University of Waterloo
Department of
Civil Engineering

CIV E 353 - Geotechnical Engineering I

SOIL MANUAL

WINTER 2006

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CIVE 353 - Geotechnical Engineering I
Instructions for Preparing Laboratory Reports

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This guide sets expectation levels for the laboratory reports. ***A professional and complete report is expected for each exercise.***

<u>Each lab MUST have</u>

1. Title page
2. Letter of submittal
3. Table of contents
4. Introduction
5. A reference to the procedure
6. Discussion (with answers to lab questions)
7. Clear, concise, and labelled figures in accordance with the lab requirements
8. Clear, complete and neat sample calculations
9. Raw data
10. Conclusions
11. References (if applicable)

All members of the lab group must sign the letter of submittal, and the letter should be consistent with current engineering standards. The Table of Content must include a list of tables, figures and appendices. Raw data and sample calculations should appear in separate appendices.

The introduction should contain an overview of the topic, the applicability of the laboratory to geotechnical problems, and a summary of the topics to be investigated in the report. No data or conclusions should be presented in the introduction. If information is extracted from the text then ***it must be referenced***.

Questions are to be answered clearly in the discussion section in paragraph form. *Yes* and *No* answers are not acceptable. Questions of a numerical nature should be summarized with a quick overview of how the calculations were performed and either a table of values or a reference to a spreadsheet included in the lab report. Always state your assumptions. In the case of drawing best-fit curves always discuss your reasoning for the shape of the curve, or why any outlier data points were excluded (if applicable). Be wary of polynomial trend lines they can appear to "fit" almost any data.

The statement "experimental error" does not address inconsistencies between lab data and theoretical data. If there were specific errors in the performance of the lab (and there always are) state them, but also use your engineering judgement (and available text book resources) to assess if there could be other reasons for data not matching. Avoid sweeping generalizations, and you cannot quote your TA.

Conclusions should summarize all of the results from the discussion. No new information can be presented here. The conclusion section should be readable and flow in paragraph form.

Extra information:

- Do not waste a lot of time imbedding figures in the text.
- Sometimes a compass and graph paper are better than Excel.
- Neat handwritten sample calculations are a lot quicker than using the Equation Editor.
- There are no bonus points for the title page and staples are as good as spiral bindings.

Plagiarism is a serious offence. It only takes a few minutes to reference a section properly.

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CIV E 353 - Geotechnical Engineering I
Consolidation

Purpose

Determine the magnitude and time rate of settlement for a compressible cohesive soil.

Required reading Das 2006 Sections 10.4 to 10.16 (pages 312 to 358).

Theory

Bringing soil samples into the laboratory and subjecting them to a series of loads and measuring the corresponding settlement can determine the compressibility of undisturbed or remoulded samples of fine soils such as clays and clayey silts. The test is known as a one dimensional consolidation test (also known as an oedometer test). The compressibility of granular soil is usually estimated empirically from in situ (field) tests, such as the Standard Penetration Test.

The consolidation test generally consists of placing an undisturbed sample in a consolidometer, also known as an oedometer, where it is subjected to a constant vertical load. This vertical load causes an increase in pore water pressure within the sample that is greater than the static pore water pressure (u_s) due to the water bath. The component of pore water pressure above the static pore water pressure is known as excess pore water pressure (u_e). Thus, excess pore water pressure is the pressure due to the applied load.

Excess pore water pressure causes water to flow out of the sample towards the drainage boundaries (upper and lower porous stones). If the sample is considered to be 100 percent saturated and the soil grains are incompressible, settlement will occur only when water flows out of the sample voids and the soil particles rearrange to create a lower void ratio (tighter packing). The rate that a sample consolidates (decrease in volume due to the dissipation of excess pore water pressure) will depend on several factors: permeability, thickness, compressibility, pore fluid, initial void ratio, and degree of saturation.

Monitoring the compression of the sample due to an applied load increment is achieved by plotting log time vs vertical settlement curve, as shown in Fig. 1. This figure shows that sample compression can be divided into three parts

- Initial compression attributed to load seating, elastic expansion of the oedometer ring, compression of the porous stones, or the presence of air
- Primary compression due to consolidation resulting from the dissipation of excess pore pressures
- Secondary compression that is thought to be caused by a gradual readjustment of the soil particles

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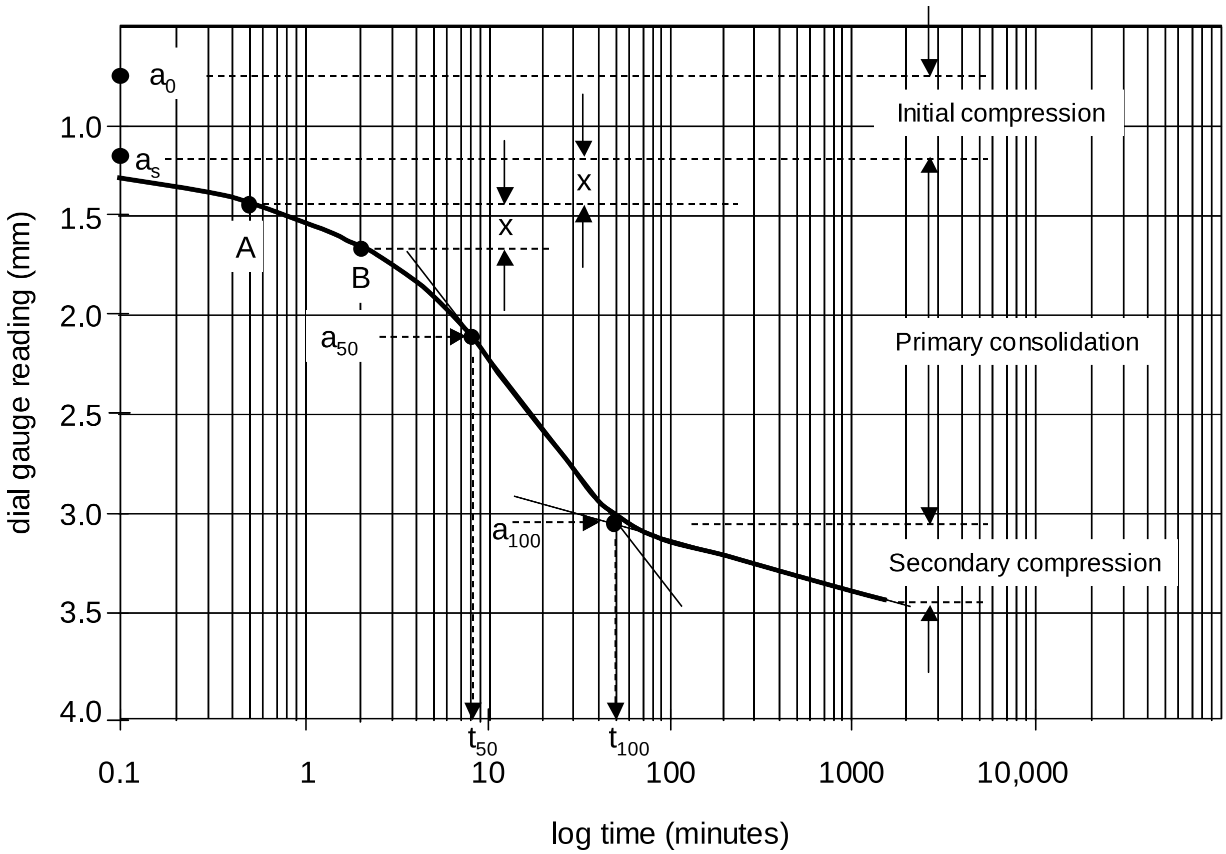

Fig. 1 Log time method

After a period of about 24 hours another increment of vertical load is applied. Data from several load increments is used to plot void ratio (e) vs. effective stress (σ′) or e - log σ′ curves (Das Fig. 10.13). The change in void ratio is computed using only the primary consolidation portion of the test data and phase relationships shown in Section 10.6 pages 317 to 318 and Das Fig. 10.12.

The e-log σ′ plot is used to determine the Preconsolidation pressure (σ_c'), and the clay compressibility defined by Compression Index (C_c) and Swell (Recompression) Index (C_s), shown in Das Figures. 10.18 and 10.19. Example 10.3 shows how to calculate the Compression and Swell Index.

Unloading the sample can cause the soil to rebound elastically and to swell. Data obtained from unloading increments are treated in the same manner as loading increments.

Procedure

1 Determine the size and mass of the oedometer ring.

2. Trim the soil to fit and completely fill the ring. Determine the mass of the ring and the soil sample.

3. Assemble the apparatus using filter papers between the soil and the porous stones, balance the lever arm, and set the dial indicator to zero. The usual procedure is to start with a small stress and to **double** the stress for each load increment. Normal loadings will be 6.25, 12.5, 25, 50, 100, 200, and 400 kPa; then unload to 100, 25, and finally 6.25 kPa.

4. Add the first prescribed load to the hanger at the back of the machine. Record dial readings at the times shown on the data sheet.

   Note: Compression is likely to be minimal and rapid under this first load. When insignificant movement is occurring, fill the cell with distilled water that is at room temperature. If the sample starts to swell, stop the test and add another increment of load.

5. Continue taking readings at the suggested times given on the data sheet for the first hour. Allow the load to remain on the sample for 24 hours before applying the next load increment.

6. Following the load/unload sequence, allow the sample to swell for 24 hours at the final load of 6.25 kPa. Remove it from the cell, blot surplus water, and determine the mass of the ring plus sample.

7. Place both the ring and sample in an oven, dry, and obtain the mass of the dry soil plus ring.

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Results

1. Calculate the water content at the end of the test from:

$$w = \frac{M - M_s}{M_s} = \frac{Mass\ of\ soil - Mass\ of\ dry\ soil}{Mass\ of\ dry\ soil}$$

Express as a percent.

2. Calculate the following:

a) Height of solids (H_s) using:

$$H_s = \frac{M_s}{A G_s \rho_W}$$

where A is the sample cross sectional area
Warning: Use compatible units.

b) Void ratio at the start of the test (e_o) using:

$$e_o = \frac{H_o - H_s}{H_s} = \frac{H_o}{H_s} - 1$$

c) Height of voids (H_v) at the start of the test using:

$$H_v = H_o - H_s$$

3. Explain why changes in void ratio (during the consolidation test) can be expressed in terms of changes in sample height.

4. From the plots of compression vs. log time obtain the changes in height (ΔH) due to primary consolidation, and calculate the void ratio for each load increment using:

$$e_i = \frac{H_v - \sum_{i=0}^{i=n} \Delta H}{H_s}$$

5. Plot e-log σ' at the end of primary consolidation (Fig 1) and determine the Preconsolidation pressure (σ_c') using Casagrande method. Calculate the Swell Index (C_s) by taking the slope of the expansion line before the Preconsolidation pressure and the Compression Index (C_c) by taking the slope of the line after the Preconsolidation pressure.

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6. Using the empirical equations provided in Das Sections 10.11 and 10.12 and the clays Atterberg limits, insitu water content, and initial void ratio (e_0), estimate the clay Compression Index and Swell Index. Please note that symbol C'_c is used to represent the Compression Index of clay in the **remoulded** state. Compare these values with the Compression Index determined from the consolidation test and comment on applicability of the best-fit equations.

7. Calculate the Coefficient of Consolidation (c_v) for 50 percent consolidation for each increasing load increment only using

$$c_v = \frac{Tv\, d^2}{t_{50}}$$

Where: *d* is the length of longest drainage path computed by taking one - half the average sample thickness (double drainage) for each load increment and time factor (*Tv*) for 50 percent consolidation is 0.196.

As shown on the calculation sheet, the net height of the sample is computed by subtracting the initial compression from the total change in height. Plot c_v vs log σ' for the loading sequence only. Why does the Coefficient of Consolidation change with increases in stress?

Selected References

1. ASTM D698 - 78 and D1557 - 78.

2. Bowles, J.E., 1979, Physical and Geotechnical Properties of Soils, McGraw-Hill.

3. Craig, R.F., 1978, Soil Mechanics, Chapman and Hall.

4. Holtz, R.D., and Kovacs, W.D., 1981, An Introduction to Geotechnical Engineering, Prentice-Hall, Inc. New Jersey.

5. Koerner, R.M., 1984, Construction and Geotechnical Methods in Foundation Engineering, McGraw-Hill.

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Consolidation

Material Data Sheet

Soil Description________________________________ Date ____________

Specific Gravity of Solids, G_s ________________

Natural Water Content (from trimmings) __________

Atterberg Limits   LL _________   PL __________

Oedometer Ring No. ______   Height, mm ______ Diameter, mm _______

Area, mm^2 _______

	Before Loading	After Loading
Mass of Soil + Ring, g		
Mass of Dry Soil + Ring, g		
Mass of Ring, g		
Mass of Soil, M, g		
Mass of Dry Soil, M_s, g		
Mass of Water, M_w, g		
Water Content, w, %		
Height of Solids, H_s, mm $H_s = M_s/A\ G_s\ \rho_w$		
Height of Voids, H_v, mm $H_v = H - H_s$ Where H = sample height		
Degree of Saturation, S_r, % $S_r = (M - M_s)/H_v\ A\ \rho_w$		
Void Ratio, e $e = H_v/H_s$		

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Time-Compression Data Sheet

Project ______________________ Group Number ______________________

Load Applied ___________ kPa
Date Applied ___________
Applied by ___________

Load Applied ____________ kPa
Date Applied ____________
Applied by ____________

Time	Δ Time	Dial Rdg.	x.002 = mm	
	0			
	0.1			
	0.25			
	0.5			
	1			
	2			
	4			
	8			
	15			
	30			
	60			

Time	Δ Time	Dial Rdg.	x.002 = mm	
	0			
	0.1			
	0.25			
	0.5			
	1			
	2			
	4			
	8			
	15			
	30			
	60			

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Void Ratio (Primary Consolidation)

Sample ______________________ Date __________

Height of Solids, H_s, mm ______________

Initial Height of Voids, H_{vo}, mm ________

Applied Stress kPa (1)	Primary Consolidation mm (2)	Height of Voids $H_{vo} - \Sigma(2)$ mm (3)	Void Ratio (4)

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Coefficient of Consolidation

Soil Sample ______________________________ Date ______________________

Initial Height of Sample, H_0, mm ___________________________________

Applied Stress kPa (1)	Total Change in Height mm (2)	Initial Compress mm (3)	Net Change in Height (2) - (3) mm (4)	Sample Height $H_0 - \Sigma(4)$ mm (5)	Drain Path *d* (6)	t_{50} minutes (7)	c_v m^2/yr (8)

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DETERMINATION OF CONSOLIDATION PARAMETERS USING THE LOG TIME METHOD

(refer to Fig. 1, page 2)

1. Mark the initial dial indicator reading at time 0 for the current load increment as a_o on the vertical axis.

2. Select two points on the curve (A, B) for which the values of time (*t*) are in a ratio of 4:1. (e.g. 0.25 and 1 min. or 0.5 and 2 min.)

3. Measure the vertical distance between A and B (X).

4. Measure out the vertical distance found in Step 3 above A and mark as a_s horizontally across to the vertical axis.

5. To check if completed correctly, steps 2 to 4 could be repeated using different pairs of points.

6. Label the distance between a_o and a_s as the region of initial compression. This difference between these two points is due mainly to the compression of small quantities of air in the soil and the degree of saturation being marginally below 100%.

7. Draw a best-fit linear line in the steep portion of the curve as shown in the Figure 1.

8. Look for where the experimental curve beings to curve at a slower rate on an almost horizontal plane.

9. Make another best-fit linear line running along this part of the curve to where it intersects with the line made in Step 7. Label this intersection as point a_{100}.

10. Label the region from a_s to a_{100} as the region of primary consolidation.

11. Beyond the point of intersection at a_{100}, the compression of the soil continues at a very slow rate for an indefinite period of time and can be labelled as the region of secondary compression.

12. Measure the vertical distance between a_s and a_{100} and divide this distance in half. Label this point as a_{50} where it intersects the curve.

13. Extend vertical lines down from a_{50} and a_{100}, labelling them t_{50} and t_{100}, respectively.

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DETERMINATION OF CONSOLIDATION PARAMETERS USING THE LOG TIME METHOD

(refer to Fig. 1, page 2)

1. Mark the initial dial indicator reading at time 0 for the current load increment as a_o on the vertical axis.

2. Select two points on the curve (A, B) for which the values of time (*t*) are in a ratio of 4:1. (e.g. 0.25 and 1 min. or 0.5 and 2 min.)

3. Measure the vertical distance between A and B (X).

4. Measure out the vertical distance found in Step 3 above A and mark as a_s horizontally across to the vertical axis.

5. To check if completed correctly, steps 2 to 4 could be repeated using different pairs of points.

6. Label the distance between a_o and a_s as the region of initial compression. This difference between these two points is due mainly to the compression of small quantities of air in the soil and the degree of saturation being marginally below 100%.

7. Draw a best-fit linear line in the steep portion of the curve as shown in the Figure 1.

8. Look for where the experimental curve beings to curve at a slower rate on an almost horizontal plane.

9. Make another best-fit linear line running along this part of the curve to where it intersects with the line made in Step 7. Label this intersection as point a_{100}.

10. Label the region from a_s to a_{100} as the region of primary consolidation.

11. Beyond the point of intersection at a_{100}, the compression of the soil continues at a very slow rate for an indefinite period of time and can be labelled as the region of secondary compression.

12. Measure the vertical distance between a_s and a_{100} and divide this distance in half. Label this point as a_{50} where it intersects the curve.

13. Extend vertical lines down from a_{50} and a_{100}, labelling them t_{50} and t_{100}, respectively.

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Hydraulic Conductivity Tests (Permeability Tests)

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Purpose

Determine the hydraulic conductivity (coefficient of permeability) of sand using the constant-head and falling-head permeameters.

Required reading Das 2006 Sections 6.1 to 6.6 (pages 156 to 177).

Theory

For the constant-head test the hydraulic conductivity (k) can be determined using

$$k = \frac{Q\,L}{h\,A\,t}$$

where Q is the quantity of water flowing through the sample of length (L) and A is the cross-sectional area for the period of time t and subjected to a hydraulic head h.

For the falling-head test the hydraulic conductivity can be determined using

$$k = \frac{aL}{At}\ln\frac{h_1}{h_2}$$

where a is the internal cross-sectional area of the standpipe, h_1 and h_2 are the hydraulic heads at times t_1 and t_2, respectively.

A schematic sketch of the apparatus showing various dimensions is posted in the laboratory.

Hydraulic conductivity is influenced by many factors: grain size and shape, void ratio, temperature, soil fabric, degree of water saturation, viscosity, and type of permeant. In this laboratory exercise water is the only permeant. Thus, only the influence of temperature and void ratio are considered.

Since viscosity of the water decreases as the temperature increases hydraulic conductivity is usually standardized at 20^{o}C using

$$k_{20} = k_T \frac{u_T}{u_{20}}$$

where u_T and u_{20} are the viscosities of water at the test temperature (T) and at 20^{o}C, respectively.

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Hydraulic Conductivity Tests (Permeability Tests)

Attempts to correlate k with void ratio (e) are made using expressions such as

$$k_2 = k_1 \left(\frac{e_2}{e_1} \right)^2$$

and

$$k_2 = k_1 \frac{e_2^{\ 3}/(1+e_2)}{e_1^{\ 3}/(1+e_1)}$$

Apparatus

Constant-head and falling-head permeameters, containers, scale, stop watch, beakers, and measuring tape.

Procedure

a. Determine the initial mass of the beaker plus dry sand sample.

b. Pour the sand into the permeameter using the funnel fitted with a hose at the spout. Maintain a constant height between the end of the hose and the top of the sand in the test apparatus.

c. After placing sufficient sand in the permeameter, determine the mass of the beaker and remaining sand. Compute the mass of sand in the permeameter.

d. Measure the height of the sand.

e. To saturate the sand, apply a vacuum to the top of the permeameter and allow deaired water to rise slowly. Note any changes in the height of the sample.

Constant-head test

1. Connect the permeameter to the constant-head tank.

2. Start flow by opening the valve. After waiting a moment for steady state flow, measure Q using a graduated flask for an arbitrary time t.

3. Measure the temperature T of the effluent.

4. Complete at least three flow measurements and compare the results.

5. Densify the sand by tapping the side of the permeameter with a rubber hammer.

6. Measure the new height and repeat steps 1 to 4.

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Hydraulic Conductivity Tests (Permeability Tests)

Falling-head test

1. Connect the permeameter to the standpipe as indicated by the instructor.
2. Open the valve to start the flow.
3. Record the time required for the water in the standpipe to drop from height h_1 to h_2.
4. Measure the temperature of the effluent.
5. Complete at least three flow measurements and compare the results.
7. Densify the sand by tapping the side of the permeameter with a rubber hammer.
8. Measure the new height and repeat steps 1 to 5.

Results

1. Calculate the void ratio of the sample before, and after, tapping. Are these values reasonable?

2. Discuss the hydraulic conductivity results with respect to the influence of temperature and void ratio.

3. Calculate the hydraulic conductivity using Hazen's expression:

$$k = C\, D_{10}^{2}\, (cm/s)$$

where C is Hazen's coefficient that varies from 0.8 and 1.2 and D_{10} is expressed in mm.

4. Discuss the validity and limitations of Hazen's formula.

5. Discuss limitations of the constant head and falling head tests.

6. How do your test results compare with published data?

Selected Bibliography

1. ASTM, D2434 - 74

2. Bowles, J.E. 1979. Physical and Geotechnical Properties of Soils, McGraw-Hill.

3. Craig, R.F. 1978. Soil Mechanics, Chapman and Hall.

4. Head, K.H. 1981. Manual of Soil Laboratory Testing Vol. 2, Pentech.

5. Holtz, R.D., and Kovacs, W.D. 1981. An Introduction to Geotechnial Engineering, Prentice-Hall.

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Hydraulic Conductivity Tests (Permeability Tests)

Constant-head Permeability Test Data Sheet

Sample________________________________ Date____________________________

Specific Gravity of Soil, G_s ____

Initial Mass of Dry Soil + Container, g ___________________________

Final Mass of Dry Soil + Container, g ______________________________

Mass of Dry Soil, M_s, g _________________________

Volume of Solids, $V_s = M_s/G_s\rho_w$ ___________________

Permeameter Internal Diameter, mm ______________ Area, mm^2 ________

For the Constant-head Permeameter $k = Q/i\ A\ t = Q\ l/\Delta h\ A\ t$

LOOSE SAMPLE: Length, L, mm ____________ Volume, V, mm^3 ____________

Void Ratio, $e = (V - V_s)/V_s$ ___________________________

Elapsed time t, (sec)	Flow Q, (ml)	Head h_1 h_2 h_3	Gradient $i = \Delta h/l$	Water Temp (°C)	k_t cm/s	k_{20} cm/s

DENSE SAMPLE: Length, L, mm____________ Volume, V, mm^3 ____________

Void Ratio, $e = (V - V_s)/V_s$ __________________________

Elapsed time t, (sec)	Flow Q, (ml)	Head h_1 h_2 h_3	Gradient $i = \Delta h/l$	Water Temp (°C)	k_t cm/s	k_{20} cm/s

Note any change in sample height during tests: _______________________________

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Hydraulic Conductivity Tests (Permeability Tests)

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Falling-head Permeability Test Data Sheet

Sample________________________________ Date___________________________

Specific Gravity of Soil, G_s ____

Initial Mass of Dry Soil + Container, g __________________________

Final Mass of Dry Soil + Container, g _____________________________

Mass of Dry Soil, M_s, g _________________________

Volume of Solids, $V_s = M_s/G_s\rho_w$ _________________

Permeameter Internal Diameter, mm ______________ Area, mm^2 _________

For the Falling-head Permeameter $k = \frac{aL}{At} \ln \frac{h_1}{h_2}$

LOOSE SAMPLE: Length, L, mm ____________ Volume, V, mm^3 ___________

Void Ratio, $e = (V - V_s)/V_s$ _________________________

Elapsed Time t(sec)	Burette Area (mm^2)	h_1 (mm)	h_2 (mm)	Water Temp. °C	k_t cm/s	k_{20} cm/s

DENSE SAMPLE: Length, L, mm____________ Volume, V, mm^3 ____________

Void Ratio, $e = (V - V_s)/V_s$ _________________________

Elapsed Time t(sec)	Burette Area (mm^2)	h_1 (mm)	h_2 (mm)	Water Temp. °C	k_t cm/s	k_{20} cm/s

Note any change in sample height during tests ____________________________________

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Moisture – Unit Weight Relationships (Compaction Test)

Purpose

The purpose of the Standard Compaction Test is to determine a relationship between moisture (water) content and dry unit weight for a soil that is being considered for use in an engineered fill, e.g. an earth dam, road embankment, site development, back fill, etc. The relationship may also be used to determine the suitability of the soil and also to determine the optimum moisture content and the corresponding maximum dry unit weight.

Required reading Das 2006 Sections 5.1 to 5.6 (pages 106 to 125).

Apparatus

(1) Hammer:	mass	= 2.5 kg (5.5 lb)
	diameter	= 50 mm (2 in.)
(2) Mould:	diameter	= 102 mm (4 in.)
	Volume (V)	= $9.44 \times 10^{-4} m^3$ (1/30 ft.^{3})

Procedure

1. Break up the dry soil sample until approximately 3,000 g passes a 4.75 mm (No. 4) sieve.

2. Determine and record the dry mass of the soil sample (M).

3. Compute the mass of water (M_w) to be added to obtain the following cumulative moisture contents (w):

   $$w = 7, 9, 11, 13, \text{ and } 15 \text{ percent}$$

4. Determine and record the mass of the mould (M_m) (without the collar).

5. Add seven (7) percent water to the dry soil and mix thoroughly.

6. Attach the collar to the mould and form a specimen by compacting the prepared soil in three equal layers to give a total compacted depth of about twelve (12) cm. Compact each layer by imparting 25 evenly distributed blows from the hammer dropping from a height of 305 mm (12 in.) above the soil. During compaction, the mould should rest on a uniform rigid base weighing not less than 90 kg (200 lbs.).

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Moisture – Unit Weight Relationships (Compaction Test)

7. Remove the collar and trim the excess soil extending above the mould with a straightedge. Determine and record the mass of the moist soil plus mould ($M + M_m$).

8. Subtract the weight of the mould and compute the (bulk) unit weight.

$$\gamma = \frac{W}{V}\left(\frac{kN}{m^3}\right)$$

9. Remove the soil from the mould, cut the specimen vertically through the centre and take a sample for a moisture content determination from one of the cut faces. Weigh this sample **IMMEDIATELY** and place in the oven for at least 12 hours at 110^0 C.

10. Return the specimen to the remainder of the sample and break up the material until it will pass a 4.75 mm (No. 4) sieve. (It is not necessary to actually pass the material through the sieve; use your judgement).

11. Add water to increase the moisture content to the next cumulative moisture content (Step 3) and mix the sample thoroughly.

    NOTE: The amount of dry soil will decrease each time a sample is taken for a moisture content determination. However, for the purpose of Step 3 calculations assume that the dry mass (M_s in Step 2) is constant.

12. Repeat the procedure (Step 6) until there is either a decrease or no change in the unit weight.

13. Calculate (the next day) the moisture content and the dry unit weight for each compacted soil trial using

$$\gamma_d = \frac{\gamma}{1 + w}$$

    where γ_d is the dry unit weight.

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Moisture – Unit Weight Relationships (Compaction Test)

Results

To complete the lab

- Finish filling out the data sheet
- Plot dry unit weight vs moisture content curve along with the 80 and 100 percent saturation curves using

$$\gamma_d = \left[\frac{Gs}{1 + \frac{w\,Gs}{S}} \right] \gamma_w$$

where S is degree of water saturation in decimal format and Gs is specific gravity of solids.

- Determine the optimum moisture content and dry unit weight
- Plot the total unit weight curve vs moisture content
- Answer the following questions.

1. Comment on the differences between the bulk unit weight and dry unit weight versus moisture content plots and describe why they are different.

2. Contract specifications require a relative compaction of at least 95 percent standard Proctor. To achieve these specifications, what field moisture content and dry unit weight does the contractor require?

3. Why is soil compaction an important part of earthwork construction in engineering earthworks?

4. Why do most geotechnical engineers specify greater compactive effort than the standard Proctor test?

5. Assume that a modified Proctor was performed instead of the standard Proctor. Comment on expected values of optimum moisture content and maximum dry unit weight.

6. Why does maximum dry unit weight decrease at water contents greater than optimum?

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Moisture – Unit Weight Relationships (Compaction Test)

Selected Bibliography

1. Akroyd, T.N.W. 1957. Laboratory Testing in Soil Engineering, London, Soil Mech. Ltd.

2. ASTM D698 - 78 and D1557 - 78.

3. Bowles, J.E. 1979. Physical and Geotechnical Properties of Soils, McGraw-Hill.

4. Craig, R.F. 1996. Soil Mechanics, Chapman and Hall.

5. Holtz, R.D., and Kovacs, W.D. 1981. An Introduction to Geotechnical Engineering, Prentice-Hall, Inc. New Jersey.

6. Lambe, T.W. 1951. Soil Testing for Engineers, N.Y., Wiley.

7. Lambe, T.W., and Whitman, R.V. 1969. Soil Mechanics, N.Y. Wiley.

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Moisture – Unit Weight Relationships (Compaction Test)

Sample ____ Date_______________

Compaction: Blows/Layer_______ No. of Layers ______ Mass of Tamper _____

Mould: Height ________________ Diameter __________ Volume ___________

UNIT WEIGHT DETERMINATION

Trial Number	1	2	3	4	5
Mass of Soil + Mould, kg					
Mass of Mould, M_m, kg					
Mass of Soil, M, kg					
Unit weight, γ, kN/m^3					
Dry Unit weight, γ_d, kN/m^3					

MOISTURE CONTENT DETERMINATION

Moisture Tin Number					
Mass of Soil + Tin, g					
Mass of Dry Soil + Tin, g					
Mass of Tin, g					
Mass of Water, M_w, g					
Mass of Dry Soil, M_s, g					
Moisture content, w, %					

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CIV E 353 - Geotechnical Engineering I
Direct Shear Test

Purpose

Determine the shear strength of dry cohesionless sand with the direct shear box apparatus.

Required reading Das 2006 Sections 11.1 to 11.7 (pages 374 to 389).

Theory

The general shear strength equation (Mohr-Coulomb failure criterion) in terms of effective stresses is

$$\tau_f = c' + \sigma'_{f'} \tan \phi'$$

where τ is shear strength, c' is the effective apparent cohesion, ϕ' is the effective angle of friction, and σ' is the effective stress $(\sigma - u)$ and subscript f represents shear stress at failure.

For cohesionless soil (sand, gravel and some silt) the effective cohesion (c') is zero and the shear equation reduces to

$$\tau_f = \sigma'_f \tan \phi'$$

The direct shear test set up consists of placing a soil sample in a split box having a cross-sectional area (A) and subjecting the test sample to a vertical normal load (N). Testing proceeds by displacing the lower half of the split box and measuring the horizontal shear forced (T) transmit through the soil to the upper portion of the box. Testing continues by displacing the lower box horizontally until the shear force increases to a maximum value and then decreases or remains essentially constant.

During testing it is often assumed that the sample cross-sectional shear area (A) remains constant. Therefore, the normal stress σ'_f on the failure plane may be calculated using

$$\sigma_{f'} = \frac{N}{A} = \frac{vertical\ normal\ \ force}{cross\text{-}sectional\ area}$$

The shear stress (τ) on the shear plane may also be calculate using

$$\tau = \frac{T}{A} = \frac{shear\ \ force}{area}$$

The maximum shear stress on the shear plane may be determined using

$$\tau_{\max} = \frac{T_{\max}}{A}$$

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Direct Shear Test

Testing consists of determining the maximum shear for at least three test samples with three different applied normal stresses that are selected to be representative of anticipated field stresses. Since a decrease in the sample void ratio will increase the soil internal angle of friction, test specimens are initially placed to the same density (unit weight). Shear strength parameters c' and ϕ' are determined by determining a best-fit line (y-intercept and slope) of the σ'_f (abscissa) vs τ_{max} (ordinate) plot.

Apparatus

Three direct shear boxes can be mounted on a single frame and they can be loaded independently or together. Each box is provided with a hanger system to permit the application of normal stress. Weights may be applied directly through the vertical hanger for small stresses or through a lever system for large stresses. The horizontal load is applied through a motor-driven gear system and the magnitude of the load is determined using a load transducer. Vertical and horizontal deformations are measured using displacement transducers with an electronic measurement system to collect and store the data.

Procedure

1. Fasten together the two halves of the shear box by tightening the vertical lock screws and insert the bottom plate, porous stone and serrated grating plate in the shear box.

2. Determine the area, *A*, of the sample box and the distance from the grating (bottom of serrations) to the top of the box.

3. Weigh out the amount of dry sand required for the sample as given by the instructor.

4. Pore the sand loosely in one smooth layer approximately 20 mm thick. Tamp the sample as directed by the instructor.

5. Measure the distance from the top of the sample to the top of the box and calculate the height of sample.

6. Put the upper grating plate, stone and loading block on top of the sample. Be careful to avoid vibration if a very loose sample is being tested.

7. Put the ball and hanger on the loading block and attach the transducers to measure the vertical and horizontal displacement of the sample.

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Direct Shear Test

8. Repeat steps 1 to 7 for the other two samples and apply the appropriate normal load for each sample as given by the instructor. **Note the amount of compression of the sample when the normal load is applied; subtract this value from the original height of sample.**

9. Separate the upper and lower parts of the shear box by removing the vertical lock screws and raising the upper frame by turning the thumbscrews one revolution. This will raise the upper frame approximately 1.5 mm to provide sufficient clearance. Back off the thumbscrews. The entire vertical load is now being transferred from the top to the bottom of the shear box through the sample.

10. Bring the loading piston into contact with the shear box. The test is now ready to begin. The test should be run at a shearing rate of approximately 1.3 mm/min. (0.05 in./min)

11. Continue the test until a maximum reading on the load transducer has been passed and the readings have begun to decrease to a constant value. **Do not continue the test past a horizontal displacement of 10 mm**.

<u>Results</u>

1. Calculate the shear stress on the sample. Assume that the area of the sample did not change during the test and use the original area for all calculations.

2. Plot shear stress (τ) vs. horizontal shear strain (ε_h) and sample volume change (ε_v) due to vertical settlement vs ε_h for each sample tested by your group. Plot all three normal stress curves on the same graph.

$$\varepsilon_h = \frac{\delta_h - \delta_{h0}}{L_0} * 100$$

where δ_{ho} is the horizontal displacement reading at the start of the test, δ_h is the horizontal displacement reading at a given time, and L_0 is the initial length of sample in the direction of shear at the start of the test.

$$\varepsilon_v = \frac{\delta_v - \delta_{v0}}{\mathrm{H}_0} * 100$$

where δ_{vo} is the vertical displacement reading at the start of the test, δ_v is the vertical displacement reading at a given time, and H_o is the height of sample at the start of the test.

3. Calculate the dry unit weights and void ratios of the samples tested by your group.

4. Plot on one graph peak shear strength (maximum shear stress) against normal stress for each test. Use the same scale for each axis.

5. Determine the effective angle of internal friction, ϕ' by drawing a best fit straight line through the data points. Hint: Assume an appropriate c' for dry sand.

6. Did the samples tested by your group increase in volume during the test? Why?

7. a) How does increasing the normal stress influence peak shear strength and horizontal strain at peak shear stress?

   b) How would the soil friction angle change if failure were defined as one percent horizontal strain (ε_h)?

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Direct Shear Test

Direct Shear Test Data Sheet

Sample Identification____________________ Date _______________

Sample: Dry Mass, M_s, ___________ g; Height, H_o,____________mm

Sample: Length, L_o, ____________ mm, Width, W_o, ____________ mm

Area, A_o, ____________ mm^2

Specific Gravity of Solids, G_s, __________

Applied Normal Stress, σ'_n, ______________ kPa

Sample Identification____________________ Date _______________

Sample: Dry Mass, M_s, ___________ g; Height, H_o,____________mm

Sample: Length, L_o, ____________ mm, Width, W_o, ____________ mm

Area, A_o, ____________ mm^2

Specific Gravity of Solids, G_s, __________

Applied Normal Stress, σ'_n, ______________ kPa

Sample Identification____________________ Date _______________

Sample: Dry Mass, M_s, ___________ g; Height, H_o,____________mm

Sample: Length, L_o, ____________ mm, Width, W_o, ____________ mm

Area, A_o, ____________ mm^2

Specific Gravity of Solids, G_s, __________

Applied Normal Stress, σ'_n, ______________ kPa

CIV E 353 - Geotechnical Engineering I
Shear Strength of Soils (Triaxial Test)

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(To be read in conjunction with the video)

Required reading Das 2006 Sections 11.1 to 11.3, and 11.8 to 11.12 (pages 374 to 378 and 389 to 409).

Introduction - Purpose

This presentation describes the triaxial test and related equipment. The following geotechnical problems illustrate the application of the triaxial test.

Earth dams are used to retain reservoirs of water. The water may be used for the generation of electric power, irrigation, recreation, flood control or municipal water supply. The internal core of an earth dam usually consists of a fine-grained soil such as clay. The core minimises the quantity of water that seeps through the dam. The outer shells of the dam provide stability and usually consist of sand, gravel and rock fill. The upstream and downstream slopes of the dam are determined by stability analyses, which make use of the shear strength of each dam material. The strength characteristics are usually determined with the triaxial test.

There is often the impression that quicksand is a special type of sand. This is untrue. Rather, quicksand is a condition that is caused by the upward flow of water. As water flows upwards, it decreases the forces that act between soil grains. If the upward gradient is large, it can cause a quick condition, or in other words, the effective stresses within the sand mass approach zero and the sand is not capable of supporting any load. This phenomenon is illustrated using the model where a toy truck is seen to sink into the sand when a quick condition occurs. Earthquakes impose cyclic loading. In a saturated loose sand mass, such loading causes a gradual build-up of pore pressure. If this cumulative build-up of pore pressures is large, the effective stresses may approach zero, and a phenomenon known as liquefaction can occur. Liquefaction is similar to a quick condition in that the soil mass provides very little resistance to load. The photo illustrates the destruction caused by the 1964 Alaska earthquake. Contractive behaviour and the consequent pore pressure build up can be studied in the triaxial test.

The photo showing the leaning Tower of Pisa is a successful failure because it has attracted tourists from all over the world. The lean in the tower is caused by large uneven settlements. In the extreme case, failure occurs by bearing capacity. Stress/strain behaviour can be studied in the triaxial test.

These examples show us the importance of understanding the strength and deformation characteristics of soils. The loadings associated with these examples, and the corresponding stress history can be simulated in the triaxial test.

<u>Equipment Description</u>

The major components of the triaxial equipment are the Triaxial Cell, the Universal Testing Machine, and the Pressure Control Panel.

The first major component is the <u>Triaxial Cell</u> that is usually transparent so that the sample can be viewed during the test. The cell has three port connections to the exterior and each port is controlled with a valve.

One port is used to fill the cell with water and to apply an all-round confining pressure to the sample, which is jacketed with a thin rubber membrane. The other two ports are connected to the top and the bottom of the sample; these are used for drainage, backpressure application or pore pressure measurement.

NOTE: Water may go into or out of the sample depending whether the sample tends to dilate or contract. There are situations in nature where no drainage will take place because the rate of loading is much higher than the rate of pore pressure dissipation. To model this case, both top and bottom valves should be kept closed. In this case, pore pressure should be measured to allow calculation of effective stresses. When a sample is allowed to drain, the sample will experience a decrease in volume proportional to the loss of pore water, i.e. an increase in volume gauge readings equates to a decrease in sample volume.

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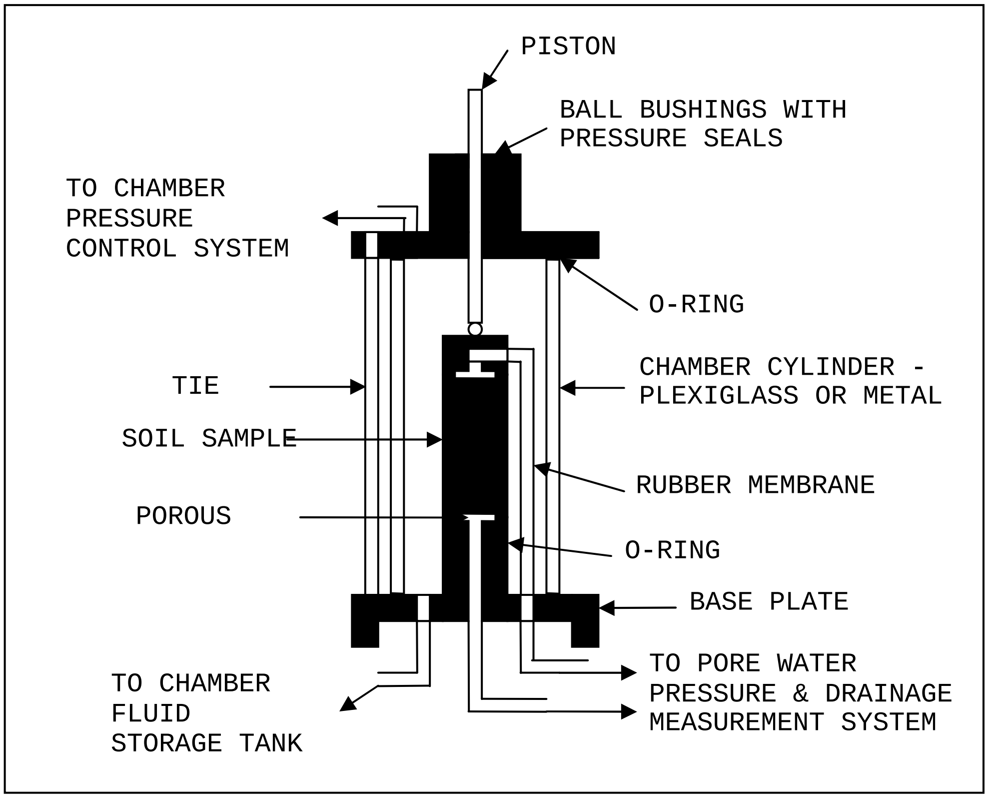

Figure 1: Schematic of a Triaxial Apparatus.

Ports that connect the sample at the top and bottom are useful not only during the performance of the test but also during sample preparation. For example, applying a vacuum to hold sand-samples, or applying a gradient to saturate the soil. This technique allows us to apply pressure to dissolve the air trapped in the void space of the sample, and achieve a full degree of saturation.

The second major component is the Universal Testing Machine that consists of a loading frame and a loading platen. The upper crosshead on the loading frame can be adjusted to accommodate triaxial cells of various sizes. The triaxial cell is placed on a platen which can be moved up or down by either of two hand wheels or by an electric motor. The rate of movement of the platen can be changed and controlled by using different combinations of gears.

As the loading platen is raised, the piston at the top of the cell is forced downwards and this action applies an axial load to the top of the sample. The magnitude of the load is determined by taking dial readings on a calibrated proving ring and the axial deformation of the sample are measured with a second dial gauge.

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The third major component is the Pressure Control Panel. The panel is connected to a water supply system and to a pressure system. Each of the three vertical sections on the right hand side of the panel is connected to a port on the triaxial cell. A three-way valve in the upper left corner is used to select a particular port. The pressure to each port is regulated through a regulator that is located at the top of the panel and the pressure is monitored or controlled with a digital display.

Another three-way valve (located immediately below the regulator) allows three options: to open the port to atmospheric pressure, to apply a vacuum or to apply a pressure. A device consisting of an annular pipette is located below the three-way valve. The pipette is used to measure volume changes that occur in the sample during drained tests.

The two-way valve immediately below is used to select large or small volume changes. Along the same serial line of valves that control each of the channels we find another three-way valve which permits incorporating water into the line, or venting it to the atmosphere without going through the volume change measurement system. The last valve permits connecting the line coming from the cell to the panel. The quick-disconnect fitting at the bottom of the panel attaches the tubing to the triaxial cell ports.

The panel is equipped with another three-way valve in each section so that the volume-change measuring device can be bypassed. Connections between the panel and the triaxial cell are made through quick-disconnect fittings, which are located at the base of the panel. Table 1 shows the four main types of triaxial tests that are commonly performed.

Table 1: Variations on the Triaxial Test

	UNCONSOLIDATED UNDRAINED TEST UU	CONSOLIDATED TESTS		
		CONSOLIDATION PHASE	CU	CD
σ_3	HELD CONSTANT	HELD CONSTANT	HELD CONSTANT	HELD CONSTANT
σ_1	GRADUALLY INCREASED FROM σ_3	EQUAL TO σ_3 *	GRADUALLY INCREASED FROM σ_3	VERY GRADUALLY INCREASED FROM σ_3
u	DRAINAGE LINES CLOSED	DRAINAGE LINES OPEN **	NO WATER PERMITTED TO ESCAPE. PORE PRESSURE MEASURED FOR EFFECTIVE STRESS TESTS	DRAINAGE LINES OPEN

* UNLESS ANISOTROPIC CONSOLIDATION IS TO BE EFFECTED
** IN BACK PRESSUREED TESTS, PRESSURE IS SUPPLIED TO PORE LINES, BUT DRAINAGE IS PERMITTED

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In summary, there are three main components: the cell with 3 pressure ports and axial plunger; the control panel which permits applying pressure to the sample as well as monitoring pressure and volume changes in each of the three connections from the cell; and finally the loading frame, which provides a constant rate of advance.

Test Procedure

Because unconfined clay samples retain their shape by negative pore pressure, they can be readily handled and trimmed to a particular size, if appropriate. The sample is placed on top of a porous stone and then enclosed in a thin rubber membrane.

The diameter of the membrane is slightly smaller than the diameter of the sample; therefore, a membrane stretcher is used. This membrane is sealed to the base pedestal and the top cap with rubber O-rings. The membrane isolates the sample from the cell fluid and allows the application of an all-round pressure to the sample.

The membrane is slid over the sample and lower platen. Then an O-ring is used to seal the membrane and secure it in place. The membrane stretcher is removed. The top cap is put in place on the sample and the membrane rolled onto the top cap. With a split ring tube, the O-ring is snapped onto the top cap to seal and secure the membrane.

The preparation of samples using granular materials that do not hold together such as sand, involves a slightly different procedure. First the weight of the sand is obtained. Then a split mould is used to form the sample during preparation. The membrane is already located and secured to the lower platen using the O-ring. The split mould is placed around the membrane and a gear clamp is used to hold the three pieces together. Next, the mould is filled with sand that is introduced using a funnel to achieve a very loose condition.

A densely packed sample can be achieved by roding moist sand. The porous stone and top cap are put in place and the membrane is sealed to the top cap.

Finally, the split mould is dismantled while applying a small vacuum holds the sample. This vacuum is maintained until a cell pressure is applied.

For any particular test, the sample weight, diameter and height are measured and recorded. The height should be measured, at two or more diametrically opposed locations. The diameter is measured at several locations with a vernier calliper, averaged, and corrected for membrane thickness.

Once these initial measurements are completed, the cell is replaced, bolted, and filled with water. Pressure is applied to the water and the vacuum inside the sample is released.

The test must be conducted following the stress path that closely simulates the stress history of the sample in the field. The most common stress path consists of applying the confining pressure (by means of the control panel) followed by the deviatoric stress. The deviatoric stress is defined as (see Figure 2):

$$\text{deviatoric stress} = \frac{P}{A} = (\sigma_1 - \sigma_3)$$

where:
P is vertical force applied to the sample
A is the cross sectional area of the sample;
σ_1 is the major principle stress (vertical in a triaxial test); and
σ_3 is the minor principle stress (confining stress in a triaxial test).

The dial indicators used for monitoring the force and deformation are zeroed. Valves connecting the top and bottom of the sample are kept open for drained tests or closed for undrained conditions.

In the simplest case, only the dial indicator (for the force) and the dial indicator (for the deformations) are read at predetermined increments.

For more elaborate testing, we must also monitor volumetric changes as indicated on the control panel using the pipettes as well as changes in pore pressures if they are allowed. Remember that soil behaviour is primarily determined by the mean state of confining stress, by deviatoric stress and by the void ratio. Therefore, we must continue to monitor these parameters as the test evolves.

Test samples are usually consolidated prior to the application of an axial load. The consolidation is monitored with the volume-change measuring device. After consolidation is complete, the piston is brought into contact with the top of the sample, the dial gauges are zeroed or read, and the drive gears are adjusted to give the desired rate of loading.

The test ends when failure or deformations exceeding 20% is reached. At completion of the test the specimen is unloaded, the confining pressure is reduced to atmospheric, then the cell dismantled and the sample is removed. Final measurements of weight can be made to verify the saturation of the sample.

Electronic Monitoring

The triaxial system can be enhanced using electronic instrumentation. Force is measured using a force transducer or load cell. This transducer is located inside the cell to cancel the friction effect.

LVDT (linear voltage differential transducers) are used to monitor the deformations.

Three pressure transducers are mounted at the base of the test cell to monitor the confining pressure and the pore pressure in the sample.

The load cell is an electronic device that correlates the deflection of a beam to the force applied to this axis. This transducer incorporates the use of strain gauges to relate deflection to force.

The LVDT replaces the dial indicator used to measure deformations. It is a transformer made up of two external coils and a central core. The change in position of the core relative to the coils produces a variable voltage change calibrated to a deformation measurement.

The pressure transducer uses a strain gage to measure the strain in a metal diaphragm as a result of pressure acting on it.

The volume measurement device makes use of an LVDT to measure the rise or fall of a piston within the volumetric cylinder. This change in movement is calibrated to the volume of water the sample will take in or push out. The volumetric indicator now used by the university was specifically selected to precisely measure very small flow volumes.

All these transducers can be read with a digital voltmeter. Alternatively, an analog to digital converter can be employed to measure the voltages and use the computer to collect the data.
New software was developed for the University of Waterloo soils lab to monitor the triaxial apparatus in May 2000. The new software is based on a National Instruments LabVIEW platform.

The new data output files no longer look like the ones you will work with. The new software performs all necessary calculations and conversions on the fly. However, the triaxial cell is still and will remain manually controlled, even though the LabVIEW software could control the experiment.

Whereas electronic systems have advantages, they do not give physical meaning to various procedures and measurements. Therefore, hands-on experience with manual devices is preferred for teaching purposes.

Additional Information

The shear strength of a soil is defined by the Mohr Coulomb failure criterion, which related the apparent cohesion (c'), effective angle of internal friction (ϕ') and the effective stress normal to the plane of failure (σ_n'). The parameters c' and ϕ' can be obtained graphically from a Mohr Coulomb (M-C) plot.

Plotting the principle stresses on the σ_n axis, and connecting them with a circle constructs a Mohr circle. The circle represents all of the possible stress shear (σ_n,τ) combinations measured on any plane through the sample.

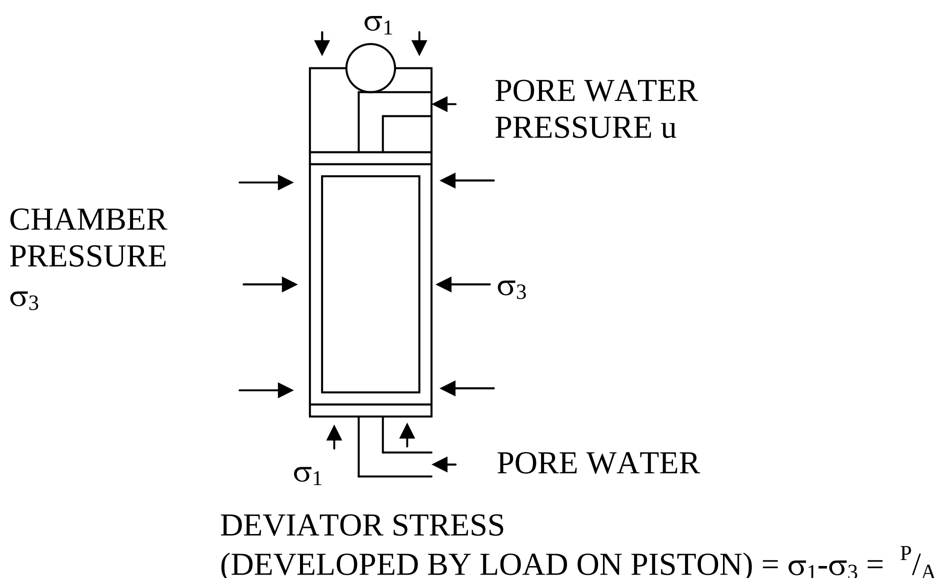

Figure 2: Principle Stress Orientations on a Triaxial Cell

By testing the sample to failure in several stress regimes, several circles can be constructed. The principle stresses are σ_1 (major principle stress), σ_2 (intermediate principle stress), and σ_3 (minor principle stress). The principle plane or no-shear plane is the σ_n axis. In a triaxial test σ_2=σ_3, but this is not always the case so be sure to check. The confining stress in a triaxial cell acts all around the sample so:

$$\sigma_1 = {}^{load}/_{area} + \sigma_3$$

Failure is defined when a stress combination (σ_n, τ) exists above the failure envelope. The failure envelope can be constructed by plotting the failure results from several triaxial tests (σ_f, τ_f). The parameters c' and ϕ' define the envelope. Phi (ϕ) can be determined by calculating the slope of the failure envelope. Phi is the internal angle of friction, or the angle at which grains slide past each other in dilatent behaviour, or break under conditions of grain shearing. Apparent cohesion or c' is the intercept of the failure envelope with the τ axis.

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To plot a Mohr circle from triaxial data, obtain σ_1, and σ_3 from the triaxial test and draw a circle connecting the two points centred at $(\sigma_1+\sigma_3)/2$. Repeat for every stress state tested. In a standard triaxial test σ_3 is fixed and σ_1 is gradually increased from σ_3 until failure occurs (a circle intersects the failure envelope). Plotting the results from three or more triaxial tests on the same graph, drawing the circles, and drawing a tangent to the circles constructs the failure envelope. The failure envelope is stress dependent, i.e. if the triaxial tests are performed at significantly different confining stresses the failure envelope will not be a straight line. For analysis a straight line may approximate the failure envelope within a given stress range. For this reason it is important that all triaxial tests be performed in the anticipated field stress range (Figure 5).

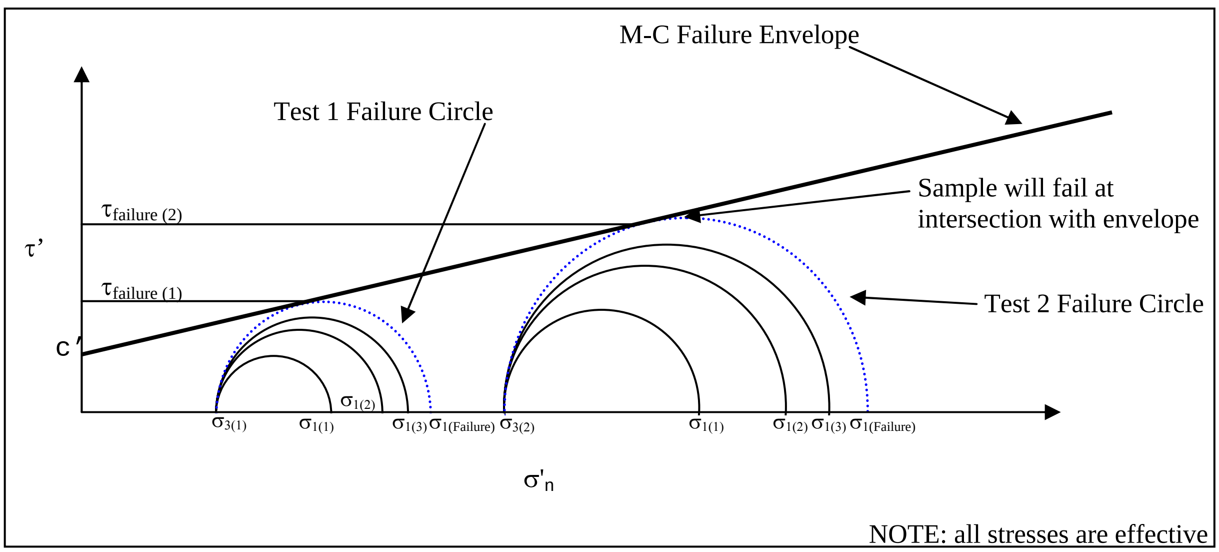

Figure 3: Mohr Circle Construction

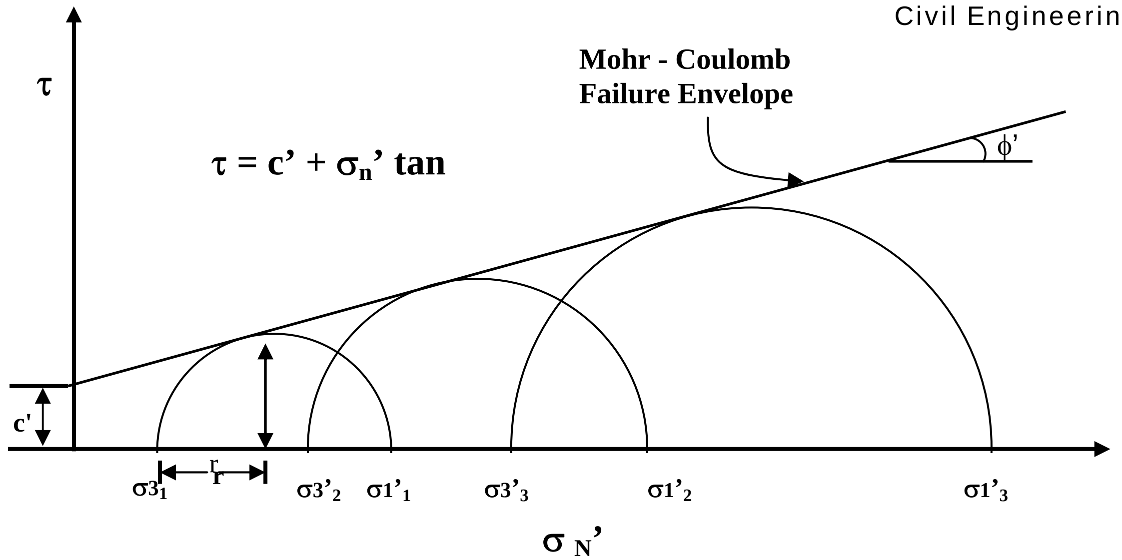

Figure 4: Final Mohr Coulomb Plot

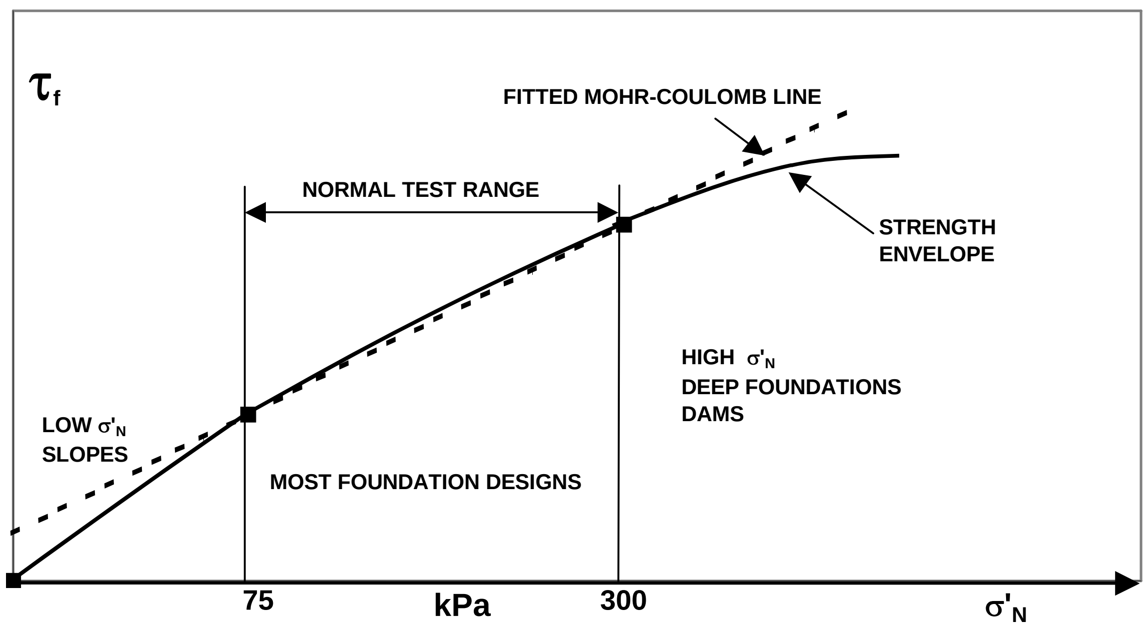

Figure 5: Changes in Strength Envelope with Changes in Stress State

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There are other methods for construction of a failure envelope. A t-s (or p-q depending on the reference) plot can also be developed using the centre and radius of the Mohr circles. The t-s relations can be transformed to convention M-C relations through the equations presented on Figure 6. Traditionally, t-s plots are used to track the stress path of a sample, when conventional M-C plots would be too cluttered. A stress path is a line that connects a series of points, each of which represent a successive stress state experienced by a soil specimen during the progress of a triaxial test.

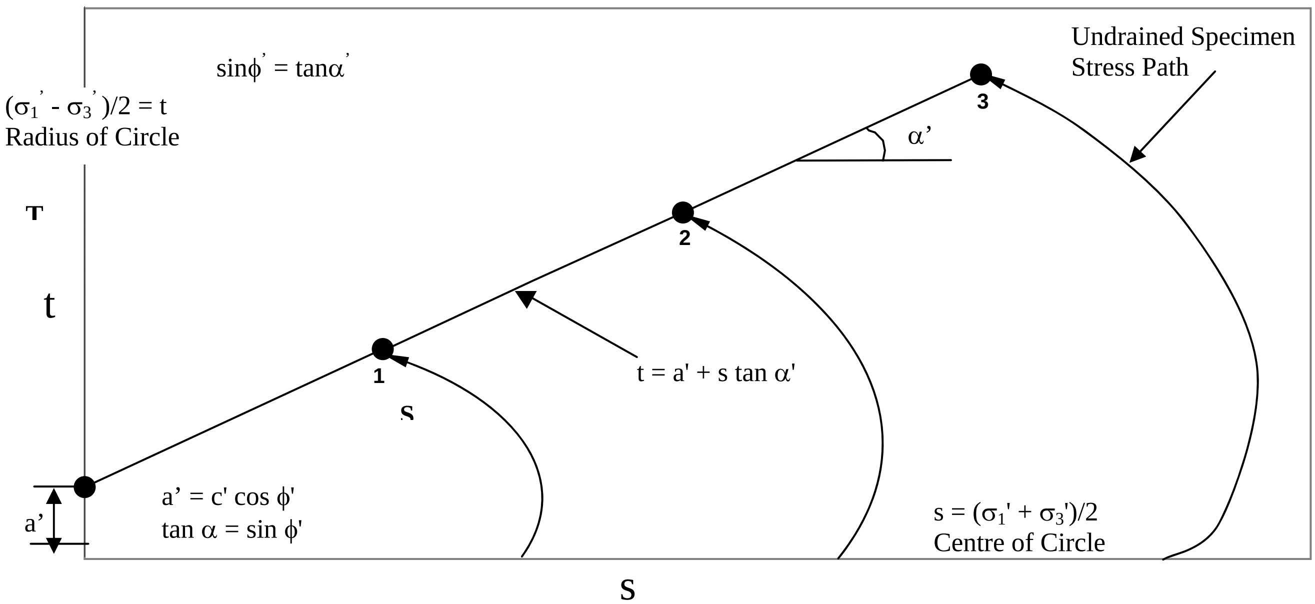

Figure 6: t-s Plot with Stress Paths

Other Parameters

There are two other pore water parameters of interest in a triaxial test, Skempton's *A* and *B* pore water coefficients.

Skempton's A criterion is used to quantify changes in pore pressure resulting from changes in σ_1. The stress on the sample resulting from the change in vertical stress is transferred from the soil skeleton to the pore fluid. When the sample fails the *A* parameter is called the Skempton's pore water coefficient at failure or A_f.

$$A_f = \frac{\Delta\mu_f}{\Delta\sigma_{1(f)}}$$

Note: the value of *A* will usually increase to A_f when loading is increased. Figure 7 shows the relationship of A_f to the over consolidation ratio (OCR) of the soil.

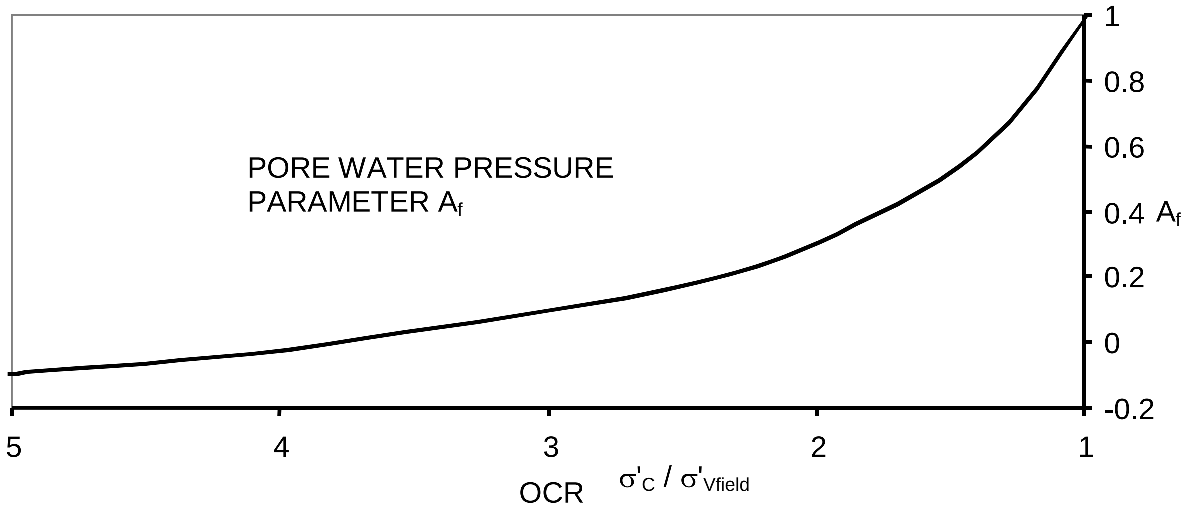

Figure 7: Relation Between A_f and OCR

The Skempton's *B* criterion indicates the degree of sample water saturation. If a sample is 100% saturated and no pore water drainage is allowed to occur an increase in the confining stress (σ_3) on the sample should result in an equivalent change in pore pressure (u). The B pore water coefficient can be expressed in the non-dimensional form:

$$B = \frac{\Delta\mu}{\Delta\sigma_3}$$

For saturated soft soils B is approximately 1. For saturated stiff soils *B* may be less than 1. A triaxial test specimen with a *B* value of 0.95 or greater is usually considered saturated.

Conclusion

The triaxial test is essential to understand soil behaviour. We can measure strength and stiffness, monitor the internal response of the particulate medium, monitor pore pressures as they build, and watch volume changes taking place during the test.

Proper understanding of material behaviour followed by the proper assessment of its characteristics allows the Engineer to improve designs and to reduce the risk of failures.

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Triaxial Test

Instructions for Laboratory Report

The consolidated-drained CD triaxial test is a popular test in geotechnical design. Engineers and consultants use parameters derived from this test in the design of foundations, retaining systems, slope stability analyses, etc. Deformation parameters at different levels of strain and confinement are determined from stress-strain plots (σ-ε), and the Coulomb failure envelope is obtained from Mohr circles or from a t-s plot (p-q).

The data set is from four CD (Consolidated Drained) triaxial tests on Winnipeg clay.

Include in your report:

Using the test data available (Civ 353 triaxial test data.xls) in the public directory.

- Plots of σ_1-σ_3 vs. ε_a and $\Delta V/V$ vs. ε_a.
- Plot Mohr Circles at maximum σ_1-σ_3 and compute c′ and ϕ'.
- Plot the t vs s graph (or p-q) and determine a’ and α’. (include the stress path)
    - Using a‘ and α’ compute c’ and ϕ’
- Compare the values of c′ and ϕ' obtained from the Mohr Circles with the values of c′ and ϕ' obtained from a’ and α’. Are these values reasonable?
- Comment on the post-peak behaviour of the Winnipeg clay tested.
- What is the physical meaning of parameters c’ and ϕ’?

Note:

$$\varepsilon_v = \frac{V_0 - V_1}{\mathrm{V}_0}$$

where V is the specimen volume and subscripts 0 and 1 represents initial and final specimen volumes respectively. It should be noted that triaxial testing is usually performed in compression ($V_0 > V_1$).

$$\varepsilon_a = \frac{H_0 - H_1}{\mathrm{H}_0}$$

where H is the specimen height and subscripts 0 and 1 represents initial and final specimen height respectively. It should be noted that triaxial testing is usually performed in compression ($H_0 > H_1$).

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Triaxial Test Data for Consolidated Drained Compression Tests on Winnipeg Clay

		SAMPLE 1			SAMPLE 2			SAMPLE 3			SAMPLE 4		
σ_3	kg/cm^2	0.25			0.77			1.40			3.00		
Vo	cm^3	80.68			81.15			80.70			77.70		
Ho	cm	8.04			8.01			7.99			7.92		
Ao	cm^2	10.03			10.12			10.10			9.81		
		Load kg.	Axial Deform. mm	Volume Gauge cc	Load kg.	Axial Deform. mm	Volume Gauge cc	Load kg.	Axial Deform. mm	Volume Gauge cc	Load kg.	Axial Deform. mm	Volume Gauge cc
		0	0	2	0	0	2	0	0	2	0	0	2
		1.11	0.037	2.02	0.9	0.01	2.18	0.99	0.04	2.1	1.65	0.079	2.02
		1.57	0.08	2.09	1.79	0.181	2.2	1.68	0.083	2.15	27.2	1.745	3.1
		4.51	0.318	2.19	2.42	0.232	2.29	2.83	0.154	2.13	29.3	1.982	3.11
		5.36	0.369	2.24	5.2	0.475	2.49	7.07	0.485	2.29	33.2	2.602	3.3
		5.52	0.402	2.28	6.01	0.56	2.5	8.35	0.602	2.4	34.9	3.183	3.45
		5.99	0.448	2.29	6.5	0.608	2.57	9.06	0.685	2.5	35	3.68	3.47
		8.27	0.698	2.34	8.04	0.802	2.66	12.53	1.038	2.68	26.4	4.303	3.42
		9.11	0.788	2.37	8.65	0.861	2.69	13.68	1.162	2.72	25.6	4.508	3.4
		9.54	0.838	2.4	8.94	0.906	2.7	14.27	1.248	2.79	22.4	5.242	3.3
		11.38	1.092	2.39	10.56	1.11	2.79	16.82	1.622	2.9	22.1	5.334	3.47
		11.66	1.118	2.4	11.12	1.178	2.82	17.53	1.75	2.98	22.8	5.383	3.57
		11.97	1.16	2.4	11.49	1.228	2.89	17.59	1.763	2.9			
		12.1	1.179	2.4	11.6	2.282	3	17.69	1.781	2.9			
		12.32	1.219	2.41	12.12	2.552	2.99	17.77	1.798	2.93			
		12.34	1.242	2.41	10.34	2.86	2.8	17.92	1.833	2.97			
		12.57	1.521	2.29	9.8	3.032	2.72	19.23	2.217	2.96			
		12.03	1.6	2.23	9.09	3.338	2.62	19.3	2.267	2.97			
		11.33	1.705	2.17	9.06	3.481	2.63	19.34	2.288	2.97			
		5.56	2.959	1.27				19.34	2.302	2.98			
		5.55	2.978	1.26				19.34	2.338	2.9			
								19.33	2.382	2.92			
								19.33	2.448	2.99			
								12.83	4.232	2.68			
								12.78	4.342	2.68			
								12.73	4.437	2.7			

Note: kg/cm^2 *98.07 = kPa

An increase in volume gauge reading means a **DECREASE** in sample volume.

$$A_{corr} = A_O\left[\frac{1+\dfrac{\Delta V}{V_O}}{1+\dfrac{\Delta H}{H_O}}\right]$$

Caution: Use the correct sign in the **Δ!!! + or -?**