of the rate of settlement by increased precision in the identification of potential drainage layers within the soil profile. Secondly, it has been demonstrated, for this embankment case history, that a simple, one-dimensional analysis can adequately predict settlements.

For the south approach embankment of the Arthur Laing Bridge, predicted performance paralleled the observed performance with a degree of accurarcy not often found in the prediction of settlement for large structures founded on compressible soils. For the McConachie Way Overpass embankment, performance predicted by in-situ test methods proved to be an improvement over that predicted by conventional methods, however, the predictions did not parallel observations as closely as in the case of the Arthur Laing Bridge south approach embankment. The original findings outlined in Bertok (1987) also were indicative of poorer performance predictions for the McConachie Way Overpass embankments.

Bertok (1987) stated that reliable prediction of the rate of settlement to be expected at this site was very difficult, adding that the reliability of predictions was tenuous due to sampling and testing problems in the soft, cohesive soil, and to lack of understanding of drainage effects from only one deep test hole. By contrast, the simplicity and versatility of in-situ tests such as the CPTU and DMT are clear advantages. These in-situ tests provided excellent stratigraphic detail and definition of potential drainage seams, enabling rational decisions to be made on site geometry and drainage. As a result, when combined with pore pressure dissipation test information, rate of settlement was predicted with surprising precision.

In relating soil properties determined in the laboratory with those determined from field measurements, Olson (1985) comments that the ultimate check on the usefulness of laboratory data is a comparison between predictions and field measurements. The same may be said for soil properties determined from in-situ tests. The ultimate check, in the case of the present performance prediction, provided a favourable endorsement for using in-situ tests to predict both rate and magnitude of settlement.

From this analysis, in-situ testing emerges as a viable alternative to the traditional approach of obtaining geotechnical parameters required in the prediction of settlement. While interpretation of in-situ test data is, by and large, empirical in nature, the large amount and diversity of the data obtained enables the engineer to obtain a better sense of site conditions and variability, leading to a generally more reliable geotechnical solution.

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