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Cone penetration with enlarged tip in cohesive soils

Pénétration avec un cône agrandi dans les sols cohésifs

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SYNOPSIS Cone resistance in weak cohesive soils is often negligibly low and lies within the accuracy of the measuring device. To get more reliable information measurements are taken with an electric cone penetrometer with enlarged tip, diameter 79.8 and 112.8 mm and a more sensitive transducer. This method of soil-investigation appears to be a quick, relatively cheap and reliable tool in advising practice.

INTRODUCTION

Flevoland is a freshly reclaimed polder in the central part of the Netherlands. This polder is reclaimed from a part of a former inside sea.

The toplayer of the subsoil in this polder consists of marine deposited clay and peat layers. The clay deposits are some times more or less sandy and sometimes more or less humous. The thickness of the layers varies from 0 to 6 m. The clay and peat layers have a high water content and are compressible, the undrained shear strength is low.

Electric soundings show a cone resistance of 0.1 to 0.2 MN/m². The usual range of an electric cone penetrometer is 0 - 50 MN/m2. The cone resistance of the clay and peat layers is within the range of the accuracy and the drift of the signal of the transducer.

Earlier investigations have shown the relation between cone resistance and undrained shear strength or other soil parameters. The need to get quick and reliable information about undrained shear strength also excists in this weak soils. Therefor a testprogram is carried out with a cone penetrometer with an enlarged tip and a more sensitive transducer.

TEST SET UP

A comparison between different methods of measuring the undrained shear strength is made at three test locations. These locations are representative for the different types of subsoil and soilparameters.

Location 1 represents the soil profile with a rather thick layer of marine clay. Location 2 represents the soil profile with peat and peaty layers and the third location represents the soil profile with sandy clay layers.

At every location the following tests are carried out (see fig. 1).

- 3 soundings with a straight electric cone penetrometer, diameter 36 mm;
- 4 soundings with an electric cone penetrometer with enlarged tip, diameter 79.8 m;
- 28 soundings with an electric cone penetrometer, with enlarged tip, diameter 112.8 mm;
- 2 field vane tests;
- 4 borings with undisturbed Ackermannsamples for direct shear, triaxial and cell tests;
- measurement of water content, volumeweight, content of organic matter and clay particles.

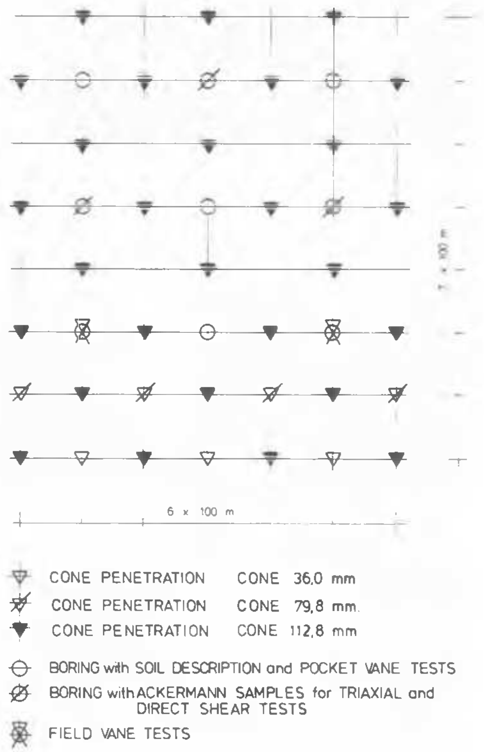

Fig. 1. Test set up

Cone penetration is done with a straight electric Fugro-cone with a transducer of 50 kN, diameter 36 mm and tip angle of 60°. For the enlarged cone an electric Fugro-cone with a transducer of 10 kN is used. The cone tip is enlarged from 36 mm (surface 10^3 mm²) to 79.8 mm (surface 5×10^3 mm²) and to 112.8 mm (surface

10×10^3 mm^2). In this way the signal is made 50 times more sensitive. Pulling out the penetrometer with enlarged cone tip after the sounding will cause a negative pressure on the transducer. To avoid this negative pressure the tip is prestressed with two tension rods (see fig. 2).

Fig. 2. Cone with enlarged tip

The prestress of the cone used during the tests is 2 kN, under this prestress the tension rods have an elongation of about 0.082 mm. The impression of the load cell inside the cone is much less, for the maximum load the impression is about 10% of the elongation of the tension rod. When the load cell of the cone is loaded at his maximum, the prestress will be reduced 10%. This can also be seen in the test graph of the cone with and without prestress in the testbank. The results of this test are given in fig. 3.

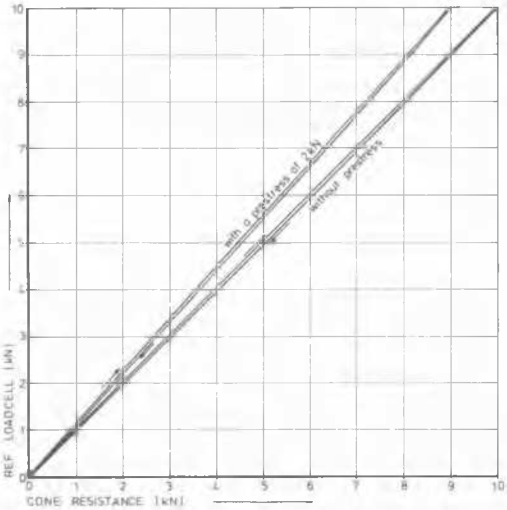

Fig. 3. Test results in the testbank.

From this result it can be seen that a correction of 10% should we made at the results of the cone penetration. The cone penetrations given in this article are made without correction and the presented correltation is also made without correction. The prestress is a safety against negative stress of the transducer, but it also limits the maximum cone resistance that can be measured with this transducer.

Field vane tests are carried out after the soundings. The blades are 50 mm wide and 75 mm high, the rotation velocity was 6° per minute.

A correction was made for friction along the rods. This correction was measured with a dummy rod of the same length.

The borings are made at the end. From this borings samples were taken for the direct shear, triaxial and cell tests. The direct shear samples were Ø 60 mm and 20 mm high, the shearing velocity was 0.3 mm/minute. The samples for the triaxial test were Ø 38 mm and 76 mm high, the deformation ratio was 0.076 mm/minute. The samples for the cell test were Ø 65 mm and 150 mm high. The tests were consolidated, the sheartest and triaxial test were undrained. In the triaxial test also pore water pressure was measured.

The undrained shear strength was calculated from

$Cu = C + \sigma' \tan \varphi$, in which
Cu = undrained shear strength (kN/m^2)
C = cohesion
φ = angle of internal friction
σ' = grain stress (kN/mm^2)

TESTRESULTS

The relationship between cone resistance and undrained shear strength for cohesive soils is investigated by many authors. The relation generally fluctuates between $qc/10 < Cu < qc/20$ (Genevois e.o.). Soft or sensitive clays show generally a higher ratio (Brand, e.o.) than stiff and fissured clays. The soft Dutch clays have a ratio of 14 to 15.

A bigger cone gives a relatively lower cone resistance per unit of surface. De Beer has found a decrease of 30% when the cone is 5 times bigger and a decrease of 45% when the cone is 10 times bigger.

The results of the borings and the composition of the soilprofiles are given together with the results of the soundings with the biggest cone tip in the figures 4, 5 and 6.

The cone resistance of the 28 soundings are drawn together to give an impression of the variation in cone resistance. The relation between cone resistance and undrained shear strength from the three locations is given in fig. 7.

For this relation the average cone resistance and the average of the shear strength from the different tests at the same depth, if more tests were excecuted at the same depth are taken. For each type of test the best curving line is calculated, assuming this line goes through the point 0.0 and assuming that it is a straight line. This leads to the following relations:

- field vane $qc = 10\ Cu$ (11)
- laboratorium vane $qc = 13.3\ Cu$ (14.8)
- triaxial test $qc = 11.5\ Cu$ (12.8)
- cell test $qc = 14\ Cu$ (15.5)
- direct shear test $qc = 8.33 Cu$ (9.3)

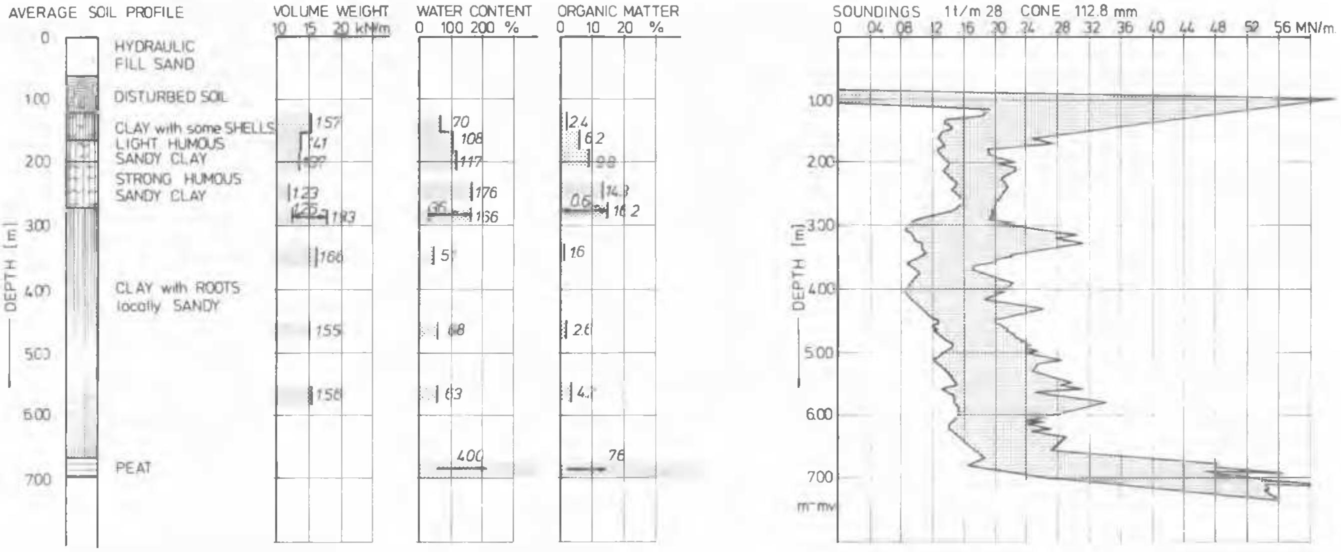

Fig. 4. Soil description and cone resistance location 1

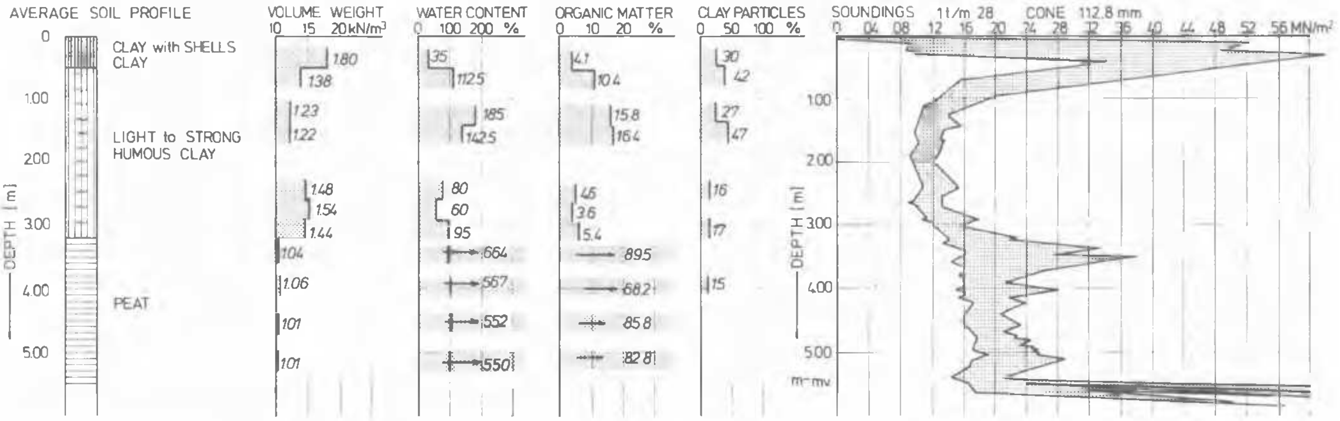

Fig. 5. Soil description and cone resistance location 2

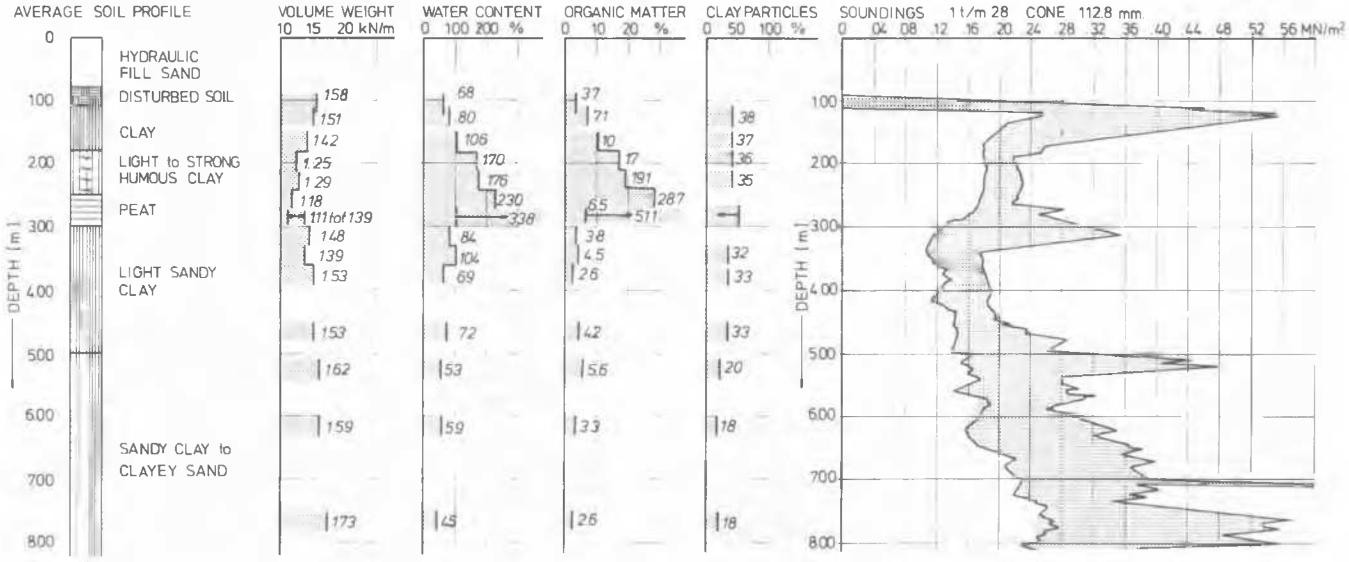

Fig. 6. Soil description and cone resistance location 3

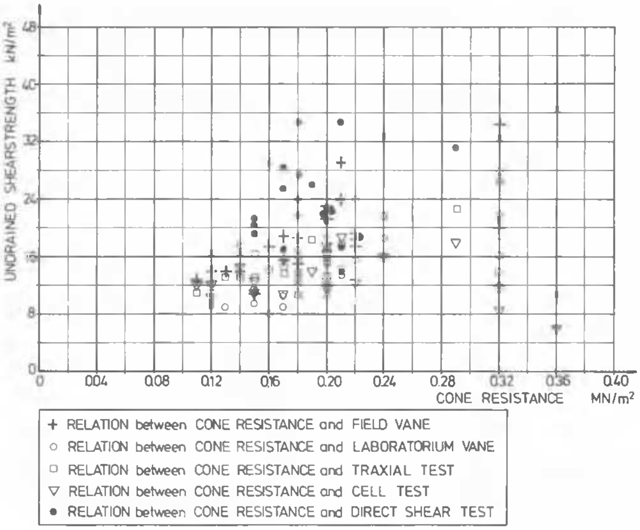

Fig. 7. The testresults

No difference was found between the three locations in this relation. The relatively higher undrained shear strength measured with the field vane in comparison with the laboratory vane is caused by the higher horizontal pressure in the field and due to the fact that the laboratory vane is used on a horizontal plane. The relatively high shear strength in the direct shear test is caused by the fact that the failure zone in the direct shear test is fixed by the geometry of the test apparatus.

In the triaxial and cell test the failure zone is not fixed and will therefore develop in the weakest zone.

When the correction of the cone resistance caused by the prestress of the cone is taken into acount the above mentioned correlation must be taken 10% higher (the numbers between brackets).

EXAMPLE

The relation mentioned above has been used in practice during the past two years. One of the cases is given here. Two bridge-approaches with the same geometry were under construction at a distance of about 1.000 m in the same area and with a comparible soil profile. At a height of the approach of 4 m and slope of 1:2, one of the approaches started to fail and the other stayed stable. The results of two representative soundings are given in fig. 8.

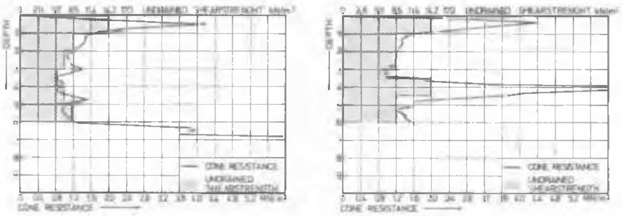

Fig. 8. Cone resistance of two approaches

A substantial difference is noticeble. The cone resistance is schematiced. The undrained shear strenght is taken as qc/14. This value is taken as input in the stability calculations according to the method of Bishop. The angle of internal friction is put at 1°. For the first unstable bridge-apporach the savety-factor was 0.79, for the second approach the safety-factor was 0.97. For the unstable approach the stability of the approach with a height of 6 m and a support with a height of 2 m and a width of 10 mm was also calculated. In that case the safety-factor was 1.08. This support was made before the last 2 m of sand were brought in place; the approach stayed stable. The results of the traiaxial tests, made in advance in this area, did not show so much difference. It will be clear that these differences at relative short distances and in the same layer are overlooked easily.

CONCLUSION

In very soft layers cone resistance is too low to give sufficient information about the strenght of the soft layers. With a cone penetrometer with an enlarged tip and a more sensitive transducer this information can be derived. Tests at three different locations showed a good relation between cone resistance and undrained shear strength. This relation depends on the type of test used for the measurement of the undrained shear strenght. However the cone resistance is not capable of giving information about the strength at a higher future stresslevel. It therefore gives no information about the ultimate shear strength under a future embankment or approach. In practice it appeares to be a quick and reliable tool.

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