DESIGN OF CROSS SECTION OF EARTH EMBANKMENTS

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This report relates chiefly to subsoil conditions as found in the western part of the Netherlands. Here the subsoil consists for the greater part of a peat and clay layer with little bearing capacity, having a thickness of 0 to 18 meters and resting on the pleistocene sand. As a rule, a layer of old marine clay resting on the sand is covered by a thick layer of peat.

Because the reclamation works were undertaken in regions, where the peat-layer had been removed, the old marine clay is often the toplayer in the "polders." In other areas the surface consists mostly of peat; except in places in the neighbourhood of the great rivers or near the sea, especially in the northern and southern part of the country, where the peat has been covered by a layer of clay. Often in the latter case the peat has been removed previously by the sea.

From this short description it is obvious that the nature of the toplayers and soil-conditions differ greatly.

As a rule the ground-water level is found at a depth of some centimeters (in the peat districts) to some decimeters below the ground surface.

Thus the different layers have been subjected only to a very small load and as a result they have very low densities. The compressibility of the soil is very high (values of C in the formula

$$C = \frac{\text{height of sample}}{\text{compression}} \times \lg \frac{p + \Delta p}{p}$$

amount to 3 to 10); likewise the shearing resistance is very small (0.05 to 0.1–0.2 kg. per cm²).

It is assumed that the top width and the height of the embankment above the original ground-surface have been fixed by other considerations than subsoil conditions.

If there are no special requirements for secondary purposes of the embankment (such as flood protection) the slopes, depth and bottom width of the fill are determined by the mechanical properties of the subsoil and of the material of the embankment itself. There is no doubt that the economical aspect of the project and the importance of the projected highway on top of the embankment, will have a great influence on the design.

SUBSOIL INVESTIGATIONS

Before starting the field survey, the geological conditions at the location of the project can be studied by means of the geological map and by visiting the field.

After this it is recommended that the investigations be started by use of the hand sounding apparatus in order to get an impression of the consistency and uniformity of the clay and peat layers. The sounding resistance can be measured in a short time; the costs are small and many observations can be made at several locations on the building site.

The hand sounding apparatus consists of a cone (with a top-angle of 60 deg. and a basic area of 10 cm²), which is fixed to the lower end of a 1-cm. diameter bar. This bar is guided by the inside wall of a pipe so that friction between the bar and the surrounding soil can be avoided in pressing down the cone. The force required, to press down the cone, is exerted by means of a hydraulic plunger (with the same diameter as the cone) and measured by a Bourdon gauge fixed to an oil chamber.

The readings on the Bourdon gauge

give the pressure on the cone directly in kg. per sq. cm., except for the weight of the bar, which must be added.

The results of the sounding resistance are plotted in diagrams against the depths below soil-surface. In this way a general survey of the cone-resistance along a longitudinal profile can be obtained.

The proper locations for sampling can now be determined by comparing the geological longitudinal profile with the sounding results. The advice of a laboratory of soil mechanics will be necessary in order to avoid superfluous borings and unnecessary loss of time.

Before starting the laboratory investigations the following data must be obtained:

1. The depth and thickness of the strata.
2. The locations and depths of undisturbed samples.
3. The ground water level.
4. The hydraulic pressure in the deeper sand-layers.

For investigation of the material of the embankment itself, a quantity of disturbed soil (for tests regarding maximum density and optimum moisture content), and some representative undisturbed soil-samples from the borrow pit should be secured.

TESTING OF SOIL-SAMPLES

Undisturbed Samples

The samples are taken in tins with a length of 44 cm. and a diameter of 7 cm. After removing the tin from the sampling apparatus, both ends are sealed with paraffin to prevent drying-out.

In the laboratory the samples are stored in a room with a high degree of humidity and a nearly constant temperature.

The investigation is for the determination of the following data:

1. *Volume-weight and Moisture Content:* The volume-weight is determined by weighing a known volume of the sample. After drying this part of the sample in an electric stove at 105° C and weighing it again, the moisture content can be calculated.

If the specific gravity of the mineral matter is known, the air content also can be determined.

From the volume-weight of the different layers and the water pressures in the soil, the grain pressures at several depths can be determined.

2. *Cone-Resistance:* A cone with a top-angle of 90 deg. is placed with its top on the surface of the sample and loaded every two minutes with a constant load. The resistance of penetration can be calculated from the quotient of the load P and the surface of the area of penetration πz^2; z being the depth of penetration.

Especially in the case of cohesive soils (clays, f. i.) a relation exists between the cone-resistance and the shearing-resistance of the undisturbed sample.

From this relation the magnitude of the loads, which can be applied to intended soil-layers, without danger of loss of equilibrium can be ascertained.

3. *Compressibility:* A part of the sample 2 cm. high is placed in a bronze ring between two porous stones. The compression of the sample caused by a load, acting on the topmost porous stone, is observed during a long time by means of a dial.

Then a new load is applied and the compression is again measured.

As the "secular" compression is especially of great importance, in the settlement of embankments, the above mentioned consolidation test must not be carried out too fast.

Settlement-results, obtained with these long duration consolidation tests can be plotted against time on a semi-logarithmic diagram.

With the aid of these data, expected settlements in course of time, can be determined.

4. *Properties of Internal Friction:* A part of the sample with a length of 12 to 15 cm. is placed in the so-called cell-apparatus and loaded vertically. A new load has to be applied several times and for every load the minimal horizontal pressure, is measured. The results of this test can be analyzed with the aid of the stress-circles of Mohr. When calculating the vertical grain pressure in the subsoil with the aid of volume weights and water pressures, a Mohr stress-circle, corresponding to this vertical pressure, produces the shearing resistance, at the same depth in the subsoil.

At the same time the inclination of the envelope of the stress-circles, belonging to pressures higher than the preconsolidation load gives an impression of the speed of the effect of consolidation.

Samples of Materials for Embankments

Usually in the Netherlands sand dredged from the rivers or from fields, bought for the purpose, is used for road embankments. This sand is generally of very good quality; it contains only a small quantity of fine particles and is densified easily during the construction of the embankment by the cars used for hauling. Moreover the upper layer is often especially treated (explosion rammer).

If sands with clay or clays with sand are used, it is desirable to determine the greatest density, which can be attained at a certain moisture content. It is desirable to test the samples in the cell-apparatus, to determine their shearing resistance and also their compressibility when high dams are to be built.

CROSS SECTION DUE TO THE EXPECTED SETTLEMENTS

With the aid of the required height of the road above the original surface of the ground, the volume-weight of the embankment, the existing vertical grain-pressure in the subsoil, the height of the ground water level, the consolidation properties and the thickness of the different layers, it is possible to calculate the settlements, to be expected in course of time.

The settlement, which takes place after construction, is of especially great importance, as it is this settlement that gives most trouble. If this settlement is too great or if the settlements on short distances differ considerably, particular measures must be taken. Then it is necessary to remove the layer, which causes the trouble (by dredging or squeezing out) and replace it by sand; or a pile foundation may be made for the road.

Sometimes the embankment is built to an extra height in order to promote the largest part of the settlement during the construction of the road.

Sometimes a substratum is used, which acts as a bridge over the soft places, such as a fascine mattress. Also the embankment can be built of light weight material.

However the choice of the solution depends for a great part on the cost and importance of the road. A few examples will be discussed hereafter.

CROSS SECTION DUE TO LOSS OF EQUILIBRIUM

To get an impression about the equilibrium of the embankment, circular shearing surfaces (Petterson) are applied.

For this computation it is necessary to know the volume-weight of the material, and the phreatic water level. Preliminary settlement computations determine the bottom of the embankment.

A comparison of the computed shearing resistance along the circular sliding plane, with the shearing resistance of the samples measured by the cell-apparatus determines the possibility of loss of equilibrium.

The settlements will be greater as computed by the results of the consolidating, if the loss of equilibrium is hardly reached.

If the comparison between the computed and the determined shearing resistances shows that no equilibrium exists, different measures can be taken:

1. Slower construction of the embankment for the purpose of increasing the shearing resistance by consolidating the subsoil may be utilized.
2. The slopes may be flattened or berms may be built to decrease the acting moment and to increase the shear-resistance along a greater part of the sliding plane by surcharging the under-ground.
3. The soft clay-layers may be replaced with sand, which has a greater shear-resistance. This can be done by dredging or by squeezing out the soft layers.
4. Light weight material may be used for the fill.
5. The road may be given a pile foundation.
6. A fascine mattress may be applied.

The choice of the method again depends chiefly on the cost and importance of the road.

Loss of equilibrium can also occur due to thin layers of very low consistency beneath a dry earth crust. As a rule, in this case, circular slip-surfaces will not occur, but the soft soil under the embankment will squeeze out. If there are ditches along the road, the squeezed soil appears there at the surface. As soon as the tensile strength of the hard crust is surpassed this cover will move horizontally and the form of a slide appears.

To prevent squeezing out, we can confine the soft layer laterally or we can apply a fascine mattress.

In this way the tensile strength of the embankment is increased enough for the high acting forces.

CONSTRUCTION

To verify the results, obtained by the tests of the samples, it is desirable to measure the settlements during construction of the road with the aid of special settlement marks. These data are important for the future.

By placing apparatus for measuring water tensions of good construction in the soft strata warning is given against the loss of equilibrium while it is still possible to take corrective measures.

PRACTICAL APPLICATIONS

In Figure 1 some of the possibilities as described herein and executed in recent years in the Netherlands, are shown.

The height of embankment A above the original surface had to be 5.5 m. The subsoil consists of sandy marine clay with a small shearing resistance so that it was not possible to construct the embankment at once to full height. After digging away the top-layer of black earth necessary for covering the slopes of the future embankment, the fill was erected to a height of 3.5 m. in the first year.

After a rest of some months it was possible to finish the embankment without loss of equilibrium. The measured settlements were about 50 per cent larger than the computed ones due to small lateral displacements.

Embankment B with a height of 10.60 m. was to be constructed on peat and clay layers with a thickness of 6 m. (shearing resistance 0.16 kg. per cm.^{2}).

Computations have shown that loss of equilibrium should occur when the construction without special measures was executed in the required time. The bad layers could have been removed, but it seemed more economical to construct berms with a height of 4.6 m. and a width of 30 m. Also a flatter slope as drawn on the right side would have been a possibility.

Embankment C was constructed partially by hydraulic fill (sand) partially by car transport after the 4 m. peat-layer meter was removed by dredging.

Afterwards it was found that the shoulders were still compressing the peat more than 4 m., those layers were squeezed out by the load of the embank-

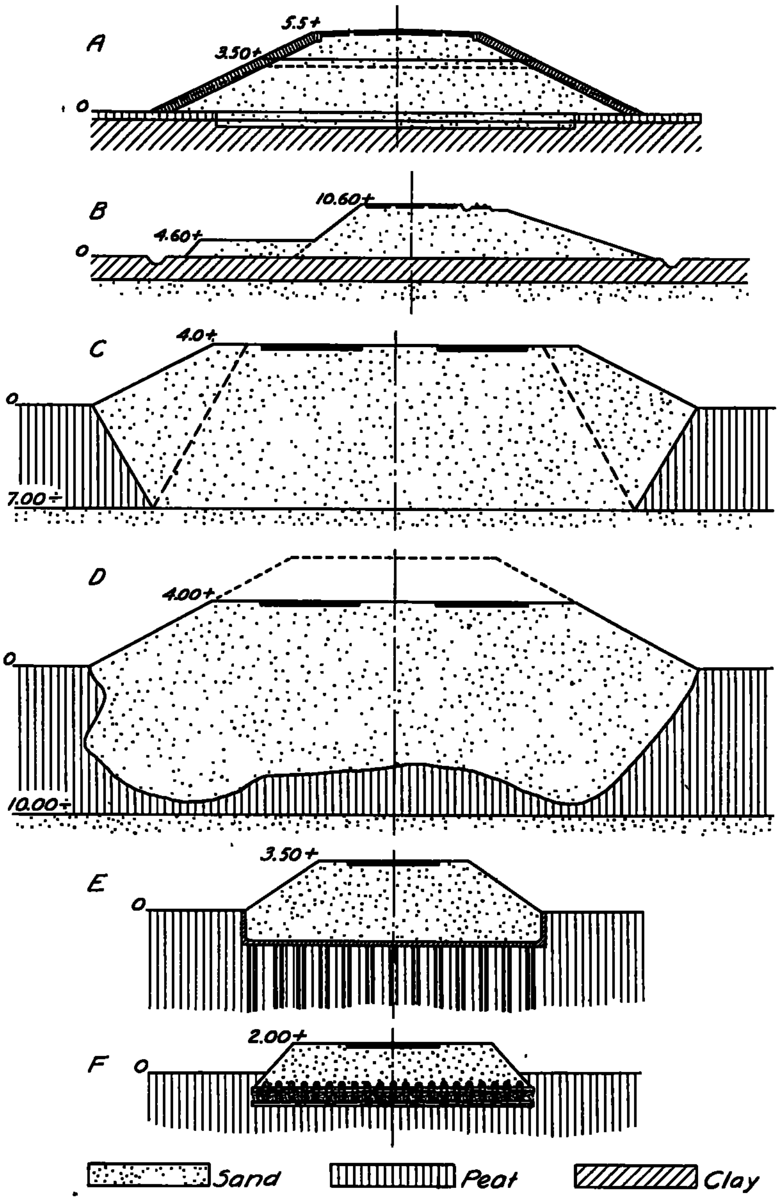

Figure 1

laterally so that fissures appeared near the roadside.

By constructing embankment D on clay and peat layers with a thickness of ments. In order to compress the remaining soft soil an extra height is applied to it.

For secondary roads a pile foundation

as drawn in E, is provided. The wooden piles bear a concrete trough filled with sand.

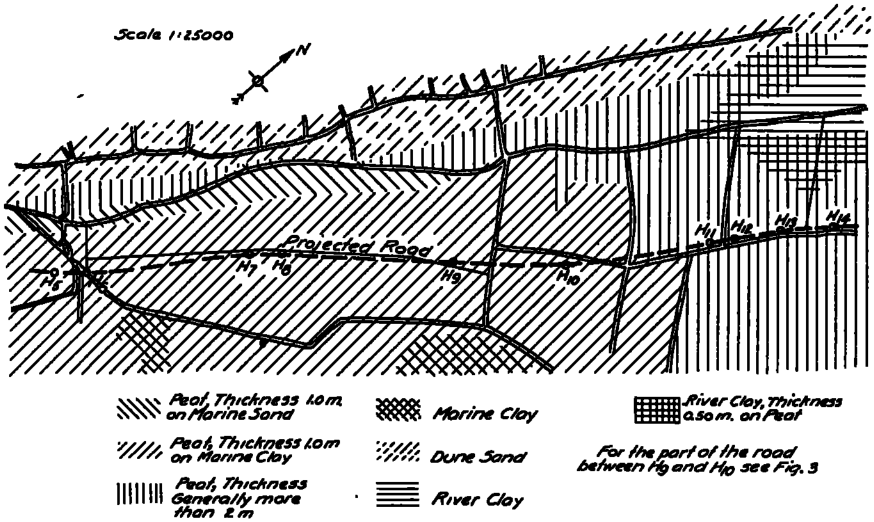

Figure 2. Geological Situation

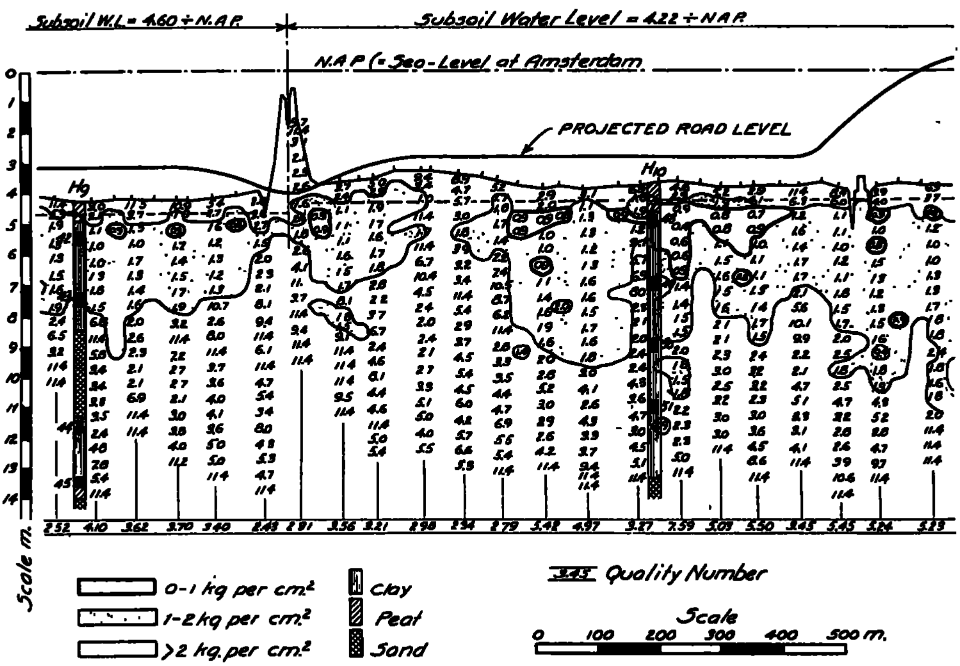

Figure 3. Sounding Resistance Profile

Embankment F is constructed on a mattress made on top of the compressible peat-layers (thickness 8 m.). Then sand is washed in and the embankment is completed. By its weight the soil layers are compressed and so the mattress sinks beneath the ground water level.

NUMERICAL EXAMPLE

A numerical example is now given following through the whole investigation.

TABLE 1

RELATION OF α_p AND α_s TO SUPERIMPOSED LOAD

Thickness of Soil Layers Meters	New Vertical Load 0.18 kg. per cm²				New Vertical Load 0.48 kg. per cm²			
	α_p	$\alpha_p d$	α_s	$\alpha_s d$	α_p	$\alpha_p d$	α_s	$\alpha_s d$
2.20	0.29	0.64	0.054	0.119	0.290	0.64	0.055	0.121
2.00	0.25	0.50	0.077	0.154	0.245	0.49	0.049	0.098
3.40	0.10	0.34	0.033	0.112	0.109	0.37	0.024	0.082
Σ		1.48 m.		0.385 m.		1.50 m.		0.301 m.

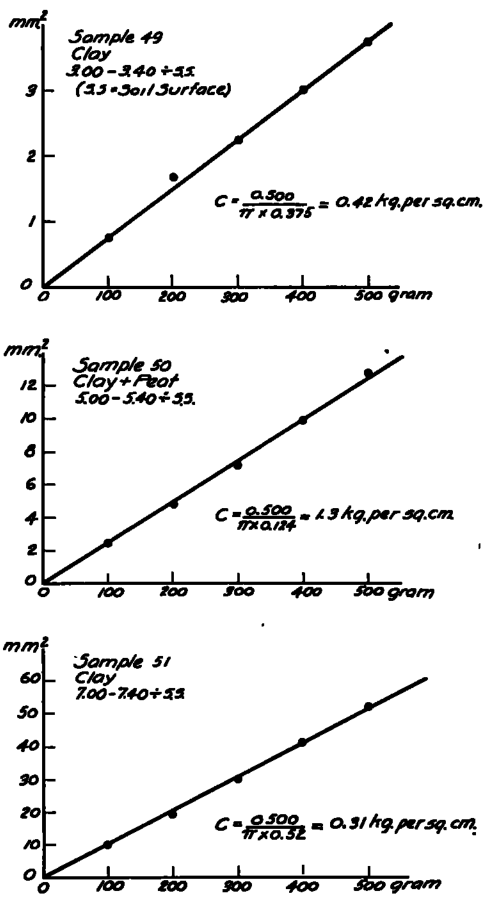

Figure 4. Cone Resistance Tests, Boring H_{10}

The axis is given by other considerations than those following from the consistency of the subsoil.

In Figure 2 the projected road is drawn on the geological map. Top layers are as usual peat on marine clay. The hand

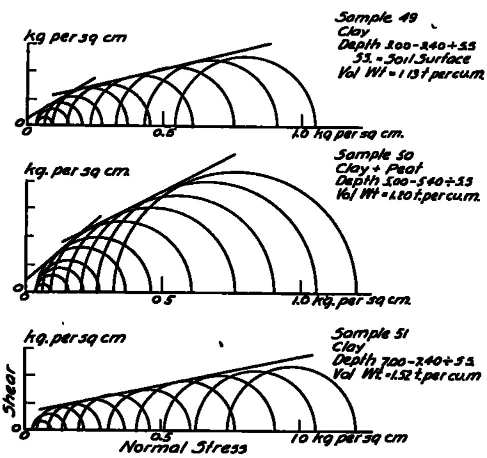

Figure 5. Cell Tests, Boring H_{10}

sounding results are plotted for a part of the length on Figure 3, on which the projected road level is also drawn.

The soundings were made at distances of about 50 m. The readings from the pressure indicator are plotted, and lines of equal resistance are drawn.

From these readings rough estimates can be made of the expected settlements and the existing shearing resistance of the soil and accordingly it is decided if further tests on soil samples must be made.

The lines of equal resistance give a good idea of the regularity of the subsoil. Further quality numbers (z) are calculated from the readings (s) by the formula

$$z = \frac{1}{n}\sum\frac{1}{s}$$

(n = number of readings per meter depth).

Now the places where undisturbed samples shall be taken are fixed. Auger

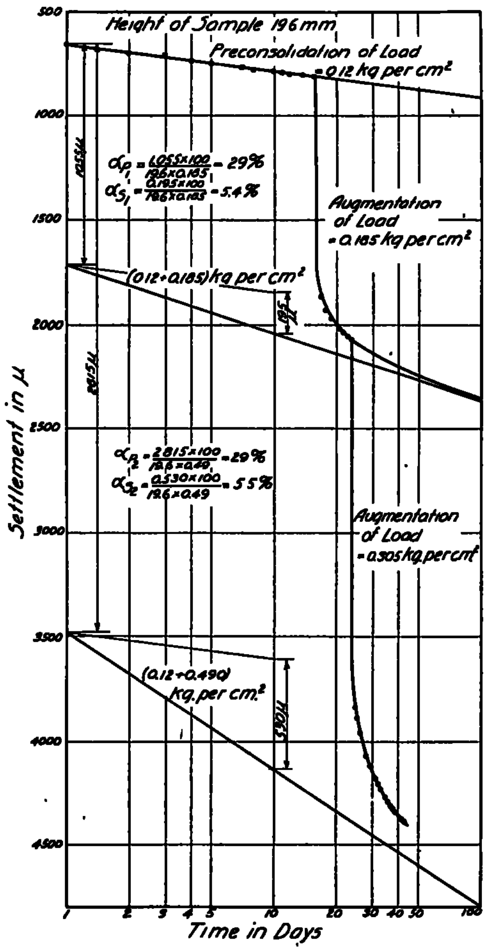

Figure 6. Consolidation Test on Sample 49, Clay 3.00 3.40 S.S.

borings are made while special samplers are used for obtaining undisturbed soil samples. These boring results are shown on the same resistance profile.

A sample investigation in the laboratory can now start. The results of 3 samples from boring H 10 are given

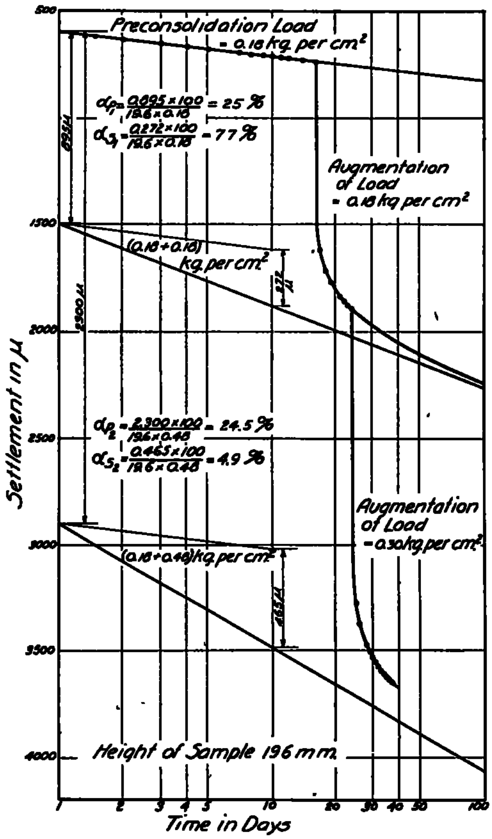

Figure 7. Consolidation Test on Sample 50, Clay Peat 5.00 5.40 S.S.

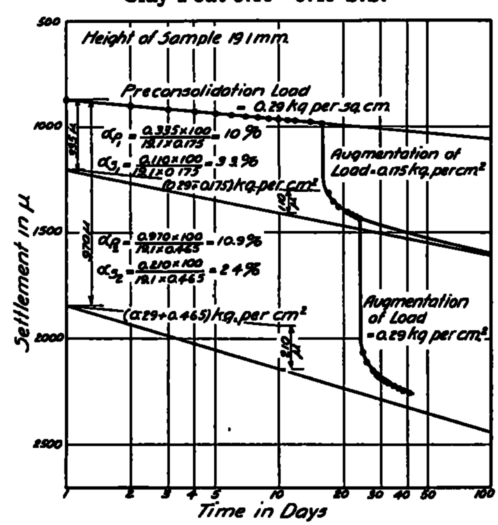

Figure 8. Consolidation Test on Sample 51, Clay 7.00 7.40 S.S.

in Figures 4 (cone-resistance), 5 (cell-test, volume-weight), 6-7-8 (consolidation test).

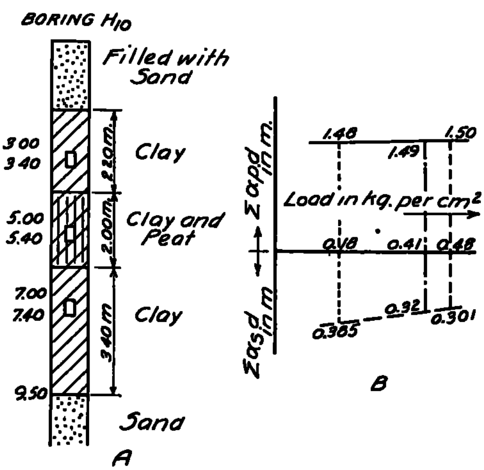

Figure 9

The settlement to be expected is computed as follows:

A diagram is erected having on the horizontal axis the load in kg. per cm.² and on the vertical axis $\Sigma\alpha_p d$ and $\Sigma\alpha_s d$ in meters, as in Figure 9-B.

$\Sigma\alpha_p d$ and $\Sigma\alpha_s d$ due to the load of the road embankment p follow from this diagram. Near the boring H_{10} is $p = 0.41$ kg. per cm.² so $\Sigma\alpha_p d = 1.49$ m. and $\alpha_s d = 0.32$ m.

The settlement z due to the load $p = 0.41$ kg. per cm.² is =

$$Z = p\,[\Sigma\alpha_p d + \Sigma\alpha_s d \log t],\ t = \text{time in days.}$$

$$Z = 0.41\,[1.49 + 0.32 \log t]$$

After 100 days Z will be 0.87 m.
After 1000 days Z will be 1.00 m.

After this, computations are made regarding the possibility of loss of equilibrium along a circular sliding plane as is drawn in Figure 10. For this purpose the most dangerous sliding plane is to be found by examining different circles.

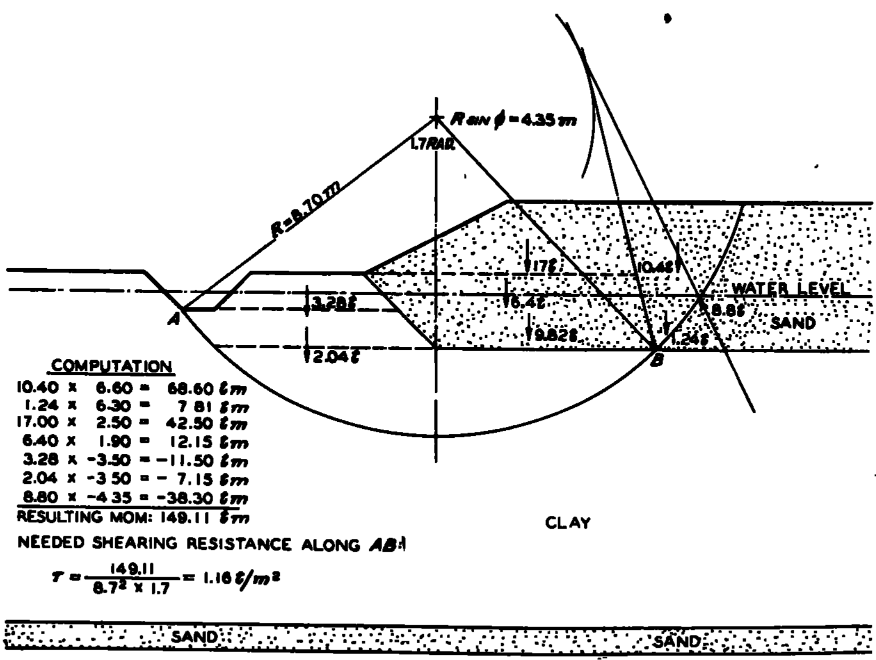

Figure 10. Sliding Plane for Profile Near Boring H_{10}

In Figure 9-A and Table 1 the relation of the constants α_p and α_s with the superimposed loads are given.

DISCUSSION ON THE DESIGN OF CROSS SECTION OF EARTH EMBANKMENTS

Harry H. Hatch, *Division Engineer, Municipal Water Works, Springfield, Mass.:* Discussion of the fundamental principles involved in the design of cross section of earth embankments is timely and very important.

There are numerous studies already available on this subject. Hogentogler, on page 220 of his "Engineering Properties of Soil", gives a table for the critical height of fill for various kinds of soil. A very good paper on the subject, by L. A. Palmer and E. S. Barber, was presented at the 1937 Annual Meeting of the Highway Research Board.[1]

The problem is that of equilibrium of forces. In the light of many experiments and theories, the forces acting in the soil at any given point of embankment, due to dead and live loads, can be computed or estimated. This acting resultant unit force is balanced by the resultant reacting unit force in the soil structure. The latter depends upon the cohesion and the internal friction of the material.

It is evident that the cohesion and the angle of internal friction are the important factors. In current practice these factors are considered to be constant and independent of each other and designers are assigning or assuming, by test or otherwise, a constant value for each of them, irrespective of the height of the structure. The writer doubts the validity of this assumption or principle.

It may be fair to ask whether we know enough of the behavior of the cohesion and angle of internal friction values of the same material with various moisture contents and under different loadings, to be able to set a tentative standard design of cross section of earth embankments for various heights. For instance, what reliable data have we showing that the cohesive strength and angle of internal friction of the material of an earth embankment will remain constant with increasing moisture and under increasing pressures? Through rain, snow and flood or action as a water barrier, an embankment will have different moisture contents in the material at various elevations; and the higher the embankment the greater is the vertical pressure at its lower levels.

The theory that the cohesive strength of a soil is independent of its angle of internal friction, or coefficient of friction, simplifies the discussion.

Some recent writers appear to be making allowances for variation in the cohesive unit value of the material by considering its small fraction only as effective during the design. Is this due to the difference in moisture content or in vertical load? No definite reason, however, is given for this assumption. In some cases the cohesive strength value and the small fraction thereof used in the design is so small that this factor could be entirely neglected without any appreciable difference.

It appears that the most important factor in these designs is the cofficient of friction or the angle of internal friction. It is of utmost importance, therefore, to know whether the angle of internal friction of a material remains constant with increasing moisture and increasing vertical loads.

With coarse and fine cohesionless soils it is a proven fact, at least to the writer's satisfaction, that the coefficient of friction of the same material undergoes an appreciable change with increase of its moisture content and under increasing vertical pressure. See Table 1.

Undoubtedly the decrease in the coefficient of friction due to increasing mois-

[1] *Proceedings*, Highway Research Board, Vol. 17, p. 503.

ture content is on account of the latter's lubricating effect.

Some writers have argued that the decrease in the coefficient of friction due to increasing vertical pressure, is the result of the pulverizing of the grains, and that this does not constitute any change in the angle of internal friction. The writer cannot agree with the last part of the argument. It is true that the decrease of the reacting force of the material is due to its particles being crushed and being made smaller, but we must not forget that it is this same material for which design is being made. We know that the smaller the particle sizes, other things being equal, the smaller is the angle of internal friction.

This raises the question of a very serious lack of coordination between the old and the new schools of thought in soils mechanics. The American pioneers, in their studies attached great importance to determination of grain size. They realized the importance of gradation and made much practical use of it with admirable results. There are yet many practicing engineers who cling to the gradation curve and use it to good advantage, in spite of the fact that it seems to have lost its importance with the new school. The writer knows of one large laboratory at the site of a big construction job where intelligent interpretation of gradation curves would have saved at least 50 per cent of the routine laboratory work.

The writer believes that the values for both the cohesion and angle of internal friction should be definitely stated for the moisture content and the load under which they were obtained and that these values should be applied in the design only when they coincide with those in the structure at the point of consideration.

Since cohesion and angle of internal friction are independent of each other, the writer does not see any reason why the angle of internal friction of a cohesive material should not obey the same law as that of a cohesionless soil.

There may have been some experiments made on cohesive materials to determine their angle of internal friction under varying loads, but the reports show that these experiments were made on very small samples, not subject to very high vertical pressures. It would be interesting to have the results of a sufficient number of reliable experiments on the same cohesive material with larger samples under different pressures up to 20 or 30 tons per sq. ft.

TABLE 1

Results of Coefficient of Friction Experiments with One Cubic Foot Samples of Cohesionless Core Materials of Very Similar Character and from the Same Borrow Pit

	Sample 5	Sample 7	Sample 10	Sample 15
Moisture, per cent	2.0	3.7	1.7	14.4
Voids, per cent	38.0	35.0	37.0	40.0
Effective Size, mm	0.007	0.006	0.007	0.007
60 per cent Size, mm	0.043	0.052	0.048	0.036
Load: lb. per sq.in.	132	198	66	132
Tons per sq.ft.	9.5	14.2	4.7	9.5
Coef. of Friction	0.77	0.66	0.82	0.56
Angle of Internal Friction	37°-36′	33°-25′	39°-21′	29°-15′

Edmund F. Preece, *Chief, Sanitary, Hydrologic and Research Engineering Division, National Park Service:* An editorial appeared in one of the trade journals recently in connection with the partial failure of the Fort Peck dam that should cause everyone who is deeply interested in the science of soil mechanics to pause and give some thought to the present state of development and use of that very valuable tool. In effect, the science was charged with being a champion come to dispel the uncertainties of earth embankment construction which, after being suitably feted and caparisoned was soundly unhorsed at one of the first major encounters.

To this speaker the idea that soil mechanics is at fault in this instance is disturbing, not because that science has been found wanting but because it has been considered sufficiently rigid in application to warrant complete responsibility. Perhaps in our enthusiasm we have put the boy in long pants and expected him to act the man too soon. Or it may be that those who are less acquainted with him have mistaken him for a man because of his lusty growth. Whatever the reason, it is unfair to the youngster to suffer censure for not being more than can reasonably be expected of him.

We cannot now and very likely never shall be able to tabulate soil behavior after the manner of a structural steel handbook. Unfortunately, this is not recognized by some of the contemporary practitioners. Very recently I was called upon to review a report on the design of a rolled earth dam. The problem concerned a stratum of material that had been questioned. One laboaratory had determined the unit shear and cohesion values of the material in a partially saturated state; another laboratory had determined the values for the same characteristics but in a saturated state. The material is predominently silt. Using the two sets of values the report arrived at a factor of safety of 1.54 for one set values and 0.89 for the other, completely disregarding the lack of relation between the two sets of values upon which those factors were based. While there is much here that might be criticized the most serious error would appear to be the use of the technique in a manner sufficiently rigid to justify the use of fractional factors of safety.

The young practitioner, especially if he be more or less on his own, must accompany his study of soil mechanics literature and its application by observation and study of the practical problems of construction and maintenance. In particular must he realize that the extreme accuracy of measurement required by soil mechanics laboratory technique leads to a mathematical solution which must be applied to the actual job in terms commensurate with the practical absence of the idealized conditions that were assumed in order to make the laboratory solution possible.

Soil mechanics technique like a drafting instrument is a tool, and, like the drafting instrument, the product of its use will depend upon the user. As an aid to broad experience and sound judgment it has an important place even now. It will never however, be an alternate for experience and judgment.