Vancouver Geotechnical Society

Symposium on Soft Ground Engineering:

Keynote lecture, June 8, 2012: Peats and Creepy Soils

List of figures (PowerPoint slides)

2-3. Determination of type (nature) of soil (e.g. inorganic, organic,peaty, creepy). This replaces attempts to classify the soil into, for example, fine-grained soils, organic, peaty organic, peaty siils peats on the basis of organic content. This approach seems more meaningful and is much less dependent on special experience and training.

4. von Post #, O_c and mineral content vs. w

- A better term than "ash" content is mineral content (since "ash" is just the residue after burning)

5. Escuminac borelog

- No obvious relationship seemed to exist between q_c (300 mm dia. cone) for the depth range 0 – 4 m, but below 4 m depth, q_c increased with decreasing w and increasing content of *Carex,* fine and coarse fibres and wood and shrub remnants. The same applies to plate load tests, under which a conical wedge developed, similar in shape to a cone. Note, however, that the mode of failure of a cone or a plate in peat bears no relationship to the modes of failure of structures on peat, except perhaps that under local loads such as wheel loads.

6. Väsby borelog

- Note the very high consistency limits, particularly w_L. Note also the surprisingly low organic content O_c.

7. Miramichi Channel borelog

- Very high water content but G_s equal to that of a mineral soil. Reason: very high content of diatoms, which are hollow silica shells filled with water. Relatively rapid consolidation to be expected – and no or little creep.

8. Hall's Creek borelog

- High Δu after 3 years because of creep and hence self-generated pore pressure, i.e. field u does not reflect degree of consolidation U. F = 2.5 based on vane strength. F = 1 for shear strength close to remoulded and to unconfined compressive strength (obviously highly disturbed). Almost certain that the failure was progressive.

9. Valley of Mexico borelog

- High to extremely high w_P, w and w_L. s_u/p = 0.5 – 2.9 (!), but $p_c/p_o \approx$ 1.1 – 2.9 also. Still, very large (excessive) settlement. p_o, p_c and s_u would suggest reasonably normal soil, but w_P, w and w_L would <u>not</u>.

10 - 12. Peat and filter paper SEMs

- Peat structure is reinforced, but clearly very porous. Consolidation = expulsion of interparticle and intraparticle water simultaneously, accompanied by plastic creep of the peat particles.

13. Coefficient of permeability (hydraulic conductivity) as a function of void ratio

- Even at a void ratio e as high as 5 (i.e. a porosity of 83%), k is extremely low; obviously, the drainage channels are almost closed.

14-15. Miramichi Bay and Mexico City SEMs

- High content of diatoms, therefore $G_s \approx 2.7$ but low unit weight, high w_P, w and w_L

16-18. Miramichi Wastewater Treatment berm: porewater pressure, berm height and settlement as a function of time or log time

- High rate of dissipation of Δu and large settlement, both as might be predicted from 14-15 above.

19. Escuminac phase diagram

- The peat mat at Escuminac, NB, was about 0.3 m thick, consisting of a highly fibrous plant growth. Nevertheless, relatively heavy loads (trucks and vans parked or moving directly on mat) were supported by a material (peat) having a porosity as extremely high as 96% (gas and water)!

20-21 Escuminac peatland: ε vs log p, undisturbed and remoulded

- Much effort was expended developing a peat sampler capable of obtaining undisturbed samples. It was found however – much to our surprise – that the consolidation of completely remoulded peat (P-82, slide 21, sample remoulded from a slurry of water content > 3000%) gave the same result as consolidation of undisturbed peat (P-66, P-67 and P-68).

22-23. Escuminac peatland: ε and settlement vs log time, lab and field

- c_α = 0.020-0.025 of undisturbed peat samples (slide 22) was roughly of the same order of magnitude as that under the Escuminac test fills (slide 23, 0.028-0.033). However, as seen from slide 22, in the lab the timing of the start of a constant value of c_α (0.020-

0.025) <u>increases</u> with increasing applied stress - whereas in the field the start of a constant value (0.028-0.033) <u>decreases</u> with increased applied stress.

24. Escuminac test fills TF6a and TF9

- TF6a was 3.7 m thick and TF9 2.5 m thick, yet the surface of both ended up at the same level (about 0.3 m above the original ground level).

25. Escuminac test fills, sections through TF8 and TF7

- TF8 was no less than 6.8 m thick, being constructed on 7.1 m of peat. Tf7 was about 0.8 m thick, also on 7.1 m of peat. Both ended up with their surfaces about 0.3 m above original ground level. Compare with slide 24.
- For TF8 there was some shear deformation (bulging) of the peat.
- The swelling of the peat outside the toes of the fill (the "after heave" surface) was originally assumed to be due to shear deformation (mudwave). However, it was later (after a few years) observed to have settled to its original elevation; the swelling was found to be one caused by hydraulic fracturing (water squeezed out laterally into the unloaded peat by the fill).

26. ε vs Δp, embankments on peats and organic soils from many locations

- As expected, the higher the applied stress, the greater the vertical strain. However, the range of strain is seen to be extremely large. For example, at a stress of 20 kPa, corresponding to about two metres of fill, the strain varies between 0.10 and 0.45; for a peat thickness of say 7 metres, as at Escuminac, this corresponds to a settlement range of 0.7 to 3.2 m.

27. Δp vs settlement with and without preconsolidation (modified from Fig. 6.9, Muskeg Engineering Handbook)

- Creep settlement can not be eliminated, just delayed. The rate of creep strain is practically independent of the magnitude of applied stress (refer to slides 22 and 23).
- Rule of thumb (Samson and La Rochelle 1971): the time taken for the swelling of the peat after removal of the surcharge is about the same as the duration of the surcharge stage (this slide: 28 days).

28. Settlement of embankments on organic soils, with and without strip drains (Koda et al. 1993)

- Slide 28 shows an interesting case of embankment settlement on organic soil (dy and gyttja) with and without strip drains. Koda et al. (1993) report that the drains apparently improved the bearing capacity of the organic soil.
- Plotting the observed settlement as a function of log time (slide 28) suggests that the dissipation of excess porewater pressure may have taken about 100 days. Creep (also referred to as secondary or delayed) settlement took place at a rate of $c_\alpha \approx 0.023$, starting immediately on application of the load. An extension of the lower creep line

(with drains) suggests that the total settlement after about 4 years (1500 days, slide 28) would be the same as that occurring if no drains had been installed.

29. Autoroute 40 (Quebec-Montreal), phase diagram

- Again – as in the Escuminac case (slide 19) – the traffic loads were supported by a material (peat) having a porosity as extremely high as 84% (gas and water)

30. Autoroute 40 (Quebec-Montreal), applied stress and strain vs log time

- Full-line curves = original observations (Samson 1985; broken-line curves have been superimposed on the original curves. Indications are that – although the pre-loading technique was successful – the long-term settlement was reduced by only a relatively small amount

31-32. Consolidation models I and II

- Terzaghi's consolidation theory is based on the consolidation model shown in slide 30 (model I). Hence if the drainage is cut off (i.e. the drainage valve closed), the forces in the springs and in the water remain at the values they had when closing the valve. With elasto-plastic springs – simulating an organic skeleton structure – the force in the springs at cut-off will start decreasing because compression due to plastic creep will continue; as the force in the springs thus decreases, the loss of spring force will be compensated by an increase in the force taken by the water. This simulates the self-generation of pore pressures occurring in creepy soils.

33. Väsby test fill: expected rate of settlement (Terzaghi 1946)

- Terzaghi was consulted by the Swedish Geotechnical Institute to give an opinion about expected settlement under embankments on the soil foundation shown in slide 6. Curve C_1 shows – in principle – the settlement expected under q_1 (embankment + surcharge) and C_2 the settlement under only the embankment. The composite curve C_3 shows the settlement to be expected under q_1 up to time t_1 and the creep settlement after time t_2. Between t_1 and t_2 it was assumed that the settlement would be negligible. This settlement behaviour would be similar to that in peat under a surcharged fill (slide 27).
- Since the amount of creep was unknown and could not be determined, Terzaghi recommended against constructing a flying field at Väsby until further field testing had been carried out.

34. $c_{\alpha\varepsilon}$ vs $C_{c,}$ NBR 2 and 3

- This diagram is included to demonstrate the very large variation in values of $C_{\alpha\varepsilon}/C_c$ with increasing values of C_c in organic soils.

35. Instant and delayed settlement model, Bjerrum 1967 (suitable for creepy soils)

- Compare with slide 28 (settlement behaviour of dy and gyttja). It is generally assumed that secondary settlement starts after dissipation of the pore pressure during primary consolidation. Bjerrum's model, on the other hand, shows that "secondary" settlement really starts at the same time as the primary settlement, i.e. at t = 0. If the pore water in the voids of the soil were incapable of retarding the compression, the applied pressure would be transferred instantaneously to the soil structure as an effective pressure. This stage is therefore referred to as *instant* settlement. Any additional settlement (creep) is referred to as *delayed* settlement. This concept is useful when dealing with highly compressible soils exhibiting significant plastic creep ("creepy" soils).

36. Plate loads tests, Escuminac

- The very considerable difference between the tests in intact and precut peat ($SH_4B_4F_1R_1$) shows clearly the influence of the fibres, even with the relatively low content of fibres (F_1R_1) in the zone of influence below the plate.

37-38. Ring shear failure patterns and results (explicable, but not applicable to field modes)

- One of the conclusions drawn by Morgenstern and Tchalenko (1967) on explaining the behaviour of kaolin in direct shear is that the central portion appears to be in simple shear.
- Reference is also made here to Landva (2008) on the simple shear testing of Norwegian quick clay and his demonstration that the many different observed failure planes (*y-z* to *s-t)* at decreasing inclination to the horizontal (such as those described by Morgenstern and Tchalenko) represent <u>attempts</u> to fail. However, no actual sliding failure can occur until such failure is kinematically possible, i.e. until the failure planes are located entirely within the soil sample and not crossing both the sample and the top and bottom filter stones.

39. Comparison of simple shear and direct shear on peat (Rowe et al. 1984)

- Rowe et al (1984) performed one set of simple shear tests and one set of direct shear tests on peat as shown in this slide. These were very carefully performed tests and can be considered to show a realistic behaviour of undisturbed peat in these modes of shear. It can be concluded therefore that fibrous peat does not lend itself to the mode of direct shear. The simple shear results, however, are very close to those obtained from large ring shear tests on peat and may therefore be representative of the shear behaviour of fibrous peat along planes parallel to the direction of the fibres (generally more or less horizontal).

40-41. Tensile tests on peat

- The term "floating the road" almost certainly originated from the fact that the upper layer of peat bogs is generally a highly fibrous, living mat of relatively high tensile strength that can support loads by suspending them in a hammock-like mode. However, in time the mat is likely to yield and thus give rise to further settlement, if not failure.

- Tensile tests of the mat do not seem practical – if at all possible. In any case, no references have been found that deal with this problem. The tests illustrated in slides 40 and 41 were carried out on Escuminac peat on prisms (rectangular parallelepiped shape) from below the mat.
- Some idea of the thickness and strength of the mat – if a mat does exist – can be obtained indirectly from “plate penetration” tests (Landva 1993), a test that was developed while assessing the trafficability of Newfoundland peatland. It is based on the fact that the resistance to penetration of a plate (a 150 mm diameter plate was used) is considerably greater through the mat than through the underlying peat.
- For an embankment on firm foundation soils shear stresses will exist under the side slopes, but not under the embankment between the shoulders and the centre-line. Hence tensile stresses will be zero or insignificant under the main body of the embankment. However, for embankments on peat, there will be considerable settlement of the embankment, increasing toward the centre-line. The resulting stretching of the peat mat into a concave configuration will create tension and, if the tensile strength is exceeded, tensile failure will result - at or close to the centre-line.

42-43. Mexico City and Miramichi Bay, triaxial and simple shear tests (very large difference)

- The difference between the triaxial and the simple shear tests is due to the existence of a very large number of flat diatoms. In the triaxial mode these diatoms provide a reinforcing action by contributing to the lateral resistance, while in simple shear such reinforcemen is practically non-existent along the mostly horizontal failure planes.

44. Probable failure mechanism of embankment on peat over soft subsoil

- There can be no doubt that the tensile strength of the peat mat and the underlying peat contributes to the stability of the embankment. However, the magnitude of such contribution remains indeterminate.

45. Embankment on peat over rigid subsoil (left: severing of mat; right: recommended construction)

- The provision of corduroy reinforcement, although intended to reinforce the embankment structure, may also have a detrimental effect by tending to sever the mat as fill is added at the shoulders after the occurrence of settlement and centerline cracking. To counteract this, the fill may be extended out past the shoulders, thus also providing space for a drainage ditch.

46. Embankment on peat before and after widening (widening causes longitudinal cracking)

- Longitudinal cracking at and near the centerline is not uncommon for embankments on peat following a widening of the embankments. The reason is simply that the additional pressure on the peat is greater outside the original shoulders than that at the centre, in which case the resulting convex bending causes a stretching of the upper part of the road

47. Case record: Cush Road (Ireland) section before and after widening

- This is an extreme (but actual) case of settlement and convex bending of the embankment and longitudinal cracking.

48. Escuminac old road on corduroy

- The 0.4 m thick corduroyed 1870 Escuminac embankment on about 7 metres of sphagnum peat did support considerable heavy traffic, as shown.
- All test fills and roads at Escuminac ended up with their surfaces at or slightly above the original (adjacent) terrain, regardless of the thickness of the embankments – which varied between 0.4 and 6.8 metres. It follows that construction specifications should not require a final elevation or height above original ground level, but rather should specify thickness.

49-50. Ditto (corduroy exposed)

- The old Escuminac road withstood traffic without apparent problems. However, eventually it was damaged by wave erosion, and a considerable length of it was washed away. The corduroy had been laid without spacing and consisted of 5 to 15 cm diameter trunks underlain by three trunk stringers.

51. Consolidation by suction (Kjellman)

- Kjellman's method of suction consolidation – used successfully in Sweden - may be a viable alternative to preloading with fill.

52 - 53. ALLU mass stabilization

- Mass stabilization is a relatively new soil improvement method where stabilizer is mixed into peat, mud or soft clay. The method has been used in Europe for a number of years. The Finnish company ALLU now has a branch in BC (e-mail: miker@allu.net).