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HIGH CLAY ROAD EMBANKMENTS

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Summary

A short discussion is presented on the use of clay and silt materials for the construction of road embankments.

Some emphasis is placed on the practical aspects of using these materials, the technical difficulties and also the more theoretical aspects of soil mechanics.

The problems are discussed as they arise, planning and design very briefly, and excavation transport and the actual fill construction more thoroughly.

Measurements of stresses, porepressures and shear strengths are presented from actual field cases.

Tentative conclusions are drawn about shear strength parameters to be used in stability analysis.

Due to the complexity of these problems, the data in this report should only be used as rough guidelines.

The number of variables involved in construction of compacted road fills are so high that theoretical soil mechanics will not be able to give any definite solutions to all these problems for a long time yet.

Introduction

This report is an attempt to describe and discuss the various problems associated with the use of clay and silt materials in the construction of road embankments.

These types of materials are universally recognized as «problem materials» both for the soil mechanics consultant, the contractors and the clients, and with good reason too.

There are serveral occations where projects have been brought to a standstill owing to unanticipated problems with excavation, transport or the actual placing and compaction of these materials in the fills. The results are invariably loss of time and money.

As the technique of handling difficult materials improves, other problems become increasingly important, such as the safety and economy of the actual construction.

Even if we are able to handle these materials it is by no means ceratain that we can construct fills up to any height. The answer to these questions is naturally sought in soil mechanics theory, but regrettably, no solution to these very complex problems is to be found to-day.

As in most areas of soil mechanics, the design and construction rests on a foundation of intuition and experience combined with a theory that seems reasonable in that particular case.

There can ble little doubt, however, that this is an area where soil mechanics is in need of increased knowledge, perhaps more than in other areas.

Both safety, time and economy would benefit greatly even from small advances in this field.

It is the author's belief that real advancement of knowledge in this field best can be found in increased emphasis on field measurements. Such measurements would include field pore pressures, horizontal and vertical stresses and strains. These data could then be used to simulate axial tests and in this way produce parameters more relevant to practical problems than routine tests do today.

Dry crust clays and their properties

The upper crust, say 2 to 6 meters of the normally consolidated clay can be used in the construction of road fills. This upper crust can often be seen to have a lighter colour than the clay has at greater depths and the moisture content of the upper few meters are usually lower. The lower moisture content in this clay makes it possible for standard machinery to handle these materials, to excavate, transport and compact it in a road fill.

The questions associated with the use of these materials are briefly centered around a forecast of the suitability of the materials on the basis of field investigations and the behaviour of the materials when placed and compacted in the road fill.

In order to give a picture of the material discussed in this paper, a brief geotechnical description is given in the following.

Geological history

Very briefly, the clay material is a result of a chemical and mechanical breaking down of larger particles to gradually smaller ones. The clay fraction is taken as particles smaller than 2 μ.

As these clay deposits became dry land by the gradual land ascent, the formation of dry crust clay started.

When any soil dries out a shrinkage process will start. For soils with coarse particles the shrinkage will be small, but for materials like clay the shrinkage will be appreciable. The shrinkage in clay results in a volum change, the particles will be drawn closer together, and the various bonds between the particles will gradually become stronger. During this process cracks will develop and a common characteristic of a dry crust clay is a hard material with larger and smaller cracks in all directions.

The depth to which this process will occur will vary, normally from 2 m to 6 m in this country.

The mineralogy of the dry crust clay in Norway will vary somewhat, but an example is given in fig. 1.

Mineralogical analysis

Three samples of dry crustclay representative of the clay used in this investigation was analysed by the Norwegian Geotechnical Institute. Method of investigation: X-ray diffraction for the minerals and chemical analysis for the determination of iron-oxide.

Test no.	Quarts %	Chloride %	Illite %	Plagioclase %	Fe_2O_3 %	Not identified %
1	10	25	45	15	3.6	5
2	35	20	15	25	3.5	5
3	25	20	25	20	4.2	10

Fig. 1. Mineralogical analysis of three samples of dry crust clay.

Geotechnical data

Even if the variation of parameters from the dry crust clay may be very wide, it may be helpful to give some values of average properties.

Below is a list of the most common parameters encountered in these materials.

a) Grain size

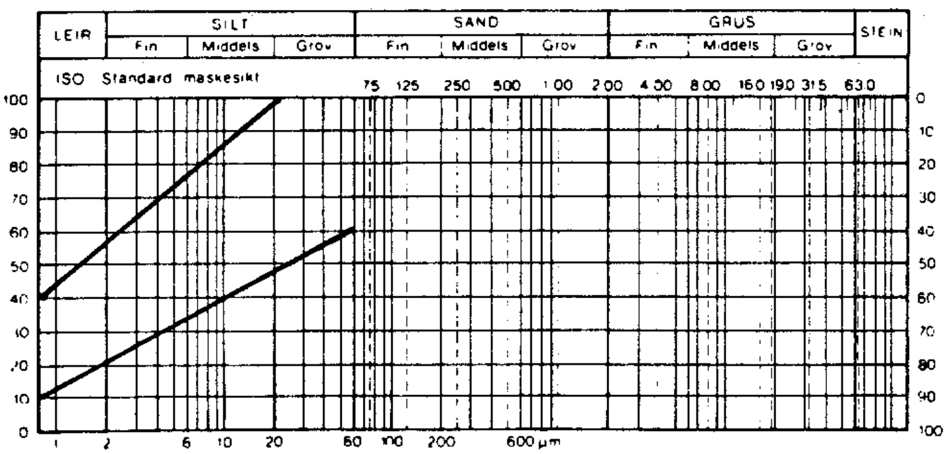


Fig. 2. Limits of grain size of common dry crust clay.

b) Atterberg's Limits

The plastic limit w_p normally varies from about 20 to 30% and the liquid limit w_L, from about 40 to 60%.

The majority of dry crust clay will typically have values of w_p/w_L 20/40 (%).

The natural moisture contents varies from about 20 to 40% and may well be higher in some instances.

Roughly speaking the natural moisture content of dry crust clay often varies between the plastic and liquid limits.

The position in Casagrande's Soil classification system of these materials can be seen in fig. 3.

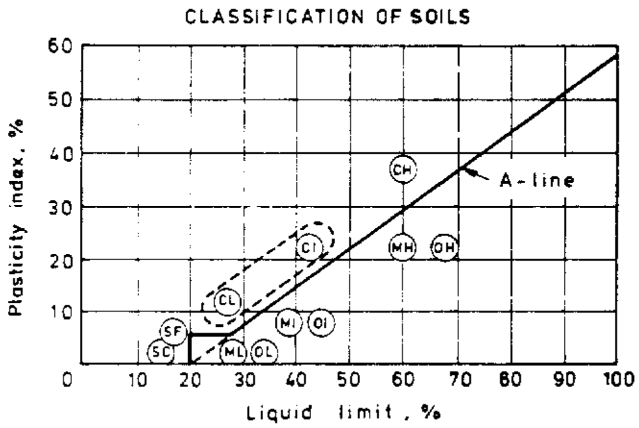


Fig. 3. Plasticity chart used in Casagrande soil classification.

In the extended Casagrande soil classification table the clay is termed «clays (inorganic) of medium plasticity, or clayey soils (inorganic)».

c) Shear strength

The dry crust clay may have very high undrained shear strength depending on the moisture content. Measured by vane, the values may range from more than 200 kN/m² down to 10 kN/m² or even less than that.

The extensive cracking of the dry crust clay will give problems on how to measure the actual shear strengths to be used in design as the permeability of the material may be governed by the cracking system and also the development of shear planes.

This brief introduction of the dry crust clay properties will serve as a platform for the further discussion of the use of this material in road fills.

The specification for the construction of a road fill of these materials are outlined below.

The various requirements are based on both experience and laboratory investigations and are a result of a period of about 15 years of experience.

A standard cross section of a motorway fill is shown in fig. 4.

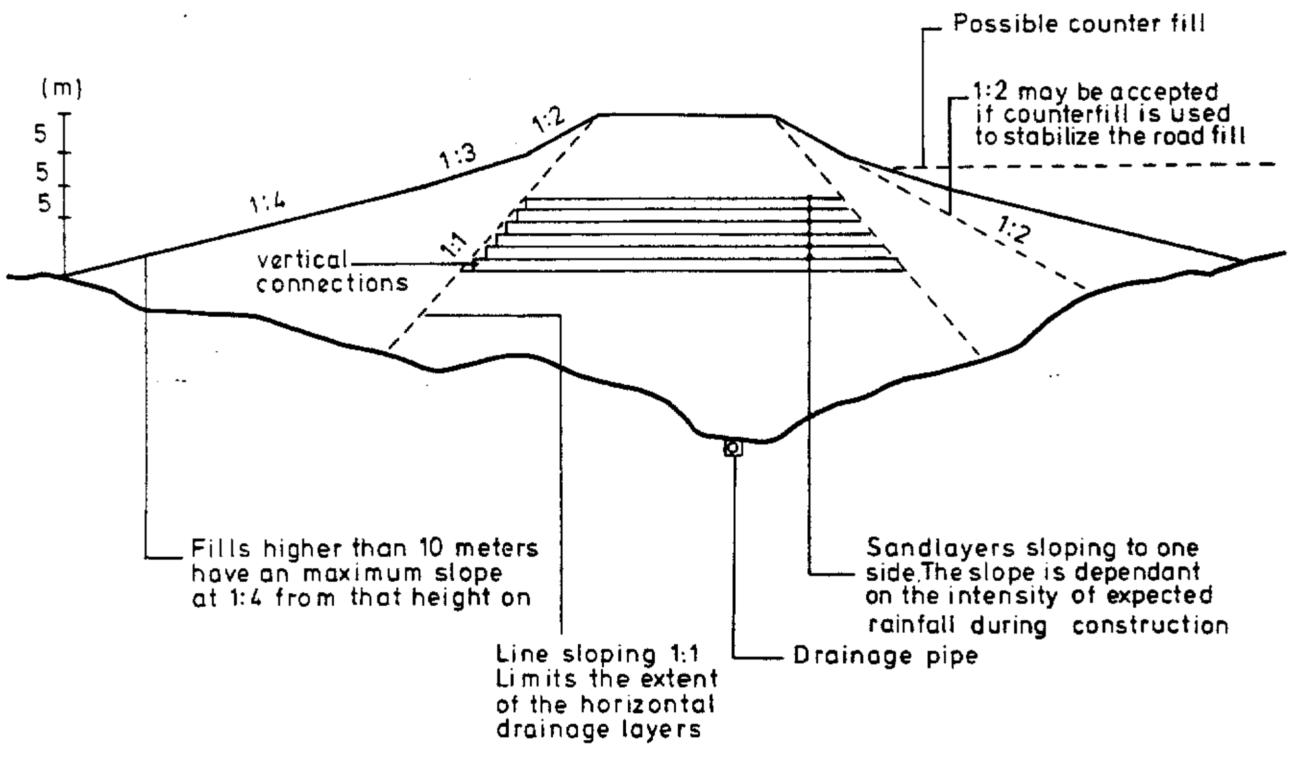


Fig. 4. Example of cross section of road fill.

The specifications call for thinckness of each compacted layer, usually 20 cm, moisture content and density control of a specified volume material planced in the fill. Usually these tests are carried out for each 200 m³ material compacted. An upper limit of moisture content is usually also stipulated.

The spacing and thickness of sand layers are also specified.

The specifications may also contain alternative causes of action if the weather is unstable, in the case of rain or snow, or if the temperature is low.

In the following a discussion will be made of the various types of measurements that have been registred over the past few years. The discussion will mainly be centred around pore pressures, general earth pressures, stability, settlements.

Pore pressures and earth pressures

Pore pressures in compacted clays are largely governed by the placement moisture content and subsequent loading and compaction. Generally speaking, if a wet soft clay is placed and compacted, subsequent layers of clay will induce pore pressures nearly equal to their weight, in the preceding layers. So if a layer of 1 m clay is placed on top of a soft clay layer, this induces a pore pressure of about 20 kN/m² in this layer. The wet weight of this type of compacted clay will usually be 19—21 kN/m³.

If the compacted clay is dry and unsaturated after compaction, the induced pore pressures from subsequent layers are a different matter.

Compaction may induce large negative pore pressures and added weights may only give small positive values and perhaps no positive values of pore pressure at all.

Such a clay has an artifically high effective stress level, and as the fill material gradually takes up water in its later life, this will cause a decrease in perhaps both strength and volume.

The negative pore pressures may be of a considerable magnitude. Values as low as 170 kN/m² have been reported. (R. E. Olson, 1965). These very low pressures were registered with just 5% dry of optimum moisture content.

N. V. R. L. N Rao, 1971, reported that no significant pore pressures developed in samples compacted near optimum moisture content. These experiments were performed on a clay not unlike the dry crust clay discussed in this paper.

The author performed series of tests on a dry crust clay and the conclusions from these tests were that for isotropic loading, the pore pressure parameter B varied as shown in fig. 5.

These findings confirm the picture that pore pressures in compacted clays are very sensitive to placement moisture content. The hypotesis may be put forward that roughly speaking the short term development of pore pressures on loading a compacted clay varies from B = 0 at the plastic limit, P_L, or optimum moisture content (OMC) and B = 1 at 1.3 — 1.4 P_L. The moisture content 1.3 — 1.4 P_L...... is often taken as a rough guide to the suitability of a clay as road fill material.

It seems that the value of B varies over a wide range with comparatively small variations in moisture content.

This is not making it an easy task to calculate the stability of such material, espesially on an effective stress basis.

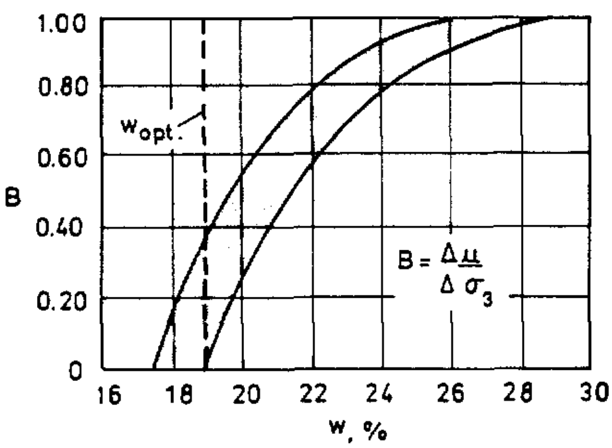


Fig. 5. Variation of B with moisture content or wet density.

Clays compacted in a road fill may vary over a wide range both in grading and moisture content. When performing any geotechnical calculations on such materials, this certainly must be taken into consideration.

Development of pore pressure during and after construction

Pore pressure measurements from two actual fills are presented and discussed. The first fill is a 12 m high motorway fill on the E 6 road, northeast of Oslo, profile nr. 23300. Counterfill is 130 000 m³.

On fig. 6 the arrangement of pore pressure meters and curves of pore pressure development can be seen.

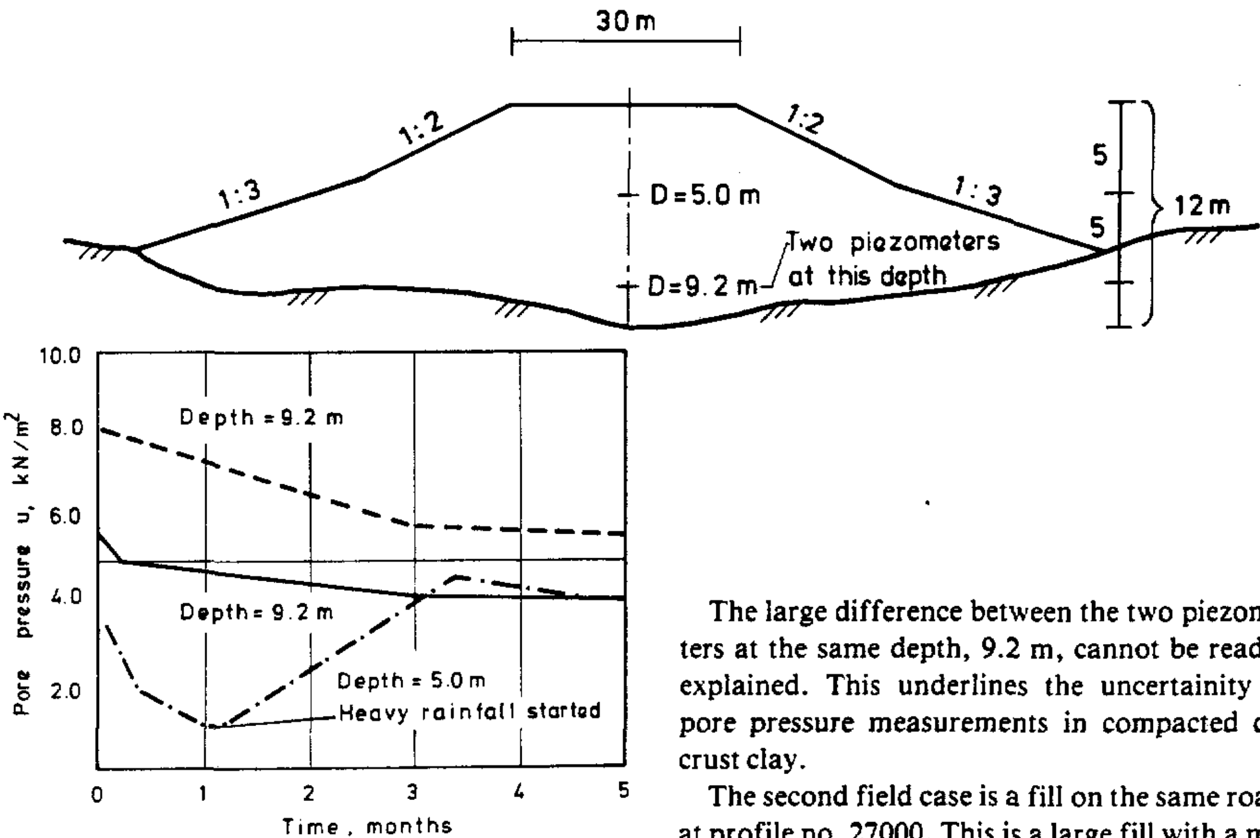


Fig. 6. Pore pressure measurements from three piezometers installed in a road fill.
Note the difference in values at the same depth 9.2 meters.

It is interesting to note that rainfall in fact has a direct effect on the pore pressure down to a depth of 5 meters.

The effect was immediate (few days) in this case and this means that comperatively large cracks at least, penetrate down to this depth.

The large difference between the two piezometers at the same depth, 9.2 m, cannot be readily explained. This underlines the uncertainity of pore pressure measurements in compacted dry crust clay.

The second field case is a fill on the same road, at profile no. 27000. This is a large fill with a maximum height of 26 meters. Total volume of this fill including counterfills and some landscaping amounts to 800 000 m³.

This fill was instrumented extensively with pore pressure meters and a specially designed steel cube was installed in order to measure both horizontal and vertical stresses.

The fill and instrumentation can be seen in fig. 7.

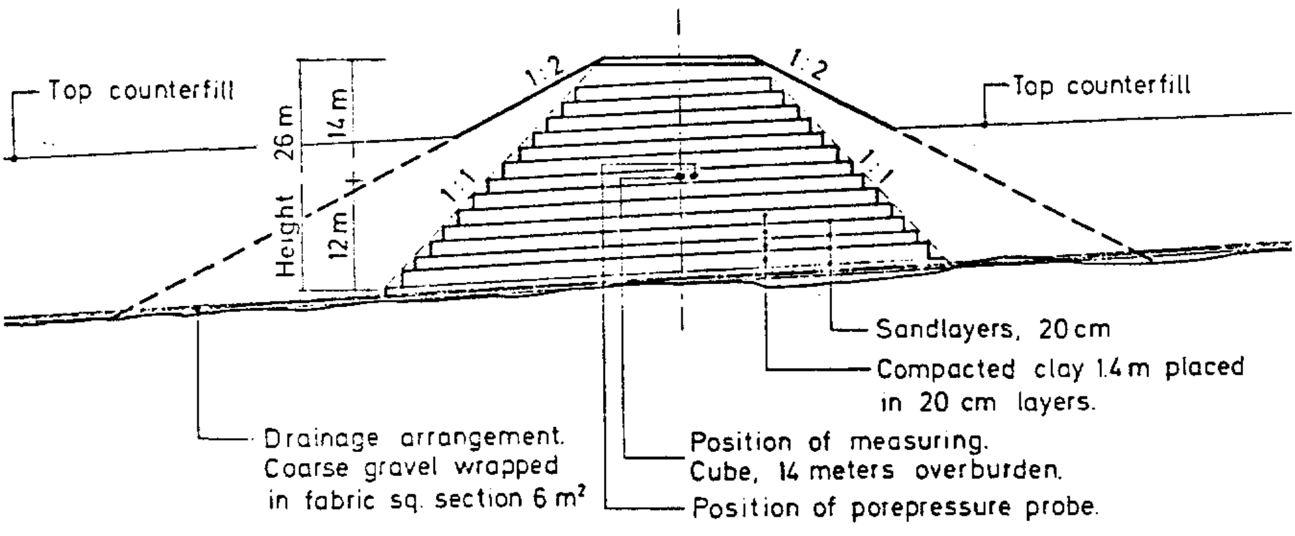


Fig. 7. Cross section of compacted fill with cube of the measurement of horizontal and vertical stresses.

The cross section of the fill is shown in the figure. The construction is of the sandwich type with 20 cm thick horizontal sand layers spaced at 1.40 m vertical distance.

These sand layers are used to keep porepressures down during construction and to increase the rate of settlement.

Materials in road fill: Mainly silty clay with an upper limit of moisture content w = 30%
w_p/w_L 20/40 (%), w_{opt} = 18—20%.

The horizontal stresses were not quite equal in both directions.

The stresses along the centre line were about 15 to 20% higher than the stresses across the fill.

This effect may be caused by lateral deformations.

The higher stresses along the centre line may also be caused by an increase in stresses due to consolidation, the valley has a very sharp V-form.

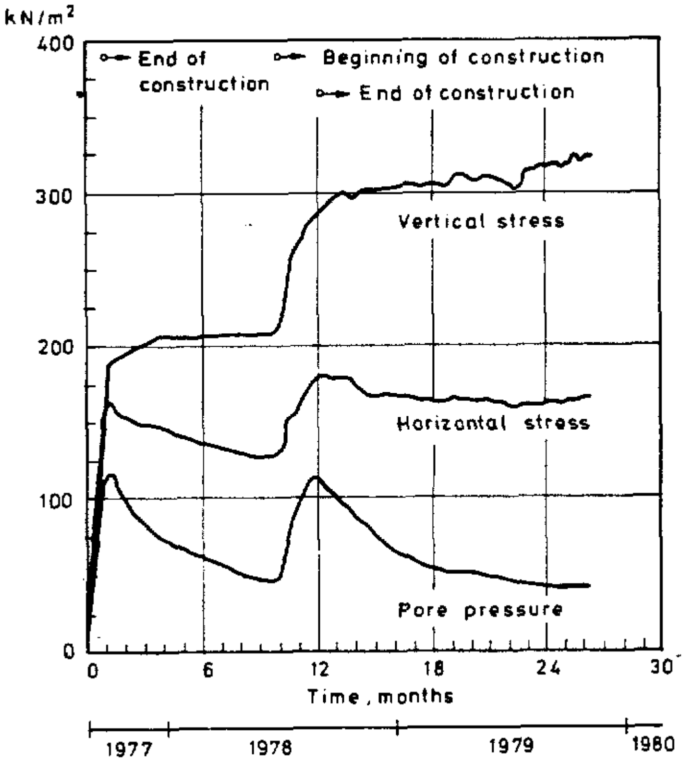


Fig. 8. Average stresses during and after construction of road fill.

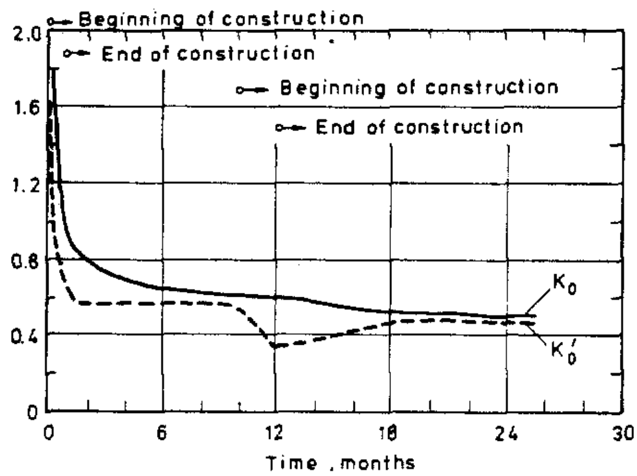


Fig. 9. Development of K_0.

The vertical stress increased roughly liniarly with the increase of the fill height and a similar behaviour is seen in the horizontal stress development.

When construction was stopped before the winter, the increase in both the horizontal and the vertical stress stopped. The steady increase in porepressure also stopped.

During the winter season very little work was done on the fill and the stresses are nearly stationary, the vertical stress increase slightly and the horizontal stresses and the porepressure decrease.

The dissipation of porepressure may be caused by the sand layers, lateral deformation or redistribution of pore pressure.

When work was resumed in summer (1978), all the stresses reacted immediately and the recorded values seem to be reasonable.

Note that the vertical stress increases more than the horizontal stress and that the general stress development changes sharply when the filling operation stops.

Fig. 9 shows the development at K_0 at various stages at the fill construction.

Efficiency of drainage arrangement

In order to keep the pore pressures down during construction a special construction of gravel wrapped in fabric was made. The principle can be seen in fig. 10.

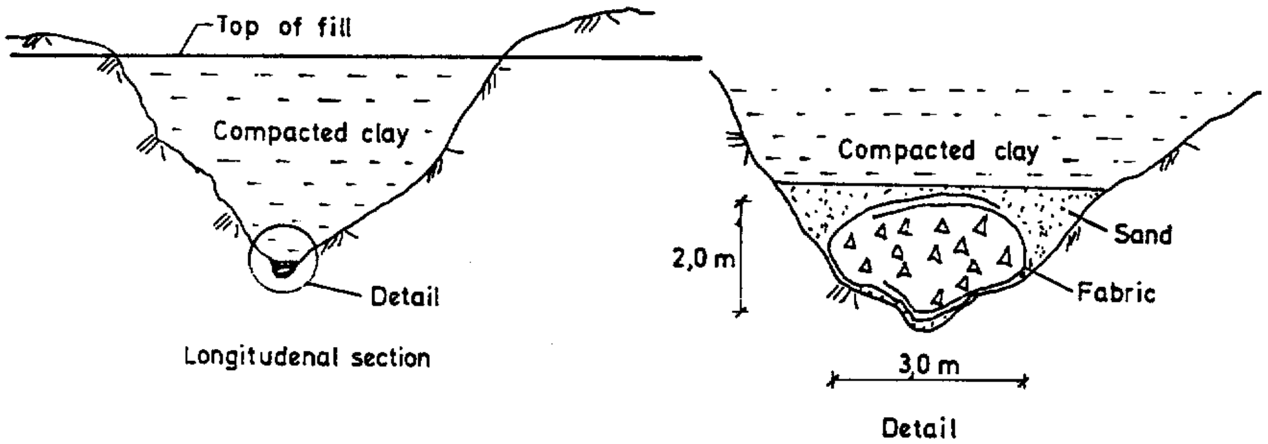


Fig. 10. Coarse gravel wrapped in fabric placed at the bottom of the fill. The purpose of this installation was to keep porepressures down both in the fill materials and in the adjacent natural ground.

The efficiency of this drainage arrangement was checked by installing pore pressure meters, and readings were taken at intervals. The result can be seen in fig. 11.

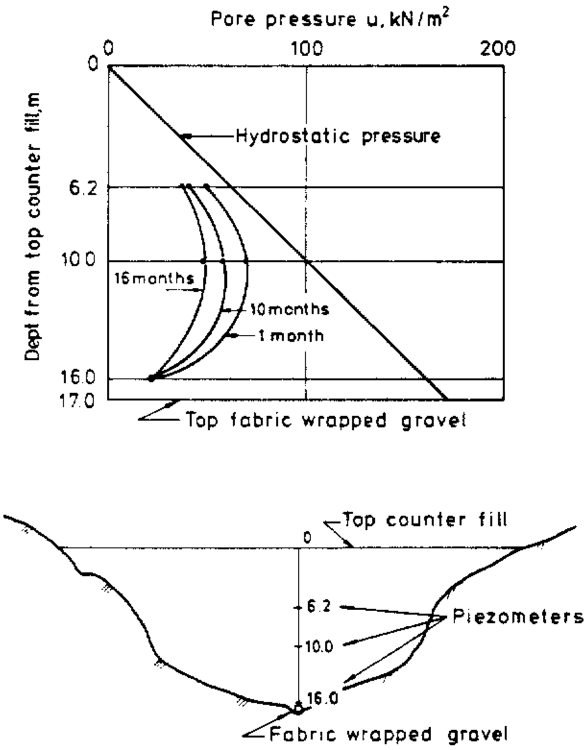


Fig. 11. Pore pressure / distribution in counterfill materials.

One question about the whole arrangement was whether the fibre wrapping would gradually become clogged.

So far, after about 2 years, the results are good, the pore pressures in the vicinity of the drainage arrangement are low and at the outlet, there is no evidence of transportation of fines to suggest erosion through the fibre wrapping.

The efficiency of the horizontal sandlayers is more complex.

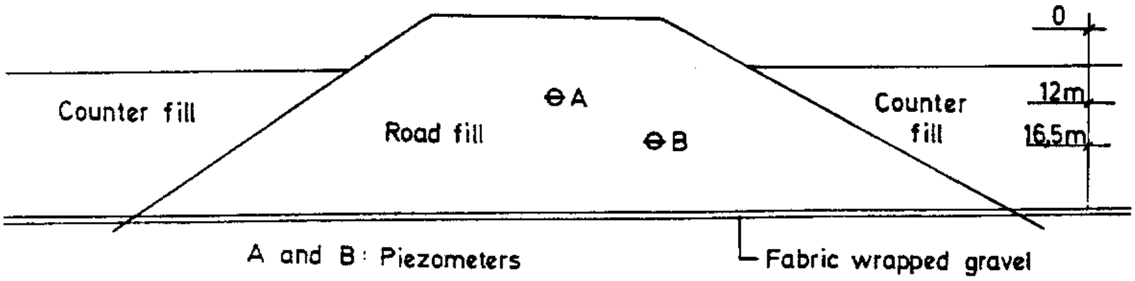
Looking at the pore pressure curve in fig. 8, the pressure is about 100 kN/m². The actual overburden pressure is about 300 kN/m².

The material in this fill was soft silty clay with moisture content ranging from 22—23% to more than 30%.

Looking back to fig. 5 it seems that values for B may range from about 0,8 to 1,0. As the induced pore pressure in fig. 8 is never higher than about 50% of the weight of the overburden, this suggests that some form of quick drainage is involved.

It is reasonable to assume that the rapid dissipation of pore pressure is an effect from the horizontal sand layers.

The future development of the pore pressure will clarify this point further.



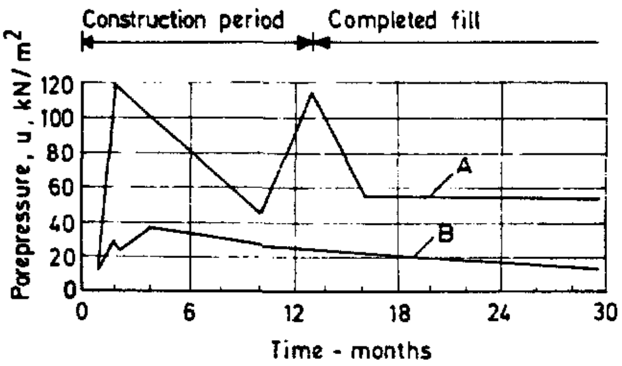


Fig. 12. Pore pressure development in road fill.

Stability

The stability of compacted clay embankments is extremely complex and there is no general theory available that readily can solve these problems.

The reason for this complexity arises mainly from the variation in compacted properties and the consequent difficulty of selecting the right parameters for performing calculations.

Before any larger road fill project can be undertaken there must be some preliminary calculations to show that the project is feasible.

Such calculations are made on the basis of knowledge of the parent material and specifications to ensure that the finished fill has certain minimum values of densities and controlled development of pore pressures.

The construction of a large road fill may take from months up to a year or two, and various incidents are almost ceratain to take place that were not originally foreseen.

In our part of the world there is the problem of low temparatures, ice and snow on one hand, and on the other hand, the summer may be very warm. Both these conditions may give rise to unexpected problems during construction.

The first condition with low temperatures, creates problems with compaction. Clay having some of its water frozen cannot be compacted to an acceptable standard.

The problem with high temperature is identical — dried clay forms lumps which cannot be broken down sufficiently to give acceptable compaction. The resulting open structure of this compacted clay may prove disastrous at a later stage when water absorption eventually will take place. In any case, the settlements in such a road fill would be quite unacceptable.

The leading principle in compaction can therefore be outlined as follows:

> A finished compacted fill should have a density close to what the parent material had when excavated, perhaps with not more than 2—4% air included.

To obtain the same density of the fill material as the material had when excavated will usually not be possible on a short term basis, some specified percentage of air has to be accepted.

In this paper the discussion of stability is restricted to the actual fill and not the stability of the fill and underground together. That problem is even more complex and at present such calculations are only giving rough indications of the magnitude of the factor of safety.

Stability of fill during construction

The stability of a fill during construction may be divided into two groups:

a) stability against bearing capacity failure, under wheels or tracks
b) overall stability of the fill itself

a) Stability against bearing capacity failure

As construction proceeds and if softer material is used, there will be an increasing problem both to place and compact the clay.

The use of machinery on wheels will have to stop and only tracked vehicles can be used. The material may be dumped on «dry land» at the start of the fill and small bulldozers may push the clay on to the fill and compact it in layers.

The problems with low bearing capacity will start in these materials at a moisture content w = 26—27%. At this point the undrained shear strength may be around 20—30 kN/m². The table fig. 13 gives some indication of the methods be used.

The prediction of bearing capacity during the fill operation is very complicated and cannot at present be calculated. These predictions have to be made based on experience from previous work with similar materials.

An important point in this connection is the skill and experience of the operators of the machinery, especially the operators of the wide belted bulldozers. When the material becomes so soft that only this type of machinery can be used both for placing and compaction, the entire fill operation is dependant on the ability of the operator and the general planning of the actual work sequences.

In order to try to give a rough idea of the equipment and methods used, a table is shown in fig. 13.

The values in this table are more a result form experience than from any calculations of bearing capacity and should only be considered as a rough guide.

The table only applies to the materials described previously in this report.

Dry crust clay of medium sensitivity

w%	S_{uv} kN/m²	Triaxial kN/m² at 15% strain	Types of equipment: Excavation	Transport	Types of equipment: Placing	Compaction	General remarks
20	250	180	Heavy dozer 30—40 t scraper	dozer scraper	dozer scraper	Heavy rubber wheel equip. 30 t·	Difficult to reach high enough densities
23	110	120	Dozer	»	»	Medium heavy rubber wheel (dozer) 15—25t	Heavy dozer *may* often be used for compaction
26	60	30	dozer 15—30t	dozer trucks	dozer LGP	Dozer as heavy as possible pref. 20t	Rutting becomes more and more a problem. Large deformations under belts.
30	20	10	back hoe dragline etc.	trucks	often very difficult LGP only	Dozer small 10t LGP	If conditions become impossibleadd drier clay or a sand layer and continue
30	10	—	»		not suitable in high quality fill	In counterfill this material may flow under a small gradient	Bearing capacity too low for any type of site equipment. May gain strength 100—300 % in a year or so

Fig. 13. Table based on experience of field operations and various equipment used in construction of road fills. Materials as described earlier in this report.

Some work has been done in this to connect the site construction problems with theoretical soil mechanics, but so far there seems to be more useful data from practice than from theory.

What is quite clear is that at present nothing can replace a field trial where combinations of various machinery and techniques are tried out. After such a trial the questions are not necessarily sloved, but the results will provide a good basis for further actions.

b) Overall stability of fill

This problem will be divided into two main groups: Short and long term stability.

To illustrate the short term stability problem and associated problems an example will be used.

In fig. 14 fill has been constructed up to a certain level when work has to stop due to rainfall. After rain has ceased, the fill is completed.

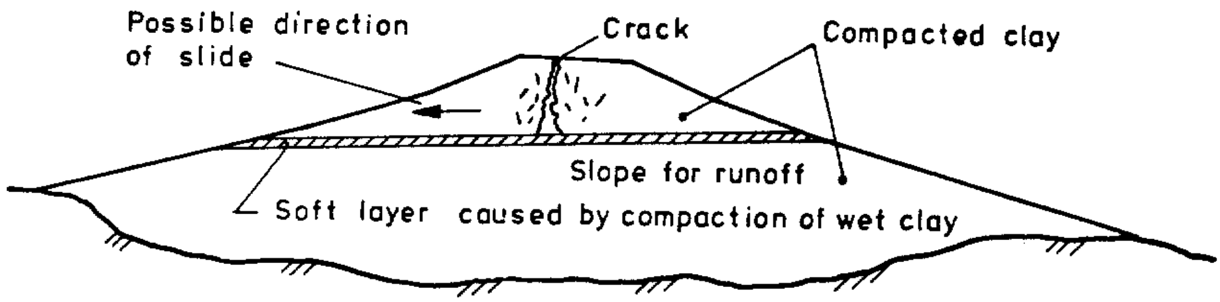


Fig. 14. Effect of heavy rainfall on stability on a short term basis.

Courses of action that will minimize the risks are:

a) After heavy rain, remove the soft layer or,

b) put in an extra sand layer on top of the wet clay.

c) Use clay with low moisture content directly on top of the soft layer or,

d) wait until the excess water has dried out.

If a calculation of the stability is to be performed, some assumptions have to be made.

Any standard proceduere can be used, as the errors made in the assumptions will more than likely outweigh the difference between various stability theories. See for instance John Lowe III 1967.

The assumptions made beforehand, can be checked during construction, for example by installing piezometers in any soft layer formed during construction. Subsequent readings will then either confirm or reject the basis for the stability analysis.

The presence of a soft layer in a fill may give rise to the forming of deep cracks. The horizontal stresses may be of a considerable magnitude in the early construction stage and horizontal movements of the compacted clay may be appreciable.

Looking back to fig. 6 it is clear that, with rainfall, the pore pressure reached immediately down to a depth of 5 meters. This does not mean that there is a crack down to that depth, but in design it would be reasonable not to rely on any strengths of the compacted material above this depth. This is not a rule in all cases for compacted clays, but it serves to illustrate that perhaps the cracking is more extensive than is generally assumed.

Another example of how instability can be brought about during or sometime after construction is shown in fig. 15.

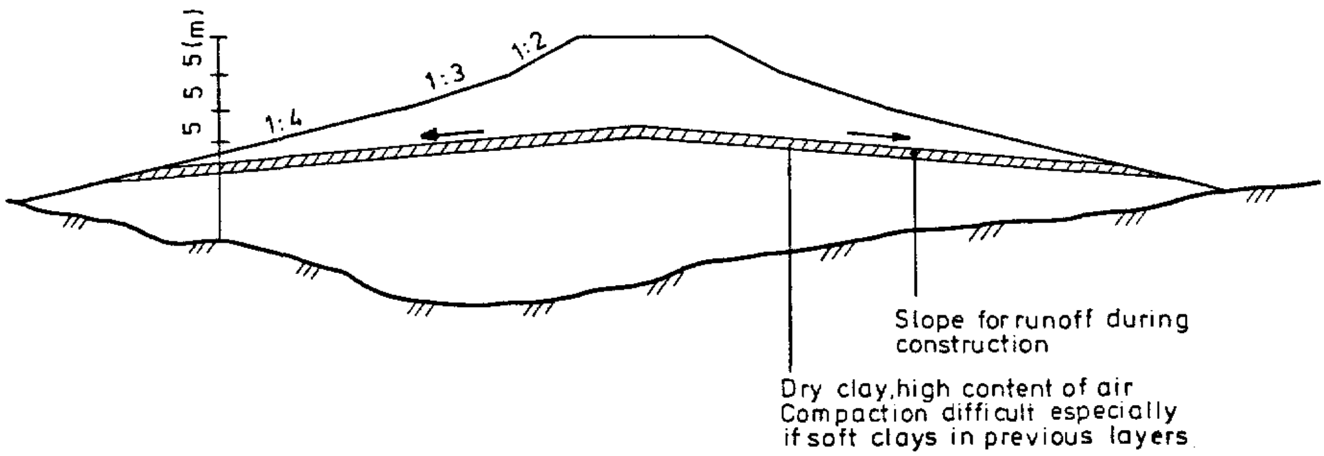


Fig. 15. Layer of dry clay in embankment. High strength when dry, radical drop in strength with volume change when wetted up in later life.

If a layer of dry clay is included in the fill, the compaction of this material is difficult especially if the previous layers are soft.

The result is a material which has a high apparent strength due to negative pore pressures, but also a high content of air.

If water enters this zone at a later stage a type of delayed failure might occur.

One question in this connection is: Is the gradual saturation caused by cappillary rise, evaporation and condensation sufficient to bring about such a situation? The answer to this is not clear at present, but the possibility of such a mechanism should be kept in mind and attention should be drawn to this, for example in geotechnical reports.

The intermediate or short term stability during or soon after construction should be treated separately for each field case. There is, however, a good reason for issuing a warning of not treating these problems only theoretically. The large numbers of variables which still have no place in theoretical soil mechanics, must also be taken into account.

One particular point may be of interest and that is the development of pore pressure on a long term basis.

The long term stability of a compacted clay fill is by no means less complicated than the short term one.

Some measurements have been done as can be seen in figures 6, 8 and 9.

If a compacted fill starts out with negative pore pressures, this «artificial» effective stress will dissipate as the clay gradually takes up water from the surroundings.

It is therefore to be expected that the stability of such a fill would decrease with time. On the other hand consolidation may take place in other and wetter parts of the fill, tixotropic strength gain will alsto take place and the overall result may be either positive or negative.

Other factors may also enter into these problems such as disruptions of old drainage paths in the original ground may give rise to water entering the fill and the clay. Erosion is another aspect. Freezing of a outlet of a culvert may give water access to the fill.

There are many other factors, but these may serve as an illustration that it is not necessarily the more academic aspects of soil mechanics that will govern the long term stability of a compacted clay fill. The problem is to combine the knowledge in soil mechanics and the effects any incidents in practice may have.

Settlements

To calculate the settlement of a compacted fill (not any settlements from the underground) is difficult for several reasons.

The various layers have different properties and the settlements will go on during construction.

From a road engineer's point of view the interesting part is the settlement occuring after the fill is completed. At present there is no procedure available even to forecast the total future settlement and certainly not the settlement rate.

So far, the only reliable source of information is actual measurement of settlements of completed fills.

Looking back at the settlement process in a compacted fill, this may be divided into serveral stages and some of these will be discussed in the following:

a) Elastic deformation

A certain amount of the total settlement is caused by elastic deformation. This occurs, however, directly on loading and will have no measurable effect after the fill is completed. The magnitude of this deformation is uncertain as the compression of air will play a part in this connection.

b) Air diffusion

When a clay containing air is subjected to stresses, the air will gradually go into solution with the pore water. The result of this process will be a decrease in overall volume.

This process takes a short time in relation to the construction rate and it is reasonable to expect most of this process to be finished shortly after the end of construction.

The statement that this process is a quick one and takes place during the construction of the fill, is merely an assumption of the author and is by no means experimentally verified.

c) Lateral deformation

Most compacted fills of soft clays will have some lateral deformations during and also sometimes after construction.

A slow lateral deformation after complection will cause a vertical settlement and mobilize strength gradually.

This is a situation not far from a factor of safety equal to 1 it seems, but in actual fact this may not be the case.

Compaction will introduce horizontal stresses which frequently are higher than the vertical stresses. Looking at the gradual dissipation of horizontal stresses this may be caused by slow lateral movements of the fill material. See fig. 8.

d) Consolidation

Consolidation is usually defined as the gradual expulsion of water from the soil, caused by an increase in effective stress.

Assume that a nearly saturated clay layer is placed in a fill, and as the next layers are placed the stresses gradually increase in this layer. Because the saturation and densities will vary over very short distances, the result is a redistribution of both moisture contents and densities and an increase in both the pore water and the pore air pressures. When all these stresses and densities

have readjusted sufficiently, consolidation may take place. One small volume of compacted clay may consolidate in the true sense of the word, by giving off water perhaps air, to a less saturated volume of soil nearby.
turated volume of soil nearby.

These redistributions of stresses, strains densities and moisture contents, will be a continuous process, and the net result will be a settlement. When a larger body of compacted material has an excess pore pressure, this will dissipate into the horizontal sand layers and also result in a settlement.

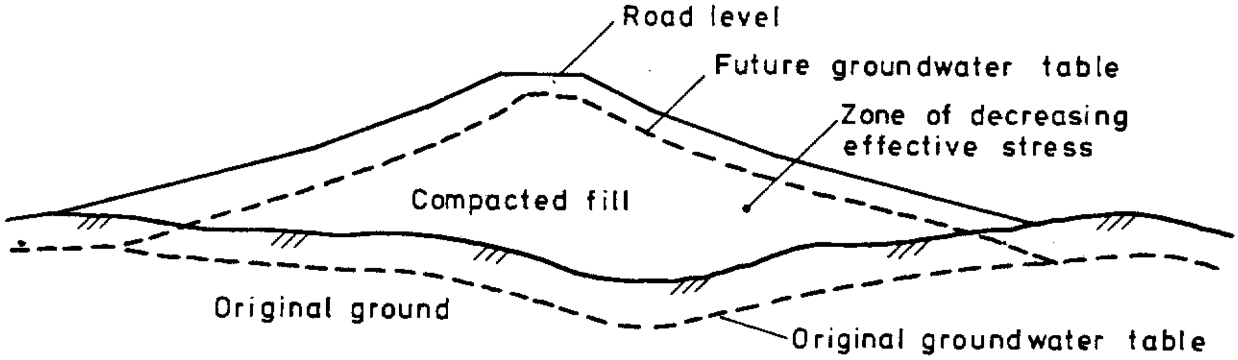


Fig. 16. Change of groundwater level after construction of fill.

Practical problems

Suitability of fill materials
Clay and silt materials excavated and used to construct road embankments have to meet certain specifications in order to be suitable for a particular site.

These specifications aim to take care of stability and settlements both during and after construction. Many places throughout the world the resources of rock, gravel and sand are diminishing and therefore there is a clear need to use materials that previously was considered unsuitable. Another point in this connection, even large embankments may in many instances be a competitive alternative to bridgebuilding. From both points of view the use of cheap fill materials will become more and more attractive in future. The construction of a large embankment may also facilitate landscaping for agricultural purposes.

Clay materials
It is convenient to divide these materials into three main groups; dry, intermediate and wet clays. It is possible to make a more «scientific» division by bringing in factors such as Atterberg's Limits, shear strengths, chemical composition and grading, but at this point in the discussion it is judged to be an unnecessary complication.

Dry clays
These group includes stiff clays forming hard lumps when excavated. In Norway this would be a dry crust clay a few per cent below optimum moisture content.

This material presents no problem either on excavation or bearing capacity during transport of the excavated material. The problem is compaction of this material and evaluation of stability on a long term basis.

The compaction problems arise from the difficulty of breaking down the hard clay lumps and producing a fill dense enough not to allow excessive water absorption.

This water absorption will cause a decrease in stability (effective stress) and this decrease cannot be allowed to endanger the safety of the construction.

Given time a compacted fill of dry clay material will almost certainly become saturated with water, as the new ground water table gradually will rise within the fill body. This wetting process may also cause settlements. This is especially serious adjacent to bridge abutments.

In order to compact this material properly the most efficient equipment in the writers opinion is heavy rubber tyred shovels. The use of heavy rubber tyred shovels (weight greater than say 20 tons) as compacting equipment seems to give good results on dry crust normally consolidated or slightly overconsolidated clay, but may be not so good on clays having a high overconsolidation.

The suitability of fill material of this kind is consequently decided by whether compaction can be done satisfactory or not.

The term «satisfactory compaction» is a somewhat vague specification and for practical purposes a specification curve is needed. The curve presented here is a result of both practical experience of what is possible and theoretical calculations of the various contents of solids, water and air. Furthermore some assumptions have to be made as to how later water absorption will affect both settlements and stability of the fill.

During the construction period, taking from a few months up to perhaps a few years, a compression of previously placed fill material will occur. This lowers the air content of the material before any water absorption can occur and consequently have a positive effect on the stability. On the other hand heavy rainfall sometime during the construction stage will leave a layer of soft and perhaps supersaturated clay layer in the completed fill. On subsequent placing of new layers of clay, porepressures will develop with a magnitude equal to the weight of the new clay layers. To remedy such a situation specifications may require such layers to be removed before construction proceeds, or for example call for an extra sand layer to be placed on top of the soft clay. This increases the consolidation and also excess pore pressure will dissipate more quickly.

The latter method may be preferable in many instances as only a thin sand layer (say 10 cm) would be necessary and besides, first placing a clay layer then removing it is one step forward and then one step back with loss of both production and time.

The problem of a wet clay layer in between unsaturated dry clay can also be looked upon in a different way. It may be argued that the excess moisture quickly will be absorbed by the drier clay above and below and therefore no special action is necessary. Whether to adopt one procedure or the other has to be left to decision of the soil mechanics engineer, as both practical experience and soil mechanics theory is involved in making such a decision.

There are numerous other practical details, that have to be taken into account when determining the suitability of fill material. This very brief discussion merely serves the purpose of illustrating the complexity of problems in this area of soil mechanics.

The specification curve for a particular type of marine, normally consolidated, Norwegian dry crust clay is shown at the end of this chapter.

Wet clays

By «wet clays» it is to be understood clays with moisture contents from 1,3 x P_L and upwards.

These clays are characterised by low bearing capacity, only small tracked tractors can be used for the actual placing of clay in layers in the fill. The low bearing capacity presents a number of problems like excavation problems and stability of haulage roads.

In addition to this a decision about whether this material is suitable or not must be made.

The problem is, as earlier stated, stability and settlement of the fill construction. Stability problems during and after construction should be checked by calculations of factor of safety by perhaps using various methods. Computer technique makes this comparatively an easy task. Far more difficult is the selection of representative parameters to feed into the computer and at present this is left more or less to the individual soil mechanics engineer.

For many reasons this is a highly unsatisafctory state of affairs and some attempts have been made to measure parameters in completed fills in order to try and get a clearer picture of what actually happens within a fill after completion.

These measurements have so far been consentrated on the following:

a) Development and dissipation of porepressures
b) Stresses during and after construction
c) Shear strengths during and after construction
d) Settlements.

The gradual collecting of these data has lead to a first attempt to present calculation of safety factors for various moisture contents of a particular clay. Calculation of settlements have also been attempted, but the real quality of these is doubtful. Settlements do not seem to be a serious problem in these types of compacted fill. The use of sand layers probably speeds up the settlement to such an extent that at the end of construction only small settlements will occur.

The suitability of a potential borrowpit material can therefore be judged on basis of a combination of past experience and theoretical considerations based on soil samples and testing from a future borrowpit.

Incidently, having stated that settlements rarely presents any problem in compacted fills, this refers only to the fill material itself. A high fill presents a considerable additional weight on the ground and this may of course give rise to settlements of considerable magnitude.

In road construction, however, one is fortunate in that crossing smaller valleys on fills, these valleys have often been eroded and consequently have some degree of overconsolidation in the clay deposits at the bottom.

•When excavating wet clays there is usually a haulage road problem. At least part of the transport routes will pass over excavated areas where the bearing capacity will be too low. In these instances special transport road has to be built.

Bark from fir and spruce trees placed directly on the soft clays in thinckness of say 50 cm with sand or gravel on top might be a practical solution.

In countries with low temperature during winter, the frost action will make it possible to locate the haulage roads nearly everywhere. The construction of fills during such low temperatures require a special organization and control of all operations on site. Even if this is difficult it can undoubtedly be done in practice.

In an attempt to summarize it might be said that having dealt with the technical aspects of stability and settlements for a particular project, the result should give a clear economic advantage if the fill istead of bridge is going to be constructed in practice.

The reason being that an economic forecast in bridgebuilding is usually more accurate than for a fill construction, working with clay materials is all dependent on weather conditions, during rainfall work has to be stopped altogether. The other very important factor is as mentioned earlier, the skill and experience of each operator on his machine. There is a general tendency to overlook this, that is the reason why it is mentioned several times in this report.

Conclusions

1. The use of «problem materials» such as silt and clay for road fill construction will increase in future. The diminishing resources of sands and gravels will make this an economic necessity.

   Combined with landscaping for agricultural purposes, this means that planning of large roadprojects will have to look at the projects in a broader sense than just on the road-traffic side.
2. The major problem in using these difficult materials lies in the lack of knowledge of the suitability of materials in the planning stage. The other great problem is connected to the behaviour of the completed fill both on short and long term basis.
3. The increased use of large road fills of silt and clay over the past few years, has facilitated the collection of important data from the field. Toghether with experience from various projects it would be possible to construct fills up to a height of say 40 to 60 m. The need of steadily using more and more difficult and soft materials will of course necessitate further development of both machinery and methods, as well as the soil mechanic theories.
4. The materials discussed in this report represents a real challenge to the operators at various types of machinery. Excavation, transport, placing and compaction are all operations that need careful planning and excecution. The soil mechanics engineer would benefit greatly in following this work in practice. He might then be able to make his design to a better fit than sometimes is the case today.

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Appendix:

Development of the use of clay fills in road works

Below is a record over some road fills of dry crust silt clays. A few data is supplied in the table to give an idea of the development of these constructions with time. All the fills are at the sandwhich type, the number totalling about 30.

Year of construction	Maximum height m	Volumes Main road fill m^3	 Counter fill m^3	 Total sum m^3
1963	10	—	—	12000
1964	15	—	—	15000
1968	15	—	—	21000
1971	16			26000
1978	19			200000
79	18	55000	60000	115000
79	17	35000	15000	50000
79	26	250000	550000	800000

Requirements of Geological Studies for Undersea Tunnels

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Summary — Zusammenfassung

Requirements of Geological Studies for Undersea Tunnels. The Norwegian coastline is dissected by deep fjords that create large problems for modern landbased transport communications. Although road tunneling has a tradition in Norway, and many tunnels have been driven under lakes and rivers, no road tunnel project in rock under a fjord has been studied seriously.

A project now under study may initiate a new phase in Norwegian road construction. Studies of a bridge project at Vardø on the Arctic Sea coast at the extreme North of the European continent, revealed such high costs that it was decided to look into an undersea tunnel project.

Studies of construction records from those of the Severn railway tunnel, built a hundred years ago, to the ongoing Seikan tunneling of today indicated that unforeseen difficulties tend to take place causing costs which exceed the estimates. It was realized that our present procedure for geological studies does not give sufficient guarantee against water inflow and cave-in.

A closer study of all information, possible methods and the costs of achieving them has therefore been undertaken. Firstly, mapping of the geology of the area is quite inexpensive compared to the amount of information given provided the rock is not covered by overburden. Echo soundings furnish a detailed contour map of the bottom topography, from which some geological interpretations may be drawn.

For mapping on the sea bottom frogmen geologists are employed, but the information obtained is dependent on the extent of exposed rock, and costs involved are reasonable.

Next, by means of acoustic soundings, the position of the rock surface and the thickness of the overburden may be determined with a good degree of accuracy, and at an acceptable cost.

Seismic measurements are useful to confirm the rock surface topography and to identify eventual low speed zones, but due to prevailing storms and heavy currents, such measurements are time-consuming and accordingly relatively expensive. Careful planning of the seismic profiles is therefore essential for good economy.

The only kind of investigations that provide exact and concrete information on the ground conditions are drillings. They may be started from land, bottom, platforms and barges. Drillings from barges and from land have been studied in particular. Geological mapping, acoustical and seismic soundings are just finished, and drillings will proceed in July 1977.

Anforderungen an geologische Studien für Unterseetunnel. In die norwegische Küste schneiden sich tiefe Fjorde ein, die für moderne Transportverbindungen an Land große Probleme bringen. Obwohl der Bau von Straßentunneln in Norwegen nichts neues ist und viele Tunnel auch schon unter Seen und Flüssen gebaut wurden, hat man sich bisher noch nie ernsthaft mit der Möglichkeit eines Straßentunnelprojektes in Fels unter einem Fjord befaßt.

Es ist möglich, daß ein Projekt, das zur Zeit näher untersucht wird, den Beginn einer neuen Phase im norwegischen Straßenbau darstellt. Untersuchungen haben

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ergeben, daß ein Brückenprojekt bei Vardø an der Küste des Nördlichen Eismeeres im extremen Norden des europäischen Kontinents mit so hohen Kosten verbunden ist, daß beschlossen wurde, die Möglichkeit eines unterseeischen Tunnelprojektes in Erwägung zu ziehen.

Untersuchungen der Bauunterlagen für den vor hundert Jahren gebauten Severn-Eisenbahntunnel bis zu den derzeitigen Bauten am Seikan-Tunnel lassen darauf schließen, daß immer mit unvorhergesehenen Schwierigkeiten zu rechnen ist, wodurch die Kostenvoranschläge überschritten werden. Es wurde erkannt, daß unser gegenwärtiges Verfahren für geologische Untersuchungen nicht genügend Garantie gegen das Einströmen von Wasser und Einbrüche bietet.

Deshalb wurde eine nähere Untersuchung aller Informationen sowie der möglichen Methoden und der damit verbundenen Kosten durchgeführt. Zunächst kann die Geologie des betreffenden Gebietes im Verhältnis zu den vermittelten Informationen auf ziemlich billige Weise kartographiert werden, sofern das Gestein nicht mit Abhub bedeckt ist. Durch die Echolotung erhält man eine genaue Höchenschichtenkarte der Bodentopographie, aus der verschiedene geologische Schlüsse gezogen werden können.

Zum Kartographieren des Meeresbodens werden Froschmann-Geologen eingesetzt aber die dadurch erhaltenen Informationen hängen davon ab, wieviel Gestein bloßliegt. Die damit verbundenen Kosten liegen in annehmbarem Rahmen.

Jetzt kann durch akustische Lotung die Position der Gesteinsoberfläche und die Dicke des Abhubs mit ziemlicher Genauigkeit und zu annehmbaren Kosten festgestellt werden.

Seismische Messungen sind zur Bestätigung der Gesteinsoberflächen-Topographie und zur Feststellung eventueller Zonen für niedrige Geschwindigkeit sehr nützlich, aber aufgrund der herrschenden Stürme und starker Strömungen nehmen solche Messungen viel Zeit in Anspruch und sind daher auch relativ teuer. Eine sorgfältige Planung der seismischen Profile ist daher für gute Wirtschaftlichkeit unerläßlich.

Die einzigen Untersuchungen, die exakte und konkrete Informationen über die Bodenart liefern, sind Bohrungen. Sie können vom Land, vom Boden, von Plattformen oder von Kähnen aus begonnen werden. Bohrungen von Kähnen und vom Land aus wurden mit besonderer Aufmerksamkeit studiert. Die geologische Kartographierung sowie die akustischen und seismischen Lotungen sind jetzt aber abgeschlossen und mit den Bohrungen wird im Juli 1977 begonnen.

Background

The population in Western and Northern Norway is concentrated along the coast. The coastline is dissected by the deep and long fjords and has high mountains rising from the shores. Traditionally communication has taken place by sea. This situation has created large problems for the modern landbased transport communication. To meet the traffic demands, about 200 ferry routes are now in service on our National Road Network (Riksveger), which comprises 25000 km of main Public Roads. The ferries serve a total distance of 2500 km of the National Roads. This service is very expensive and is subsidized by the government. Work has therefore been going on to replace the ferry routes by roades and bridges. Up to now, about 7500 bridges are in service on our National Roads, some of the newest concrete bridges have spans of up to 160 m and there are suspension bridges with main spans of 525 m. The remaining ferry routes on the National Roads require very large investments if they are to be replaced. Some of those most urgently needing replacement are at Vardø, Tysfjord, Sannessjoen, Kristiansund and Oslofjorden at Drøbak (Fig. 1).

Possibilities for Undersea Rock Tunnels

The idea of constructing undersea tunnels in rock instead of bridges is, of course not new. A milestone in the construction of such tunnels was reached with the construction of the 7 km long Severn Railway tunnel completed in 1886. At present the Japanese are constructing the 54 km long Seikantunnel under the Tsugaru Strait between Honshu and Hokkaido.

Road tunneling has a tradition in Norway, and on the National Road Network, now in service, there are about 300 tunnels with a total length of 150 km. Because the tunnels have less than 3% concrete lining, very little ventilation and good illumination, they are relatively inexpensive.

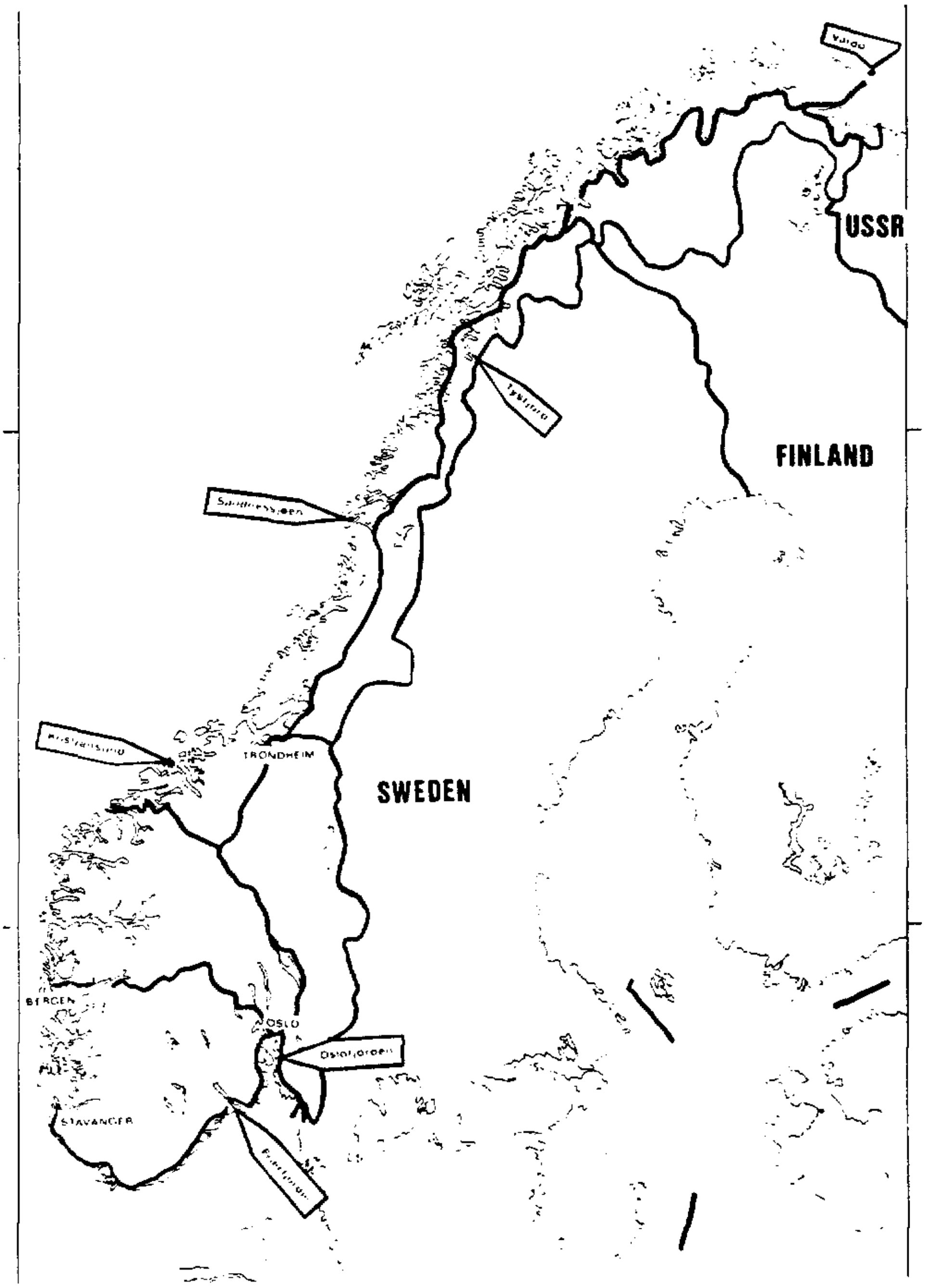

Fig. 1. The undersea road tunnel projects discussed are at Vardø, Tysfjord, Sannessjøen, Kristiansund and Oslofjorden

Die diskutierten Untersee-Straßentunnel-Projekte sind: Vardø, Tysfjord, Sannessjøen, Kristiansund und Oslofjord

Undersea road tunnels in rock have also been considered, but never studied seriously, mainly because of uncertainty as to the consequences. Construction problems, demand for waterproof lining, and expensive installations might make this design less economically attractive. On the other hand, large investments could be saved provided such tunnels could be constructed for prices of the same order as our normal road tunnels. Many types of tunnels other than road tunnels, constructed in rock under fjords, lakes, rivers and water reservoirs indicate that this may be feasible. For example a 5 km long rock tunnel under the 3 km wide Frierfjord has recently been completed with very little rock reinforcement and water proofing other than pre-injection.

The Norwegian fjords are normally very deep. The largest fjord, Sognefjorden, is 180 km long and 1200 m at its deepest point. However the continental shelf is about 200 m deep in areas close to the coast except for a trough just outside southern Norway. For a long time it has been known that the fjords have their shallowest parts at their mouth areas (Fig. 2). Accordingly, the Sognefjord is about 200 m deep where it finishes at the coast.

The shallow mouths of the fjords have been attributed to the existence of terminal moraines. Recent studies indicate to the contrary that the soil cover is relatively thin. If this is correct, some of the fjords may be crossed in undersea rock tunnels at their outer parts. This may be the case at Tysfjord which constitutes one of the two ferry connections left on E6, the main road to Northern Norway. Plans to eliminate this ferry connection are now under way, and a normal solution calls for 80 km of new road, including 30 km of tunnels and three very large suspension bridges. The fjord is very deep, but at its mouth rock may be found at a depth as shallow as 200 m. If this is correct, the whole normal project may be replaced by 6 km of undersea tunnel and about 20 km of new road.

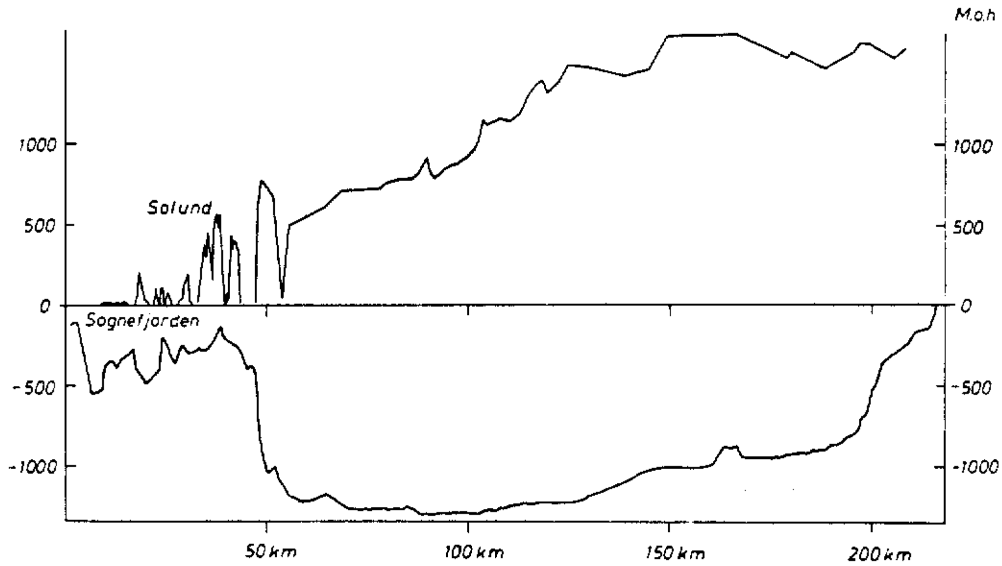

Fig. 2. Longitudinal section of the Sognefjord, showing the treshold so common in many Norwegian fjords. The level of the land area north of the fjord is also shown

Längsschnitt des Sognefjordes mit der Schwelle, welche in vielen norwegischen Fjorden angetroffen wird. Man erkennt auch das Niveau des Landstriches nördlich des Fjordes

The Vardø Project

The city and port of Vardø is situated on the island of Vardø lying in the Arctic Ocean just outside the extreme north of the European Continent.

The city has 3000 inhabitants, but is an important sea port, as well as a centre for fisheries and sea communication. Because of prevailing storms the road on the mainland is closed on an average of 12 days a year, and the ferry connection to Vardø even more often. A new breakwater has been constructed on the mainland, and the future expansion of the city is planned here.

A road project for replacement of the ferry has been studied for 20 years. From the beginning, cost estimates for a bridge were so low that the prospect of a bridge connection was accepted as a matter of course. Cost estimates made recently have, however, revealed extremely high prices for a bridge, partly because of poorer ground conditions than anticipated, and partly because of the soaring cost situation. Consequently the Authorities began to look for a tunnel project in 1977. The Authorities have decided to make a choice between tunnel and bridge in the beginning of 1978, and construction will start to take place in 1979 (Fig. 3). .

If an eventual tunneling at Vardø is successful, this experience may initiate a new phase in Norwegian road construction. The possibility of

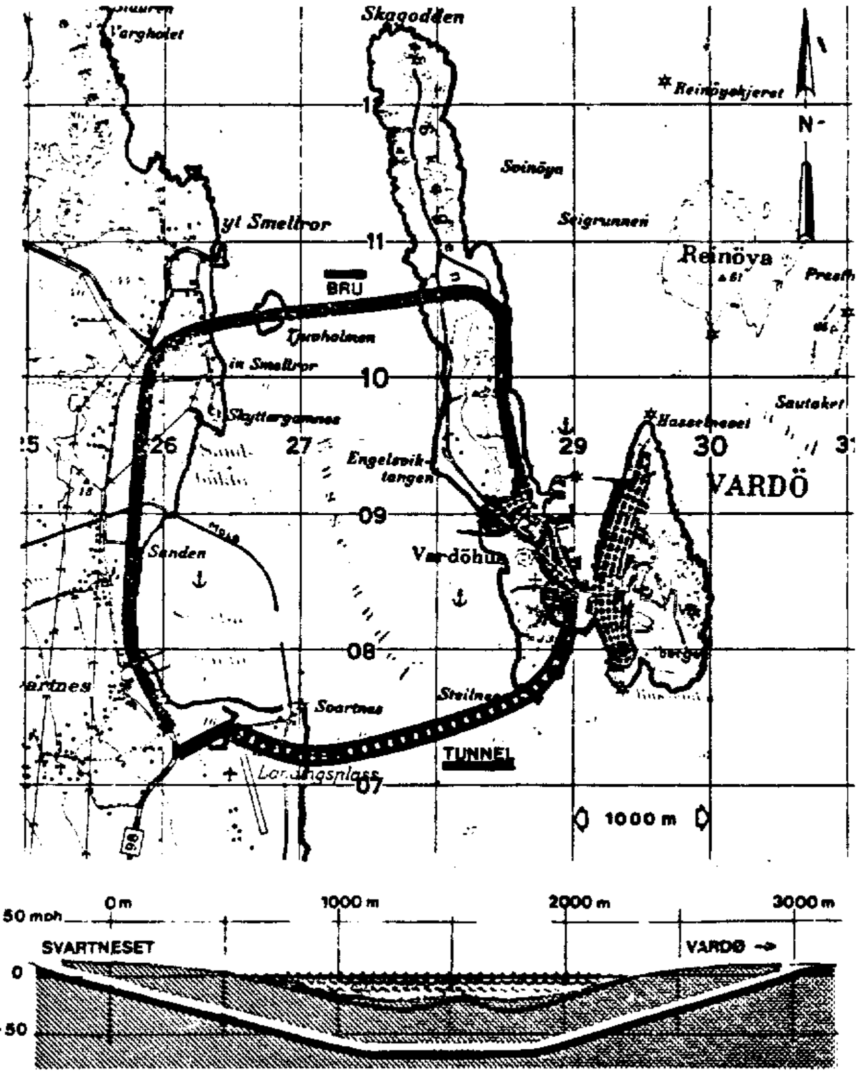

Fig. 3. The Vardø tunnel project
Das Vardø-Tunnelprojekt

constructing undersea road connections as reasonably as road tunnels on land will be received with great interest by the many small communities in areas waiting for a road connection.

Required Studies

Studies of undersea rock tunnels should of course include all aspects of the project such as installation, maintenance and service. In the following outline, only the scheme of the geological studies, consequences of tunneling and plans for investigations during driving are dealt with. In the Vardø case, a tunnel project should have a cost estimate that does not exceed that of the bridge project. In addition it should offer some advantages because it is a new design considered by many to be somewhat risky. Consequently, at the planning stage, all risks involved in the project should be forseen and measures to meet them be worked out. This situation enforces strict requirements on the studies, and extra care must be taken to make an account of the project as complete and carefully worked out as possible.

Firstly, the studies must give a reliable outline of the position of the bedrock, because it is the deepest lying surface of the bedrock that determines the depth, and consequently the length, of the tunnel.

Secondly, a great deal of attention must be paid to detect the presence of any gaps in the rock surface. Even though geological studies indicate even and homogeneous topography, experience has shown that extrapolation of geological information has had some shortcomings. In the case of undersea tunneling in rock, driving unexpectedly into overburden which is filling a gap in the rock surface would be catastrophic. Thirdly, stability of the rock is essential for the tunneling costs, and in undersea tunnels special care must be taken to avoid cave-in and to prevent a situation that cannot be controlled.

Fourthly, the permeability of the rock should be mapped to decide the extent and method of water inflow control during tunneling and a method of permanent leakage prevention and waterproofing. Normally the upper crust of the bedrock is weathered and permeable, and it gradually changes downwards to more sound and less permeable rock. Consequently a tunnel depth must be found at which the tunnel length is weighed against costs of water inflow prevention measures.

Location of the Bedrock

The rugged topography of Norway make it unreliable to assess bedrock depth from soundings or echo soundings for a judgement of an undersea tunnel project. The physical contrasts between bedrock and overburden is normally so sharp that the accoustical soundings have supplied quite reliable data on the thickness of the overburden and the depth of the bedrock.

Deviations tend to appear in areas where the bedrock surface slopes steeply, or where overburden is thick. Consequently, accoustical soundings should supply reliable information on average bedrock depth. The accoustical soundings are therefore useful at the early planning stages when only rough estimates of trace and costs are needed. When the amount of information given by the accoustical soundings is considered, the expenses are low.

More reliable information has to be supplied by other, more exact, methods. Seismic refractive measurements is a step further in this direction, firstly because the equipment is stationary and a more exact methods of location have to be applied. Experience from measurements taken on sea bed in the Vardø area reveal that in areas with sloping rock surface, seismics have resulted in larger depths to bedrock and thicker overburden than the accoustics. It is therefore unjustified to rely only on these methods.

Confirmation of the results are required at least in selected areas. However, the method give a good basis for working out a drilling programme.

Vertical drillings are normally used for a reliable verification of bedrock depth. Such drillings may be accomplished from barges, platforms or from special drilling rigs that are made for underwater operation from the bottom. At great depths, drilling from the bottom is the only alternative. Such equipment exists but operational records are scarce. It is believed that this procedure is expensive and dependability still uncertain.

Drilling from platform is favourable in shallow seas and the drilling may proceed with little influence form currents and storms. Removal of the platform from one point to the next requires better weather. Drilling from platforms has an important advantage in that inclined drillings are possible, and that closely spaced holes are drilled quickly without moving the platform.

Drilling from barges is normally the least expensive in shallow and sheltered waters. Drilling from barges has been accomplished even at Vardø this year at sea depths of 25 m and where the current goes up to 3 knots, and where storms may be strong. When the waves became higher than 1.5 m, the drilling had to be stopped.

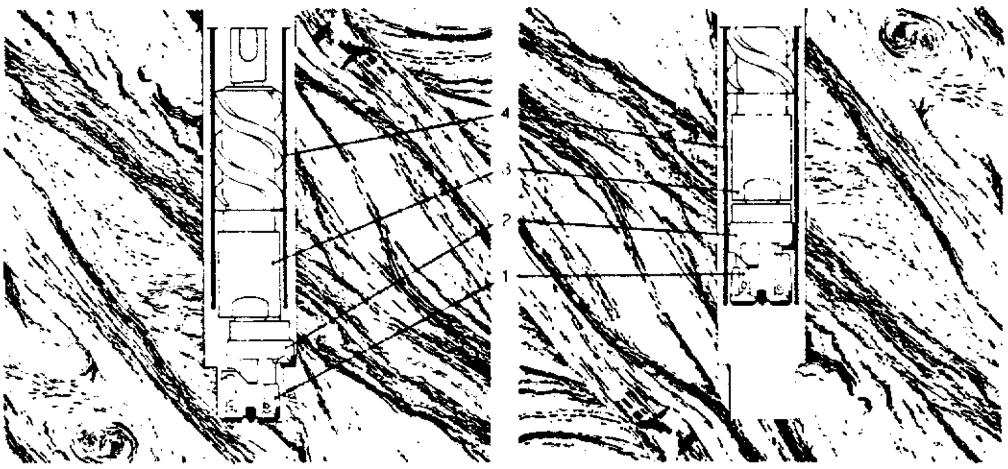

Fig. 4. The Odex drilling method. Drilling position at left, retraction at right
1. Pilot bit, *2*. reaming bit, *3*. steering, and *4*. casing (2)

Das Odex-Bohrverfahren. Links: Bohrposition, rechts: Einziehen
1 Stufenmeißel; *2* Nachnehm-Bohrmeißel; *3* Lenkung; *4* Gehäuse (2)

Experience has demonstrated that the drilling has to be accomplished with casings of 10", 5" and 3" to avoid breaking the drill rods. The 3" casing is provided with the Odex bit, an eccentric rotating bit that may penetrate rock so that the casing is tied 2—3 m in the rock (Fig. 4). Through this casing normal percussion or core drilling was undertaken. The rock surface was determined by observing the drilling and by automatic recording of drilling speed, rotation speed and feeding pressure. The rock was drilled to a depth of 10 m and a core of 1 m length was taken at the bottom of the holes in good rock. The rate of drilling gives some indication of rock strength when feed pressure is taken into consideration. Variation in the rate of drilling held together with the variation in rotational speed indicates joints in the rock. For every 3 m, some of the drill mud was sampled.

Mapping Rock Stability

Progressive rockfall and cave-in is not common in Norwegian tunneling, because the bedrock is formed by sound crystalline rocks or old, competent sedimentary rocks. Some faulting occurs, and faulted or strained zones with heavy jointing may cause instabilities.

Regional geologic maps give indications on the main structures in the bedrock. More detailed studies are required to locate rock quality and zones that may be unstable. The stratigraphy and rock types classified in units pertinent to the project should be carefully drawn on detail maps. Areas classified in terms of frequency of fracturing should also be indicated, and diagrams showing fracture directional statistics be worked out.

In the Vardø case, frog-men geologists detected very few outcrops of bedrock. Consequently information on geology could mainly be extrapolated from outcrops on land and from regional studies.

From the geological mapping, prospects of rock quality may be extrapolated. More detailed studies are necessary to obtain the information needed. Held together with the seismic information, the outlined geology gives a good basis for working out a drilling programme that will supply a maximum of information from a minimum of effort.

Mapping Rock Permeability

Water leakages constitute the most expensive feature in Norwegian road tunneling: The rock permeability, or by a more precise term, rock conductivity, have to be paid special attention dealing with an undersea tunnel project. In most road tunnels, leaks originate from limited sources. Water pressure decreases after tunneling and water inflow is stabilized or become periodical when the ground water reservoir is partly drained.

In undersea tunnels, water pressure must be expected to be the same as in underland tunnels, and dependent on the distance to the water table. However, water inflow must be assumed to be constant or even increase with time. Tunneling costs are therefore very much dependent on the type and extent of the water-proofing. A prospect of the leakage conditions is essential for a well prepared driving and water-proofing method.

Improved driving efficiency may be obtained in areas where no water proofing measures are needed, and therefore information on sections with large inflow hazards is needed at the earliest possible stage during planning. Water pumping or pressure tests undertaken in drillholes in the rock provide

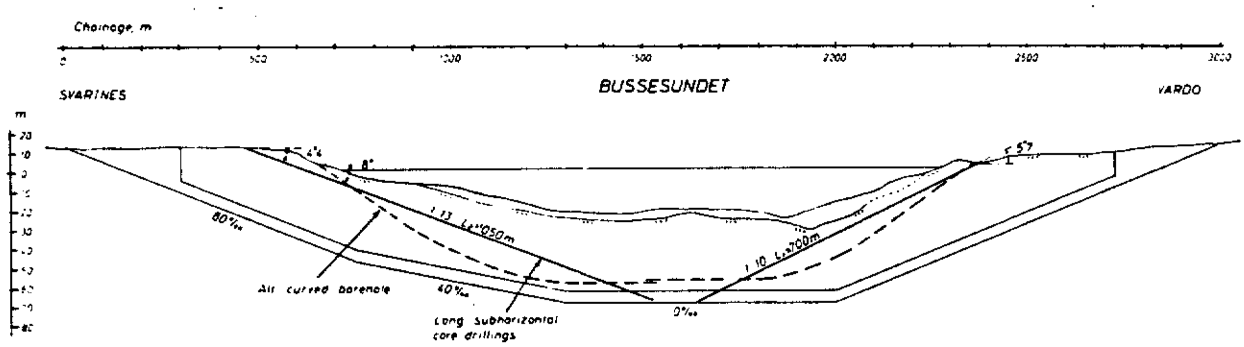

Fig. 5. Possible courses for long exploratory drillings
Möglicher Verlauf für lange Erkundungsbohrungen

a good indication of permeability and conductivity. An average estimation of the permeability may be obtained by performing such tests during the vertical drilling programme. By studying all the information from the investigations, and extending the drilling programme a good average idea of the leakage conditions may be reached.

Detection of Gaps in the Rock Surface

Normally, seismic measurements should give some indication of marked topographical features buried below overburden. However, the method has its limitations, for instance in the detection of narrow clefts or clefts directed at small angles to the seismic cable.

By performing vertical drillings, chances of striking such features are very small, even though drilling on seismic indications. The only way of obtaining complete and dependable detection of such clefts is by inclined or horizontal drillings. Inclined drillings may be executed from the shore, islets, platforms or by underwater drillings rigs. If possible, the best answer is to drill long holes in the rock above the projected tunnel. A study on the possibility of this has recently been undertaken. There seem to be few documentations on such drillings. Core drillings with length of 1000 m have been recorded, but the critical factor is the drilling deflection.

In Vardø case, two drillings are required, each of 1000 m from the opposite shores (Fig. 5). To be of use, the drillings should have a total deflection of less than 20 m. Indications have been received from drilling companies that this might be possible.

To be successful, such drillings have to be controlled by deflection measurements and course corrections. Many factors tend to deflect the drilling from keeping a straight course. These factors originate from gravitation, rock structure, equipment and drilling procedure. Normally, gravitation tend to deflect a 1000 m long borehole 10—20%. Possibilities are good for controlling deflections caused by equipment and operation, but controlling the influence from gravitation and rock structure may prove to be difficult. Several methods have been applied or suggested to direct the drilling in the wanted position.

The simplest method is to map the deflective tendency by drilling, and start new drillings in the most favourable direction. At the same time the deflective capability of equipment and operation should be adjusted to counteract the deflection from gravitation and rock structure. If deflection still becomes to large, the course may be corrected by installation of wedges that force the bit in a more favourable direction.

The most sophisticated method is to apply guided mud-turbine drilling machines.

Long, horizontal drillings with directional control are expensive. However, the quality of the rock may change over very small distances, so that the closer the drilling is performed to the projected tunnel, the more reliable the results. Shorter and less expensive drillings may of course be undertaken at intervals from the tunnel face. At that time little can be done to avoid bad ground conditions.

Driving Method

The purpose of the exhaustive studies of the project is to arrive at a cost estimate and a scheme of the driving conditions that is reliable. At

Seikan and Severn the situation was different in that the projects had to be completed whatever the costs, and that driving conditions had to be found in front of the working face because a thorough study done beforehand was insuperable. In such cases a driving method must be undertaken that includes drillings and special operations to prevent disasters.

In the Seikan project the hazard measures are threefold, firstly by driving two pilot tunnels, secondly by continuous exploration of the rock ahead of the face by drillings always undertaken 350 m ahead of the working faces, and thirdly by continuous pre-injection of 70 m long sections. In addition, water gates are constructed at certain intervals. Even with this procedure severe water inbursts have happened four times.

A procedure like this is timeconsuming and expensive. For this reason, the Norwegian undersea tunnel projects have to include a minimum of operations other than driving. On the other hand, a complete and perfect knowledge of the ground conditions is not obtained until the project has been completed. The studies of the project supply assurance of the ground conditions that is increasing with the efforts, but the last pieces of information are going to be extremely expensive. It is consequently reasonable that the driving method should include some sort of exploration and/or security operations.

The risks involved in the project are those of water inbursts and striking into the overburden. It is important to find a scheme of drilling in front of the face that will in the least possible degree interfere with the driving; for instance, drilling every week-end. It may be difficult to detect all zones of water inburst by drilling only a couple of holes in front of the face. The drilling of about 100 holes in the round would therefore serve as a last check on water leakages. When such water conductive zones are found, preinjection has to be accomplished immediately. In order to drive through overburden, freezing machinery should be on hand.

There may be a more simple driving method, at least of sections in the best quality rock. In case of cave-in or inflow, and when the inflow exceeds the pumping capacity, it will probably be very difficult to have work done by divers at a water depth of 60 m at the face of a 1 km long tunnel. If however, a transportable freezing machine could be placed at the face, it could freeze a clog of ice and permit water to be pumped out and injection work started. Such a procedure would allow for considerable savings if water leakage problems appear to be small.

Discussion on Study Schemes

There should be no doubt that drillings are necessary to establish reliable information in order to assess the ground conditions for undersea tunnel projects. Some information is needed in order to work out a rational drilling programme.

The mapping of geology may give indication of disturbance zones made up of poor rock. Soundings and accoustical soundings are relatively inexpensive and supply important information on average bedrock depth and thickness of overburden. Seismic measurements are by far more expensive, but supply better locational control, better detailed results and classification of the bedrock.

If vertical drillings are planned, seismic measurements are considered desirable for the working out of a reasonable drilling programme. There may still be undetected occurrences of poor rock, at least in narrow zones.

To obtain the highest degree of ground condition control, there are no alternatives other than inclined or horizontal drillings. Such drillings may be accomplished in selected areas indicated by previous studies. For instance, short range drillings through fault zones may supply important information about rock quality and thickness of the disturbed zone. The conditions in rock, and especially in zones of disturbance, may vary over very short distances. It is essential therefore, that drillings for ground condition control should penetrate the poor rock areas as close to the tunnel as possible.

This may not be feasible by means other than long subhorizontal drillings started from the shores. If the drillings are directed along or above the crown of the projected tunnel, they may also give continuous recording of the rock quality, as a reliable detection of harmful buried subductions or gaps in the bedrock surface, and lastly map the permeability and water conduction in the rock. Such drillings will penetrate the predominant rock in the project and eventually leave undetected small, local problem areas. Parallel structures may also be undetected for a shorter distance but this possibility should be taken into consideration when planning the drillings. The drillings are expensive, but they supply information of a more exact nature than seismic measurements. If long subhorizontal drillings are considered, seismic measurements should be required only to plan the location of the drilling. If the drillings may be planned on the basis of the accoustic soundings, expenses for seismic measurements may be saved.

Provided long horizontal drillings are possible it seems that, for feasibility and cost estimation, such drillings should be the most reasonable way of obtaining the required information.

Long horizontal core drillings are at present limited to about 1000 m in length, deviational control may not be satisfactory, and the information given may be limited if the tunnel project is curved. If the drilling strikes into overburden filling gaps and subductions in the bedrock surface, other drillings may be required to re-locate the tunnel. This procedure may therefore be expensive and parallel disturbance zones may not be detected.

If such a situation may be foreseen, a more reasonable approach would be to undertake a thorough mapping of the ground conditions by means of seismic measurements, and subsequently to apply a reduced drilling programme. The risk involved for tunneling into local water inbursts and cave-ins is considered higher by this procedure than by drilling long subhorizontal holes. For the Vardø project, discussions so far have resulted in a step by step progress in the studies, to have the opportunity to stop them immediately if and when high cost ground conditions are found. This means that the last procedure is pursued, and that eventual long horizontal drillings are postponed.

Where the sea is wide and deep, neither of the discussed procedures may be feasible for a complete exploration of the project. Large risks have to be accepted when dealing with tunnel projects in such areas. The only way of dealing with the risks of accidents is by the undertaking of continuous drillings ahead of the face. However, with this procedure, large costs may arise if the tunnel has to pass areas with very poor rock or overburden.

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Soil Exploration in a 500 m Deep Fjord, Western Norway

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Abstract A number of engineering organizations and individuals have contributed toward a comprehensive feasibility study made in 1973–1974 in connection with a submerged floating tunnel project in Norway. The tunnel is planned to cross the 500 m deep Eidfjord in Hardanger over a length of about 1.3 km. The main findings from geophysical explorations, subsoil sampling operations, and an extensive laboratory testing program on extracted soil samples are described here.

A main part of this paper is devoted to the study of a full-scale field test with a gravity anchor block weighing 180 tonne. This study reports on the behavior of the block during launching from its sloping construction ground, the sinking operation, and the behavior of the block after it reached the bottom at 450 m depth. Observations of settlement and tilt are available, and a comparison is made between the observed and computed behavior. Broadly speaking, a fairly good agreement was found.

Introduction

The very long Norwegian coast, with numerous fjords and islands, has necessitated as many as 200 ferry connections. The depth and width of the fjords will in most cases prohibit the use of conventional bridge structures. This has led to studies of alternative methods for crossings in order to obtain permanent connections on some of the main roads.

A possible solution is the construction of submerged floating tunnels (Brandtzæg, 1972; Arild, 1975). The detailed study reported herein has been made for an Eidfjord crossing on the road between Bergen and Oslo (Figure 1).

A fairly narrow site was chosen. The steep mountain sides continue under water down to a depth of 400 to 500 m. The bottom of the fjord is fairly level,

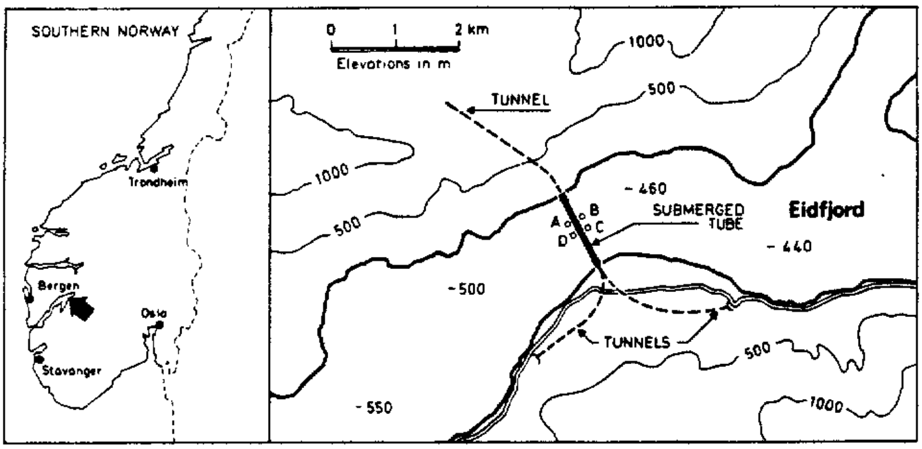

Figure 1. Site location and general topography.

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and the selected route has taken advantage of the presence of a gentle ridge which gives the shallowest depths. A typical cross-section is shown in Figure 2.

An extensive soil exploration program was carried out to establish a rational basis for the selection and design of an adequate anchor system. Furthermore, a proper knowledge of the soil conditions was required for the detailed analysis of the behavior of the full-scale in-situ tests on the selected gravity anchor.

The system for the submerged floating bridge is shown in Figure 2. The anchor groups are concentrated in two areas, partly because of the soil conditions but mainly because of the great water depths. The anchor cables, which are almost vertical, were originally inclined at an angle of about 60° with the horizontal. Gravity anchors with vertical cables were chosen as a result of the

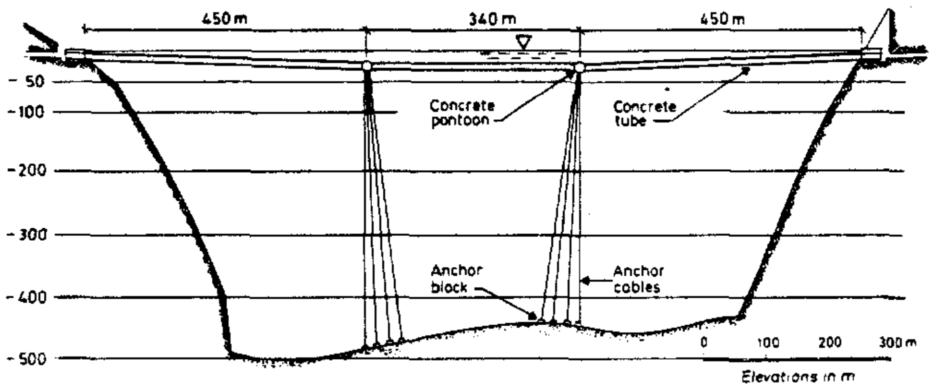

Figure 2. Cross-section along tunnel axis.

study of the soil conditions and the construction problems. A detailed description of the project is given by Arild (1975).

Field Investigations

The field investigations started early in 1973 with topographic mapping and acoustic profiling. Bottom sediment sampling and other field tests were performed in the spring. Supplementary investigations made in the fall of 1973 and in the spring of 1974 comprised additional acoustic profiling and sampling.

Geology

It is known that turbidity currents have occurred in several Norwegian fjords. This phenomenon has also been studied at certain locations of the Hardangerfjord (Holtedahl, 1965). It is believed that slides may start underwater fairly high up on the steep side slopes. A mixture of minerals and water move down toward and along the bottom. The minerals are then deposited at different locations depending on their grain size and the current velocity.

Turbidity deposits are characterized by graded and bedded sediments. The presence of such deposits may, therefore, be detected by a study of the stratigraphy and the presence of shell fragments from animal life inhabiting shallower water. Such analyses require sampling.

The extent of possible activity is a very difficult question to resolve. It is of vital importance, however, to avoid turbidity currents that will exert large forces on the anchor system, and thereby jeopardize the stability of the submerged tunnel.

Topographic Mapping

A satisfactory topographic map (scale 1:5000) was available for the south side of the fjord. Moreover, aerial photographs of a recent date made the construction of a map for the north side possible. The steepness of the mountain sides beneath sea level and the topography of the fjord bottom were not known. The depths at certain locations in the fjord were given on an existing marine map. However, further information was needed to determine the most favorable location for a crossing.

It was therefore decided to produce a map sufficiently accurate to allow for future planning, design, and construction of the project. Because the narrow fjord has steep sides and large depths, a special echo sounder with a narrow (4°) beam was utilized. Even so, vertical errors on the steep sides of as much as 50–100 m are possible. However, over the central area, which is the most important one for this project, the error is believed to be of the order of 1–2% of the water depth.

Acoustic Profiling

The profiling with the special echo sounder was run simultaneously with the acoustic profiling and the results were coordinated. The latter utilized sparker and pinger sub-bottom profiling equipment. A number of profiles were obtained over the area. The geometry problem was present all the way through, because of the great depths and the steep valley sides.

The accuracy is also influenced by the limited resolution of the sparker system. Soil thicknesses of as much as 5 m above rock were not detected. Neither did the penetration echo sounder (the pinger with high resolution) indicate any reflecting sub-bottom horizons. No satisfactory explanation has been offered.

Typical results of the acoustic profiling are shown in a cross-section of the fjord (Figure 3). It is seen that thick sediments are encountered on both sides of the ridge. The ridge itself is indicated as bedrock without soil cover. However, with the limited resolution of the sparker system, the sediment cover could easily be 2–5 m. This was later confirmed by sampling.

Sampling was made to a depth of 4–6 m over the whole area without encountering bedrock. To obtain a better picture of the situation, a high resolution boomer survey was tried although there was little past experience at such great water depths. On the basis of this survey it was concluded that approximately 2–4 m of sediments covered the ridge, with an uncertainty of ±1.5 m.

Bottom Photography

Several photographs of the fjord bottom were taken with a Hasselblad camera. All pictures showed an even, silty, clayey floor without visible coarse matter. There was evidence of innumerable traces of burrowing creatures, and the biological activity at the bottom is rather intensive.

Sampling

A variety of equipment was used to obtain samples from the bottom sediments. It was decided to obtain samples from four areas, marked A, B, C, and D in Figure 1. A few additional samples were extracted from neighboring areas and on the tunnel axis itself. Short comments on the merits of the various sampling equipment may be appropriate.

Grabs. Three types of grabs were available on board the cruise vessel: a large spherical grab (1.5 m^3) and two smaller grabs (0.1 m^3). It was believed that if all other equipment failed to do the job the grabs should at least obtain surface samples. Four attempts using the large grab in area A failed because the soft top

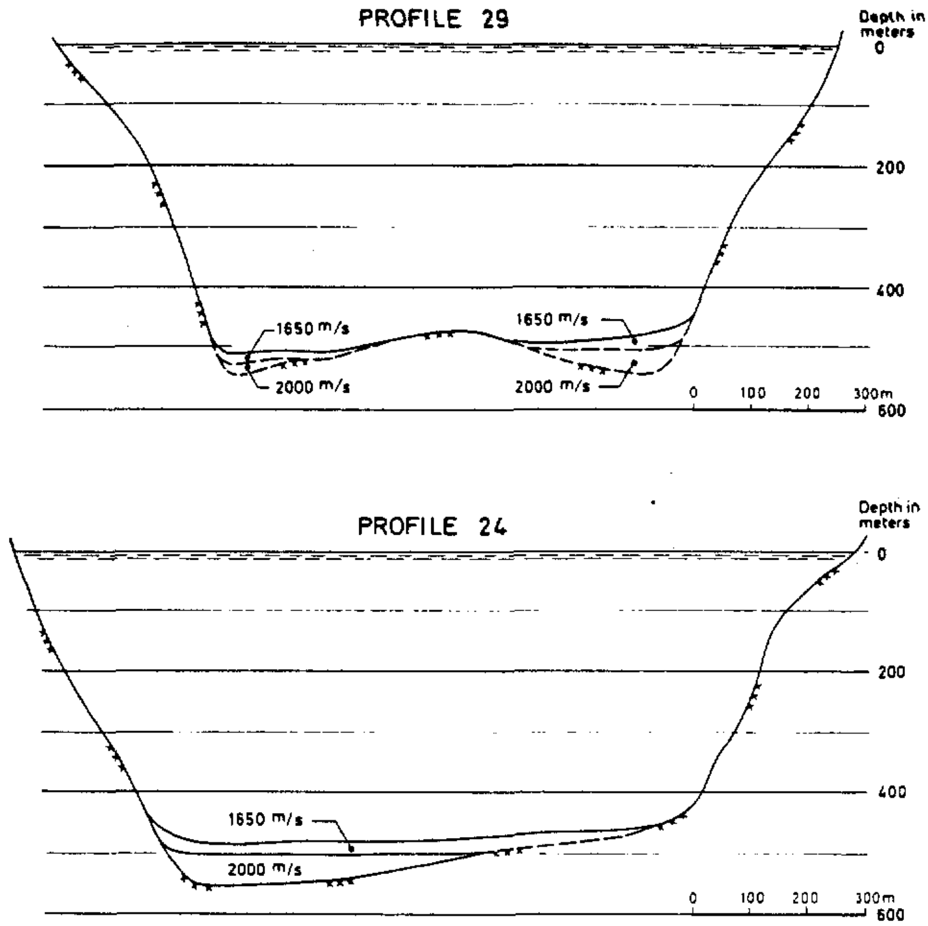

Figure 3. Results of acoustic profiling, 300 m to both sides of axis.

deposit hindered the release of the grab upon contact with the floor. After a modification of the release mechanism, the large grab produced a full bucket from which a core sample was taken.

Surface Sampler. A simple seabed surface sampler of short length was used, assuming that a short sample tube would produce good quality samples. The tubes of the standard Norwegian Geotechnical Institute (NGI) sampler had a length of 0.8 m and a diameter of 54 mm. They were open at one end, and had a simple closing mechanism at the other. A dead weight assisted in penetrating the sampler into the bottom. Three out of four tubes retained a soil sample varying in length from 0.4 to 0.7 m.

Gravity Corer. Gravity corers with 60 and 100 mm diameter were at our disposal, but only 100 mm was used (Figure 4). This sampler, equipped with a plastic inner tube, had a length of 3m. Altogether 17 samples were taken, and recovered soil lengths varied from 1.0 to 2.6 m. This equipment was rugged and expedient to operate, but occasional difficulties can be expected with retaining the sample in the tube during extraction and hoisting. Much shorter samples than expected were obtained.

Kullenberg Piston Sampler. The Norwegian Kullenberg piston sampler had a diameter of 60 mm and normally a length of 6 m (Figure 5). An additional 6 m was added in two trials bringing the total length up to 12 m. In three out of nine

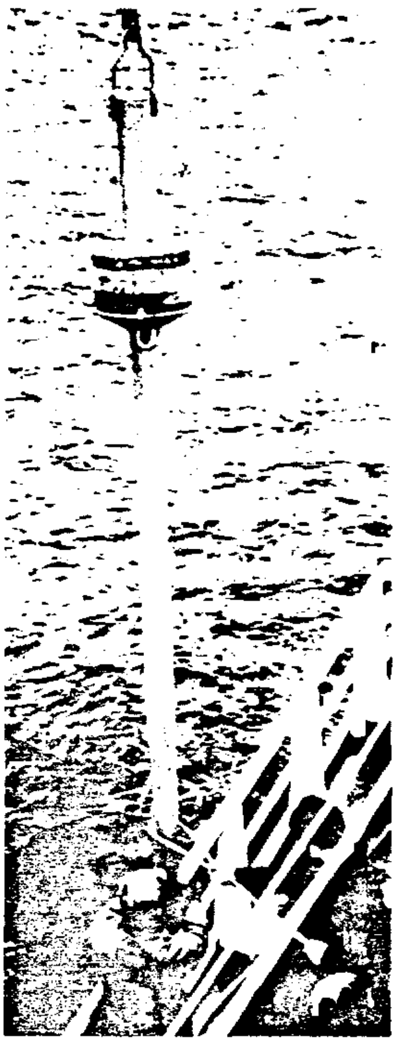

Figure 4. Gravity corer.

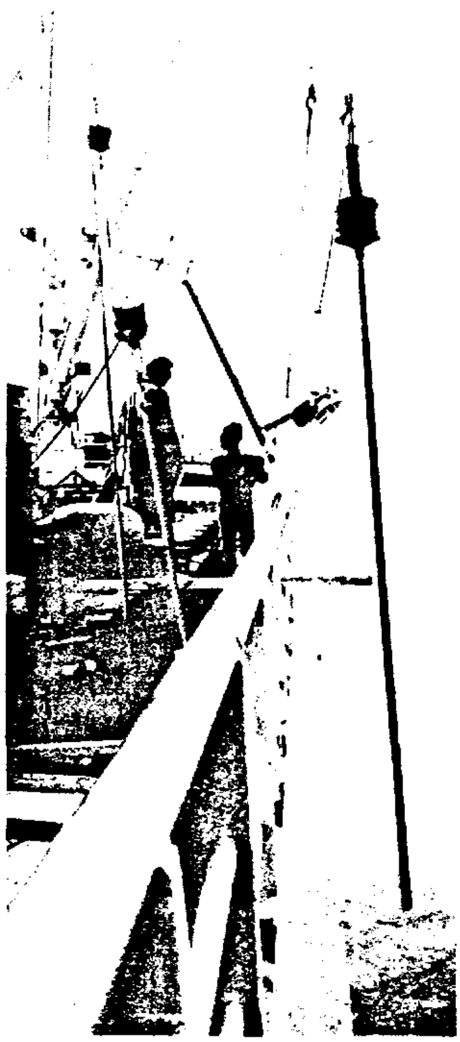

Figure 5. Kullenberg piston sampler.

cases the tubes broke, whereas five of the remaining six produced a full soil sample. The impression obtained from working with this type of sampler was in many ways positive. The expediency, the degree of recovery, and the fair sample quality led us to select only this sampler for the entire supplementary investigation in 1974. This last cruise produced five samples out of six trials. The 12-m-long tubes could not be used at this site because the maximum thickness of the soft soil seemed to be about 6 m.

NGI Gas Sampler. The gas-operated piston sampler (Andresen et al., 1965) has a diameter of 54 mm and a length of 1.5 m. After the sampler is lowered down to the bottom, a time-set charge is ignited and the gas pressure pushes the sampling tube into the soil while the piston is retained. The sampler was used four times and the recovery varied from 0.6 to 1.5 m. Some problems arose with determining the position of the sampler relative to the seafloor before ignition. The ideal starting location for the first sample would be when the tip just touched the floor. In shallow waters, the length of the supporting wire and depth to sea bottom can be measured with fair accuracy. In our case, with a 450 m depth, the depth control was much more difficult.

Laboratory Investigations

There were two main reasons for carrying out an extensive laboratory program. It was the first time that laboratory tests were performed in Norway on samples obtained from water depths of nearly 500 m. Besides, the designers were faced with a novel task: to tackle the practical problems involved in the foundation of the anchorages for the submerged tunnel.

Program

Initially, some of the samples were examined on board the cruise vessel, where also a few water contents and strength tests were made. The following samples were retained for laboratory testing:

4 Kullenberg, length up to 6 m, diameter 60 mm
10 Gravity corer, length up to 2.6 m, diameter 100 mm
4 NGI gas sampler, length up to 1.5 m, diameter 54 mm
2 Surface sampler, length up to 0.4 m, diameter 54 mm

It was decided that the Kullenberg samples, covering the largest depths, be used for the main investigations. The NGI gas-sampler samples should be used for special refined tests, assuming least sample disturbance, while the large diameter gravity corer samples were used for supplementary studies. The extent of the main part of the program is summarized in Table 1.

Sample Quality

Prior to boring operations the consequences of sample disturbance on the test results were discussed and a few precautionary measures were taken.

Table 1
Extent of Laboratory Program

Area	Sampler Type	Max. Depth, m	No. of Lab. Test at NTH[a] Triax.	Oedom.	Grain Size	s_u[b]	γ	w	w_L w_p
A	Kullenberg	6.00	9	8	18	13	15	29	6
B	Kullenberg	5.05	8	10	14	14	11	26	4
C	NGI gas	1.50	4	4	5	4	4	8	2
D	Kullenberg	5.70	6	9	16	12	10	22	5
	Sum		27	31	53	43	40	85	17

[a]NTH = Norges Tekniske Høgskole = Technical University of Norway.
[b]s_u obtained by fall-cone tests.

First, a simple pressure device was made to determine if gas developed in the sample after it was extracted from an environment of low temperature and large in-situ water pressures (5 MPa) and became exposed to air temperature (20°C) at a low pressure (100 kPa). This investigation showed that the samples did not develop gas upon exposure to atmospheric pressure and temperatures.

The effects of laboratory temperature changes were also investigated by comparing results of cooled samples vs samples tested at +20°C. No significant differences were observed.

Consequently, sample disturbance should exclusively be due to the mechanical distortions created when the tubes penetrated into the ground and when the filled tubes were pulled out and hoisted up on deck. Further disturbance could also take place during handling and transportation, but presumably to a smaller degree. The mechanical distortions of the soil layers are illustrated here by photographs.

Figure 6 shows photographs of four cross-sections of samples from various depths. Apparently sample disturbance increases with depth, possibly due to increased friction between the sample and the tube wall. The specimens are all obtained from a 6-m-long Kullenberg sample.

Two samples obtained by the 100 mm gravity corer are shown in Figure 7, along with a sample taken with the NGI gas-operated corer for comparison. The photographs demonstrate that the degree of disturbance is less pronounced for the shallow samples, whose maximum length is 1.5 m.

The study of sample disturbance can broadly speaking be summarized as follows: The shallow samples (less than 2 m in length) and the upper part of longer samples appear to have been substantially less disturbed than the middle and the lower portions of the 6-m-long samples. The degree of disturbance of the deepest portions of the long samples must be characterized as severe.

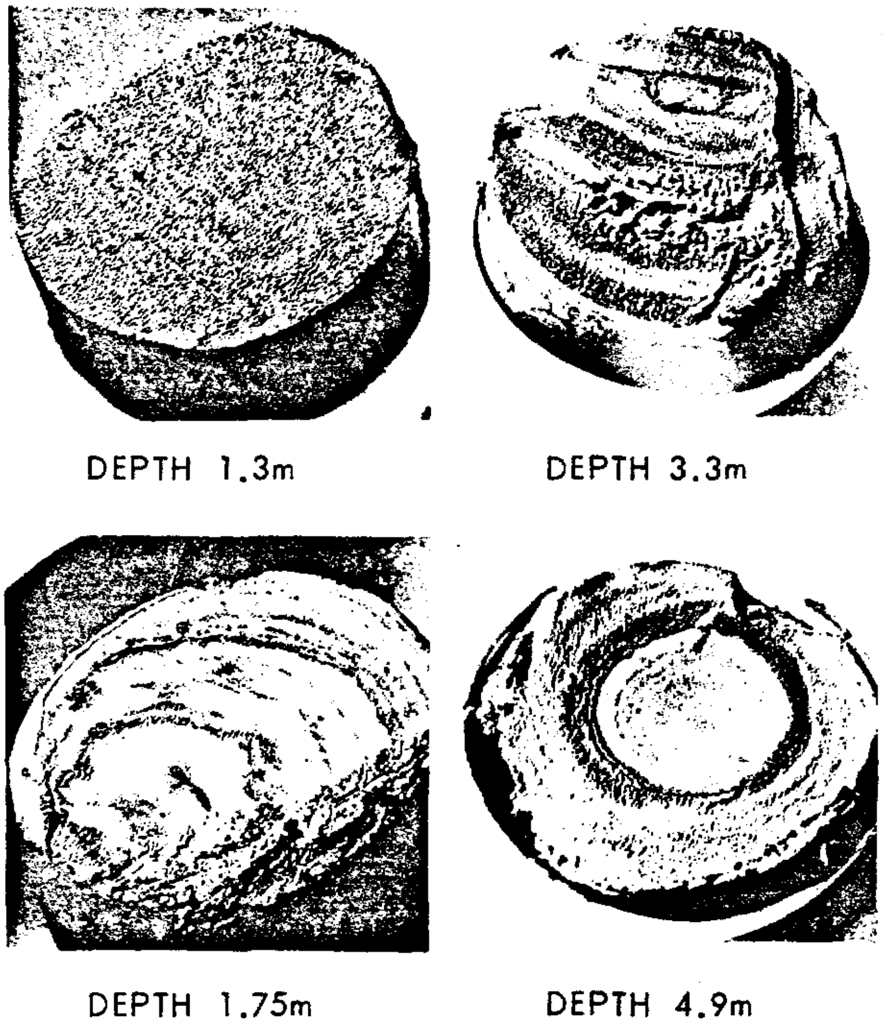

Figure 6. Cross-sections of soil samples from tube B9.

Routine Tests

The routine analyses include the following determinations: natural water content, plastic and liquid limits, undrained shear strength, grain size analyses, and organic content.

Typical results are shown for tubes A8 and B9 in Figure 8. The supplementary sampling made in 1974 produced five samples and these were subjected only to routine investigations. The results were quite similar to those shown in Figure 8. It was therefore concluded that the whole area has fairly uniform soil properties.

Figure 7. Longitudinal sections of samples.

The soft top layer (w = 60–90%) is underlain by a stratified soil of much lower water content (25–35%). Broadly speaking this soil can be classified as a lean silty clay or clayey silt with some distinct layers of silt and fine sand, and with occasional traces of organic matter. Inspection of the silt and fine sand grains under a microscope gave the impression of relatively sharp grains, the coarser grains being mostly quartz.

Shear Strength and Compressibility

A recently developed oedotriaxial procedure (Janbu, 1973) was used in addition to conventional triaxial and oedometer test. The oedotriaxial procedure yields both strength parameters simultaneously, in addition to the coefficient of lateral earth pressure at rest, K_0, and the pore pressure parameter, $\bar{B}$.

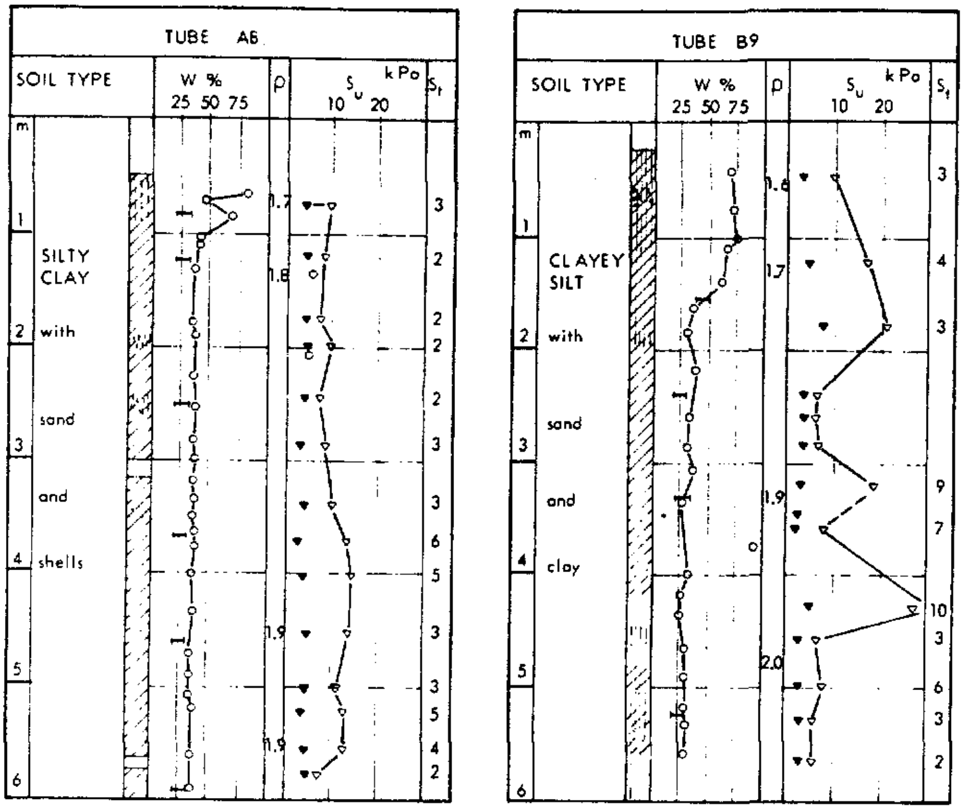

Figure 8. Typical boring logs.

The strength parameters, a and $\tan\bar{\phi}$, define the *shear strength* on an effective stress basis, expressed by the formula

$$\tau_f = (a + \bar{\sigma})\tan\bar{\phi}. \tag{1}$$

Note that attraction a = cohesion ($\bar{c}$) divided by friction ($\tan\bar{\phi}$), or $\bar{c} = a\cdot\tan\bar{\phi}$.

A summary of the range and overall average of 27 measured values of the shear strength parameters is given in Table 2. The lowest values generally were obtained for the upper soft layer.

For clays and clayey silts, it has previously been found that the *compression modulus, M,* increases almost linearly with stress for stresses above the preconsolidation pressure, hence

$$M = m(\bar{\sigma} + a) \tag{2}$$

where m is the modulus number. A summary of the measured ranges of modulus numbers as well as the average values obtained are given in Table 3. In general, the lowest values were obtained on samples from the soft top soil.

The *coefficient of consolidation,* c_v, was also determined. As a whole, neglecting the most extreme data, it has been found that the average c_v is

$$c_v = 11\ \text{m}^2/\text{year} \pm 6.$$

This value of c_v corresponds to effective stresses near and slightly above the in-situ overburden pressure. For increasing stresses c_v increases.

The *coefficient at rest* for effective stress, $\bar{K}_0$, is determined from the oedotriaxial tests. Table 4 contains a summary of $\bar{K}_0$, and a values as derived from the $\bar{K}_0$ vectors.

The excess pore pressure, Δu, under additional stress $\Delta\sigma$ can be expressed by

Table 2
Measured Attraction and Friction

Area	*a in kPa* Max.	Min.	$\tan\bar{\phi}$ Max.	Min.	No. of Tests[a]
A	3.5	0	0.78	0.73	9
B	2.5	0	0.80	0.71	8
C	5.0	0	0.73	0.67	4
D	2.0	0	0.81	0.78	6
Average	1.5±		0.75±		27

[a]Triaxial tests.

the simple equation

$$\Delta u = \bar{B}\Delta\sigma \tag{3}$$

where $\bar{B}$ is the pore pressure parameter.

From conventional triaxial tests and from oedotriaxial tests it is found that $\bar{B}$ generally varies between 0.5 and 1.0, with most of the values being concentrated around 0.8 to 1.0. The overall average value for the Eidfjord samples was found to be

$$\bar{B} = 0.85 \pm 0.15.$$

The average shear strength and deformation properties given herein are not representative for the soft top layer. Especially, the upper 1 m is very much softer than the underlaying soil.

Table 3
Modulus Numbers Obtained by Oedometer Tests

Area	*Values of m* Max.	Min.	No. of Tests	*Extreme Values* Low	High
A	31	22	8	...	...
B	44	27	7	13	79
C	28	20	3	...	67
D	59	24	7	13	...
Average	32 ± 9[a]		25		

[a]Similar results of 12 oedotriaxial tests.

Table 4
Values of $\bar{K}_0$ and *a* Obtained by Oedometer Tests

Area	*Values of* $\bar{K}_0$ Max.	Min.	Approx. Average *a*	No. of Tests	Type of Sampler
A	0.46	0.39	2±	4	Kullenberg
B	0.44	0.34	−2	4	Kullenberg
C	0.41	0.40	5±	2	NGI gas-operated
D	0.37	0.35	2±	2	Kullenberg
Average	0.40 ± 0.03		1.5±	12	

Full-Scale Load Test

It was concluded from the laboratory results that drilled-in anchors were not feasible, and that the ground was so soft that it would lead to large lateral deformations for inclined anchoring. Gravity anchors with vertical cables were therefore chosen. It was also decided to make a field test with a concrete anchor block to study construction, installation, and maintenance problems. This also gave an opportunity to check the settlement computations in full-scale testing.

Anchor Design and Instrumentation

The designer chose a test anchor block of 4.5 X 4.5 m with a height of 2m. By using heavy aggregates, the total weight in air was 180 tonne and submerged 130 tonne. This resulted in a pressure on the soil of 65 kPa, as compared to the short term ultimate bearing capacity of the top soil that is less than 80 kPa.

The instrumentation included two inclinometers to observe the tilt in two directions. The settlement is observed relative to the fjord floor by means of special equipment developed by NGI.

A three-legged device is suspended in a frame 4.0 m to the side of the anchor. A closed mercury system records pressure differences by means of a vibrating wire pressure cell. The three-legged device is suspended in such a way that it will touch the bottom first. The settlement of the device itself is negligible. Knowing the geometry of the system, the movements of the anchor relative to the bottom can be observed. The instruments are connected to an observation panel ashore by means of an 800-m-long cable.

Construction and Launching Procedure

The test anchor was constructed directly on the nearby shore. A launching pad was made on the natural ground, which consisted of sand and gravel. The anchor block rested horizontally on a triangular sled, Figure 9. The sled was made of light-weight aggregates and hollow spaces were provided to minimize weight. To make the conditions for sliding as uniform as possible, the sled was soled with a steel plate. Friction tests were made to determine the most favorable slope, and the base friction coefficient was found to be 0.45 ± 0.10.

The slope had an inclination of 1:2.5 corresponding to sliding for a coefficient of friction equal to 0.40. Therefore, only a minor pull or restraint should be needed during the launching procedure. However, this configuration of sloping ground, inclined load, and fairly loose gravel could lead to bearing capacity failures. Hence, it was decided beforehand to use artificial reinforcement of the ground. During launching the reinforcement was not effective, however, and a shallow bearing capacity failure took place. Fortunately, neither the tilt nor the settlement was large. The launching procedure could therefore continue.

Lowering Operation and Settlement Observations

The launching was terminated when the anchor reached about 10 m depth. The anchor block was then picked up and suspended at the back of a supply boat by means of a double wire from the two main winches on board. The anchor was transported to the site in this position and the lowering operation took place. This operation took a little less than 2.5 h, and the maximum speed was 4 m/min. The speed was about 2 m/min at the time when the anchor reached the bottom.

Tilt as well as settlements have been observed. The tilt has been fairly constant since the anchor reached the bottom; a little more than 3° in one

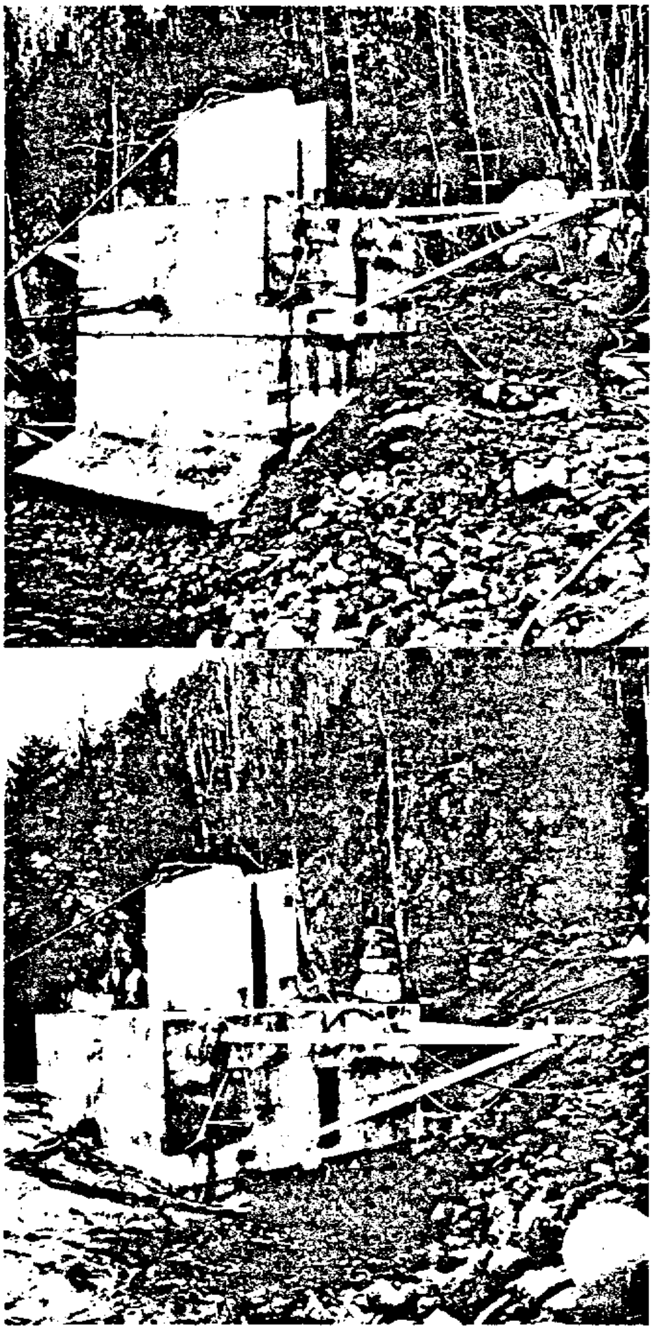

Figure 9. Test anchor before and during launching, 4.5 X 4.5 X 2 m.

direction and about 2.5° in the other direction. The observed settlement is shown in Figure 10. It is characterized by a fairly rapid settlement over the first few hours, after which it flattens out rapidly over the first month or so.

Settlement Analysis

The result of the consolidation settlement is shown in Figure 11. The analysis is based on the oedometer test results, and theories previously derived by the second author (e.g., Janbu, 1969). The surface load is 65 kPa.

Using Equation (2) for $M = d\bar{\sigma}/d\epsilon$ and integrating ϵ between stresses $\bar{p}_0$ and $\bar{p}_0 + \Delta p$ a formula for vertical strain ϵ at any depth z is obtained:

$$\epsilon = \frac{1}{m} \ln \frac{\bar{p}_0 + \Delta p + a}{\bar{p}_0 + a} \tag{4}$$

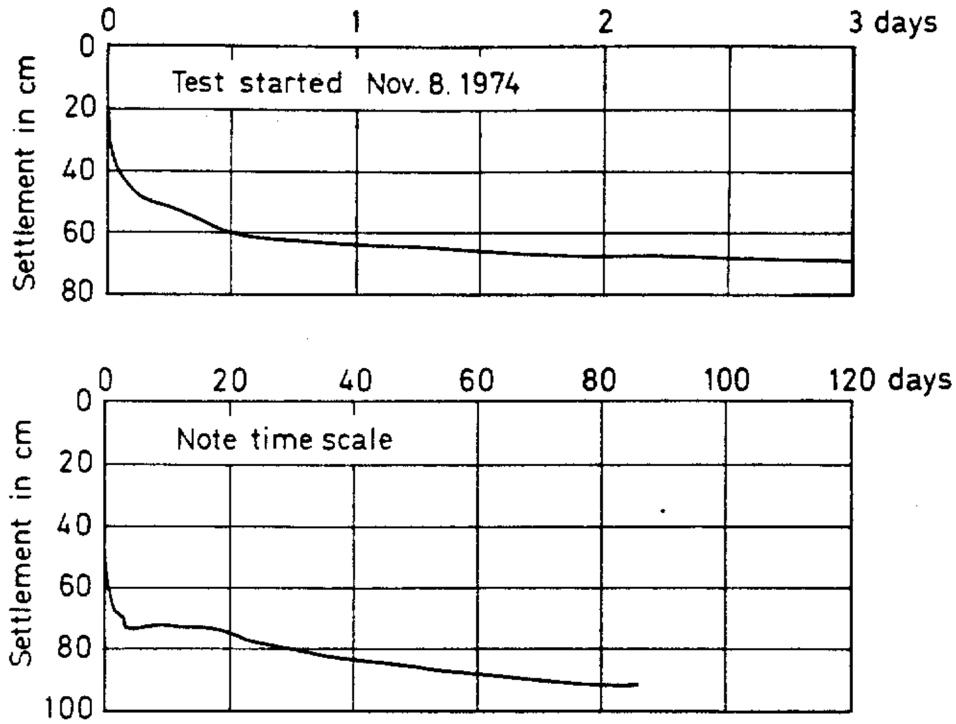

Figure 10. Settlement observations.

where

m = modulus number,
$\bar{p}_0$ = vertical effective overburden,
Δp = vertical stress increase, and
a = attraction.

The left-hand part of Figure 11 shows the variation with depth of $\bar{p}_0 + a$ and Δp, while the middle illustrates how the average modulus number m increases from about 10 to about 30 over the top 3 m. The right-hand part shows how the calculated ϵ varies with depth. From the defining equation $d\delta = \epsilon dz$, it is seen that

$$\delta = \int_0^H \epsilon \, dz = (\epsilon - z) \text{ area.} \tag{5}$$

Evaluating the area of the $\epsilon - z$ diagram in Figure 11 one finds

$$\delta_{oed} = 53 \text{ cm}$$

whereof 29 cm occurs within the upper 1 m.

The oedometer moduli corresponds to no lateral yield. Here, lateral yield will take place immediately after loading. Ordinarily, in such cases, an additional, immediate, undrained settlement is estimated by means of a simple formula

$$\delta_{iu} \cong \frac{q \cdot B}{M_{iu}} \tag{6}$$

where

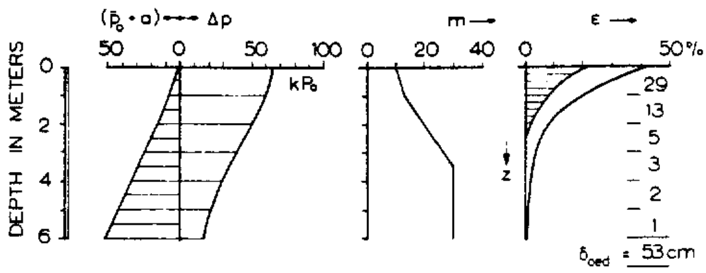

Figure 11. Settlement calculations.

q = surface load,
B = footing width, and
M_{iu} = average undrained modulus.

From experience, $M_{iu} \sim (150\text{–}250)s_u$ when the short-term safety factor is $\geqslant$ 2.0. Here, the safety factor is probably less than 1.5. Moreover, the anchor block hit the bottom with a speed of 2 m/min, thus creating an impact and a possible added plastic flow. To account for a low safety factor and the effect of impact, a smaller value, for example, $M_u \geqslant 100 s_u \cong 1000$ kPa, needs to be selected, hence,

$$\delta_{iu} = \frac{65 \cdot 450 \text{ cm}}{1000} = 29 \text{ cm}.$$

Consequently, a total settlement of 53 cm + 29 cm = 82 cm is obtained.

Since the $\bar{B}$ parameter is 0.85 in average, the immediate effective stress increase is 0.15 Δp. For such a stress increase the immediate strains ϵ_{ic} are calculated and plotted in Figure 11, the shaded area of which is 18 cm. Hence, instantaneous settlements are (29 + 18) cm = 47 cm, while the actual consolidation settlement is the remaining 35 cm.

Due to the rapid decrease of ϵ with depth, the primary consolidation occurs rapidly. For the ϵ variation shown in Figure 11 one can estimate the time needed for 60% primary consolidation by means of the formula

$$t_{60} \cong 0.1 \frac{H^2}{c_v} \tag{7}$$

where H is the length of effective draining path, or height of idealized $\epsilon - z$ diagram. With c_v = 11 m^2/year, as observed, and cutting the ϵ diagram in Figure 11 at depth $H = 3$, it is found that $t_{60} \sim 1$ month.

For soft slightly organic soils, the rate of long-term settlement is mainly governed by secondary effects. Based on the time resistance concept ($R = r_s t = 1/\dot{\epsilon}_s$), the rate of secondary settlement $v_s = \dot{\epsilon}_s H$ is

$$v_s = \frac{H}{r_s t} \tag{8}$$

where t is the time elapsed after loading and r_s is the time resistance number.

The time resistance number $r_s < 100$ for the upper 2m, but > 100 for the underlying soil. Using $r_s = 100$ as average over H = 6 m one obtains the following rate of secondary settlement:

v_s = 6 cm/month after 1 month
v_s = 3 cm/month after 2 months

v_s = 2 cm/month after 3 months

As a whole, the settlement analysis has led to the following result:

δ_i = 47 cm = immediately
δ = 82 cm = total settlement

In addition, an estimated 11 cm of secondary settlements can have developed over the 3 months observation period shown in Figure 10, making the total value ~ 93 cm if 100% primary has occurred. The rate of increase at the end of the observation is calculated to 2 cm/month. Thus the measured and calculated settlements agree fairly well.

Conclusions

Based on the variety of investigations carried out in connection with a submerged floating tunnel project across the 500 m deep Eidfjord in Norway, the following conclusions are drawn.

Mapping and Acoustic Profiling

The obtained map is satisfactory for the purpose of the project, because the area of main interest is away from the slopes. The accuracy with respect to side slope topography and the transition between slope and seafloor is not satisfactory for general purposes. Accuracy control was, in this case, limited by the funds available.

The acoustic profiling furnished valuable information that cannot be obtained economically otherwise. Its usefulness for a detailed description of the surface layers is, under the conditions at this site, very questionable. The prospect for detecting bedrock seems better, although the assumed accuracy with respect to detecting soil cover has not been confirmed.

Sampling Methods and Sample Quality

The conclusions that can be drawn with respect to sampling do not divert from previous experience with soft soil under other conditions. Samplers with low area ratio give less disturbance. A longer sampling tube gives more disturbance than a short one.

The gravity corer and the Kullenberg sampler are expedient to operate and can be fitted with fairly long sampling tubes. The advantage of being able to sample a thick layer in one operation is obvious, but the poorer quality of the sample is a problem.

The NGI gas sampler is promising for obtaining better samples. However, if thicker layers must be penetrated, a better control system for locating and positioning the sampler with respect to the seafloor and more expedient operation is desirable.

Laboratory Analysis

The sample disturbance represented a problem for accurate laboratory analysis. The influence is noticeable on the undrained shear strength. This effect can to some extent be compensated for by testing the central part of the sample, a technique which is possible with the fall-cone test.

The determination of accurate shear strength parameters, a and $\tan\phi$, is also influenced by disturbance. The most sensitive parameter is attraction, a, whereas friction, $\tan\phi$, seems to be less affected.

Disturbance has also influenced the deformation properties and the pore pressure coefficient. However, as a whole it was found that the soil properties obtained through careful handling and advanced testing techniques served their practical purpose satisfactorily.

Soil Conditions

The soil is stratified and consists mainly of alternating layers of silty clay and clayey silt. Embedded thin layers of fine sand have been observed. Some of the layers contain shell fragments, grains of gravel, and traces of organic matter.

The upper 1 to 1.5 m has a high water content (60 to 80%) while the water content of the underlying strata varies within 25 to 40%, with a tendency to decrease with depth. Maximum sample depth was 6 m.

Load Test

The construction, launching, and installation of a test anchor block has provided a unique opportunity for testing out theories in a scale seldom experienced. The launching confirmed the estimated low bearing capacity of slightly inclined loaded footings on sloping ground. Very accurate values for base friction could not be established in this case within the limited time and funds available. Nevertheless the block behaved according to predictions.

Altogether, the observed behavior of the test anchor showed that the block could be installed safely despite the high contact stress level. The deformations observed compare well with the calculated values using the soil data derived from the laboratory tests. The field test also proved that the use of gravity anchors is feasible, and that the test block had a size that can be handled economically today.

Turbidity Currents

The soil profiles indicate that the upper soft organic clay layer is not due to turbidity currents from slopes in the near vicinity. On the other hand, the underlying, layered silty clay could possibly be traced back to ancient flow slides on the slopes followed by streams of suspended minerals, but a very definite conclusion cannot be drawn from the limited evidence available. However, it is believed unlikely that turbidity currents of sufficient intensity will occur to endanger the anchoring at the sites chosen.

Acknowledgments

The study of submerged floating tunnels in Norway is being performed by a task group of four engineers appointed by the Norwegian Public Roads Administration: Director A. Arild, Professor A. Brandtzæg, Consultant Eng. O. Folkestad, and Professor A. Selberg. The first author has acted as advisor and coordinator for exploration of foundation conditions, while the second author has served as special consultant on soil mechanics and foundation problems.

The studies reported herein are the results of cooperation among a number of consultants. The topographic mapping and the acoustic profiling were performed by A/S Geoteam. This firm was also responsible for the positioning of the cruise vessel for all operations. The soil sampling was carried out by the Continental Shelf Division of the Royal Norwegian Council for Scientific and Industrial Research.

The laboratory analyses were performed at the Institute for Soil Mechanics and Foundation Engineering at the Technical University of Norway. The equipment for observation of tilt and settlement of the test anchor was developed by the Norwegian Geotechnical Institute. NGI was also responsible for obtaining the data in the field. Dr. T. Preber, formerly with the Norwegian Road Research Laboratory, took an active part in the first stages of the program.

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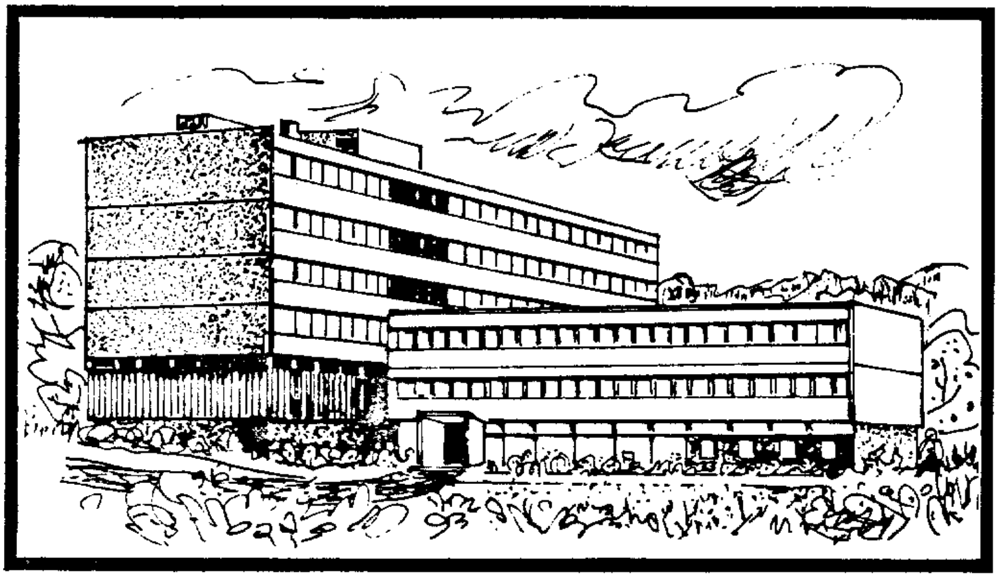

VEGLABORATORIET

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Statens Veglaboratorium ble etablert i 1938 og er administrativt en avdeling i Vegdirektoratet.

Veglaboratoriet er internt organisert i 5 seksjoner som knytter seg til fagområdene asfalt og kjemi, betong, bærelag, geologi og geoteknikk, samt sekretariat og felles verksted.

Oppgaver:

Hovedoppgavene er å drive forsknings- og utviklingsarbeide på det vegtekniske område samt å virke som rådgiver innenfor de fagområder laboratoriet dekker. I dette arbeidet inngår kurs og opplæringsvirksomhet.

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NORWEGIAN ROAD RESEARCH LABORATORY

Organization:

The Norwegian Road Research Laboratory was established in 1938 and is a sub-division of the Public Road Administration.

The N.R.R.L. is internally organized in five technical sections, Bituminous materials and Chemistry, Concrete, Pavement design, Geology and Soil mechanics, together with a secretariat and a workshop.

Field of Operation:

The main object of The N.R.R.L. is research and development in the area of highway construction and consulting within the technical fields of operation. This work includes offering courses and training programs.

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