Embankment construction of a very recent tidal deposit

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ABSTRACT
The old narrow Gunningsville Bridge across the Petitcodiac River, connecting Moncton and Riverview, New Brunswick, and built in 1915 has been replaced with a four-lane bridge, the Petitcodiac River Bridge No.1, located 80 metres further upstream. The approach embankment fills are up to about 7.5 m thick. The foundation soil consists of a silt of thickness up to about 7.4 metres. The silt is underlain by relatively competent soils. The silt was laid down recently in nearly horizontal laminae varying in thickness between about 3 and 6 mm, each lamina representing the deposit left after one tidal cycle. The laminae consist of fine silt separated by thin veneers of coarse silt. The settlement in the silt was accelerated by placing most of the embankment during the first construction season. The behaviour of the silt was monitored during fill placement through settlement measurements and piezometer readings.

RÉSUMÉ
Le vieux pont étroit de Gunningsville traversant la rivière Petitcodiac pour joindre Moncton à Riverview, NB qui à été construit en 1915 a été remplacé avec un pont à quatre voies (le Pont No. 1 de la rivière Petitcodiac). Celui-ci est situé 80 m en amont. Les terres de remplissages de la levée de terre pour l'approche sont d'une épaisseur maximale de 7.5 m. Les sols de la foundation sont constitués de limon d'une épaisseur approximative de 7.4 m. Il y des sols relativement compétents dessous le limon. Le limon a récemment été mis en place en lames horizontales d'une épaisseur de 3 à 6 mm. Chaque lame représente un dépôt placé après un cycle des marées. Les lames sont constituées de limon fin séparé par des couches minces de limon à gros grains. L'affaissement dans le limon a été accéléré par la construction de la majorité de la levée de terre pendant la première saison de construction. Le comportement du limon a été surveillé pendant le placement de la terre de remplissage en prenant des mesures d'affaissement et des mesures de piézomètres.

1. INTRODUCTION

The determination of the rate of settlement expected under an embankment is usually based on instant loading, i.e. it is assumed that the embankment load is applied instantaneously, even if the fill is placed over a period of several months or longer.

The construction of the approach embankments to the new Petitcodiac River Bridge connecting Moncton and Riverview, New Brunswick, afforded an opportunity - through settlement and porewater pressure measurements - to compare the rates of settlement due to instant (theoretical) and gradual application of the load.

The Petitcodiac approach embankments are underlain by a somewhat unusual foundation soil, viz. a loose laminated silt of thickness up to 7.4 m. The silt, which is underlain by relatively competent soils, is a tidal deposit that has been laid down relatively recently and consists of nearly horizontal laminae varying in thickness between about 3 and 5 mm. The laminae consist of fine silt separated by thin veneers of coarse silt. Each lamina represents the deposit left after one tidal cycle.

A longitudinal section of the embankment on the Moncton side is shown in Figure 1. Since the abutments are founded on piles to rock, it was important to determine the magnitude and rate of settlement of the approach embankments. The maximum thickness of the new embankment fill was about 5 metres.

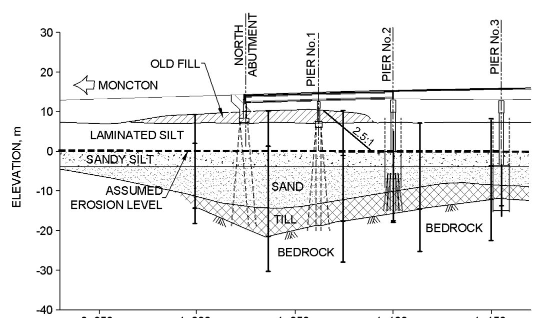

Figure 1. Longitudinal section along centerline at north abutment

2. LAMINATED SILT CHARACTERISTICS

Figure 2 shows typical 50 mm diameter silt samples after drying and after hand separation of some of the laminae. Typical geotechnical properties are given in Table 1.

(a)

(b)

Figure 2. Typical tidal silt deposit, Petitcodiac River. (a) After drying initial samples. Laminae are 3-5 mm thick fine silt, separated by thin veneers of coarse silt. (b) Hand separation of some of the laminae.

Table 1 - Typical geotechnical properties

Initial moisture content w;	28.3 - 43.6%
Initial void ratio	1.17
Specific gravity of solid portion of silt particles G_s	2.72
Initial density P;	1718 - 1874 kg/m^3
Coefficient of hydraulic conductivity k	1.7 x 10^{-7} cm/sec
Compression index = 0.25 / (1 + 1.17)	0.12
Coefficient of consolidation (laboratory values)	13 - 38 m^2/yr
Cohesion intercept c'	3 - 10 kPa
Friction angle φ'	29° - 31°

Figure 3 shows the relationship between initial moisture content w_i and initial saturated density ρ_i for different values of specific gravity G_s, as compared with measured values of w_i and ρ_i. One reason for the considerable spread in the measured values may be different values of the specific gravity, the latter being a result of the variable content of microfossils (diatoms) in the silt, as observed through scanning electron microscopy. The diatoms are hollow or perforated particles of silica (G_s = 2.66). The G_s values plotted in Figure 3 represent silt particles containing no voids (G_s = 2.7) to particles containing 10% voids (G_s = 2.3). It is noted that values of the specific gravity routinely determined in the laboratory are based on crushed samples, i.e. any crushed hollow or perforated particles will yield the specific gravity of the solid portion of the particles rather than the average specific gravity of the hollow or perforated particle.

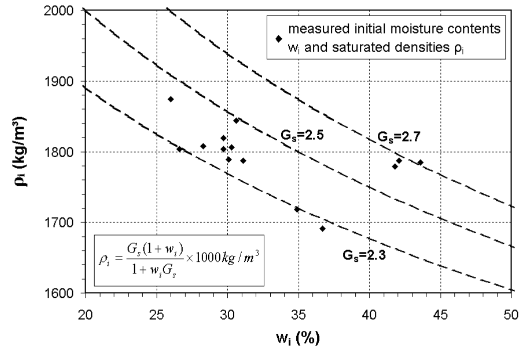

Figure 3. Relationship between w_i and ρ_i for different values of specific gravity G_s

The presence of organics also contributes to a lowering of the specific gravity. Unfortunately, the organic content was not determined for the laminated silt. However, typically the organic content of harbour mud deposits in New Brunswick has been found to be in the order of 8-13%. If it is assumed that the organic content of the laminated silt is about 5%, this would lower the specific gravity from 2.70 to 2.60.

The shear strength parameters were based on undrained triaxial tests on undisturbed fibre-glass tube samples, which yielded parameters in the ranges c' = 5-10 kPa and φ' = 30.3º-31.4º. Also, a review of the results of piezocone tests at the *Brage* and *Stjørdal* sites in the North Sea (Senneset et al. 1988) and the *Miramichi* lagoon site (Landva et al. 2006) yielded parameters of c' = 3-5 kPa and φ' = 29º-31º. Finally, the parameters were based on a study of the Gunningsville Bridge Channel Monitoring sections, for which the steepest eroded gradient was determined to be 2.5H:1V. A back calculation of the stability of such a slope gave parameters of c' = 3-7 kPa and φ' = 30º.

3. SETTLEMENT ANALYSIS

The analysis of the expected consolidation settlement was based on the compression of the 7.4 m of laminated silt, assuming a saturated unit weight of 17.5 kN/m^3 and a ground water level at ground level:

$$S_{\max} = 0.12 \times 7.4 \times \log\frac{(3.7\times7.7)+(5\times21)}{3.7\times7.7} = 0.60m \quad [1]$$

This calculation is based on the higher moisture contents determined and represents therefore a maximum value of the expected settlement. Nevertheless, it was decided to place as much of the embankment as possible during the first construction season. The behaviour of the silt would be monitored during fill placement through settlement measurements and piezometer readings, so that the stability of the embankment would be ensured at all stages.

The time to reach 90% consolidation was calculated to be 3 to 9 months on the basis of (i) a coefficient of consolidation c_v of 13-38 m^2/year, (ii) double drainage (i.e. a maximum drainage length of 3.4 m), and (iii) , as is usually assumed, an instant application of the full load. Since the time elapsed from the start of load application is proportional to the square of the drainage length, the question arose whether the coarse silt laminae were continuous under and beyond the embankment If continuous, the actual drainage length could be just a very small fraction of the thickness of the laminated silt deposit. In such a case, the consolidation would occur practically instantaneously.

4. EMBANKMENT STABILITY

Because of the unusual character of this geologically very recent silt material, it was decided to use the lower range of shear strength parameters in the stability analysis, i.e. c' = 3 kPa and φ' = 29º. It was thus determined that the proposed side slopes of 2H:1V were too steep and that the slopes should not be steeper than 3H:1V.

Considering the laminated nature of the silt, it was tentatively reasoned that the porewater pressures set up under the weight of the embankment might be transmitted via the coarse silt laminae into the adjacent unloaded areas beyond the embankment toes and possibly give rise to instability there by heaving, in a way similar to hydraulic fracturing adjacent to embankments on peat. Piezometers were therefore installed outside as well as inside the toes.

5. CONSTRUCTION BEHAVIOUR

5.1 Observed Behaviour

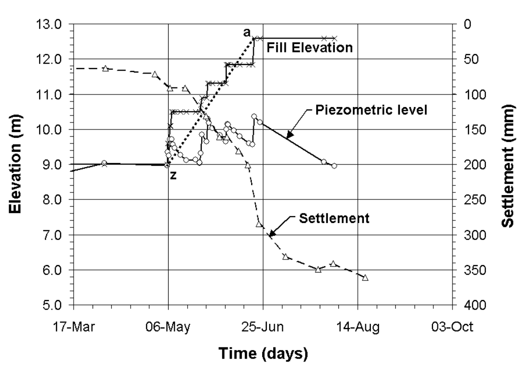

Figure 4a. Observed settlement and porewater pressure at sta.0+900m

Figure 4a shows the observed settlement and piezometric level at a representative station along the embankment (sta. 0+900). Of interest is the observation that the time to complete the period of consolidation is in the order of 5 months, showing that the silt laminae are not continuous and in fact indicating that the drainage paths may simply be vertical as if the laminated silt were a uniform deposit.

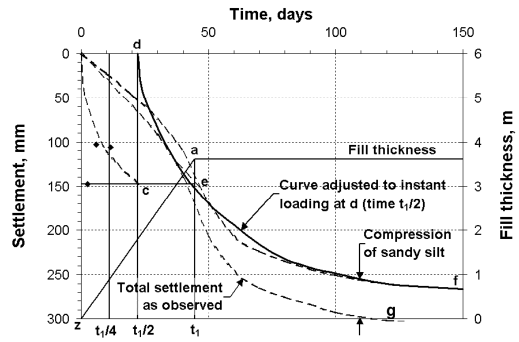

Figure 4b. Settlement of Petitcodiac embankment at sta.0+900m (*0f* = compression of laminated silt; *de* = *0c* displaced by $t_1/2$ from origin).

The lower complete settlement curve in Figure 4b (*0g*) is the same as the settlement curve in Figure 4a except that it is a smoothed version. The upper complete curve (*0f*) is the net settlement of the silt, arrived at by deducting the instantaneous compression (40 mm) of the sandy silt underlying the laminated silt. The line *za* showing the gradual application of the embankment load is the same as the line *za* in Figure 4a.

5.2 Correction for Construction Period Settlement

Figure 5 shows a method of correcting for the consolidation occurring during gradual load application as devised by Terzaghi and Frølich (1936) and as described in some detail by Terzaghi (1943, article 104).

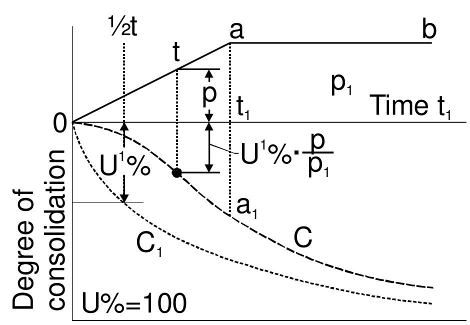

Figure 5. Correction for gradual loading (Terzaghi 1943, figure 86)

Terzaghi's (1943) method of correction is based on the assumption (Terzaghi and Frølich 1936) that the state of consolidation at time *t* is the same as if the pressure *p* had acted on the clay during a time period *t/2*. The degree of consolidation at time *t* would thus be equal to

$$U\% = U'\%p/p_1 \qquad [2]$$

in percent of the final settlement due to consolidation under the full load p_1. *U'* is the degree of consolidation at time *t*/2 after sudden application of a pressure *p*.

Beyond the time represented by the point *a* in Figs. 4a and 4b, corresponding to the time of loading t_1, consolidation proceeds as if the final pressure p_1 had been suddenly applied at a time $t_1/2$ (curve *de*, Figure 4b).

Since c_v is inversely proportional to t (time), Terzaghi's corrected curve (marked *C* in Figure 5) will affect the value of c_v significantly, whereas the theoretical curve C_1 is based on a constant value of c_v. For example, if a time t_1 of 250 days is assigned to point *a* and the drainage length is assumed to be 2.0 m, the value of c_v for curve C_1 will be found to be in the relatively narrow range 2.2-2.4 m^2/yr, i.e. practically constant. On the other hand, the values for curve *C* will be found to vary between about 0.4 and 1.7 m^2/yr. These ranges are plotted in Figure 6 (right scale) as a function of the degree of consolidation U%. It would thus seem that Terzaghi (1943) may have ignored the effect of his correction on the value of c_v. In any case, basing the calculation of c_v on the actual field curve representing settlement during the construction period could lead to a significant underestimate of c_v and also to an unrealistic range of variation of c_v.

Considering now that observed settlements during the construction period correspond to Terzaghi's curve *C*, his method of accounting for gradual load application can simply be reversed to produce the theoretical curve C_1 corresponding to the sudden application of the full load p_1 at a time t = 0. The curve *0c* in Figure 4b was produced by this reverse procedure, i.e from the upper curve *0e* to curve *0c.* The calculated values of c_v on the basis of the observed settlement curve *0f* varied between about 10 and 45 m^2/yr, as plotted in Figure 6 (left scale).

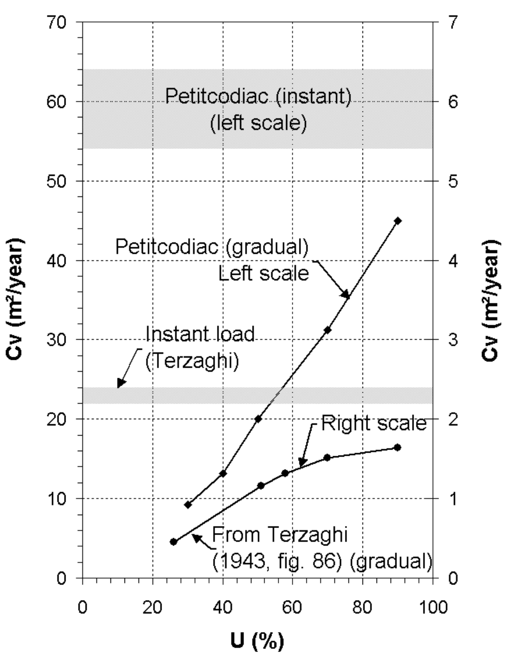

Figure 6. Comparisons of c_v values based on instant and gradual loading for Petitcodiac and for numerical example of Figure 5 (the Terzaghi gradual curve is based on t_1=250 days, 2H=6.8m)

The adjusted curve in Figure 4b is based on a combination of curves *0c*, *de* and *0f*. As shown in Figure 6, the corresponding values of c_v are in the range 54 to 64 m^2/yr. However, it must be pointed out that this is at least partly a result of some curve-fitting, because the procedure of correcting for gradual loading by reversing Terzaghi's method is quite sensitive to time, especially in the early stages when the rate of settlement is high.

It is noted that the ratio of field and laboratory values is in the range 2.5 - 2.7, i.e. $c_{vfield} \approx 2.5 - 2.7 c_{vlab.}$. This is in agreement with a previous comprehensive comparison of field and laboratory behaviour carried out by the second author for a series of case records (about forty) of rates of field and laboratory consolidation of different clays in various countries.

6. CONCLUSIONS

A new bridge has been constructed across the Petitcodiac River, connecting Moncton and Riverview, NB. The approach embankment fills are up to about 7.5 m thick. The foundation soil consists of a silt of thickness up to about 7.4 m. The silt is underlain by relatively competent soils.

The silt was laid down recently in nearly horizontal laminae varying in thickness between about 3 and 6 mm, each lamina representing the deposit left after one tidal cycle. The laminae consist of fine silt separated by thin veneers of coarse silt.

The settlement in the silt was accelerated by placing most of the embankment during the first construction season. The behaviour of the silt was monitored during fill placement through settlement measurements and piezometer readings.

The settlement observations showed that the laminated silt behaved more or less as one uniform layer of 7.4 m thickness (double drainage) , showing that the coarse and fine silt laminae were not continuous.

The determination of the rate of settlement expected, e.g. the time required to reach 90% consolidation, is usually based on instant loading, i.e. it is usually assumed that the load from the structure is applied instantly. This seems to be the case even if the structure is placed over a period of several months or even longer.

It is possible to estimate the rate of settlement for gradual application by a simplified method suggested by Terzaghi. However, this method leads to an underestimate of c_v. We have concluded that this can be dealt with by reversing Terzaghi's method; that is, by converting the observed rate of settlement curve to one corresponding to instant loading. The resulting value of c_v is then the correct value.

The corrected value of c_v for the Petitcodiac soil is about three times the value determined from consolidation tests, which is in agreement with numerous case records previously analyzed by the second author.

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