CHAPTER IV

LABORATORY INVESTIGATION

The laboratory investigation was carried out between October, 1981 and Febuary, 1982, to obtain the petrographic description, and physical and mechanical properties of the rock specimens of Overburden Claystone and Red Beds (highly weathered claystone) sequences to be employed in the stability analysis.

4.1 Microscopic Study

The microscopic study of 8 thin-sections and the X-ray diffraction study of 3 specimens from Overburden Claystone were done. Both studies were to obtain the petrographic description and the mineral composition especially the clay minerals in these specimens.

4.1.1 Thin-section study

Eight thin-sections of 4 claystone specimens were cut parallel and perpendicular to bedding plane. The four claystone samples were from Subareas 2, 3, 4, and 5 and were assumed to be the representatives of the similar-appearance claystones in the entire study area. All specimens indicate a very fine-grained characteristic with the grain size mainly in the clay range with subordinate silt size (0.01 to less than 0.004 mm). The rocks consist mainly about 40-50 % calcium carbonate (calcite and dolomite ?) and about 30-40 % clay minerals, with sub-ordinate

5-10 % of quartz and feldspar and 2-5 % of organic matters. The alteration was found along the open crack.

4.1.2 X-ray diffraction study

Figure 25 shows the X-ray diffractogram of the clayey part of a claystone sample (No. CL-2), being used as the representative of the Overburden Claystone rocks. From the study, four types of minerals, illite, kaolinite, feldspar and quartz were recorded. This study indicate low content of expansive clay mineral in the claystones.

An additional proximate determination of weight-percent carbonate content in the rocks was carried out using HCl 1:1 as a solvent. The study indicates that the carbonate percentage found in four samples is 38.7 % in average. According to the diagram proposed by Lewan, 1978 (Figure 26), the rocks should be used the root name 'marlstone'. However since the petrologic study is not a major concern in this work the name 'claystone' is still used throughout this paper as a concurrence to the previous works.

4.2 Determination of Unit Weight and Water Content

The unit weight is a significant physical property of soil and rock for determining weight of the entire soil/rock mass, i.e., the non - tectonic stress in soil/rock mass. It is the gravitational force which creates the shearing stress on the rock body. Duncan (1969) defined the term unit weight or 'bulk density' (γ) as the weight of mineral aggregate and void-filled (partially or fully) water per unit volumn.

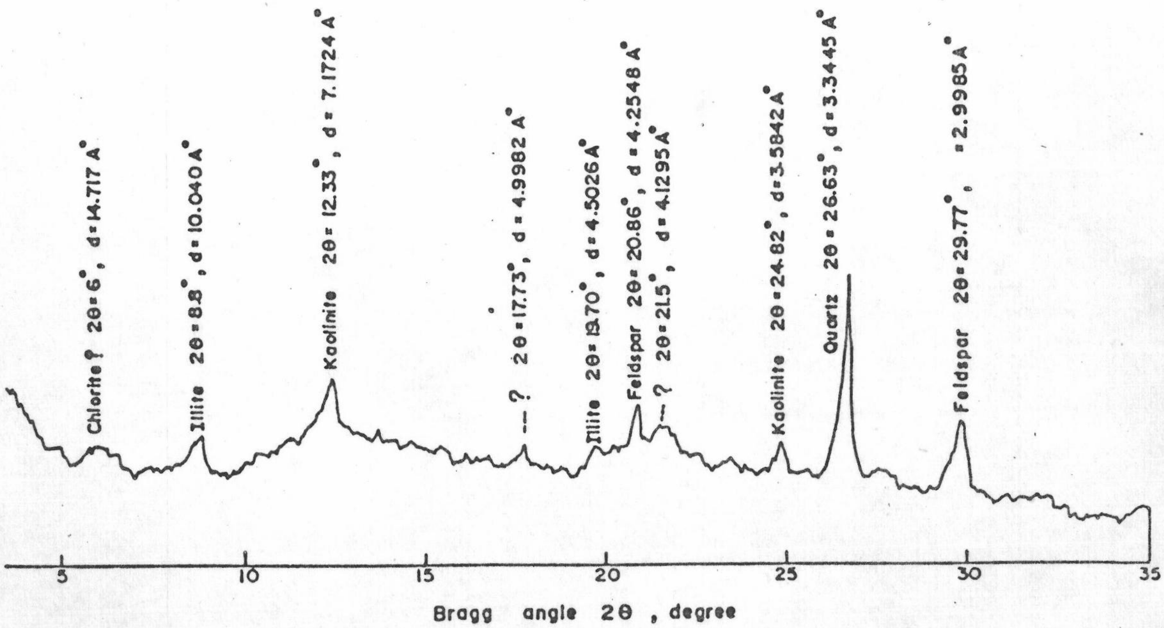


Figure 25. X-ray diffractogram of the clayey part of a claystone sample (No. CL-2). Cu K_{α} radiation (λ = 1.5405 A° 30 KV) was employed.

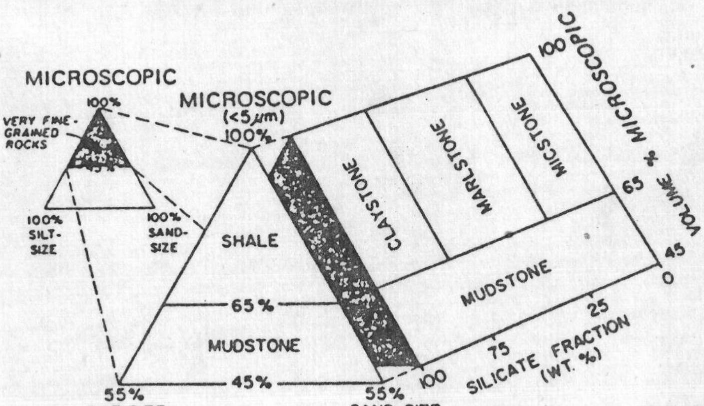


Figure 26. Lewan's (1978) proposed classification of very fine grained sedimentary rocks.

This test is meant to find the bulk density (γ) of the saturated claystones samples, and of naturally-moist, highly weathered claystones (of the Red Beds). The tests were carried out on the block specimens (5 x 5 x 4.5 cm^3) which were prepared for the further direct shear test.

Fresh to moderately weathered claystone specimens were soaked to obtain the maximum limit of water-filled condition. This is the water absorbtion or the capacity of the rock to absorb water by capillary suction when soaked for a comparatively long time at the atmospheric pressure and room temperature. It is sufficient to soak the rock samples for two days since there is not much difference in the absorbed-water content in any longer-time saturation.

Bulk density, natural water content and water content after two-day saturation are presented in Table 5. The results show that the bulk density of fresh claystones, slightly to moderately weathered claystone and highly weathered claystone (Red Beds) are slightly different, ranging from 1.95 to 2.17 metric ton/cubic meters (or gm/cm^3). The average value is 2.05 metric ton/cubic meters. Natural water content is in average 20.93 % and the saturated water content average 23.01 %.

4.3 Determination of Shear Strength

Shear strength of the block specimens was measured to find the shear strength parameters of slope materials, namely, cohesion which includes peak cohesion (C_p) and residual cohesion (C_r), and internal friction angle which includes peak internal friction angle (ϕ_p) and residual internal friction angle (ϕ_r). These parameters are used in

Table 5. The unit weight and water content of fresh (Fr), slightly to moderately weathered (Sw-Mw), and highly weathered (Hw) claystones.

Sample no.	Sample description	Natural water content (%)	Unit weight (gm/cm^3)	Saturated water content (%)
CL-1	Claystone, Fr	20.05	1.99	23.45
CL-2	Claystone, Fr	15.10	2.01	22.99
CL-3	Claystone, Fr	19.18	2.08	21.08
CL-4	Claystone, Fr	19.27	2.05	23.24
CL-5	Claystone, Fr	19.49	2.06	23.15
CL-6	Claystone, Sw-Mw	19.67	2.00	24.00
CL-7	Claystone, Sw-Mw	18.98	2.15	23.51
CL-8	Claystone, Sw-Mw	19.51	2.17	22.98
CL-9	Claystone, Sw-Mw	18.79	2.03	21.99
CL-10	Claystone, Sw-Mw	20.18	2.01	23.75
Rb-1	Claystone, Hw	18.50	1.95	-
Rb-2	Claystone, Hw	15.25	2.17	-
Rb-3	Claystone, Hw	15.80	2.10	-
Rb-4	Claystone, Hw	16.69	1.98	-
Rb-5	Claystone, Hw	17.54	2.05	-

the slope stability analysis. The principle of rock direct shearing is illustrated in Figure 27.

The direct shear strength tests used in this study are grouped of two methods, the measurement along the fabric or intact shear strength and the shear strength of rock on the planes of existing defect (or along discontinuity planes).

4.3.1 Intact shear strength test

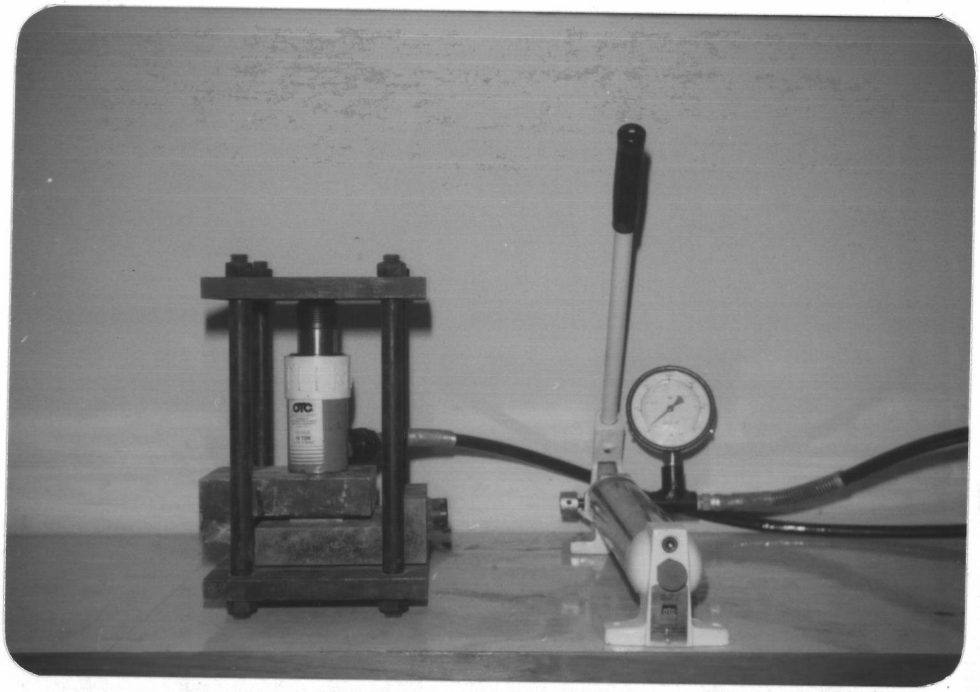
In this test the peak and residual direct shear strengths are measured as a function of normal stress on the shear plane. The test also facilitates the determination of cohesion intercept and angle of internal friction for peak and residual shear strength curves.

4.3.1.1 Apparatus and supplies

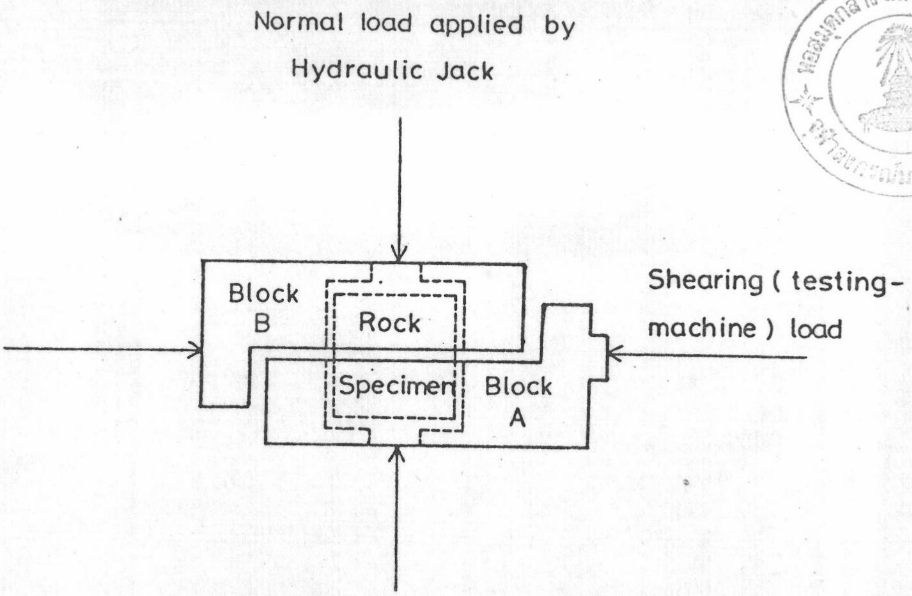
The apparatus and supplies used in the study are listed as follows.

a) Farnell's testing machine (Leonard Farnell & Co. Ltd., Hatfield, England) is used for applying compressive load to the test specimens (Figure 28).

b) The shear device to accommodate required size-specimen is shown in Figure 27. Test specimens are placed in a box made in two parts (blocks), each part having a recess of 5 X 5 cm^2 and 2 cm deep. The two blocks, A and B, assembled with the specimen, are placed within a framework consisting of two plates connected by four tie rods, each with a nut



(a)



(b)

Figure 27. Rock direct shear test.

(a) An assembly of shear device being used in this study.

(b) Principle of rock direct shearing.

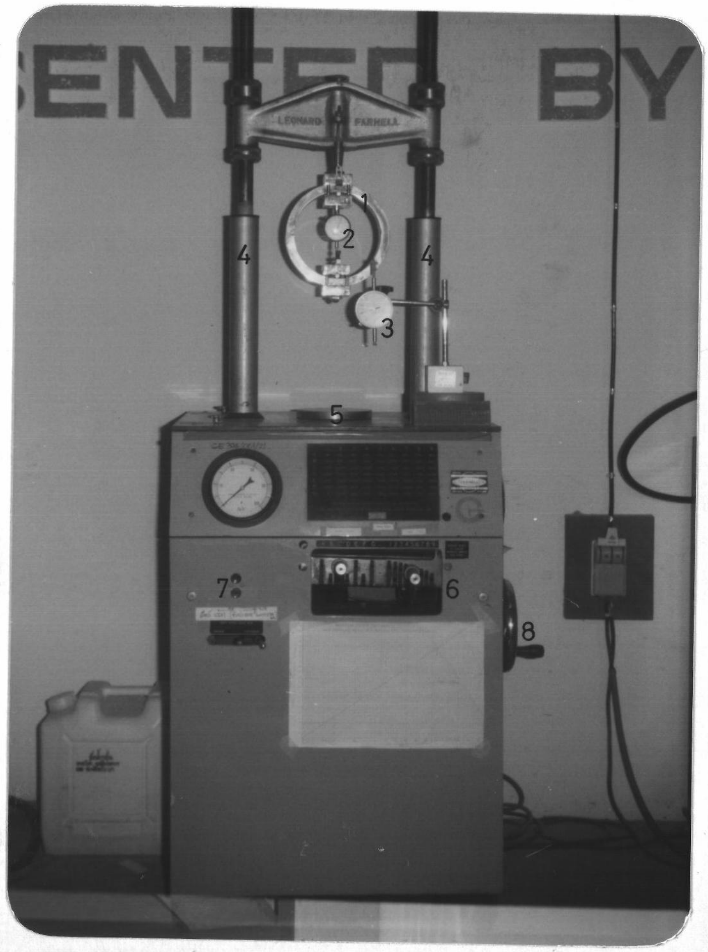


Figure 28. Farnell's testing machine. The parts of the machine are as follows.

1 - Proving ring (10 metric ton capacity)

2 - Dial gauge for load reading

3 - Dial gauge for measuring shear displacement.

4 - Loading arms

5 - Table

6 - Rate control

7 - Switches

8 - Load release wheel

on each end. One block (Block A) is fitted with a pair of ball bearings races in the baseplate for minimizing friction of the moving block. A hydraulic jack, 18 ton capacity, is inserted between the other block and its companion cover plate to apply and hold the normal load on the specimen.

c) A 10-ton (metric) capacity proving ring with dial gauge inside (reading factor = 10.538 Kg/division) is used for measuring shear load.

d) A dial gauge for measuring displacements reading to 0.01 mm is mounted between the table and load head.

e) A vernier caliper is used for measuring specimen size.

4.3.1.2 Specimen preparation

a) Nine collected samples were cut and trimmed into 72 test - specimens of a rectangular shape (5 X 5 X 4.5 cm^3). The dimensions of the test-specimens were measured to nearest 0.1 mm.

b) Thirty-six prepared test-specimens were soaked in distilled - water for two days (as mentioned in section 4.2) for testing in a saturated condition. The rest were for a natural condition test.

Each of a samples thus consist of 8 test-specimens; four specimens for naturally-moist and other four for saturated conditions.

4.3.1.3 Procedure of the test

Only three test-specimens of the same moisture condition of a collected specimen were tested to obtain one shear stress/normal stress relationship. The procedure of testing on each specimen is as follows.

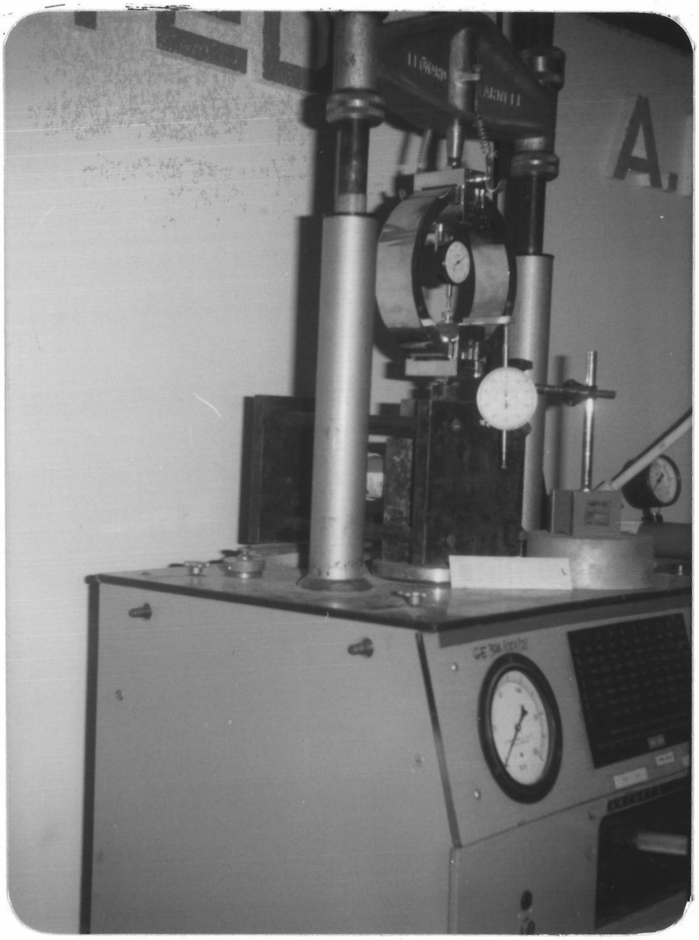
a) The selected normal load (normal stress X cross-sectional area) was applied using the hydraulic jack (Figure 27). The whole assembly was turned 90 degrees until the flange of Block B was rested on the table and the shear plane parallel to the axis of loading head, with the head ready to apply the shearing load onto the flange of Block A (Figure 29).

b) The dial gauge was mounted to measure a shear displacement.

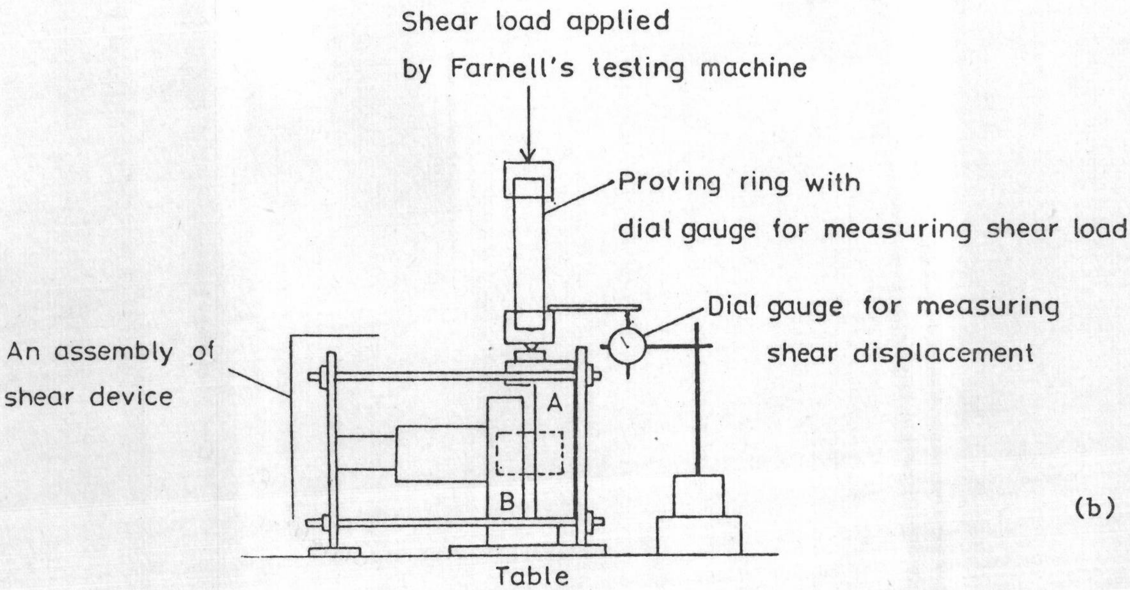
c) The shearing load was applied at a rate of 0.032 inch/minute (0.8 mm/minute). Shear load and displacement readings were taken at the intervals. After the failure occurred, shear load was still applied for one or two more minutes to read the residual shear strength value.

4.3.1.4 Calculation of the test results

The shear stress (τ) was calculated by dividing the shear load by the cross-sectional area, and the normal stress (σ_n), the normal load by the cross-sectional area. The shear stress/normal stress relationship was then plotted with the shear stress value on the ordinate and normal stress on the abscissa. A best-fitted line was drawn for the 3 test - results of each collected sample. This line marks the fracture envelope. The inclination of the envelope was measured and reported as the angle



(a)



(b)

Figure 29. Direct shear apparatus (a). The schematic diagram (b) gives the essential details of the apparatus, ready for test.

of internal friction of the rock. The shear stress where the envelope intersects the ordinate is the cohesion of the rock.

4.3.1.5 Discussions of the results

The results of intact direct shear strength test are reported in Table 6 and the normal stress/shear stress relationship of different samples are illustrated in Figure 30 (a) to (i). The test results indicate that the shear strength parameters of fresh claystone are generally greater than those of slightly to moderately weathered claystone especially for the naturally-moist condition though each group of claystones might have a large variation of shear strength parameters which includes peak and residual cohesion (C_p and C_r) and peak and residual internal friction angle (ϕ_p and ϕ_r).

For the saturated condition, rock sample CL-3, CL-5 and CL-7 could not be performed in the shear test because the prepared specimens cracked after two-day saturation. The crack occurred along the bedding trace of claystone probably due to the swelling effect. Although these three saturated samples could not be tested, the results obtained from the rest indicated a general tendency for both the cohesion (C_p and C_r) and the internal friction angle (ϕ_p and ϕ_r) of the saturated condition to be lower than those of naturally moist.

In some specimens the shear strength parameters showed abnormally high value, much higher than those from the other specimens tested in the same condition. Such is the case of Sample CL-2 which an abnormally high ϕ_p value was found in the naturally moist condition (see Figure 30(b)).

Table 6. Intact shear strength test results of claystone.

Sample no.	Sample description	Shear strength parameters: Naturally condition C_p	ϕ_p	C_r	ϕ_r	w	Saturated condition C_p	ϕ_p	C_r	ϕ_r	w
CL-1	Claystone, Fr	28.9	29.2	1.8	18.0	20.0	23.0	21.9	1.2	21.4	23.4
CL-2	Claystone, Fr	5.8	52.7	2.2	15.1	15.7	17.0	12.5	2.0	17.0	22.9
CL-3	Claystone, Fr	31.6	15.5	3.3	16.2	19.3	-	-	-	-	23.2
CL-4	Claystone, Fr	25.0	30.9	0.8	29.2	19.2	14.1	25.8	1.5	17.4	21.1
CL-5	Claystone, Fr	25.7	21.8	1.4	20.8	19.5	-	-	-	-	23.2
CL-6	Claystone, Sw-Mw	6.8	32.7	0.0	24.4	20.3	9.0	13.2	0.5	12.0	23.5
CL-7	Claystone, Sw-Mw	22.7	18.5	3.0	17.2	19.8	-	-	-	-	24.0
CL-8	Claystone, Sw-Mw	8.2	25.6	4.5	25.0	20.2	3.2	24.0	0.0	20.5	23.8
CL-9	Claystone, Sw-Mw	8.0	19.6	1.1	10.0	19.5	7.8	31.7	2.3	27.1	22.9

Notations:

C_p = Peak cohesion (Kg/cm^2)

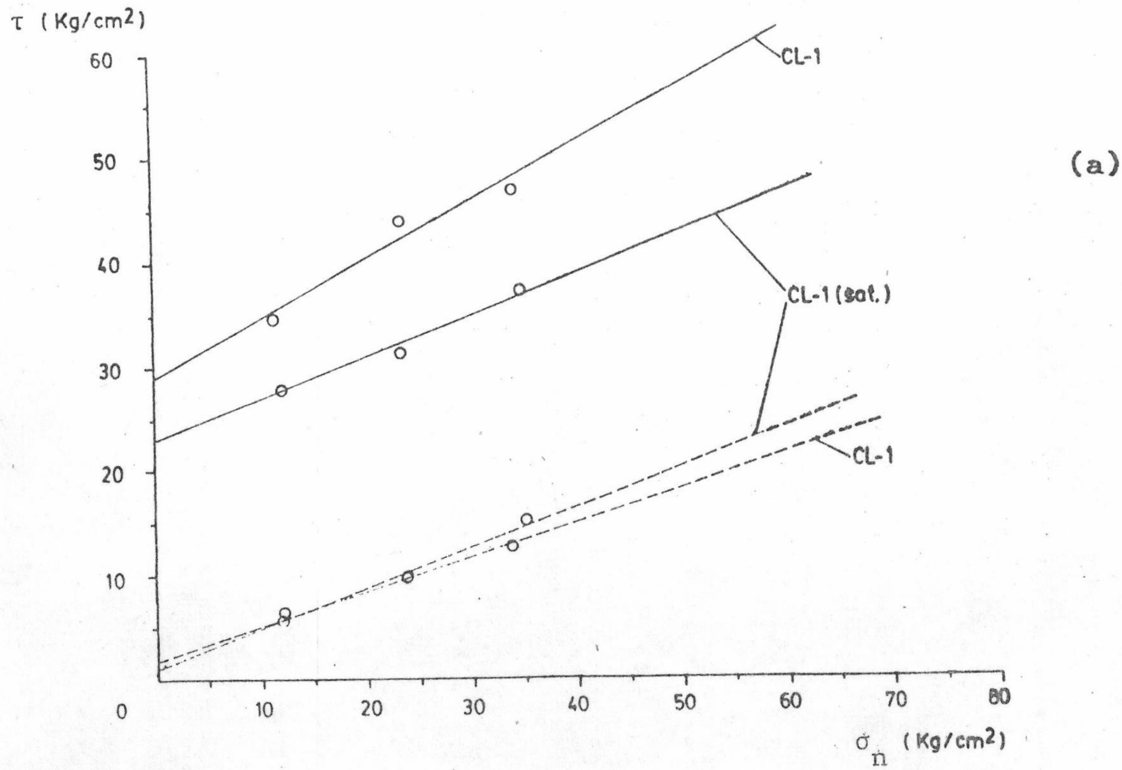
C_r = Residual cohesion (Kg/cm^2)

ϕ_p = Peak internal friction angle (degrees)

ϕ_r = Residual internal friction angle (degrees)

w = Water content (%)

Figure 30. Normal stress (σ_n)/shear stress (τ) relationship of intact shear strength on claystone. The graphs shown are the normal stress/shear stress relationship of claystone samples CL-1 (a) to CL-9 (i) respectively. The envelopes of solid line are of the peak shear strength; of broken line, residual shear strength. The saturated condition (sat.) is marked; otherwise, naturally-moist condition.



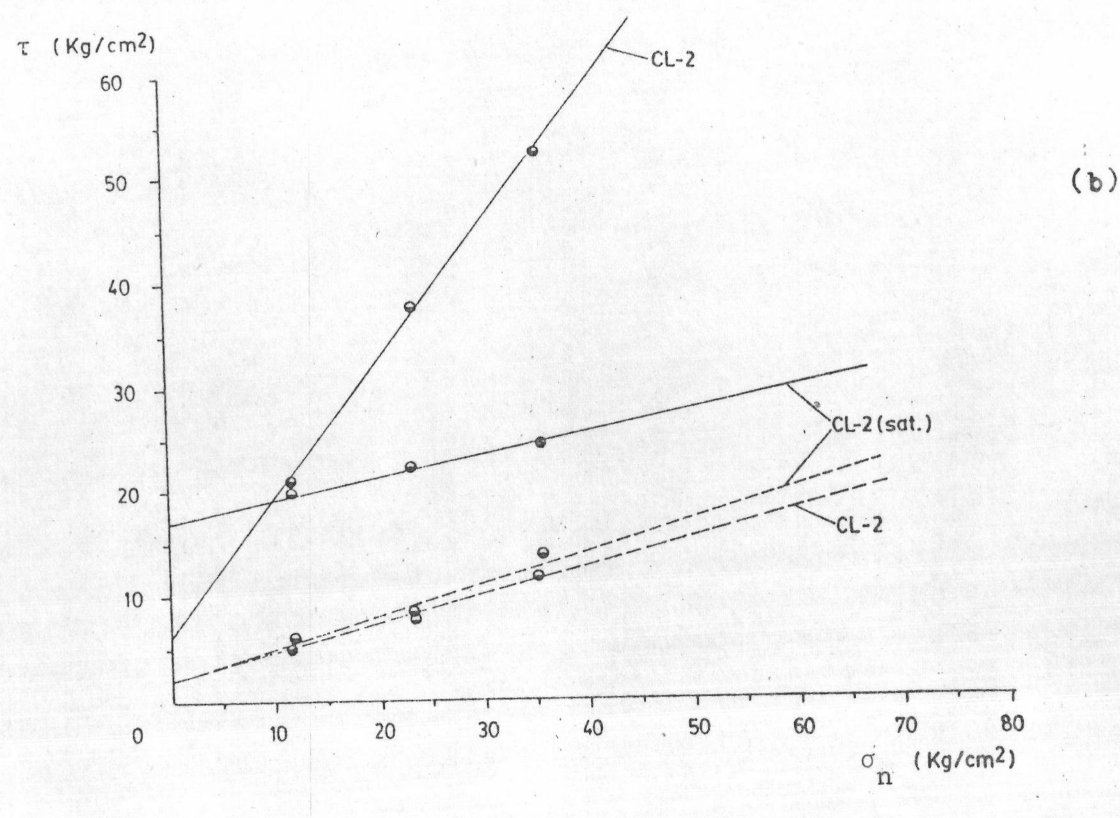
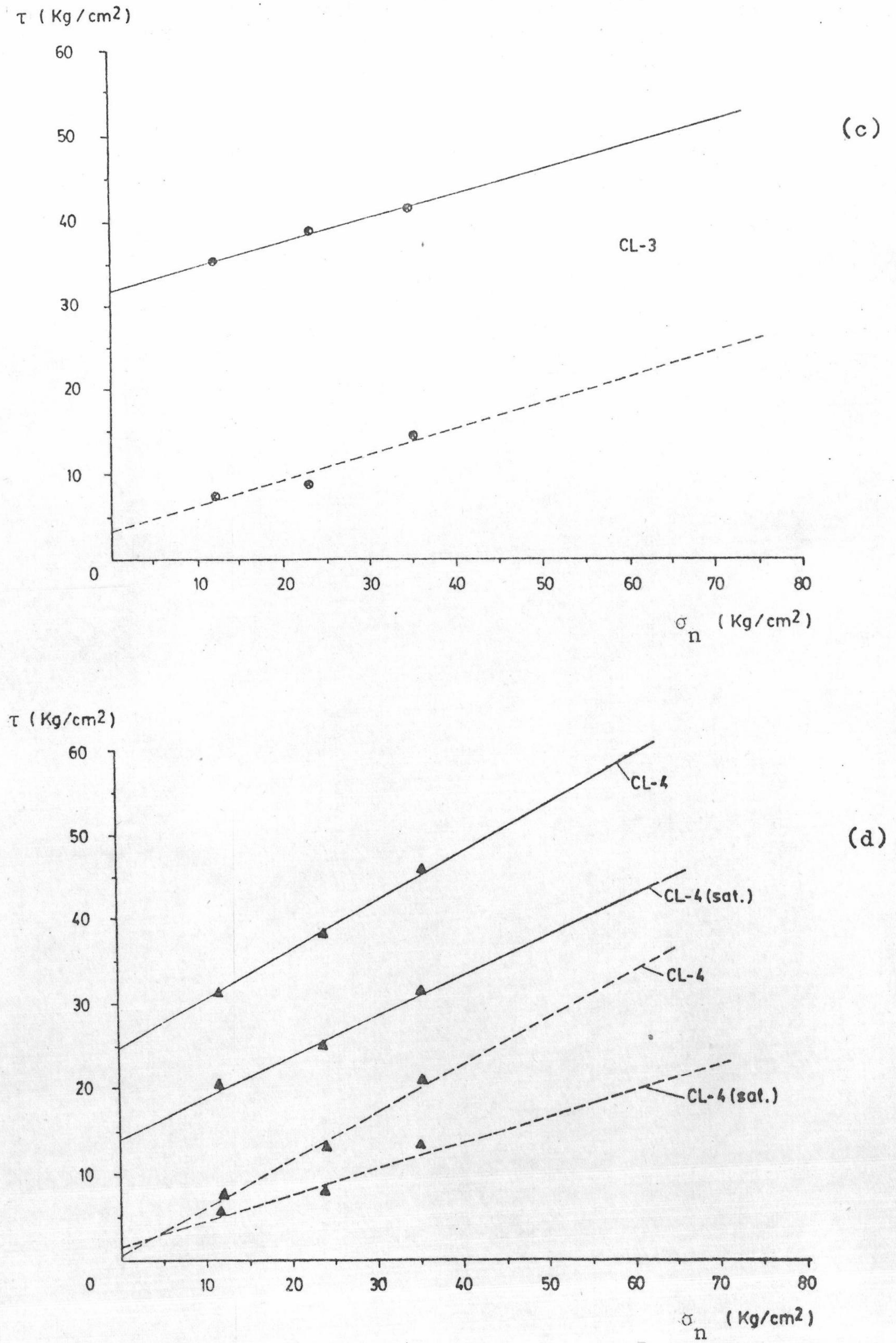


Figure 30. cont.



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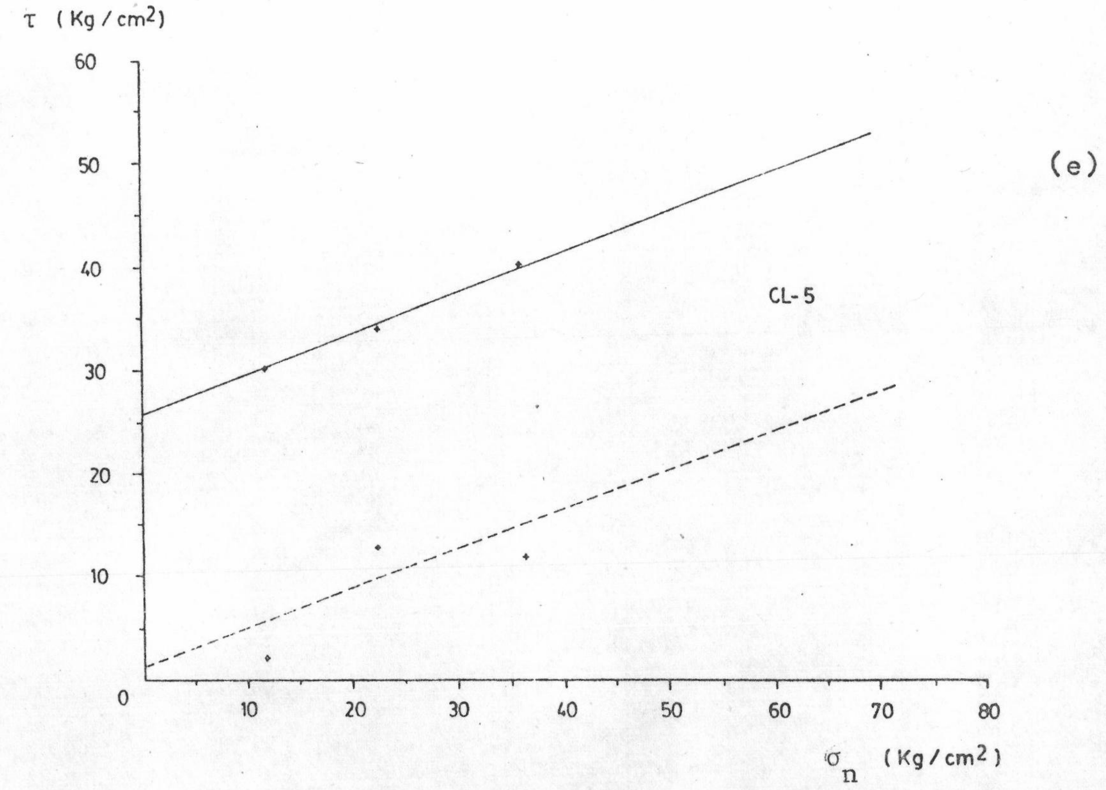

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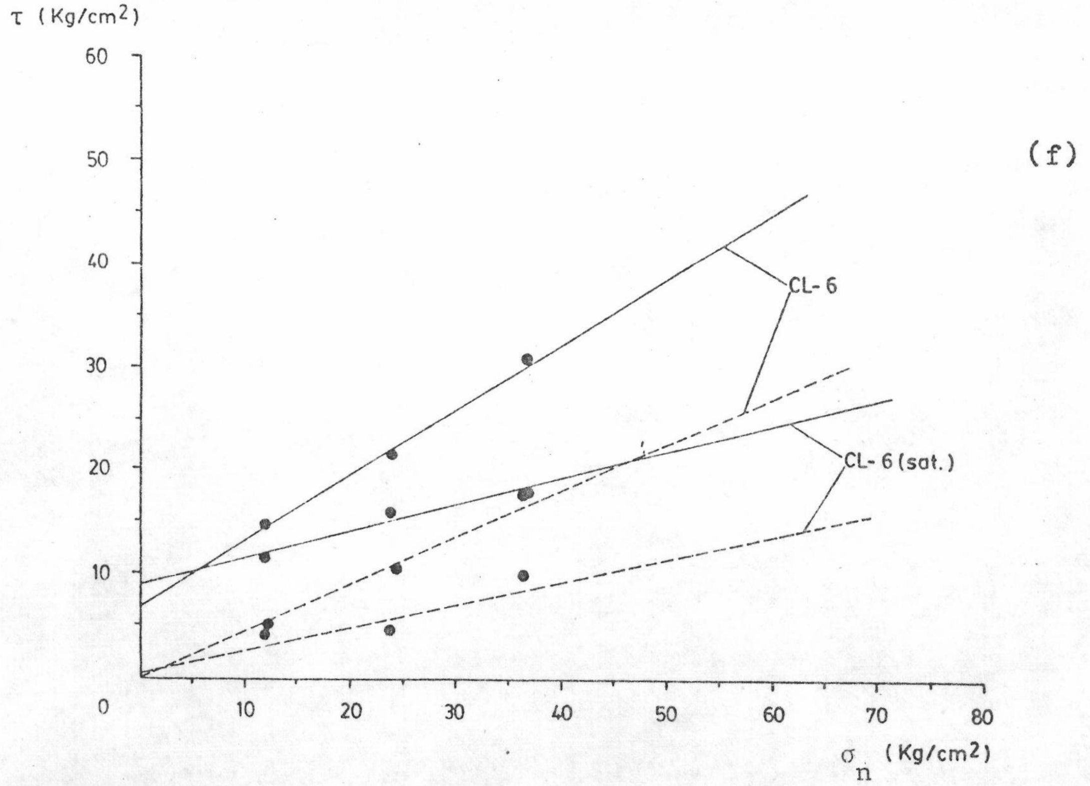

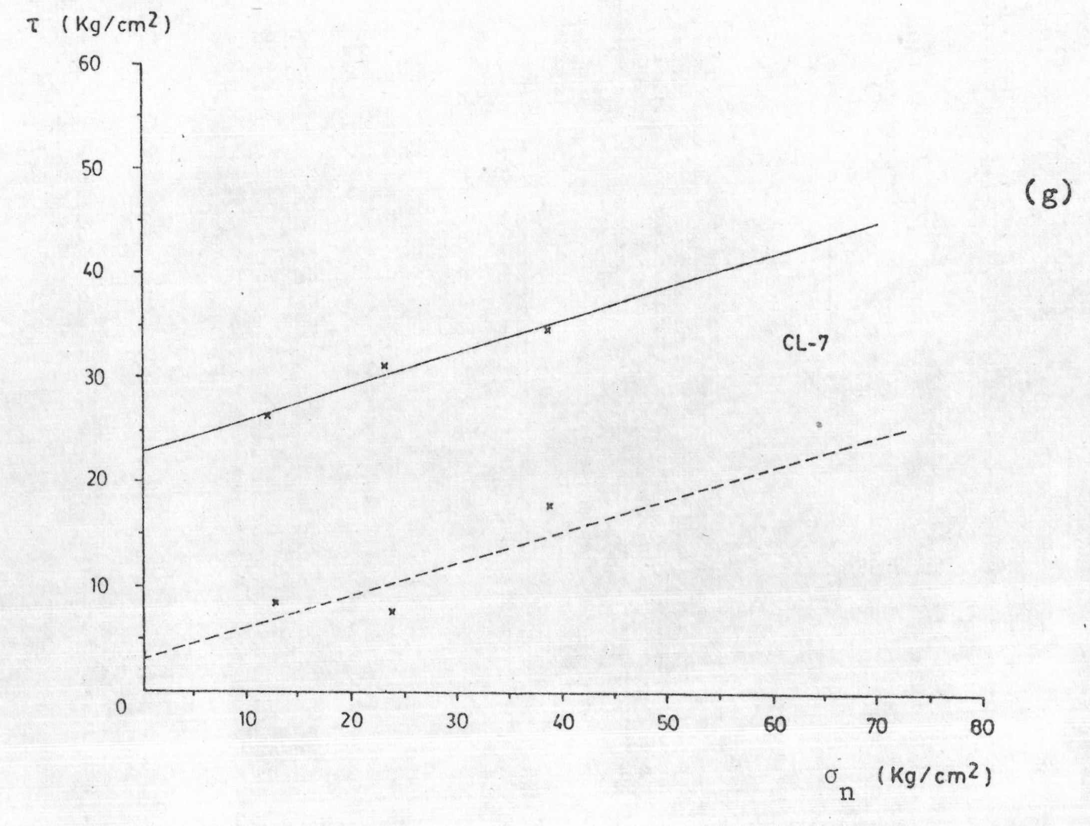

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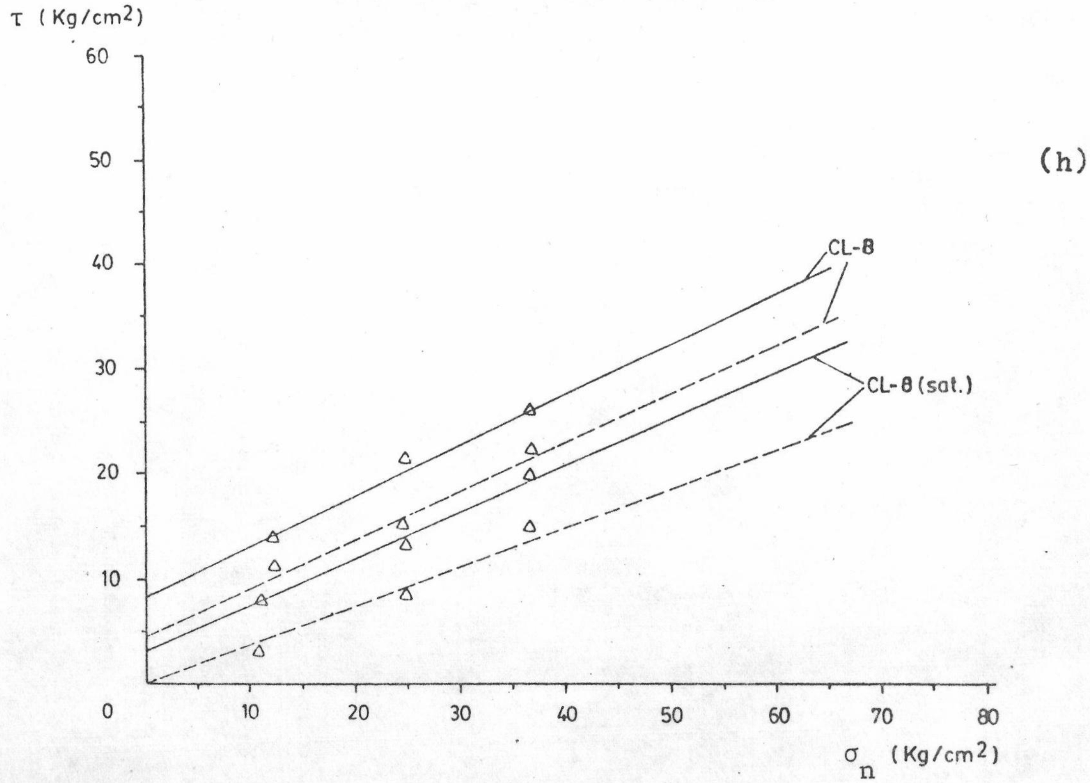


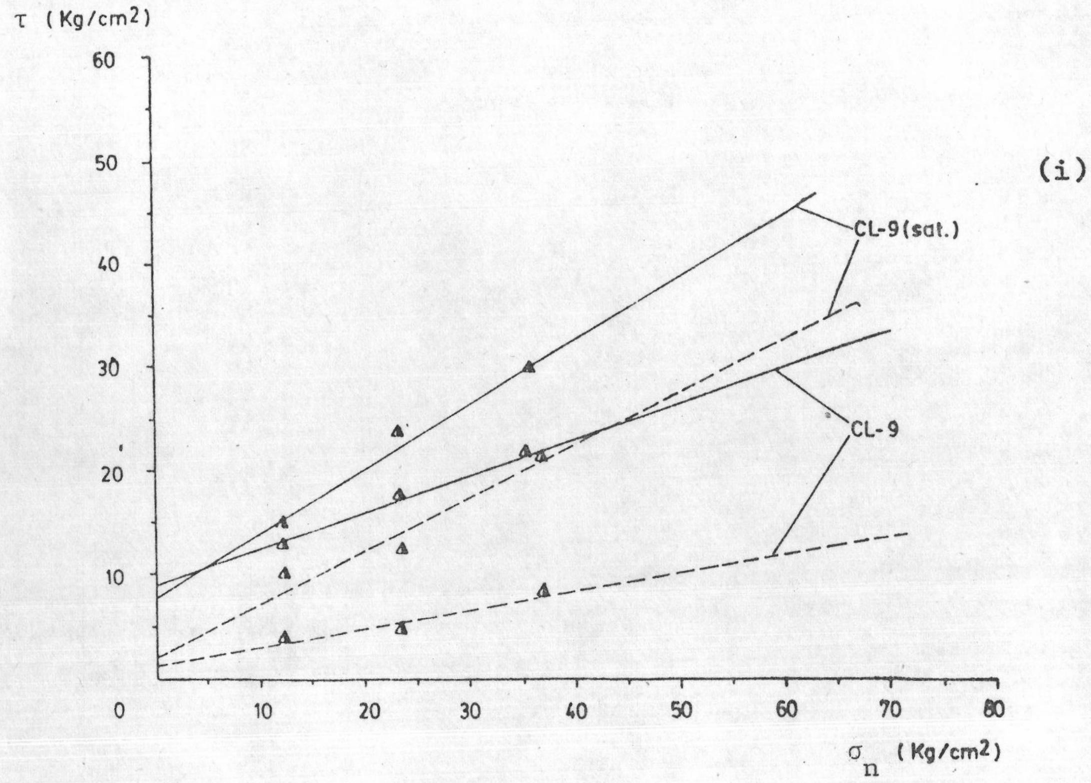


Figure 30. cont.

It is probably because the natural water content of Sample CL-2 is low relative to the others. The significant effect is thus the anisotropy of rock specimen causes the rough shear surface, then the asperities composing rough surface are interlocked at high normal load. Hence the shear strength increases.

4.3.2 Defect shear strength test

This test was to measure the direct shear strength of rock on smooth bedding plane and on natural rough fracture surface.

4.3.2.1 Apparatus and supplies

The apparatus and supplies to be used are listed below.

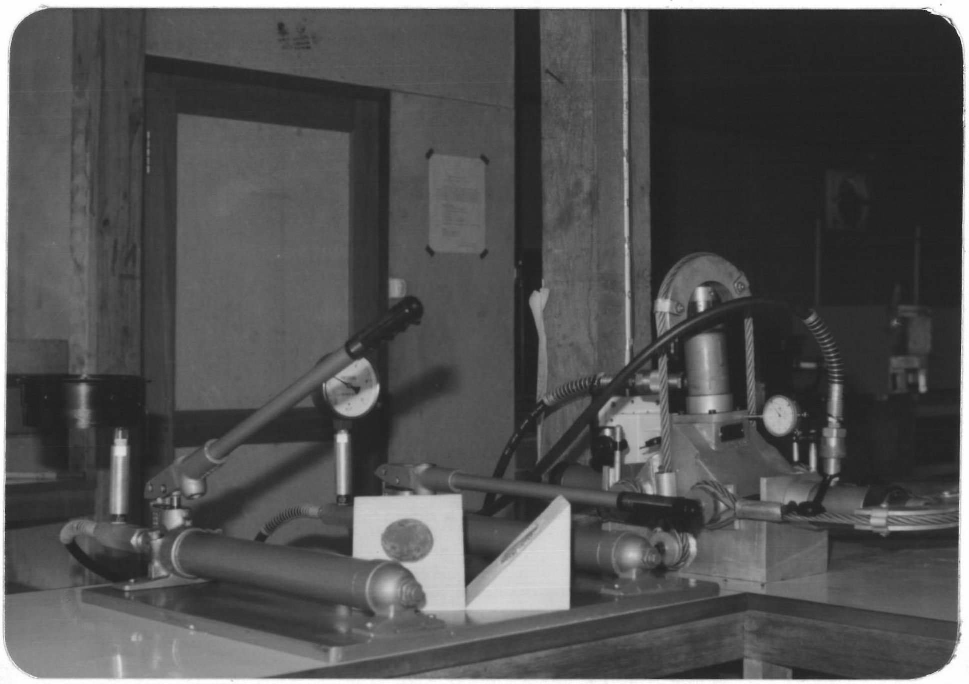
a) Ross-Brown portable shear machine as show in Figure 31(a) and (b).

b) Two prism-shaped molds suitable for specimen casting, together with plaster of paris. Each mold is 16 cm long, 12 cm wide and 8 cm high.

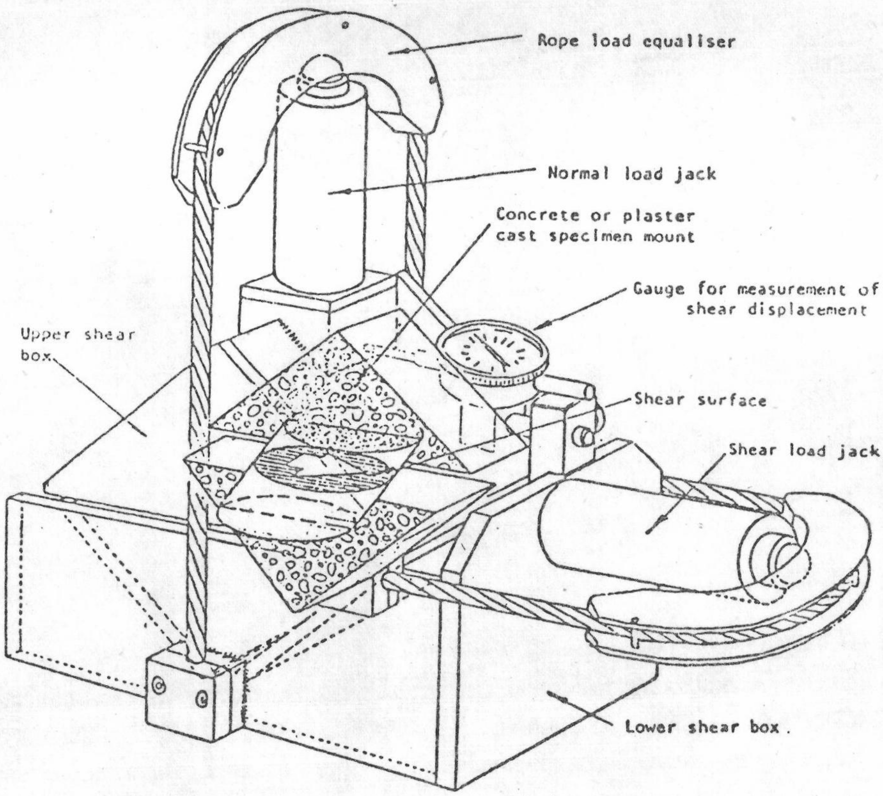
c) A vernier caliper for measuring the size of shear plane.

4.3.2.2 Specimen preparation

A sample containing a discontinuity to be tested is cut and trimmed to a size which fits into the mold, then the surface area of discontinuity is measured. The two halves are wired together in order to prevent movement along the discontinuity and the sample is then casted in plaster cement. Care must be taken that the discontinuity is positioned



(a)



(b)

Figure 31. Ross-Brown portable shear machine.

(a) Photograph of a portable shear machine with two hydraulic jacks for applying shear load (left) and normal load (right).

(b) Drawing of a portable shear machine showing the position of the specimen and the shear surface. (after Hoek and Bray, 1974).

accurately so that it lies in the shear plane of the machine. A bed of clean gravel, placed in the bottom of the first mold is sometimes used to support the specimen during setting up and casting the block by pouring the wet mix of plaster around the blocks. The gravels can be left in place to become a part of the casting without any effect in the test.

4.3.2.3 Procedure of the test

Once the casting has been set, this usually takes 24 hours, the specimen is transferred into the shear machine. The upper shear box is set in position and a small normal load is applied in order to prevent movement of the specimen. The wires binding the two halves of the specimen together are then cut and the shear load cable is place in position. The specimen is now ready for testing.

The normal load is increase to the value chosen for the test. This normal load is maintained constant while the shear load increases. The displacement during the application of the shear load is recorded through out the test.

Once the peak strength (maximum shear stress) has been exceeded, usually after a shear displacement of a few millimeters, the displacement is allowed to continue to obtain the residual shear load in the test. It was found that a lower shear load was required to sustain the movement.

4.3.2.4 Calculation of the test results

The normal load and shear load are divided by the surface area of the discontinuity surface to obtain normal and shear stresses respectively. The recorded data were treated similar as those obtained in the direct shear test (Section 4.3.1.4), i.e., the shear stress/normal stress relationship was plotted with the shear stress value on the ordinate and normal stress on the abscissa. A best-fitted line was drawn for the 3 test-results of each collected sample. This line marks the fracture envelope. The inclination of the envelope is measured and reported as the angle of internal friction of the rock. The shear stress where the envelope intersects the ordinate is the cohesion of rock on the defected-plane.

4.3.2.5 Discussion of the results

The results of defect shear strength test are shown in Table 7 and the normal stress/shear stress relationships are illustrated in Figure 32 (a) and (b).

From the results, it shows that the cohesion (C_r) and internal friction angle (ϕ_r) of fresh to moderately weathered claystones have a small variation on both groups of specimens, those sheared on a smooth bedding surface or on a rough joint surface. Higher shear strength parameters were however found on a rough joint surface with an average value of C_r = 10.3 metric ton/m^2 (1.03 Kg/cm^2) and ϕ_r = 22.12 degrees while the lower values were found on smooth surface with an average value of C_r = 5.7 metric ton/m^2 (0.57 Kg/cm^2) and ϕ_r = 16.07 degrees. These C_r, ϕ_r values will be used as maximum and minimum shear strength parameters employing in the slope stability analyses in the following chapter.

Table 7. Defect shear strength test results of claystone.

Sample no.	Sample description	w (%)	Defect types	Shear strength parameters C_r (kg/cm^2)	$\emptyset_r$ (degree)
CL-D1	Claystone, Fr-Sw	16.0		0.50	17.0
CL-D2	Claystone, Fr-Sw	18.5	Bedding	0.72	15.8
CL-D3	Claystone, Fr-Sw	18.6	plane, smooth	0.75	16.0
CL-D4	Claystone, Sw-Mw	17.5		0.30	15.7
CL-D5	Claystone, Sw-Mw	19.0		0.57	15.9
			Average	0.57	16.1
CL-D6	Claystone, Fr-Sw	14.3		1.05	24.0
CL-D7	Claystone, Fr-Sw	16.7	Jointing	1.45	20.2
CL-D8	Claystone, Fr-Sw	18.3	plane, rough	0.90	24.0
CL-D9	Claystone, Sw-Mw	15.6		0.91	21.8
CL-D10	Claystone, Sw-Mw	17.5		0.84	20.6
			Average	1.03	22.1

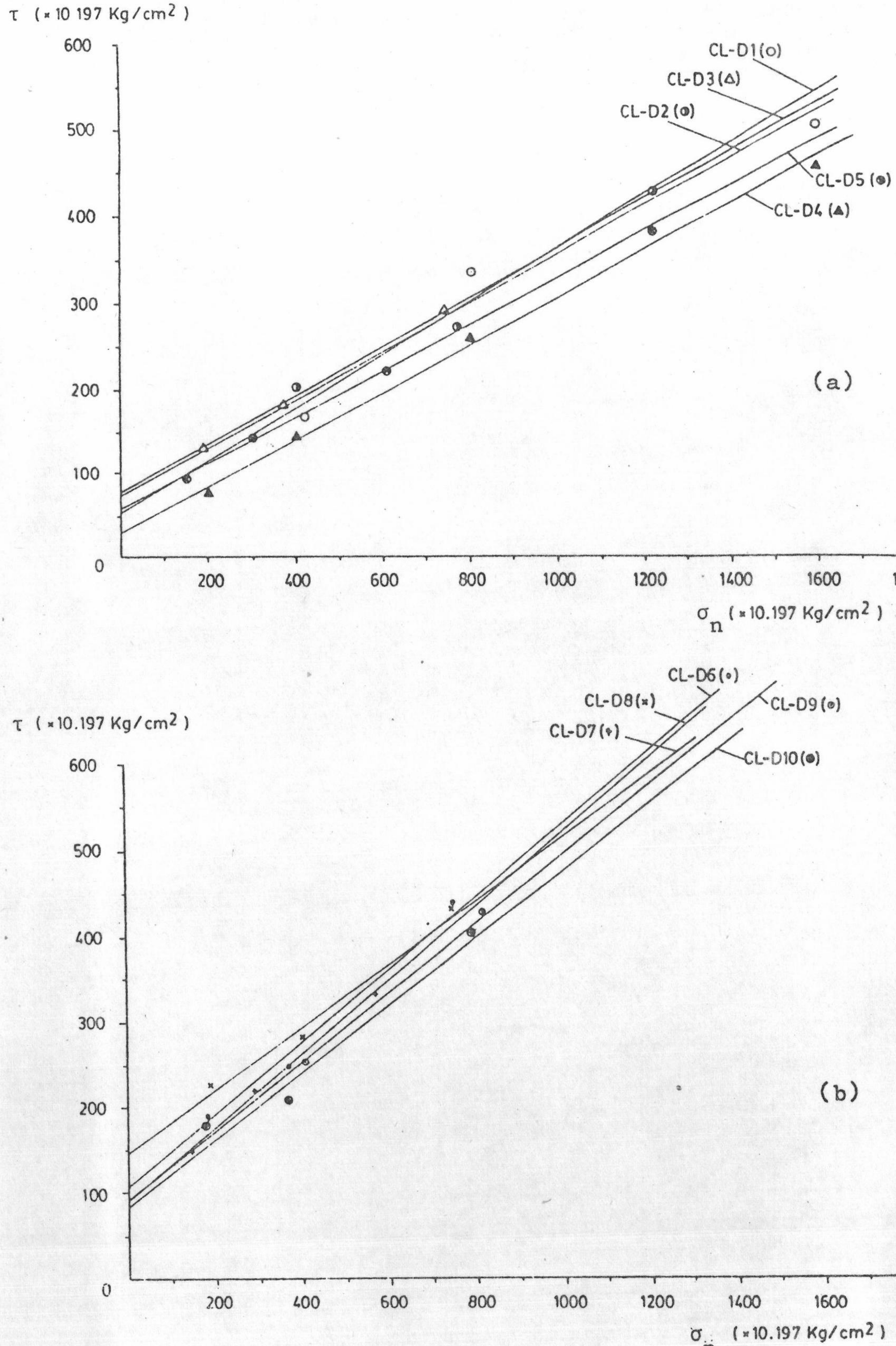


Figure 32. Normal stress (σ_n)/shear stress (τ) relationship of deject shear strength test on claystone.

(a) Test on smooth bedding plane.

(b) Test on rough jointing plane.

4.4 Other Mechanical Properties Available

Apart from the laboratory study just mentioned, the other mechanical properties essential for the slope stability analysis were collected from the previous works, especially the work of Duangduen (1978).

4.4.1 Unconfined compressive strength

Duangduen (1978) performed a laboratory test of the unconfined compressive strength of both Overburden and Interburden Claystones collected from the mine area. The unconfined compressive strength test results of overburden claystone were quoted in Table 8. The results indicate that the unconfined compressive strength in the direction perpendicular to bedding plane is relatively high compared to that in the direction parallel to bedding plane, and that the increasing in water content reduced the unconfined compressive strength in both cases.

4.4.2 Tensile strength

Tensile strength is the property of rock that resists the pulling effect. This property counteracts the driving force which tend to pull the rock mass down slope. A summary of the results of tensile (Brazilian) test on Overburden Claystone done by Duangduen (1978) is shown in Table 9.

From the table, the tensile strength of claystone when the tensile stress direction is perpendicular to bedding plane is lower than that when the direction is parallel to bedding plane. For a dry-state

Table 8. Summary of the unconfined compression test results of Overburden Claystone. (after Duangduen, 1978)

Specimen No.	Direction of Test Relate to Bedding	State of Test	L:D Ratio	W.C. (%)	Max. U.C. Strength kg/cm^2	E_t kg/cm^2	E_s kg/cm^2	Poisson's Ratio
1	⊥	Dry	1.96	1.81	354	16.670	25,180	0.12
2	⊥	Dry	1.89	1.84	363	13,690	23,380	0.11
Average				1.83	358	15,180	24,280	0.12
1	⊥	Nat.	2.00	17.32	139	12,000	12,000	0,25
2	⊥	Nat.	1.94	18.43	138	20,000	20,000	0.26
Average				17.88	139	16,000	16,000	0.26
1	⊥	Sat.	2.06	23.27	79	7,880	7,880	0.26
2	⊥	Sat.	1.99	23.32	88	6,820	6,820	0.26
Average				23.30	84	7,350	7,350	0,26
1	//	Dry	2.02	0.59	291	40,000	52,000	0.17
2	//	Dry	2.02	0.93	322	43,000	57,500	0.14
3	//	Dry	1.99	0.44	304	46,000	46,000	0.15
Average				0.66	306	43,000	51,830	0.15
1	//	Nat.	2.01	19.91	133	14,500	16,200	0.29
2	//	Nat.	2.00	20.52	133	16,750	20,000	0.28
3	//	Nat.	2.01	20.65	125	14,300	17,840	0.29
Average				20.36	130	15,180	18,010	0.29
1	//	Sat.	1.99	25.75	77	10,000	10,000	0.31
2	//	Sat.	1.99	23.30	84	12,500	15,000	0,33
3	//	Sat.	1.99	25.30	80	7,500	7,500	0.38
Average				24.78	80	10,850	10,670	0.34

<u>Note</u>

Nat. : Natural

Sat. : Saturated

Table 9. Summary of the tensile (Brazilian) test results of Overburden Claystone. (after Duangduen, 1978)

Specimen no.	Direction of Stress Relate to Bedding	State of Test	L:D Ratio	w.C. %	Max. σ_t kg/cm^2	E_t kg/cm^2	E_s kg/cm^2
1	⊥	Dry	0.48	1.13	30	46,670	46,670
2	⊥	Dry	0.50	1.29	32	36,360	36,360
3	⊥	Dry	0.50	1.10	30	66,500	66,500
Average				1.17	31	49,840	49,340
1	⊥	Natural	0.50	21.83	19	8,710	12,780
2	⊥	Natural	0.52	21.45	15	8,570	12,000
3	⊥	Natural	0.47	22.35	19	8,500	11,870
Average				21.87	18	8,600	12,220
1	⊥	Saturated	0.50	25.27	8	7,000	8,260
2	⊥	Saturated	0.50	25.72	8	5,700	6,670
3	⊥	Saturated	0.50	25.45	8	6,170	7,400
Average				25.48	8	6,290	7,450
1	//	Dry	0.49	1.80	34	37,400	37,390
2	//	Dry	0.50	2.31	34	30,650	36,960
3	//	Dry	0.50	1.64	38	36,670	36,670
Average				1.92	36	35,000	37,000
1	//	Natural	0.48	19.07	27	10,570	15,450
2	//	Natural	0.50	18.92	21	12,600	16,400
3	//	Natural	0.49	19.17	22	13,000	16,000
Average				19.00	23	12,060	15,900
1	//	Saturated	0.48	24.74	16	14,330	20,000
2	//	Saturated	0.51	25.00	13	10,800	16,600
3	//	Saturated	0.49	22.00	14	8,890	8,890
Average				24.00	14	11,340	15,200

test the tensile strength of claystone is higher than that when test on natural and saturated state.

4.4.3 Defect shear strength of Red Beds

The upper part of the northwest slope in Subareas 2, 3 and 4 consists of Red Beds. The shear strength of Red Beds was also quoted from the previous reports to be used in a slope stability analysis here.

Defect shear strength of Red Beds was collected from the work of Longworth-CMPS (1981) as Table 10. The defect shear strength test was carried out using a Ross-Brown portable shear machine. The type of defects consisted of rough joints, smooth joints and bedding planes and faults. The test was performed on the core specimens (NX-size, 54 mm in diameter) which collected from the geotechnical boreholes within the Mae Moh Basin. The representative shear strengths shown below (Table 10) were selected from the data obtained in the laboratory testing, field observation, and estimation, for each group of discontinuity (Longworth - CMPS, 1981).

Table 10. Summary of the defect shear strength of Red Beds (after Longworth-CMPS, 1981).

Defect types	C_r (Kg/cm^2)	ϕ_r (degree)
Rough joint	0	26
Smooth joint	0	21
Bedding plane and fault	0	17

The results in Table 10 illustrate a non-cohesive types of discontinuity of the Red Beds though the internal friction angle increases with the degree of roughness.

4.4.4 Swelling strain

The swelling property of the geologic materials, especially the clayey ones, is caused the absorption of water in between sheets of clay minerals, thus the total mass expands. This property is relevant to the strength reduction as well as water absorption.

The summary of the three dimensional unconfined (free) swelling test results is quoted from Duangduen (1978) and is shown in Table 11.

From the results, it can be concluded that the unconfined swelling strain in direction normal to bedding plane is greater than that in direction parallel to bedding plane. The swelling strain of claystone which started as a dry specimen is larger than that when started with the natural moisture content.

In general, the unconfined swelling strain value is very low. This corresponds the microscopic study done by the present worker who recorded a high percentage kaolinite and illite in claystone. Both minerals are regularly low-expansive.

Table 11. Summary of the three dimensions of unconfined (free) swelling test results of Overburden Claystone. (after Duangduen, 1978)

Speci-men No.	Undis-turbed or Remold-ed	Above or Below Lignite	State of Test	W.C. (%) before Test	W.C. (%) after Test	ε_v $\perp$ Bedding %	ε_{1-1} // Bedding %	ε_{1-2} // Bedding %	ε_{1-ave} %	$\frac{\varepsilon_v}{\bar{\varepsilon}_1}$
1	Un.	Above	Nat.	18.67	23.91	1.56	0.29	0.27	0.28	5.57
2	Un.	Above	Nat.	18.77	22.64	1.34	0.39	0.38	0.39	3.44
3	Un.	Above	Nat.	19.30	24.00	1.75	0.29	0.34	0.32	5.47
Average				18.91	23.52	1.55	0.33	0.33	0.33	4.70
1	Un.	Above	Dry	0.68	26.74	2.42	0.55	0.52	0.54	4.48
2	Un.	Above	Dry	1.10	27.79	1.76	0.72	0.46	0.59	2.98
3	Un.	Above	Dry	1.20	27.49	2.31	0.50	0.50	0.50	4.62
Average				0.99	27.34	2.16	0.59	0.49	0.54	4.00