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Deliberate Failure of Slurry Trench Excavations in Soft Clay

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ABSTRACT: The instrumentation used and the observations made on full scale field tests carried out in two slurry trench excavations in soft marine clay (at Vaterland, Oslo) are presented and discussed. The excavations were 1 m x 5 m and 1 m x 1.8 m in internal cross section and 20 m deep. The testing program was carefully planned to obtain information which could confirm the overall stability of the trenches using water instead of expensive special slurries. The instruments used included a precision settlement gauge, numerous settlement reference points and electrical piezometers, slope indicator casings and specially designed hydraulic gauges for instantaneous measurement of the inward movement of the vertical walls of the trenches. This project illustrates the importance of large-scale tests and instrumentation in Geotechnical Engineering.

KEYWORDS: Full-scale test, Slurry trench excavation in soft clay, Geotechnical instrumentation, Stability, Deliberate failure.

1. EXECUTIVE SUMMARY

1.1 Background and Description of the Project

One of the most fascinating construction techniques developed by foundation contractors and engineers in the early 1950s is the so-called slurry trench method of constructing deep underground concrete walls, shafts or similar structural elements. The key to this method and the fascinating feature of this technique is that the excavation is stabilized during excavation simply by keeping the trench filled with a suitable fluid that exerts a hydrostatic pressure on the walls of the trench and thus, partially compensates for the removal of the original stresses in the ground caused by excavating the soil from the trench. The stabilizing fluid is commonly referred to as “Slurry”.

1.2 Factors that led to the Full-Scale Tests

In 1970 little was known or at least written in engineering journals about the use of this method in soft clay of the type commonly found in Norway. In addition, the slurry trench method was being considered for construction of a new dual rail/subway tunnel through the center of Oslo. This project would require construction of in situ concrete walls to depths of 28 m in soft clay. Therefore, there was a definite need for initiating some basic research on the advantages and problems associated with the use of slurry trenches in soft clay.

1.3 The Vaterland Slurry Trench Test Program

The test program included 2 excavations, a 1 x 1.8 m trench and a 1 x 5 m trench, both 20 m deep. A simplified cross section of the 1 x 5 m trench and the instrumentation used to monitor performance of the excavation are shown in Figure 1 (a).

The test program, which spanned over a period of 51 days, was carefully planned to obtain information that could confirm the overall stability of the trench using water instead of bentonite slurry. The principal objectives of the test program were:

- Monitor the deformation of the surrounding soil and changes in pore pressure.
- Document the stability of the trench as a function of time and the type of stabilizing fluid in the trench.
- Initiate a deliberate failure of the trench by lowering the level of the fluid in the trench until it failed.

Bentonite slurry with a unit specific gravity of 1.077 was used to stabilize both test trenches during excavation. Samples of slurry in the trenches after excavation showed an increase in specific gravity from 1.077 to 1.173 due to the addition of a small amount of natural clay particles during excavation of the trench.

1.4 Special Instrumentation

Changes in the width of the excavated trench due to inward movement of the vertical sides are the most important indicators of the stability of the trench. Therefore, NGI designed, constructed, and used a simple system consisting of a hydraulic cylinder with anchor plates at both ends that could measure the change in width of a trench filled with slurry. As shown in Figure 1 (a), 10 of these devices were installed in the trench when excavation to final depth was completed. Two examples of actual measurements with these instruments are shown in Figure 1 (b).

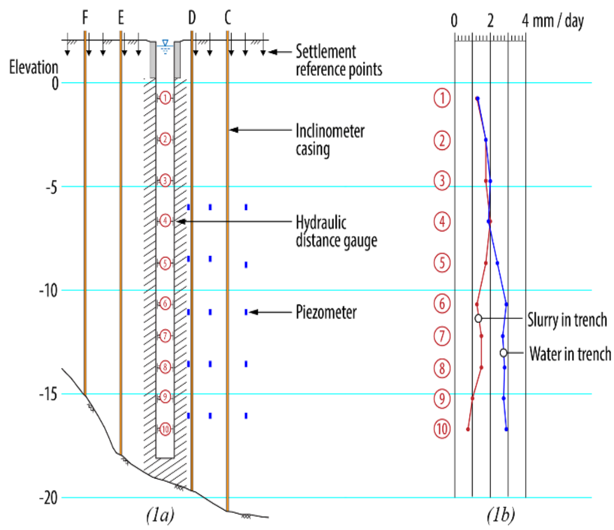


Figure 1 (a) Cross section of the 1 x 5 m trench and instrumentation details. (b) Change in width of the trench the last day it was stabilized with bentonite slurry and change per day afterwards when filled with water.

1.5 Result of Measurements and Conclusions

The 1 x 5 m trench was successfully stabilized, first with bentonite slurry for 7 days, then the bentonite slurry was replaced with water for another 2.5 days. Figure 1 (b) illustrates the small measured changes in width of the trench per day when filled with bentonite slurry or water. When the water in the 1 x 5 m trench was replaced with oil, with a specific gravity of 0.85 the stability of the trench was critical. A partial failure occurred 42 hours later. Detailed analysis of the test data by NGI led to a procedure for calculating the Factor of Safety for slurry trench excavations in soft clay.

2. INTRODUCTION

One of the most fascinating construction techniques developed by foundation contractors and engineers in the early 1950s is the so-called slurry trench method of constructing deep underground concrete walls, shafts or similar structural elements. The key to the slurry trench method and the fascinating feature of this technique is how the excavation is stabilized. This is accomplished simply by keeping the trench filled with a suitable stabilizing fluid during excavation. The stabilizing fluid is commonly referred to as "Slurry". The most important function of the slurry is that it exerts a hydrostatic pressure on the walls of the trench and thus, partially compensates for the removal of the original stresses in the ground caused by excavating the soil from the trench.

The principle slurry trench applications are for construction of in situ concrete walls in granular or sandy material where specially designed slurries are used to form a watertight membrane at the interface between slurry and the soil that prevents leakage of slurry from the trench into the natural ground. Thus, the four principal phases in the application of the slurry trench method for the construction of a section of a concrete wall for example are:

1. Excavation of the trench
2. Stabilization of the trench during excavation by means of a suitable fluid
3. Lower a reinforcing cage into the trench
4. Back fill the trench with concrete to form the wall

Concreting is done by traditional tremie method from the bottom of the trench where the slurry is displaced upwards by the concrete.

The first application of the slurry trench method in Norway was in 1969 in conjunction with the construction of a new office building in the center of Oslo. The slurry trench method was used to construct 2000 m^2 of in situ reinforced concrete walls around the periphery of the site. The walls, which were 1 m thick and ranged in depth from 10 to 21 m, functioned as a caisson or retaining structure during excavation of the soil within the foundation area. The length of each slurry trench panel was 5 m. A bentonite slurry with a relative density of 1.2 was used to stabilize the excavations. The bottom of the panel was keyed into bedrock before the slurry was replaced with concrete. The concrete slurry trench walls also served as the permanent basement walls for the new building. The slurry trench walls were constructed using special equipment leased from the French Company *Soletanche*.

Most of the problems associated with the use of this method have been solved by contractors primarily on a learn as you go basis and they have indeed been remarkably successful in solving most of their problems on their own. Although there has been so much worldwide construction activity with the slurry trench method, very little is known or at least written in engineering journals about the use of this method in soft clay of the type commonly found in Norway.

In 1969, when the above project was carried out, there were several projects being considered in Norway where the slurry trench method of construction would be suitable. One of these was a new combined rail and subway tunnel, one above the other, through the center of Oslo. It was estimated that this project would require construction of in situ concrete walls to depths of 28 m in soft clay. For this reason, there was a definite need for initiating some basic research into the problems associated with the construction of slurry trenches in soft clay.

Therefore, before the special excavating rig and slurry mixing plant were disassembled and returned to France at the end of the project described above for the new office building, it was no surprise that Laurits Bjerrum, the Director of the Norwegian Geotechnical Institute (NGI), engaged the contractor to excavate two full-scale test panels, 20 m deep in soft insensitive marine clay at a test site in Oslo. The objectives of this research project, which was financed entirely by NGI, were to study the deformations that result when an excavation of this kind is made in soft clay and to determine what effect panel length and the density of the slurry have on the deformations, creep and overall stability of the excavation.

Of primary concern was the following question: "Would it be possible to stabilize such an excavation in soft clay with plain water instead of expensive slurries?" Therefore, for these tests the relative density of the slurry varied from 1.077 for a bentonite slurry, to plain water or oil with a relative density of 0.850. The water was replaced with oil in only one of the 2 test trenches. Eventually both trenches were failed deliberately by lowering the level of the fluid in the trench to observe the extent and type of failure that occurred. These tests, which are referred to as the Slurry Trench Tests at Vaterland are the topic of this publication.

3. THE VATERLAND TEST SITE

The location of the Vaterland test site in Oslo and a map of the bedrock contours are shown in Figures 2 and 3. The test site is in an area close to the main train station where the principal soil type is soft marine clay.

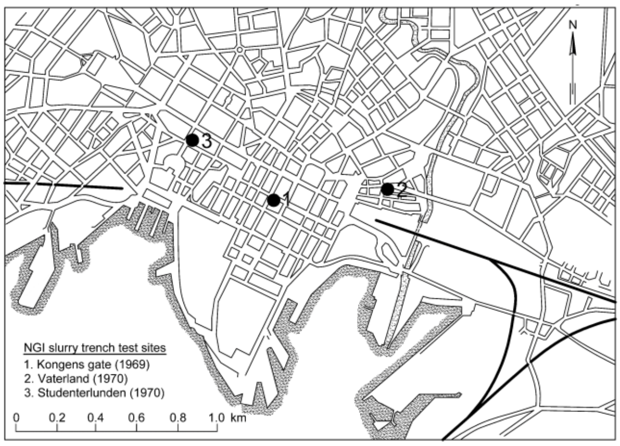


Figure 2 Location of test sites

The test trenches at Vaterland were excavated in one of the clay-filled canyons that are typical of the glaciated bedrock in the Oslo region. The bedrock topography and location of the 2 test trenches are shown in Figure 3.

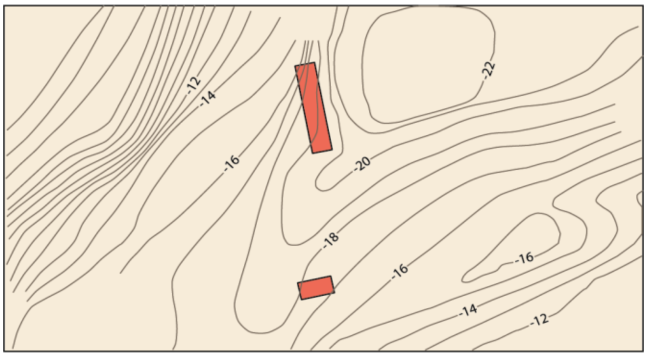


Figure 3 Bedrock profile and location of the 2 test trenches

3.1 Soil conditions at the test site

Preparation of the site for the proposed tests included vane borings and undisturbed sampling, installation of instruments and casting the concrete guide walls for the 2 test trenches. This work started in February 1970.

The ground surface is nearly level and approximately at elevation +2 m. Vane borings were taken at the proposed location of the two test trenches before the tests started. A typical vane boring is shown in Figure 4.

Undisturbed samples were taken near the proposed location of the 1 x 5 m trench. The first two meters of the sub-soil consists of miscellaneous fill material, underlain by a deep deposit of normally consolidated marine clay. The soil profile at the location is shown in Figure 5.

Due to secondary consolidation, some of the clay has developed an apparent pre-consolidation effect. As is usually the case, the upper part of the clay deposit has been weathered to form the stiff dry crust which extends to a depth of about 2 to 4 m. The sensitivity of this clay is about 5 to 8, the unit weight being 1.70 to 1.74 ton/m^3.

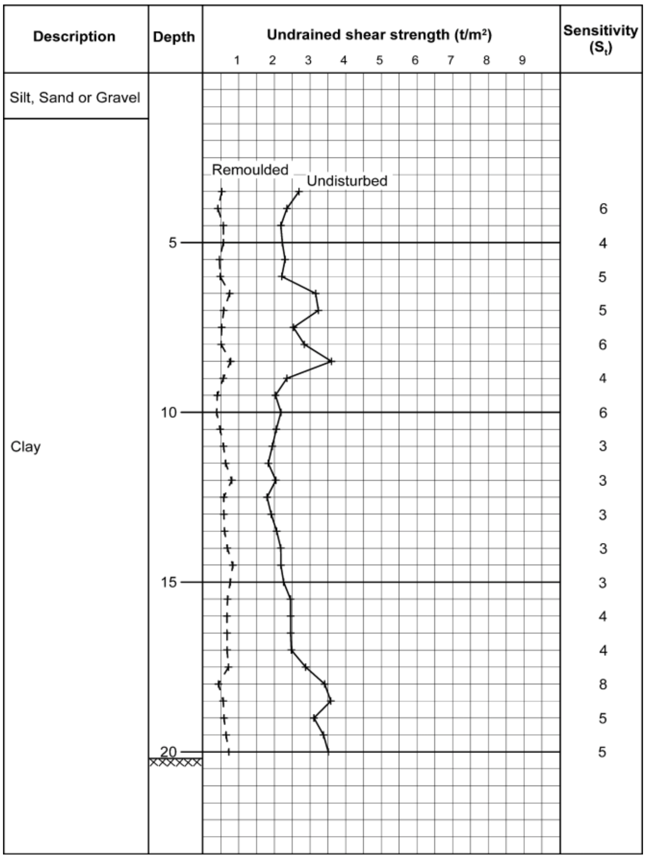


Figure 4 A typical vane boring at the test site

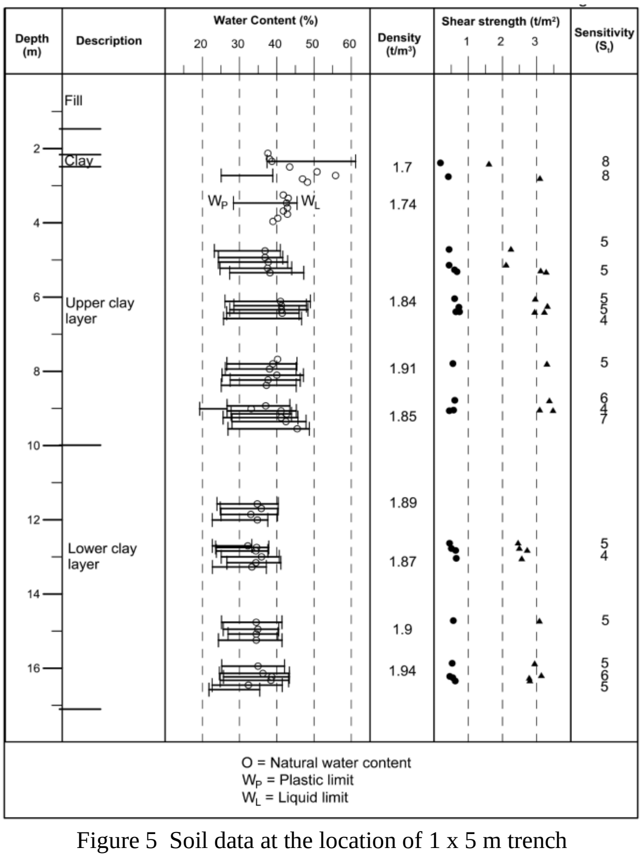


Figure 5 Soil data at the location of 1 x 5 m trench

Below the weathered crust lies an upper layer of soft marine clay to a depth of about 10 m. The liquid limit of this clay varies between 40% to 45% and generally decreases with depth. The plastic limit is more or less a constant with depth and is about 25%. The natural water content decreases with depth from about 40% to about 35%. The unit weight lies in the range of 1.75 to 1.85 ton/m^3, the undrained shear strength has a value of about 2 to 3 ton/m^2. The sensitivity is about 5. The soil properties for the lower clay layer are: unit weight 1.90 ton/m^3, liquid limit 40%, plastic limit 45%, natural water content 35%. The sensitivity is 5.

At the test site there was already installed a precision settlement gauge and 2 open standpipe piezometers that are part of the permanent geotechnical monitoring network for the city of Oslo.

3.2 Details of the test slurry trenches

The test trenches were excavated by means of a mechanical grab attached to a modified KELLY BAR rig. The measured width of the grab was 0.96 m and when fully extended, the opening between the cutting edges was 1.8 m. Therefore, the minimum size excavation was approximately 1.0 x 1.8 m.

NGI decided to have two test excavations i.e., a 1 x 1.8 m trench and a 1 x 5 m trench. A plan view and cross section of the two test trenches are shown in Figures 6, 7, 8 and 9.

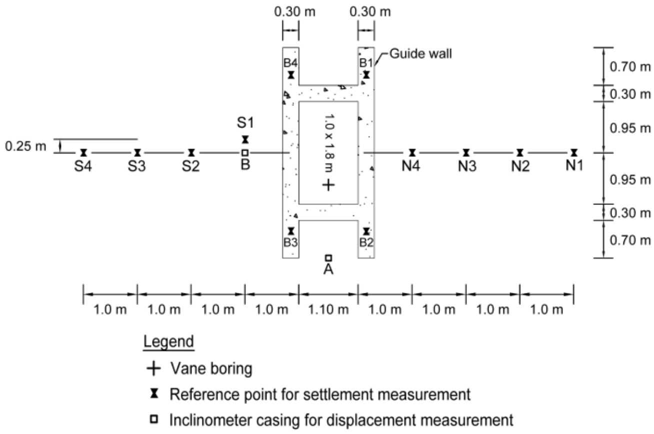


Figure 6 Plan view of the 1 x 1.8 m test trench

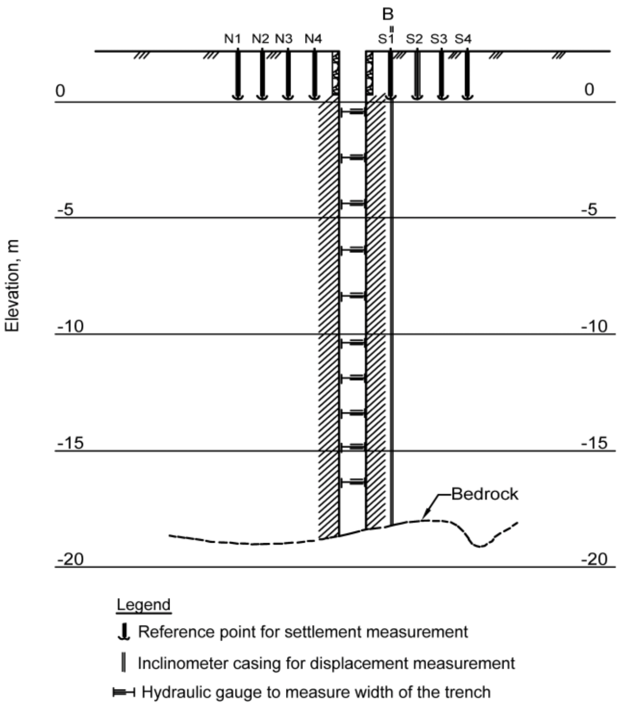


Figure 7 Cross section of the 1 x 1.8 m trench

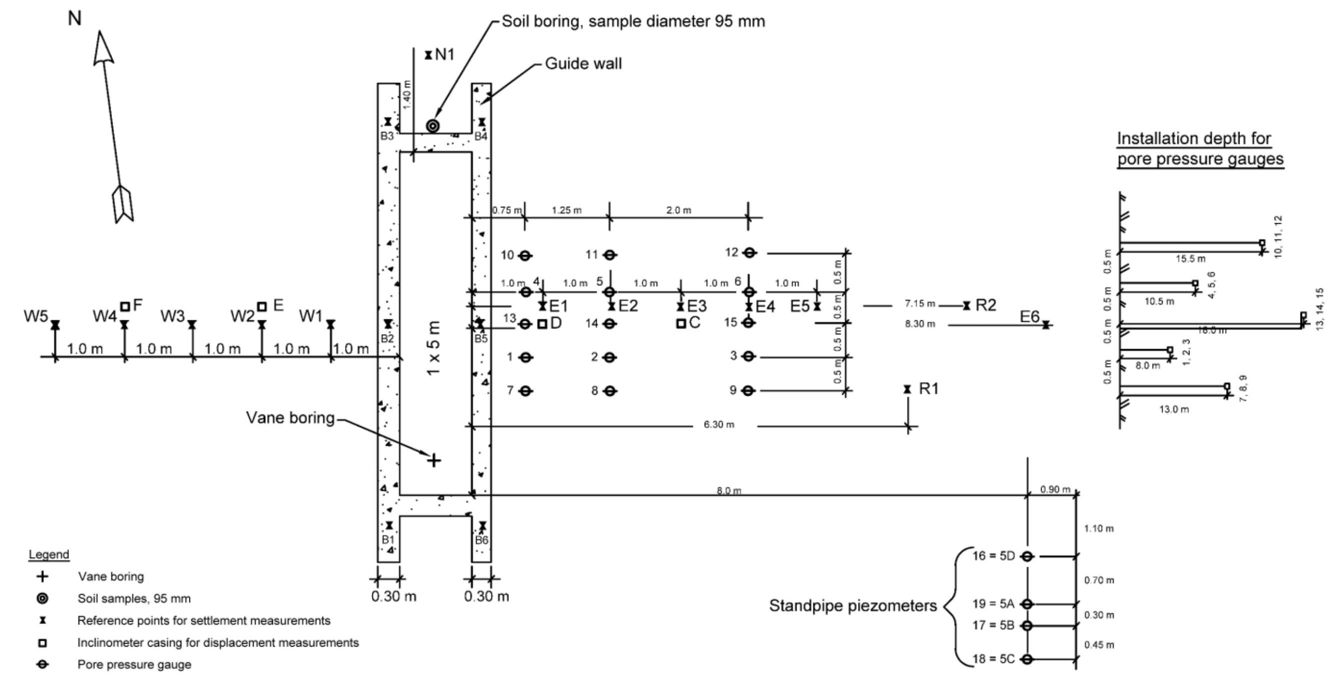


Figure 8 Plan view of the 1 x 5 m trench

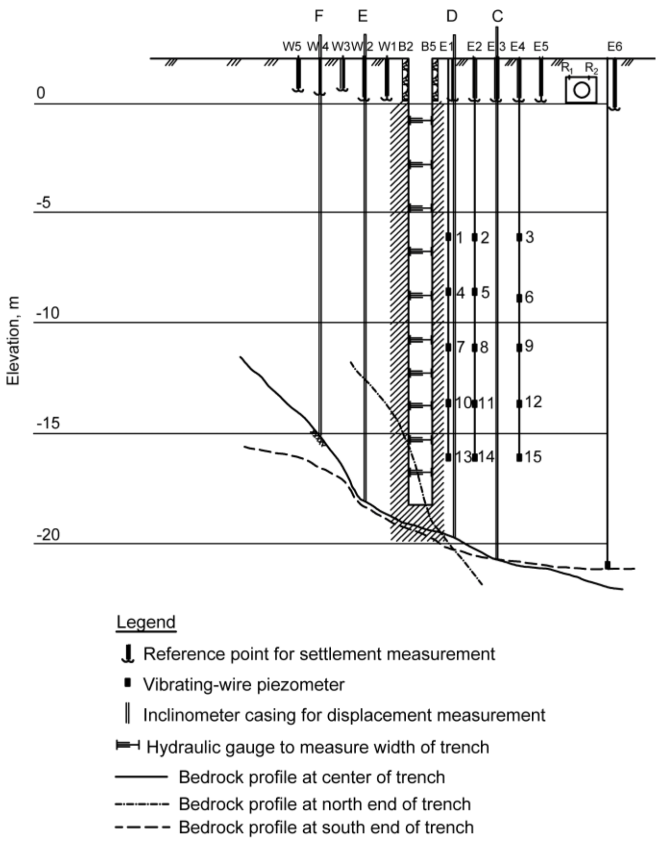


Figure 9 Cross section of the 1 x 5 m trench

3.3 Relative density of the bentonite slurry and oil used in the tests

A bentonite slurry with a relative density of 1.077 was used to stabilize both test trenches during excavation. Samples of the freshly mixed slurry and samples taken at different depths in the first trench excavated were laboratory tested to measure the increase in density caused by the addition of clay particles during the excavation process. These tests showed an increase in specific gravity from 1.077 to 1.173. A summary of the density tests is included in Table 1.

Table 1 Relative density of the bentonite slurry and oil

Sample No.	Measured values	Mean value	Mean value corrected for air buoyancy
Sample 1 Clean Mix	1.07832 1.07836	1.07834	1.0773
Sample 2 Depth 5 m	1.15502 1.15506	1.15504	1.1540
Sample 3 Depth 10 m	1.16980 1.16780 1.16850	1.16872	1.1677
Sample 4 Depth 15 m	1.17258 1.17622 1.17468	1.17449	1.1734
Relative density of the oil used in the tests: 0.850			

4. INSTRUMENTATION DETAILS

The Vaterland tests utilized standard geotechnical monitoring techniques commonly used in Norway in the 1970s to monitor the performance of the slurry trenches. However, there were two exceptions: namely, a vibrating-wire inclinometer probe for measuring horizontal displacements and a unique hydraulic system for measuring changes in width of the trench. The total number and types of geotechnical instruments are listed below.

- 15 Electrical vibrating-wire piezometers for monitoring pore water pressure,
- 4 Open standpipe piezometers for monitoring pore water pressure,
- 28 Reference points for monitoring settlement by means of optical leveling,
- 1 Precision settlement gauge that extended down to bedrock served as a benchmark for measurements.
- 10 Specially designed hydraulic systems for measuring change in width of the trenches.
- 6 Vertical casings to measure horizontal displacement of the clay.

The 15 vibrating-wire piezometers were installed in the vicinity of the 1 x 5 m trench. These piezometers were installed in groups of 3 at five different elevations. This array made it possible to draw contours of changes in pore pressure from the measurements.

The 4 open standpipe piezometers were located approximately 10 m from the center of the 1 x 5 m trench. These were used to detect if significant changes in pore pressure occurred at that distance during the various test procedures.

The settlement reference points were set at a depth of 1.5 m, so that the measurements would be independent of frost action in the ground. The precision settlement gauge mentioned above was used as the reference for optical levelling. Settlements of the concrete guide walls around the periphery of the trenches were also measured.

It is obvious that changes in the width of the excavated trench are the most important indicators of the stability of the trench. Likewise, proper implementation of the proposed test program at Vaterland required almost instant knowledge of the deformation of the slurry filled trenches. Unfortunately, the only off the shelf instrument that was available then to measure changes in width of the trench were borehole inclinometers surveys of the type described in Figure 13 below. Inclinometer measurements require a considerable length of time to take the measurements and calculate the results. For this reason, and in spite of the short-lead time before the Vaterland tests were to start, NGI decided to develop a suitable alternate system for measuring the width of the trenches.

NGI had used hydraulic cylinders, attached to reinforcing steel, to install earth pressure cells in one of the slurry trench panels of the project mentioned in the Introduction to this report. Based on this experience, NGI managed to design, construct, and test a simple hydraulic system consisting of a small diameter hydraulic cylinder with anchor plates at both ends that could measure the width of a trench filled with slurry. A sketch of the measuring system is shown in Figure 10.

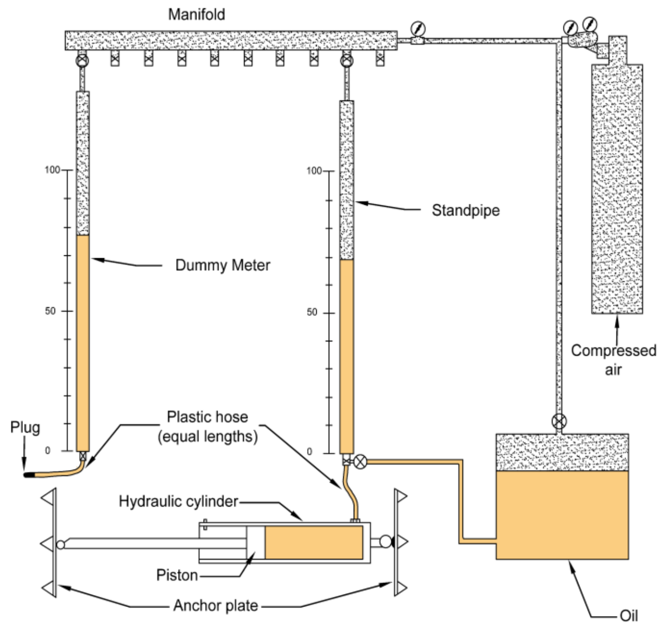


Figure 10 NGI Hydraulic system for monitoring width of the trench

The instruments were attached to the end of a special mounting jig shown in Figure 11 and lowered, one at a time into the slurry filled trench. When the gauge was at the proper depth and oriented in the right direction, oil under pressure from the large container was applied to the hydraulic cylinder to press the anchor plates into the soil on opposing sides of the trench. The oil pressure in the cylinder was maintained relatively high for a short time to firmly seat the anchor plates. Thereafter, the pressure was reduced to a constant value that provides a constant force strong enough to hold the instrument in place.

Once the gauges are in position and properly seated, they are connected to a common manifold that is kept under constant pressure with compressed air. Movement of the walls of the excavation cause a displacement of the piston in the hydraulic cylinder. This movement causes a change in the volume of oil inside the hydraulic cylinder and a simultaneous change in the oil level in the standpipe. Because of the difference in diameters of the piston in the hydraulic cylinder and the standpipe tube, a piston displacement of 1 mm corresponds to a change in elevation of the oil level of 10 mm. In order to compensate for the variations of temperature, pressure and creep in the plastic supply lines in the hydraulic system, a dummy installation was also installed. All standpipe tubes from the surface down to the gauges, including the dummy, are of the same length. The dummy installation, which is connected to the same pressure manifold, has a similar standpipe and supply line, except the supply line is sealed at the lower end. Readings on the dummy gauge, which react only to pressure and temperature changes, are used to correct the readings on the active standpipes.

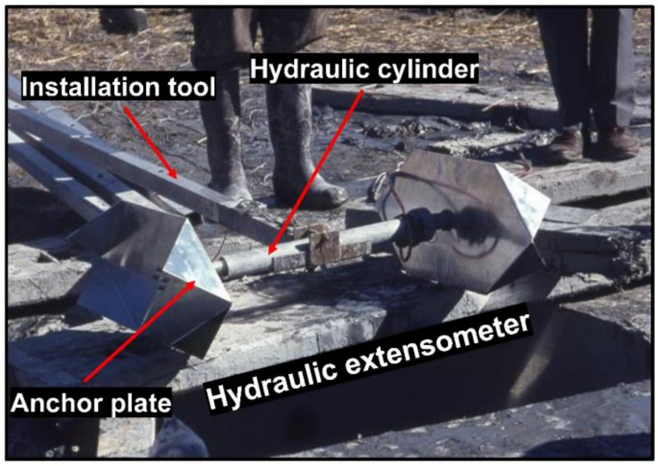


Figure 11 Installation tool attached to the instrument

The array of standpipes show immediately changes in the width of the trench in real time; and if the initial oil level in all the standpipes were the same, at the start of a test, the change in oil levels represents the actual profile of the width of the trench. Figure 12 is an actual photograph taken during drawdown of the water in the 1 x 1.8 m trench to cause a deliberate failure of the trench.

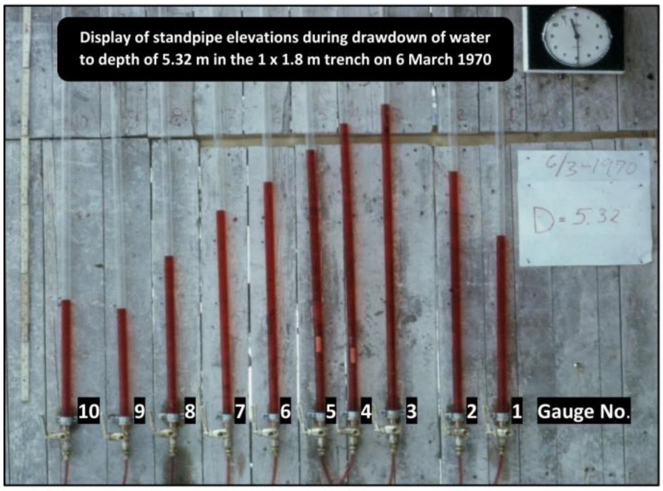


Figure 12 Array of standpipes connected to the hydraulic gauges

An inclinometer probe, based on the principles of the vibrating-wire strain gauge, Figure 13, was used to measure the horizontal displacements of the vertical casings installed close to the test trenches.

Continuous measurements were taken at intervals of 30.48 cm (12 inches) the spacing of the wheels on the instrument. There were 2 wheels at the top and one at the bottom making it possible to track properly on the flat surfaces of the square casing in the ground. The instrument had a sensitivity of approximately 0.0002 radians and a reading accuracy of about ± 0.008 radians.

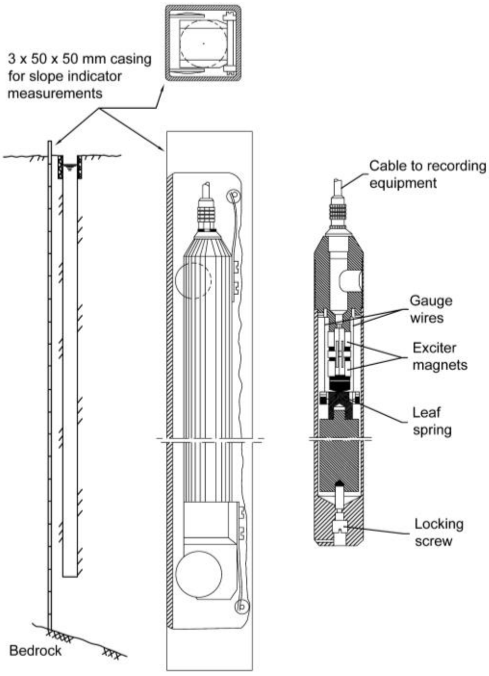


Figure 13 Vibrating-wire Inclinometer probe

5. TEST PROGRAM AT VATERLAND

The test program at Vaterland was carefully planned to obtain information which could confirm the overall stability of the trench using water instead of bentonite slurry. The three principal objectives of the test program were:

1. Monitor the deformation of the surrounding soil and changes in pore pressure during excavation of the trench.
2. Evaluate the stability of the trench as a function of time, type of stabilizing fluid in the trench, and the level of the fluid in the trench.
3. Initiate a deliberate failure by lowering the level of the fluid in the trench until a failure occurs.

With these objectives in mind, the testing program to achieve these goals is summarized below in separate tables, one for the 1 x 1.8 m trench and one for the 1 x 5 m trench.

The entire test program spanned over a period of 51 days. For convenience, time is expressed primarily as Day Number, not the calendar date. A list of Date and corresponding Day Number is given in Table 2 for the 1 x 1.8 m trench and Table 3 for the 1 x 5 m trench.

6. SUMMARY OF TEST PROCEDURES

Table 2 Test procedure for the 1 x 1.8 m trench

Date 1970	Day Number	Test procedures
26 Feb	30	Excavation of 1 x 1.8 m trench with bentonite slurry, specific gravity 1.077
	30 to 36	Observed the performance of the trench filled with bentonite slurry
4 March	36	Replaced the bentonite slurry in the trench with water
5 March	37	Lower water level to depth of 4.3 m and observe performance
6 March	38	Lower water level to depth of 8.3 m Maximum change in width 13 cm
9 March	41	Filled trench with bentonite slurry, specific gravity 1.077
	41 to 71	Observe the performance of the trench filled with bentonite slurry
25 March	56	The hydraulic displacement gauge No. 4 at depth of 6.75 m fell down

Table 3 Test procedure for the 1 x 5 m trench

Date March	Day Number	Time	Test procedures
2	34		Excavation of 1 x 5 m trench, with slurry specific gravity of 1.077
2	34	13:15	Excavation of north side of trench completed, 1.8 m wide
2	34	14:45	Excavation of south side of trench completed, 1.8 m wide
2	34	16:45	Excavation of central part to create the 1 x 5 m trench
	34 to 41		Observe the performance of the trench filled with bentonite slurry
9	41	18:30	Replaced bentonite slurry in the trench with water
11	44	11:05	Started replacement of water in trench with oil, density 0.850
12	44	14:35	Trench completely filled with oil. Total volume of oil 88.7m^3
	44 to 46		Observe the performance of the trench filled with oil
14	46		Replaced oil in trench with water
	47 to51		Observe the performance of the trench filled with water
19	51		Started lowering water level in trench to cause a deliberate failure
19	51		Water level lowered 1 m every 30 minutes to cause a failure
19	51	09:10	Water level lowered 1.0 m
19	51	09:40	Water level lowered 2.0 m
19	51	10:10	Water level lowered 3.0 m
19	51	10:40	Water level lowered 4.0 m
19	51	10:55	Water level lowered 4.7 m **The trench failed!**

7. EXAMPLES OF TEST RESULTS

A large number of technical drawings of pertinent test details and results of measurement data were produced by NGI during the analysis of the measurement date. Most of these are too large to present in this publication. Therefore, only a few selected examples are included in the text. In this publication, only the eight typical examples of measurements listed in Table 4 are included in this report to illustrate some key test results.

Readers who are interested in additional or more specific information about the slurry trench tests at Vaterland can contact NGI's Document Center and inquire about availability of the original NGI project report 55401-06 and available drawings.

Table 4 Examples of measurement for key test stages

No.	Trench	Data presented
7.1	1 x 5 m	Measured pore pressure and settlement changes during excavation of the trench
7.2	1 x 5 m	Change in width of trench, pore pressure and settlement when water replaced with oil
7.3	1 x 5 m	Rate of change in width of the trench when filled with different stabilizing fluids
7.4	1 x 5 m	Profiles showing reduction in width of the trench for various test stages
7.5	1 x 5 m	Measured creep rate for the width of the trench for different fluids
7.6	1 x 5 m	Measured pore pressure and settlement during deliberate failure of the trench
7.7	1 x 5 m	Measured pore pressure and settlement after the deliberate failure of the trench
7.8	1 x 1.8 m	Settlement and width of the trench during drawdown of water to 8.3 m

7.1 Example 1: Measured change in pore pressure and settlement during excavation of the 1 x 5 m trench

The 1 x 5 m trench was 20 m deep. It was excavated in three steps using a grab that had a width of 0.96 m and when fully extended the distance between the cutting edges was 1.8 m. Therefore, the first two steps consisted of excavating a 1 x 1.8 m wide excavation to design depth at both ends, then excavation of the central part to complete the trench. Bentonite slurry was used to stabilize the trench during excavation. The time required to complete the excavation was about 4 hours. Pore pressure and settlements were measured continuously during the excavation. A sample plot of the measurement data when the trench was completed is summarized in Figure 14.

7.2 Example 2: Measured change in width, pore pressure and settlement when replacing water in the 1 x 5 m trench with oil

On Day No. 44 the water in the 1 x 5 m trench was replaced with 88.7 m^3 of oil that had a relative density of 0.850. The operation took 3.5 hours. The measured changes in pore pressure, width of the trench and settlement are illustrated in Figure 15 for 5 stages in the operation. The maximum change in the width of the trench was only 13 mm when the trench was filled with oil. The trench was observed for 2 days. During the second day the upper part of the trench was not stable. The oil was replaced with water on Day 46. On Day 51 the water in the trench was drawn down in steps until a failure occurred. The failure occurred when the drawdown level was 4.7 m.

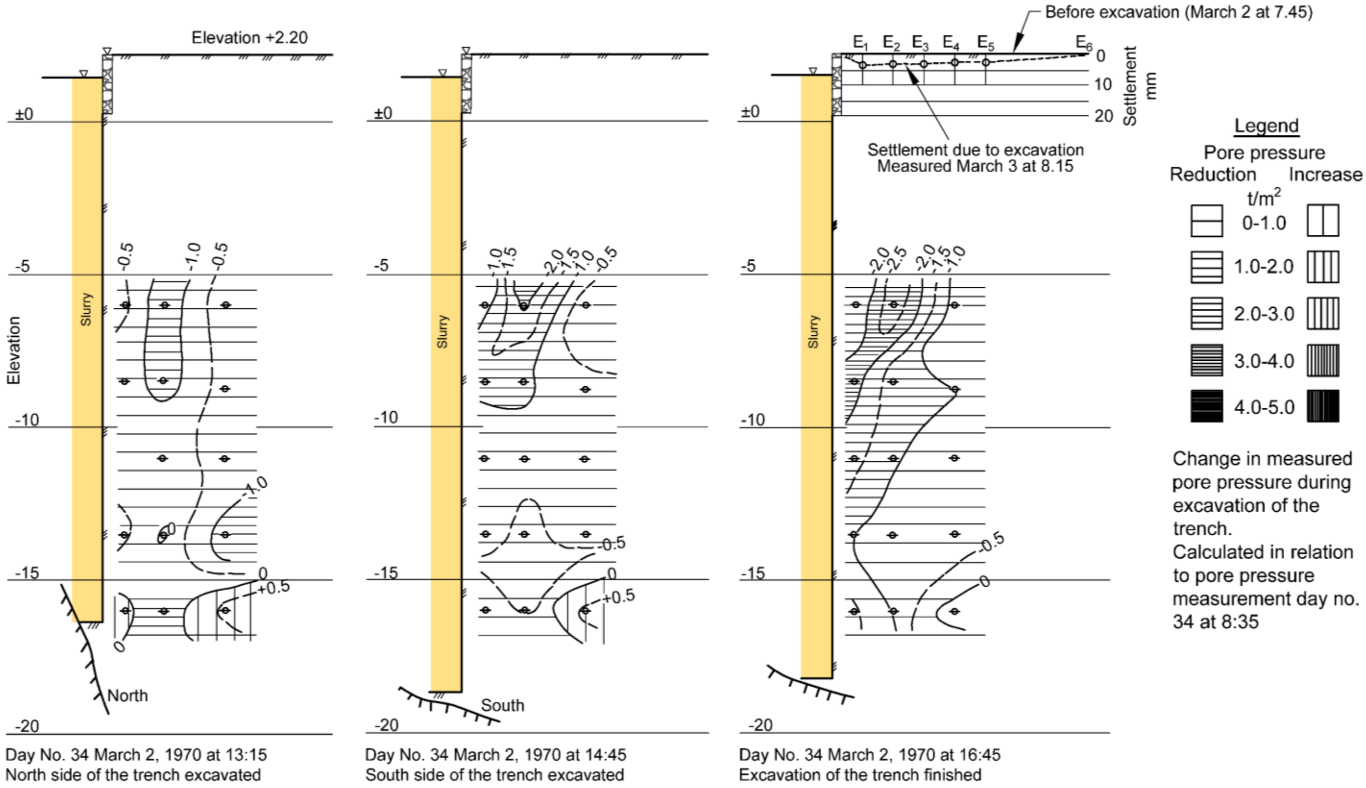


Figure 14 Measured change in pore pressure and settlement during excavation of the 1 x 5 m trench

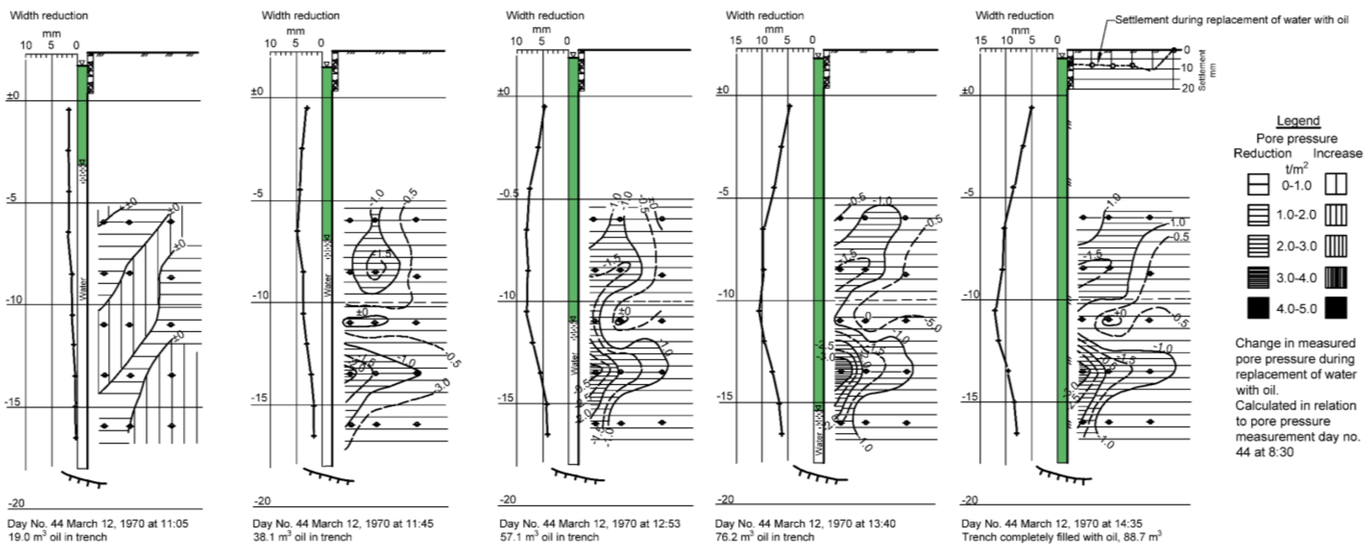


Figure 15 Change in width of the trench, pore pressure and settlement after replacing water in the 1 x 5 m trench with oil

7.3 Example 3: Rate of change in width of the 1 x 5 m trench when filled with different stabilizing fluids in the trench

Figure 16 illustrates the reduction in the width of the 1 x 5 m trench, measured with the 10 hydraulic displacement gauges, as a function of time when it was stabilized with bentonite slurry, water, or oil. *This example probably illustrates the most important results for the Vaterland tests*. The observed creep rates per day, for the three stabilizing fluids used in the tests, are determined from the time-displacement plots and summarized in Table 5 at the bottom of this page.

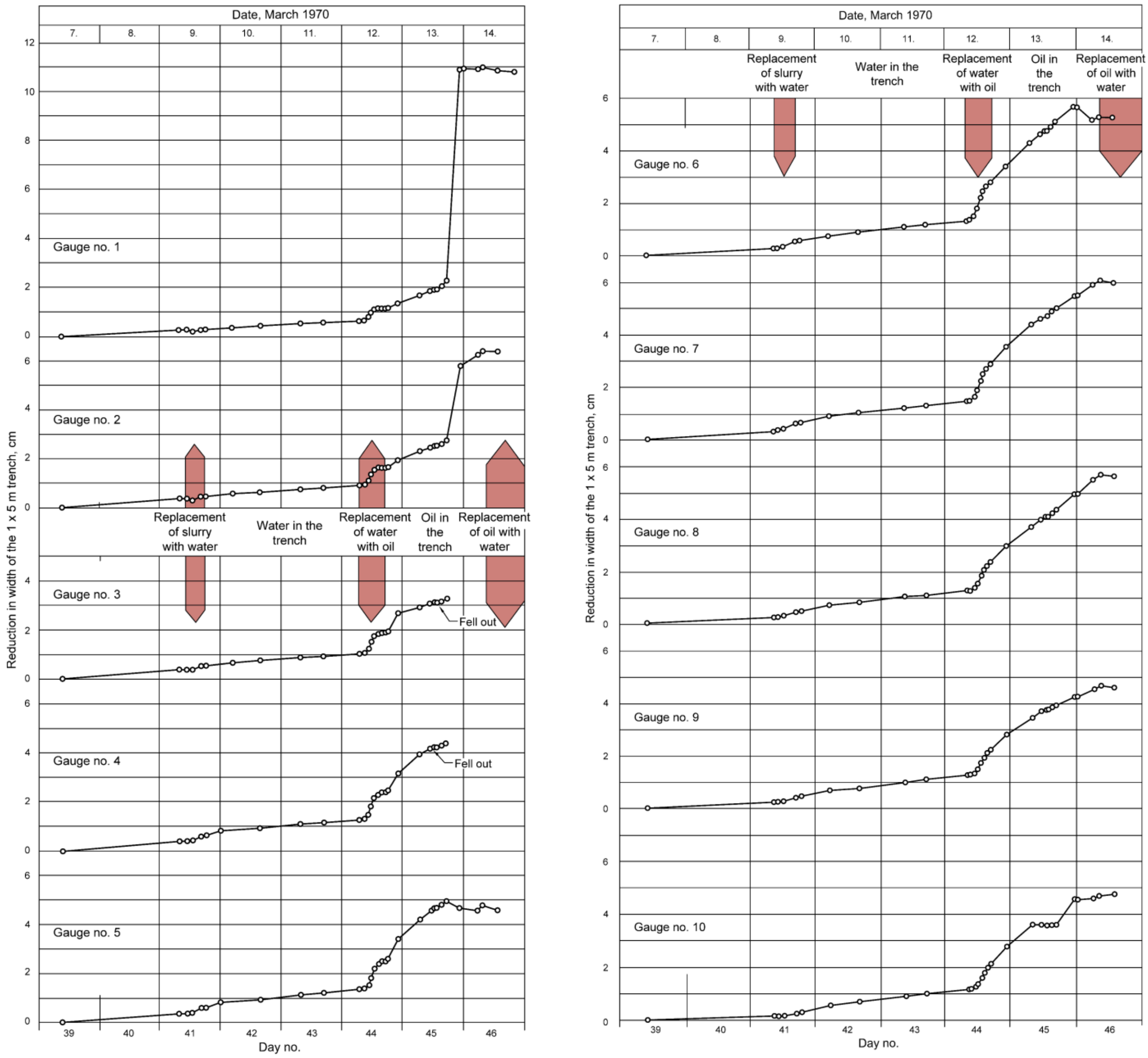


Figure 16 Rate of change in width of the 1 x 5 m trench versus time for bentonite slurry, water and oil

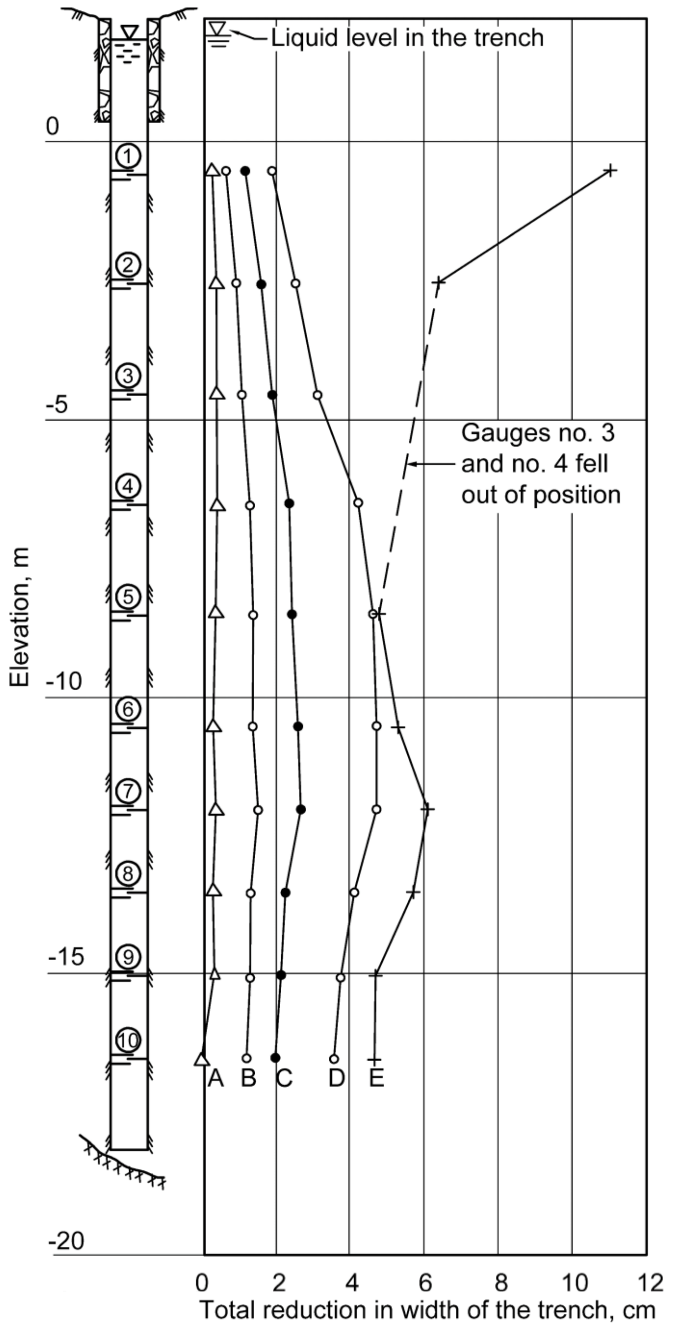
Table 5 Rate of reduction in width of the 1 x 5 m trench

Gauge number	Measured rate of change in width of the trench, mm/day		
	Bentonite Slurry in the trench	Water in the trench	Oil in the trench
1	1.25	1.50	8
2	1.75	1.75	10
3	1.75	1.75	10
4	2.00	2.00	19
5	1.75	2.25	23
6	1.25	3.00	25
7	1.50	2.75	21
8	1.50	2.75	21
9	1.25	2.75	16
10	0.75	3.00	18
Average	**1.4 mm/day**	**2.3 mm/day**	**17.1 mm/day**

7.4 Example 4: Profiles showing reduction in width of the 1 x 5 m trench for various test stages

The 1 x 5 m trench was stabilized with bentonite slurry during excavation. Five days after the excavation was completed, 10 hydraulic instruments were installed in the trench to measure changes in width. The bentonite slurry was replaced with water 2 days later. Then and the stability of the water filled trench was monitored for 60 hours. Next, the water was replaced with oil.

Profiles showing the change in width of the trench for each of these test stages are shown in Figure17.



Legend
A: Before replacing bentonite slurry with water
B: After 60 hours with water in the trench
C: Trench completely filled with oil, Day 44
D: 23 hours after the trench was filled with oil
E: 42.5 hours after the trench was filled with oil Large displacements at top of the trench.

Figure 17 Measured reduction in width of the trench

7.5 Example 5: Creep rates for the width of the 1 x 5 m trench for diferent fluids

In general the shape of the displacement curves showing the change in width of the trench have similar shapes. Figure 17 shows that the maximum reduction in width of the trench for the 4 profiles A, B, C and D were measuured by the hydraulic displacement gauge No. 7. That also applies to profile E provided one neglects the upper part of the curve where there was a local failure after the trench was filled with oil. The creep rate for the different fluids in the trench are illustrated for Gauge 7 for all the profiles.

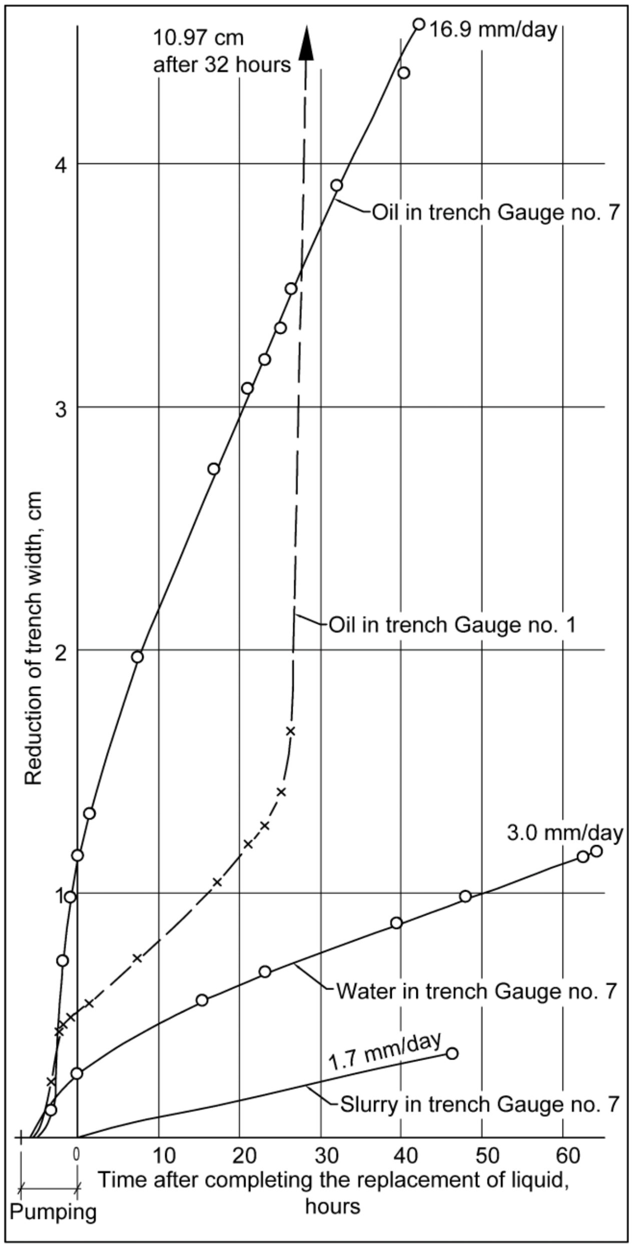


Figure 18 Example of creep rates for different fluids

Table 5 Observed reduction in width of 1 x 5 m trench, mm/day

Trench profile	Fluid in trench	Measured interval	Reduction in width		
			Min	Max	Ave.
A	Bentonite	48 hours	0.75	2.0	1.4
B	Water	60 hours	1.5	2.3	2.3
D	Oil	23 hours	8	25	17
E	Oil	42 hours		90	

7.6 Example 6: Measured pore pressure and settlement during the deliberate failure of the 1 x 5 m trench

On Day 44, the oil in the 1 x 5 m trench was replaced with water. Monitoring continued up to Day 51 when it was decided to initiate a deliberate failure of the trench. The failure was initiated by lowering the water level in the trench in steps of 1 m every 30 minutes. The first drawdown, to 1 m, was at 9:10. Settlement and pore pressure were monitored during the test.

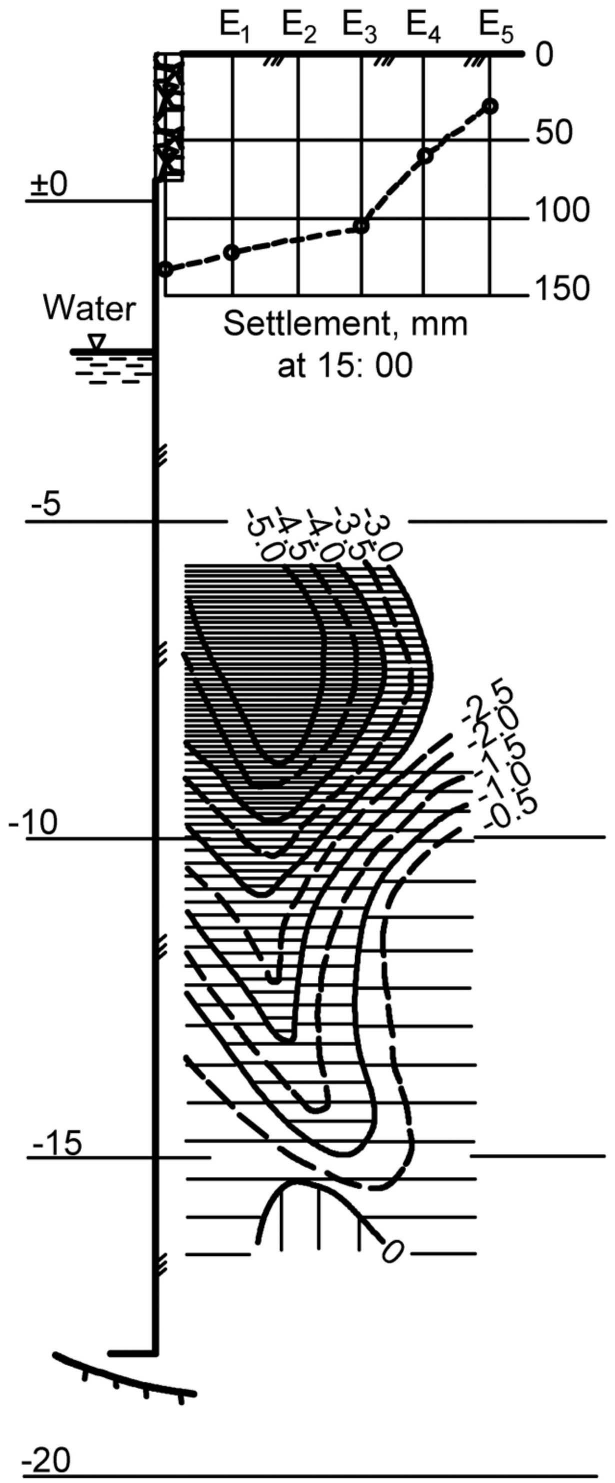


Figure 19 Measured pore pressure and settlement when the 1 x 5 m trench failed

The failure occurred when lowering the water level from 4 m to 5 m. When the drawdown reached 4.7 m, the trench failed. The time from start of the test to failure was approximately 2 hours.

7.7 Example 7: Measured pore pressure and settlement after the failure of the 1 x 5 m trench

The trench failed at 10:55, 15 minutes after the water level was lowered 4.7 m. Figure 20 shows the pore pressure distribution 4 hours after failure. The 15 vibrating wire piezometers were arranged in a grid between the depths of 8 and 18 m. Thus, it was possible to determine contours of pore pressure variations as indicated in the two figures.

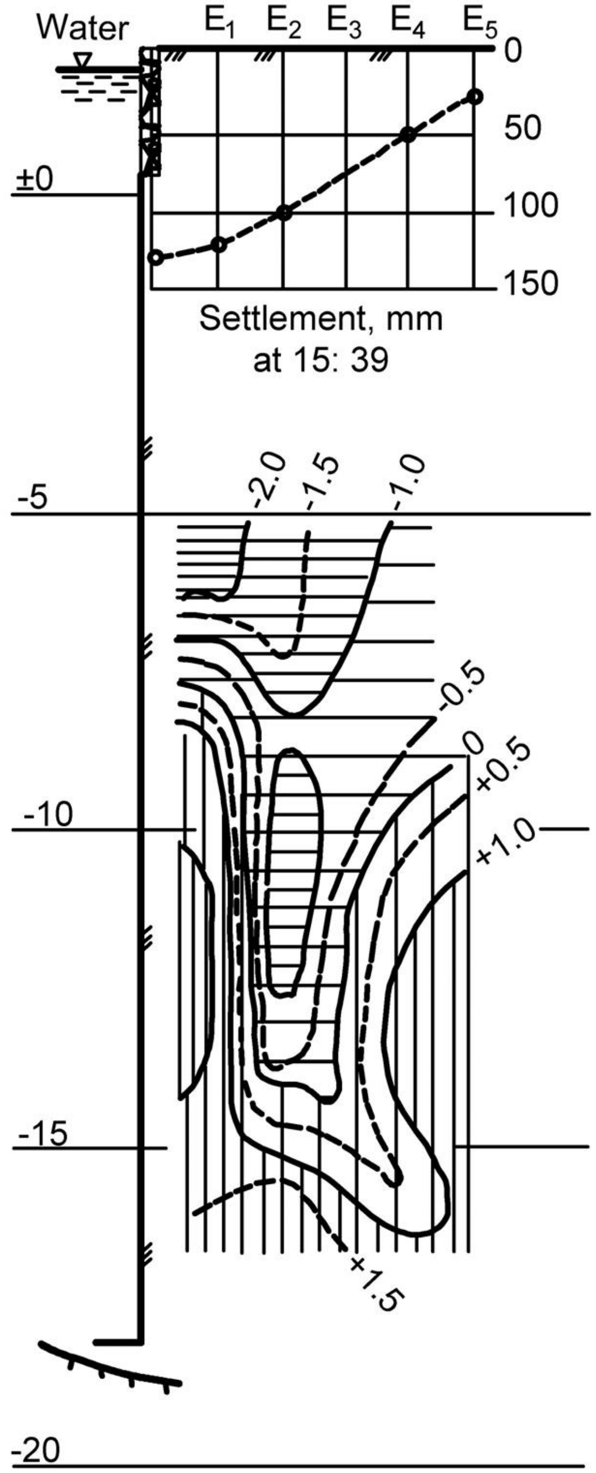


Figure 20 Measured pore pressure 4 hours after the failure of the trench

The pore pressure after the failure shown in the Figure 20 was measured approximately 4 hours after the deliberate failure of the trench. The maximum surface settlement was 13 cm.

7.8 Example 8: Measured settlement and width of the 1 x 1.8 m trench during drawdown of water to 8.3 m

The 1 x 1.8 m trench was stabilized for 6 days with bentonite slurry, then 5 days with water. During this interval, some minor tests were performed. On Day 38, the water level in the trench was lowered in steps to 8.3 m to create a deliberate failure of the trench. Figure 21 shows the settlement profiles on both sides of the trench before and after drawdown to failure. The upper settlement profile shows that the maximum observed settlement was only about 6 cm before the drawdown to failure started. Figure 22 illustrates the change in width of the trench during the test. When the water level was lowered to 8.3 m, the hydraulic displacement gauge No. 4 fell down.

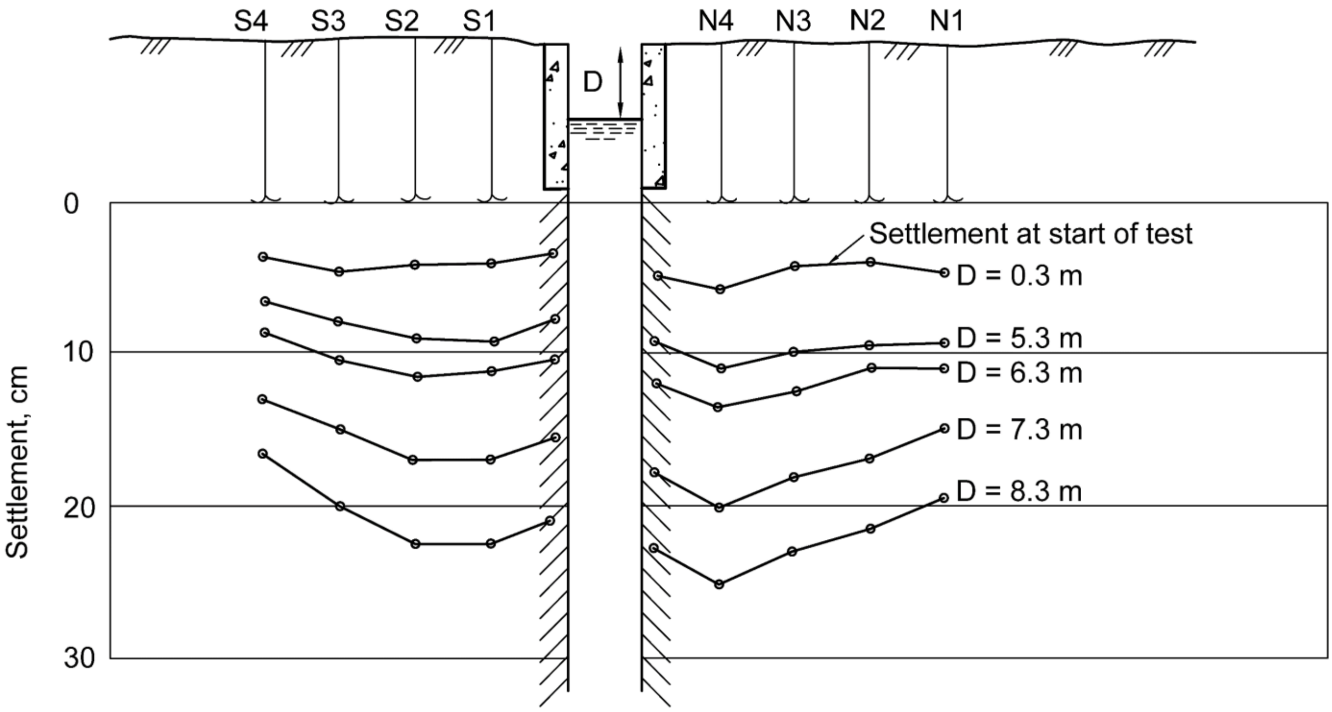


Figure 21 Settlement profiles before and during the drawdown of water in the 1 x 1.8 m trench

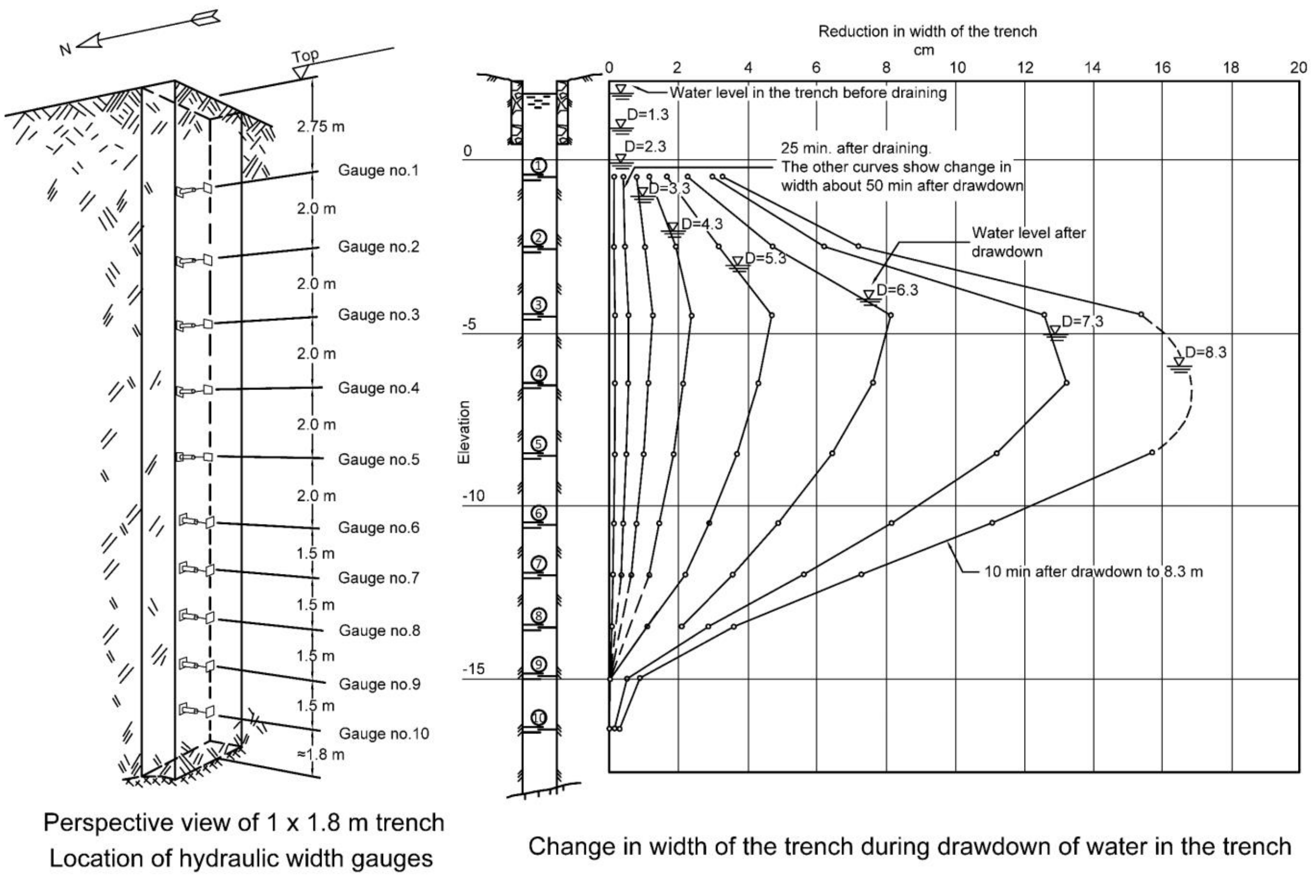


Figure 22 Change in width of the 1 x 1.8 m trench measured by the hydraulic gauges during deliberate failure of the trench

8. CALCULATING THE STABILITY OF SLURRY TRENCHES

One of the most important results from the full-scale tests of slurry trench excavations at Vaterland is a general procedure for calculating the Factor of Safety for excavations of this kind. The procedure was developed by Gunnar Aas. He was Head of the Foundation Section at the Norwegian Geotechnical Institute when the tests were carried out. The Lead Editors of this publication asked him to prepare a brief introduction to the method and to apply it to the Vaterland full-scale tests. *Readers who are interested in the complete mathematical background of the procedure are advised to consult the reference G. Aas (1976).*

8.1. Description of the stability analysis

The analysis is based on the level and density of the stabilizing fluid in the trench, the type of clay and dimensions of the excavation. The assumed failure criteria shown in Figure 23 (a) includes two separate sliding blocks. A lower wedge-shaped body, which is assumed to slide horizontally into the trench along two planes inclined at 45° and an upper body, which is assumed to move vertically downwards. Observed displacement patterns in the test trenches have confirmed that this type of failure body is relevant.

Along the two planes inclined at 45° the stress conditions correspond to an active failure state. Hence, the mobilized shear resistance along these planes is assumed equal to active triaxial compression strength at a corresponding stress level. The shear strength on the vertical surfaces of the excavation is set equal to the measured vane strength modified, if necessary, to compensate for the presence of items like open cracks in the weathered crust or the tendency of over-estimation of strength in the underlying weathered clay or the possibility of progressive failure occurring between the different contributors of shear resistance. The vane shear strength is assumed equal to the actual observed vane strength at the bottom of the sliding body (depth D). It is assumed to decrease linearly with decreasing depth with a value of zero at terrain level. This set of very simplified assumptions results of course in some uncertainty about the calculations.

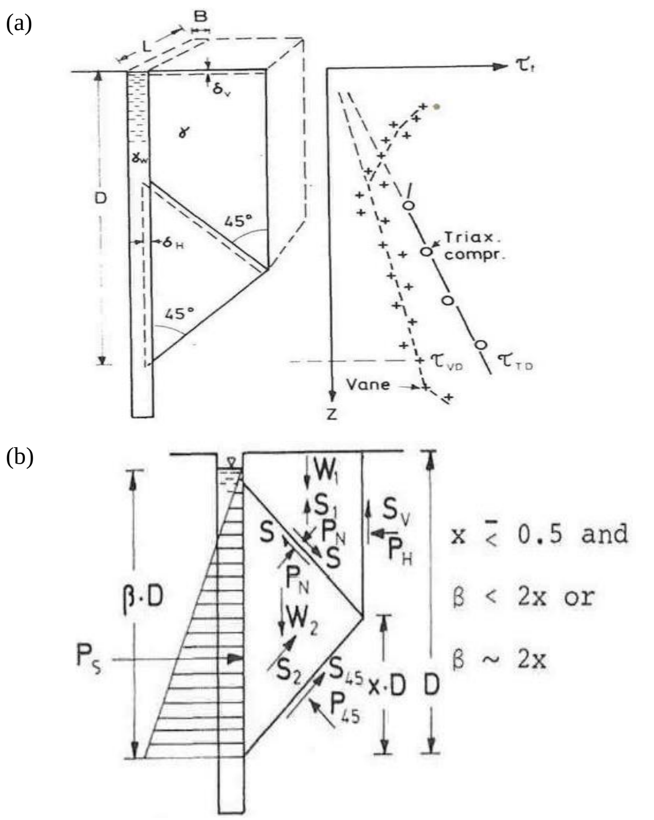


Figure 23 (a) Assumed failure condition, (b) Stability analysis

The general formula for the factor of safety for a slurry trench can be written in a form similar to the well-known expression for stability calculations for cuts and excavations.

$$FS = \frac{\tau_{VD}}{D(\gamma - \beta^2 \gamma_f)} Ns \qquad (1)$$

The factor of safety expresses the relation between vane shear strength and the stress difference outside and inside the trench, and finally multiplied with a stability number Ns. However, in this case, the stability number reflects not only the geometrical dimensions of the excavation. It also takes into consideration the anisotropic shear strength of the clay. In the formula, D is the maximum or critical depth of the trench, γ and γ_f are the densities of the clay and the fluid, respectively. The vane strength at depth D is τ_{VD} and the corresponding strength value from triaxial compression tests, τ_{TD}. For practical reasons it was felt necessary to choose a general value for the ratio between these two values, τ_{TD}/τ_{VD}.

Based on the failure criteria in Figure 23 (a) formula has been established that expresses FS for the trench as a function of the variables D/L, τ_{TD}/τ_{VD} and x. By setting the derivatives with respect to x equal to zero for this expression, one obtains a function that describes a unique relationship between D/L and x. This implies that for a given value of D/L there exists, as shown in the upper diagram of Figure 24 (a) value of x which may be used to determine the stability number Ns.

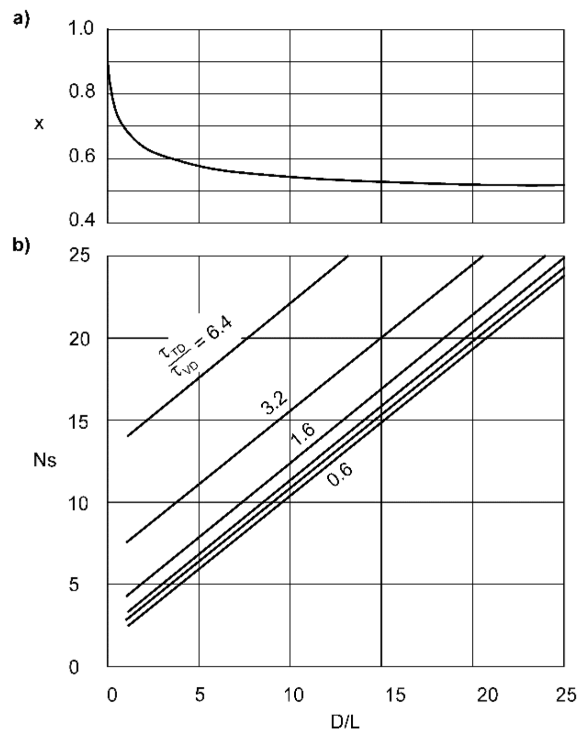


Figure 24 Graphical diagram for determination of Ns

For typical values of x, equal to 0.54 - 0.6 in Figure 24, corresponding to D/L = 10 - 4 in the lower diagram of the figure, are the calculated values of Ns quite constant and equal to $2\tau_{TD}/\tau_{VD} + 0.3 + 0.9\,D/L$. Hence the complete expression required for calculating the best possible Factor of Safety (FS) for a slurry trench at Vaterland should be the following:

$$FS = \frac{\tau_{VD}}{D(\gamma - \beta^2 \gamma_f)} \left(2\frac{\tau_{TD}}{\tau_{VD}} + 0.3 + 0.9\frac{D}{L}\right) \qquad (2)$$

Where,

FS	Factor of safety
τ_{VD}	Vane strength at depth D, t/m²

τ_{TD} Strength from Triaxial compression test, t/m²
D Maximum or critical depth of the trench, m
L Length of the trench, m
β Height of fluid/D
γ Density of the clay, t/m³
γ_f Relative density of the fluid in the trench, t/m³

Tables 6 - 10 contain the input data to use in Equation (1) for calculating the Factor of Safety.

As mentioned earlier, the Vaterland site included two test trenches with cross sections of 1 x 1.8 m and 1 x 5 m and a depth of 20 m. The soil conditions at the site are shown in Figure 25.

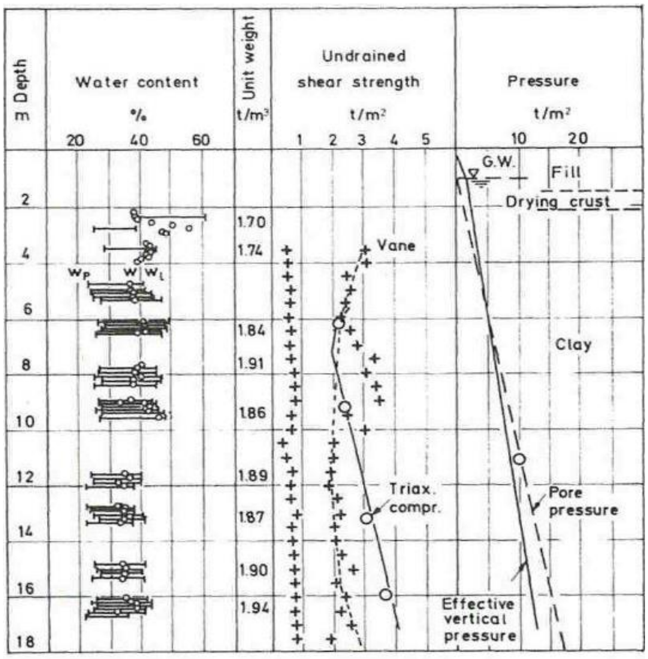


Figure 25 Soil conditions at the Vaterland site

8.2 Deliberate failure of the 1 x 1.8 m trench

The 1 x 1.8 m trench was excavated using bentonite slurry, which was replaced with water after 5 days. For the time interval the trench was filled with water, the analysis yields a minimum safety factor of 1.92 and a critical depth of 13 m.

Table 6 shows the calculated factor of safety for the 1 x 1.8 m trench filled with water.

Table 6 Stability of the 1 x 1.8 m trench filled with water

D	L	τ_{VD}	τ_{TD}	γ	γ_f	β
13	1.8	2	3.2	1.8	1.0	1.0
m	m	t/m²	t/m²	t/m³	t/m³	-
Calculated Factor of Safety = 1.92						

The next day, the water level was lowered in steps of 1 m until the trench failed after about 20 hours. The depth to the water level in the trench was then 8.3 m. The critical depth for the conditions under which the trench failed was about 9 m. The computed value of the safety factor is shown in Table 7.

Table 7 Drawdown of the 1 x 1.8 m trench to 8.3 m

D	L	τ_{VD}	τ_{TD}	γ	γ_f	β
9	1.8	2	3.2	1.8	1.0	0.07
m	m	t/m²	t/m²	t/m³	t/m³	-
Calculated Factor of Safety = 0.99						

In Figure 26 the measured decrease in width of the trench and the factor of safety is plotted as a function of the depth D to the tip of the bottom failure block. The observed total inward movement of the trench wall just before failure shown in Figure 26 clearly confirms the theoretical critical depth.

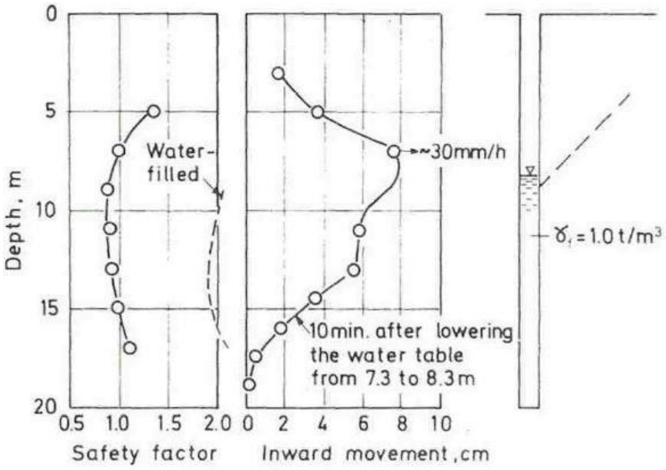


Figure 26 Calculated safety factors and observed change in width of the 1 x 1.8 m trench

7.3 Deliberate failure of the 1 x 5 m trench

During excavation of the 1 x 5 m trench and the following 7 days the trench was stabilized using a bentonite slurry which has a density of 1.17 t/m³. Then the slurry was replaced with water, leading to a critical depth of about 13 m, and the factor of safety shown in Table 8.

Table 8 Stability of the 1 x 5 m trench filled with water

D	L	τ_{VD}	τ_{TD}	γ	γ_f	β
13	5	2	3.2	1.8	1.0	1.0
m	m	t/m²	t/m²	t/m³	t/m³	-
Calculated Factor of Safety = 1.11						

After another 3 days the water was replaced with oil with a density of 0.85 t/m³. This condition turned out to be critical, as a slide took place and the trench was partly filled with clay debris. As can be seen in Table 9 the calculated critical depth for the oil filled trench was approximately 13 m and a safety factor of 0.95.

Table 9 Stability of the 1 x 5 m trench filled with oil

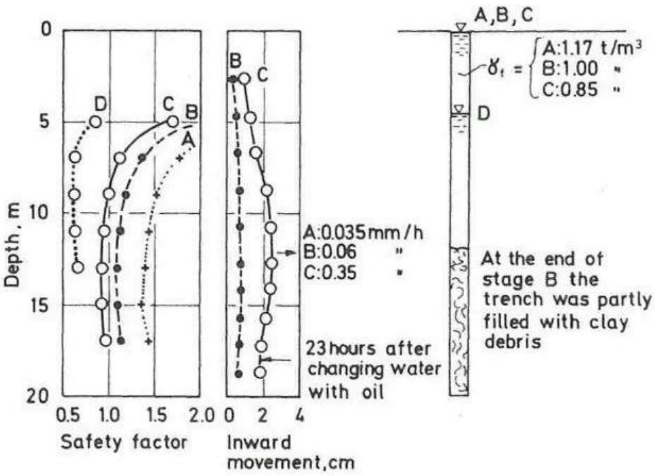
D	L	τ_{VD}	τ_{TD}	γ	γ_f	β
13	5	2	3.2	1.8	0.85	1.0
m	m	t/m²	t/m²	t/m³	t/m³	-
Calculated Factor of Safety = 0.95						

The next step was to replace the oil in the trench with water. Then, 5 days later the water level in the trench was lowered in steps of 1 m to initiate a deliberate failure. After about 2 hours, when the water level was lowered to 4.7 m, the trench failed. The results are shown in Table 10.

Table 10 Deliberate failure of the 1 x 5 m trench

D	L	τ_{VD}	τ_{TD}	γ	γ_f	β
9	5	2	3.2	1.8	1.0	0.64
m	m	t/m²	t/m²	t/m³	t/m³	-
Calculated Factor of Safety = 0.70						

Figure 27 shows the calculated safety factors as a function of the depth D, the observed inward movements of the trench walls, the depth of maximum deformation, and the creep rate for the different stabilizing fluids mentioned above.



Fluid in the trench: ***A****: Bentonite slurry,* ***B****: Water,* ***C****: Oil*

Figure 27 Calculated safety factors as a function of D and observed change in width of the 1 x 5 m trench

The slurry trench tests at Vaterland have shown that in soft clay there was no serious problem associated with excavation of the trenches and that, in this case, it was possible to use only water as a stabilizing fluid in the trenches. However, it is possible that the use of water requires that the clay is homogeneous, without layers or inclusions of coarser materials. Although the method of analyzing the test trenches in soft clay has given reliable results, applications with other types of clay may require verification by new test trenches.

It is of interest to note that the calculated safety factor for the 1 x 5 m trench filled with water is 1.11 compared to the corresponding value of 1.92 for the 1 x 1.8 m trench.

8. CONCLUSIONS BASED ON TEST RESULTS AND USE OF THE SLURRY TRENCH METHOD IN SOFT CLAYS

The instrumentation used and the observations made on full-scale field tests carried out in two slurry trench excavations (i.e. 1 x 5 m and 1 x 1,8 m) to 20 m depth in soft marine clay at Vaterland-Oslo are summarized in this publication. The overall stability of the trench when filled with bentonite slurry, water, and oil, respectively, was studied with respect to surface settlement, lateral deformation close to the trench, change in width of the trench and the development of negative pore pressures.

The performance of the trenches was quite satisfactory when filled with bentonite slurry or water. The additional settlements and deformations during the phase in which the slurry in the trench was replaced with water are insignificant and acceptable because the time intervals that slurry trenches must remain stabilized is generally only a few days. As expected, significant increases in settlement and deformations were noted when the water in the 1 x 5 m trench was replaced with oil. Likewise, the magnitude of surface settlement, lateral deformation and pore pressure began to increase considerably for both trenches when the fluid in the trench was drawn down in stages to failure.

The Vaterland tests showed without doubt that a slurry trench can be excavated in soft clay to a depth of 20 m with plain water as the stabilizing fluid instead of expensive slurries. On this basis and experience gained during other projects described herein, a semi-empirical method of evaluating the stabi1ity of the trenches was developed. Application of the method to the Vaterland test trenches is illustrated in this publication. A detailed description of the procedure used to calculate the stability of slurry trenches is given in the reference (Aas, G. 1976).

The findings from the full-scale field tests at Vaterland contributed to a basic understanding and experience when considering construction of a dual East-West subway and railroad tunnel through the centre of Oslo, in Studenterlunden and at Jernbanetorget. These two large projects would require slurry trench excavations to a depth of 28 m in soft clay and construction of about 31,000 m^2 of in situ longitudinal concrete walls or diaphragm walls. The projects also involved application of a new principal for preventing bottom heave, which was based on constructing closely spaced cross-lot slurry trench walls in between the longitudinal tunnel walls.

NGI initiated therefore, in November 1970, a successful second full-scale field test to verify that it was possible to stabilize a 28 m deep slurry trench with water. When the tests were completed, the water in the trench was replaced with concrete using the tremie-pipe method to form an in-situ concrete wall element. Furthermore, in situ pullout tests were carried out on concrete piles cast in bentonite mud and water respectively to verify that any use of bentonite mud instead of water would adversely affect the shear resistance which could be mobilized between concrete and clay, which was a necessary condition to prevent bottom heave failure.

The construction of the jointly subway and railroad tunnel through the centre of Oslo was successfully carried out during the period 1973-1977. The continuous longitudinal concrete walls and cross-lot walls of the tunnel were constructed of elements 1 m thick and 4.5 m long. The projects showed that the slurry trench method is well suited for soft to medium soft clays and that it is, to a large extent, possible to carry out such work with only water as a stabilizing fluid during excavation of the trenches. This represented a great economical saving in comparison to use of the traditional bentonite slurry.

Since a new construction method and design principle were employed for the two tunnel projects in Studenterlunden and at Jernbanetorget respectively, a thorough program for performance monitoring was established. In Studenterlunden this included one thoroughly instrumented section that included: (a) Numerous reference points near ground surface for geodetic settlement measurements and magnetic settlement devices in boreholes including some inside the tunnel (b) Inclinometer casings for monitoring horizontal displacements, of which one was cast in the concrete tunnel wall (c) Earth pressure gauges in the clay around the tunnel. Earth pressure cells and pore pressure cells on each face of the concrete wall, (d) Strain gauges on the reinforcement steel for monitoring bending moments in the concrete wall. In addition, surface settlement points and several inclinometer casings for displacement monitoring were distributed along the entire length of the tunnel in Studenterlunden.

At Jernbanetorget the instrumentation included earth pressure cells, pore pressure cells, inclinometer casings, magnetic settlement devices in the tunnel, and numerous geodetic settlement points on neighboring buildings. The additional experience and measurement data gained from the performance control program during construction work of the tunnel projects resulted in a more confident approach to design and use of the slurry trench method in soft clays on new projects in Scandinavia.

One example is the construction during 2001-2004 of the 6-lane Lilla Bommen road tunnel in Gothenburg, Sweden, which required a deep cut-and-cover excavation up to 16 m deep in a very soft clay deposit. To insure bottom heave stability and limit displacements, the excavation for the tunnel was supported longitudinally by concrete diaphragm walls and cross walls every 4.5 m. The cross-walls were used to form a grid-like pattern extending below the base of the tunnel excavation to prevent bottom heave and serve as braces. The concrete diaphragm walls were excavated using the slurry trench method. Analysis showed that the slurry trenches could be stabilized using water, but the contractor decided from his practical experience to mix bentonite into the water. The adopted design principle with cross-walls is identical to what was first developed and very successfully applied for constructing the two new railway and subway tunnels through soft clays in Oslo. A comprehensive instrumented monitoring program of the construction work at Lilla Bommen tunnel verified that the structures behaved as intended.

Another example is the construction of a 350 m long section of a new 6-lane cut-and cover submerged road tunnel for E18 across Sørenga in Oslo during 2008-2009. The ground conditions consist of soft normally consolidated clay. The excavation, which was 40 m wide and up to 19 m deep, was constructed using concrete diaphragm walls produced by the slurry trench method and stabilized with water mixed with bentonite and barite. Cross-walls extending below the base for every 6 m were used to prevent bottom heave and act as pre-bracing of the longitudinal walls. No serious problems were encountered during construction of the diaphragm wall supported excavation and the measured horizontal deformations were quite in line with what the design analyses predicted.

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