CHAPTER VIII

LABORATORY TESTS

The susceptibility of materials to failure is in part related to their physical and engineering properties. The relevant properties include texture, structure, coherence, grain size, relative density, permeability, strength etc.

Laboratory tests that include the classification tests and strength tests of soils were performed on filled materials both in 1983 and 1985-1986 failure, and natural soils of colluvium and sheared materials. The test are summarized as follows:

8.1 Tests on Engineering Properties of Soils.

8.1.1 Water content determination.

Natural water content of each soil sample was determined. The moist soil samples were weighed with the containers. After putting in an oven for 24 hours, the soil samples and containers were weighed again for the amount of water loss.

8.1.1.1 Variation of natural water content with depth.: The natural water content was determined for the soil samples collected by the split spoon sampler. The results shown in bore hole loggings in Appendix A. reveal that natural water content increases with depth. The average natural and saturated water contents of soils are also shown in Table 8.1.

Table 8.1 Results of the natural and saturated water content of soils.

Average Water content%	1983 filled material	1985 - 1986 filled material	colluvium
natural	18.3	17.85	18.25
saturated	20.6	20	-

Table 8.1 shows that the natural water content in filled materials at failure in 1983 and 1985-1986 was almost in a saturated condition.

8.1.2 Specific gravity determination.

The specific gravity of soil solids, G_s, is defined as the ratio between the weight of the solid matter of a soil sample to the weight of an equal volume of water. The test procedure used in this experiment follows the ASTM Standard Methods, Designation 854-58 (ASTM, 1981). The results of test are presented in Table 8.2.

Table 8.2 Results of the specific gravity test of soils

Materials	Range of results (G_s)	Mean
1983 filled material	2.73 - 2.74	2.74
1985-1986 filled - material	2.63 - 2.69	2.66
Colluvium	2.60 - 2.79	2.70
Shear zone soil	2.65 - 2.76	2.71

The non-uniformity in composition of the colluvium, is responsible for a rather wide range of the values of specific gravity as shown in Table 8.2

8.1.3 Atterberg limits.

The air-dried samples that passed through a U.S. Standard sieve No.40 is used for determining the Atterberg limits. The Atterberg limits tests were carried out following the procedures given by Lambe (1951), ASTM Standard Methods, Designation 423-66 and 424-59 (ASTM, 1981). The test results are presented in Table 8.3.

Table 8.3 Results of Atterberg limits of soils.

Materials	Liquid Limit		Plastic Limit		Plastic Index		Activity	
	range of result	mean	range of result	mean	range of result	mean	range of result	mean
1983 filled material	23.2-33.25	27.78	16.6-22.07	18.45	5.1-15.2	9.33	0.24-0.72	0.44
1985-1986 filled material	-	-	-	-	NP	NP	-	-
Colluvium	25.8-30.8	27.75	17.8-23.2	20.43	5.4-8.3	7.32	0.20-0.30	0.27
Shear zone soil	39.0-47.2	42.62	27.1-31.15	28.24	11.8-16.8	14.38	0.28-0.40	0.35

Note NP = Non - plastic

8.1.3.1 Swelling characteristics.: The soil activity is used as an index property to determine the swelling potential of expansive clays (Das, 1985). Skempton (1953) defined a parameter A called activity as:

$$A = \frac{PI}{C}$$

where C = percent of clay-size fraction, by weight

PI = plasticity index

Seed et al. (1962) conducted several tests on laboratory compacted sand-clay mineral mixture to determine their swell potential. Base on these tests, they arrive to the following relationship

$$S = (3.6 \times 10^{-5})\ A^{2.44}\ C^{3.44}$$

where S = swell potential (percent of axial swell under 1 - lb./ft.^{2} pressure)

Table 8.4 Potential expansiveness of soils (From Akinlabiola, 1982; Seed et al., 1962).

a) Plasticity Index	Swelling Potential
0 - 15	Low
15 - 25	Medium
25 - 35	High
> 35	Very high

b) Swell Potential	Degree of expansion
0 - 1.5	Low
1.5 - 5	Medium
5 - 25	High
> 25	Very high

Table 8.4 gives the criteria for determining the potential expansiveness of soils. From the data presented in Table 8.3 and swell potential, it is clear that the soils in the present study area have low swelling potential.

8.1.4 Grain size distribution of soils.

The test procedure is according to Lambe's (1951). To do the study, the oven-dried soil samples were first crushed with rubber hammer in order to render the natural grain size. Partition of about 500 grams of already grind samples was performed by quartering and sieving through a series of U.S. Standard Sieves No. 3/4", 3/8", 4, 10, 20, 40, 60, 100 and 200. The fractions retained on different sieved were weigh ed. If a fraction of more than ten percent by weight of soils was left on a pan, hydrometer analysis was needed. Cumulative grain size distribution curves were prepared from data obtained from sieve and hydrometer analysis.

From sieve analysis, more than 10% of soils retained in the pan with less than 0.074 mm. opening thus, hydrometer analysis were performed. The test results of soils are as follow.

8.1.4.1 Filled material of failure in 1983.: Grain size analysis showed that this filled material contains 10 to 25% gravel, about 25 to 40% fine sand, while the percentage of fines varying from 32 to 45, and clay fraction ranges from 17 to 25%. The grain-size distribution curve and its ranges are shown in Figure 8.1.

8.1.4.2 Filled material of failure in 1985-1986.: This filled material comprises mainly of fine sand ranging from 30 to 54% with a mean of 47%, gravel ranging from 2 to 7% with mean of 4%, 5% of medium to coarse sand, silt ranging from 20 to 37% with

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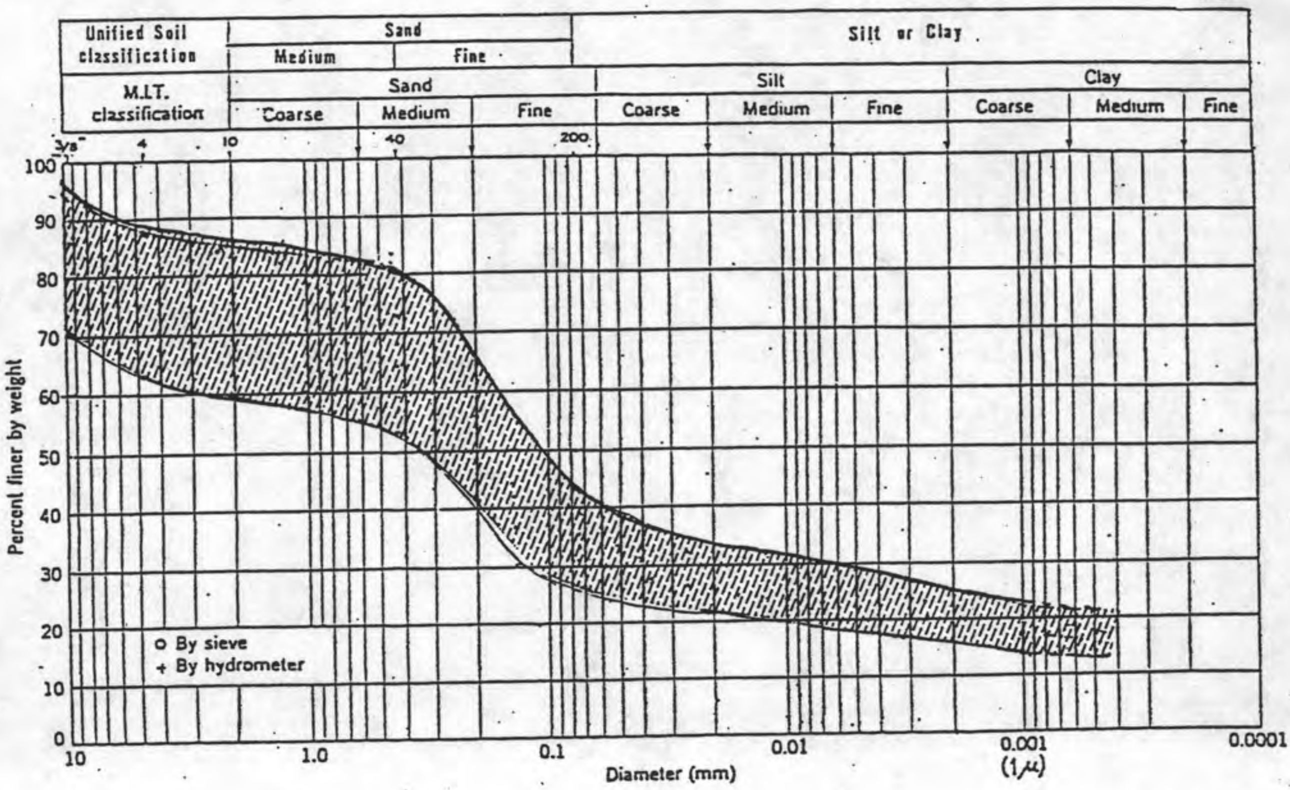

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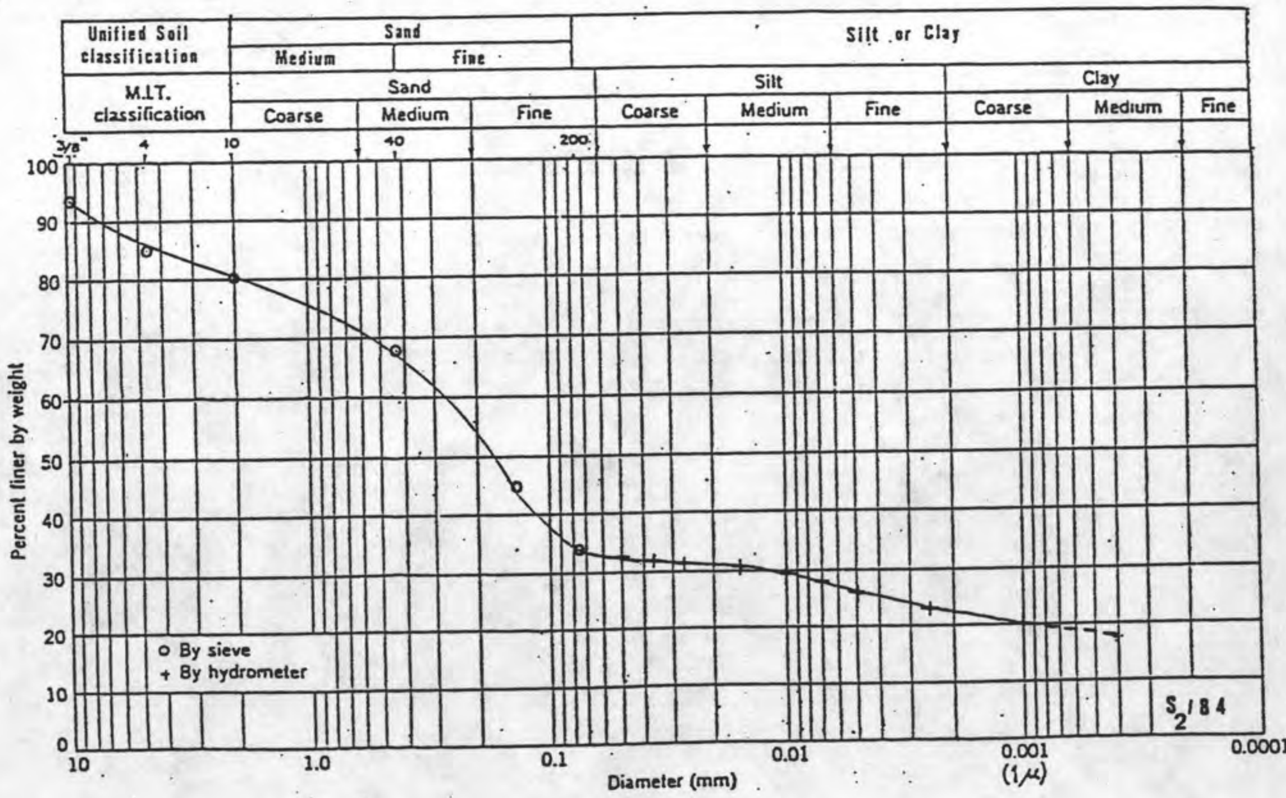
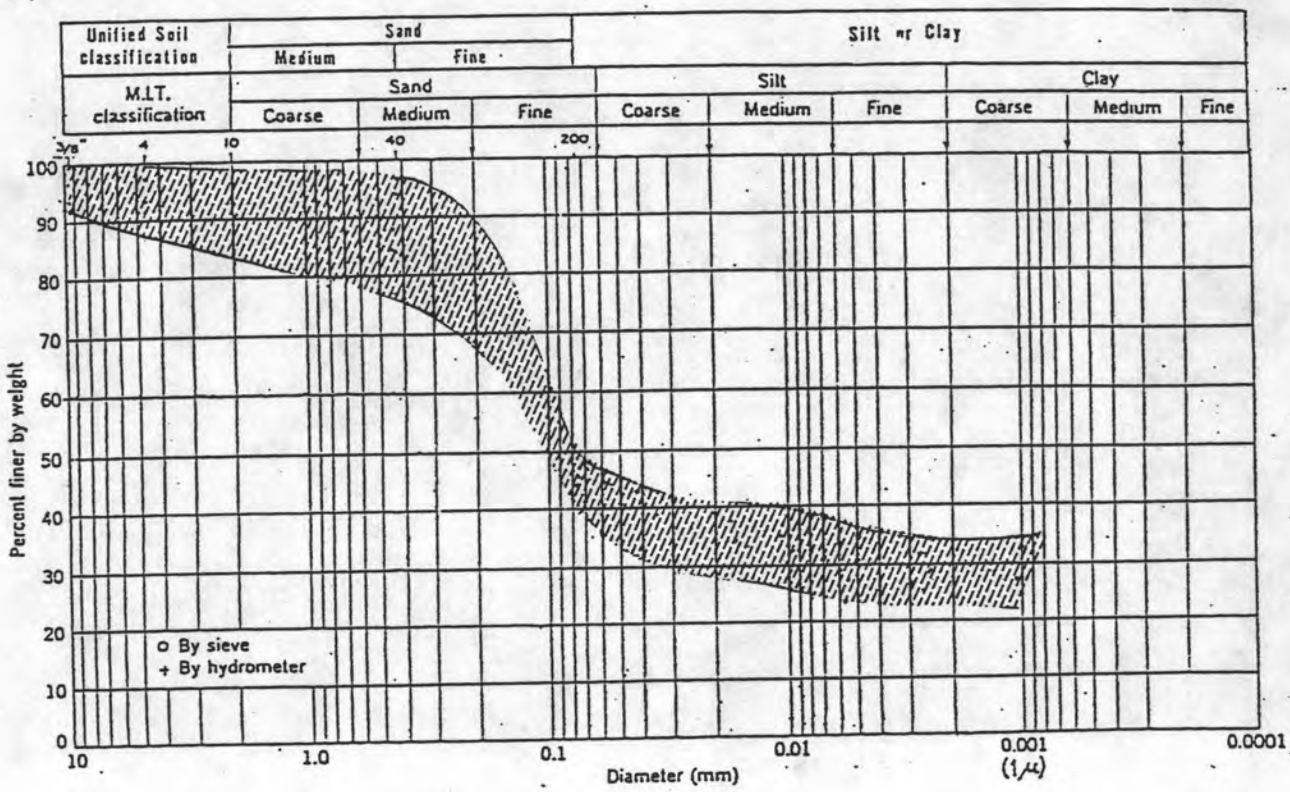


Figure 8.1 The grain - size distribution test results of 1983 filled material.

a) The grain - size distribution ranges.

b) The representative grain - size distribution curve.

a)



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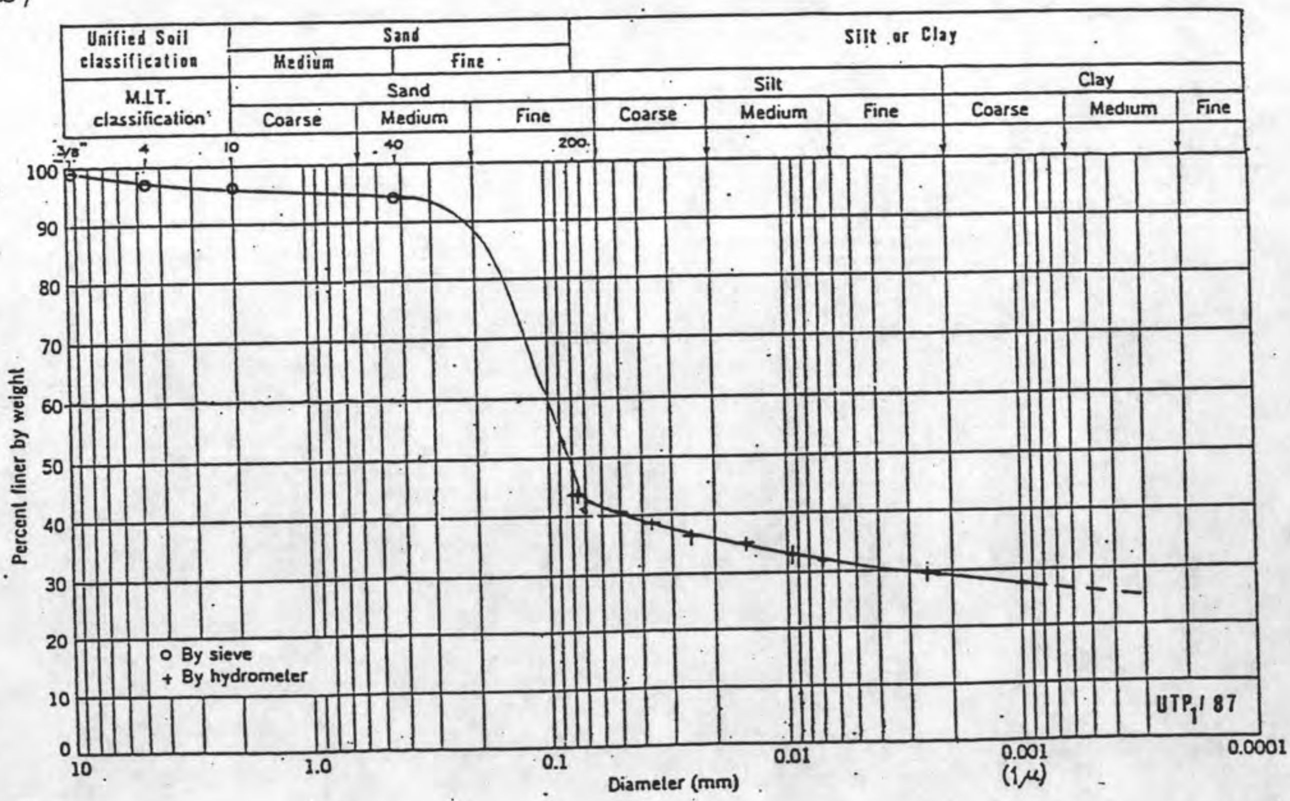

Figure 8.2 The grain - size distribution test results of 1985 - 1986 filled material.

a) The grain - size distribution ranges.

b) The representative grain - size distribution curve.

a mean of 26%, and clay ranging from 8 to 26% with a mean of 18%. This soil is poorly graded. The grain-size distribution curve and its ranges are shown in Figure 8.2.

8.1.4.3 Colluvial soil.: Grain-size analysis on the colluvial soil showed that this soil is poorly graded and contained particles of gravel ranging from 4 to 25% with mean of 11%, sand ranging from 35 to 47% with a mean of 41%, silt ranging from 10 to 29% with a mean of 20%, and clay ranging from 22 to 40% with a mean of 28%. The grained size distribution curve and its ranges are shown in Figure 8.3.

8.1.4.4 Shear Zone Soil.: This soil contains a high percentage of fines which are composed of about 31 to 51% clay and 31 to 44% silt, and the rest is sand and gravel. The grain-size distribution curve and its ranges are shown in Figure 8.4.

8.1.5 California Bearing Ratio test of filled material.

The California Baring Ratio (CBR) tests were performed on filled material of failure in 1985-1986 to obtain an indication of the strength of this subgrade soil. The soaked CBR tests were performed on four remolded samples by following the procedures given by ASTM Standard Methods, Designation 1883-73.

The soaked CBR test results at 95% standard compaction ranging from 5.5 to 8.6% mean of 6.6% rating as poor to fair subgrade (Bowles, 1978.; The Asphalt Institute, 1962).

8.1.6 Standard compaction test.

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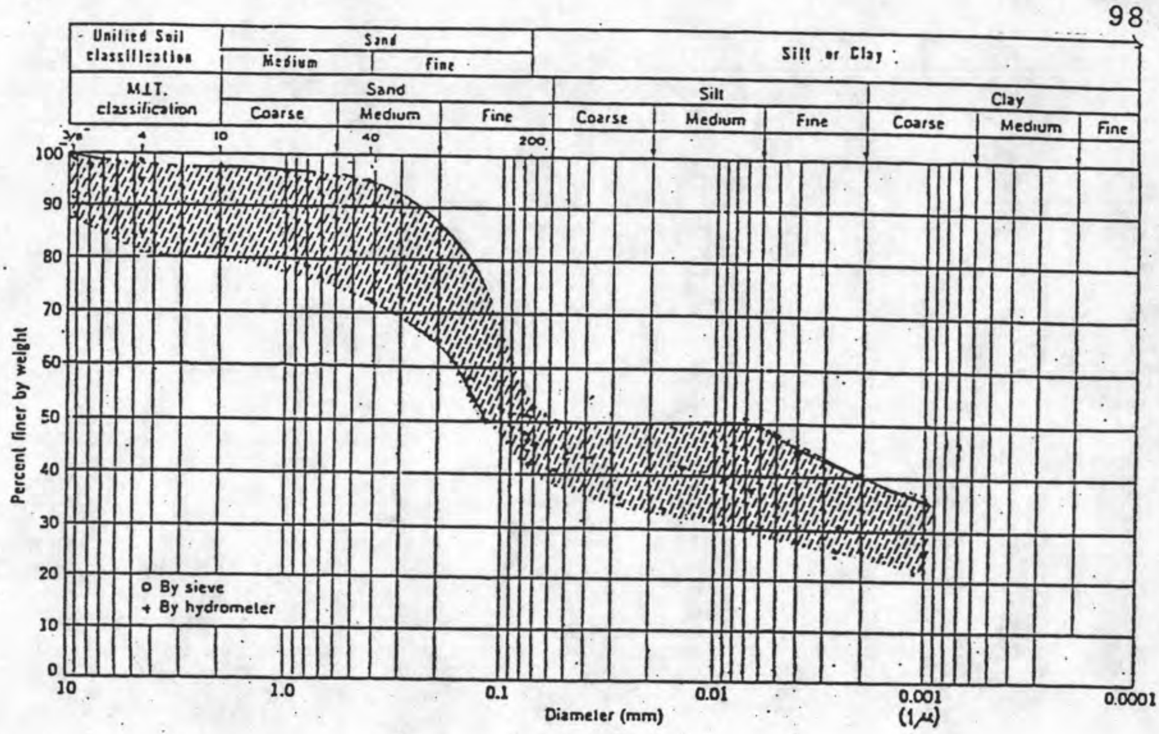

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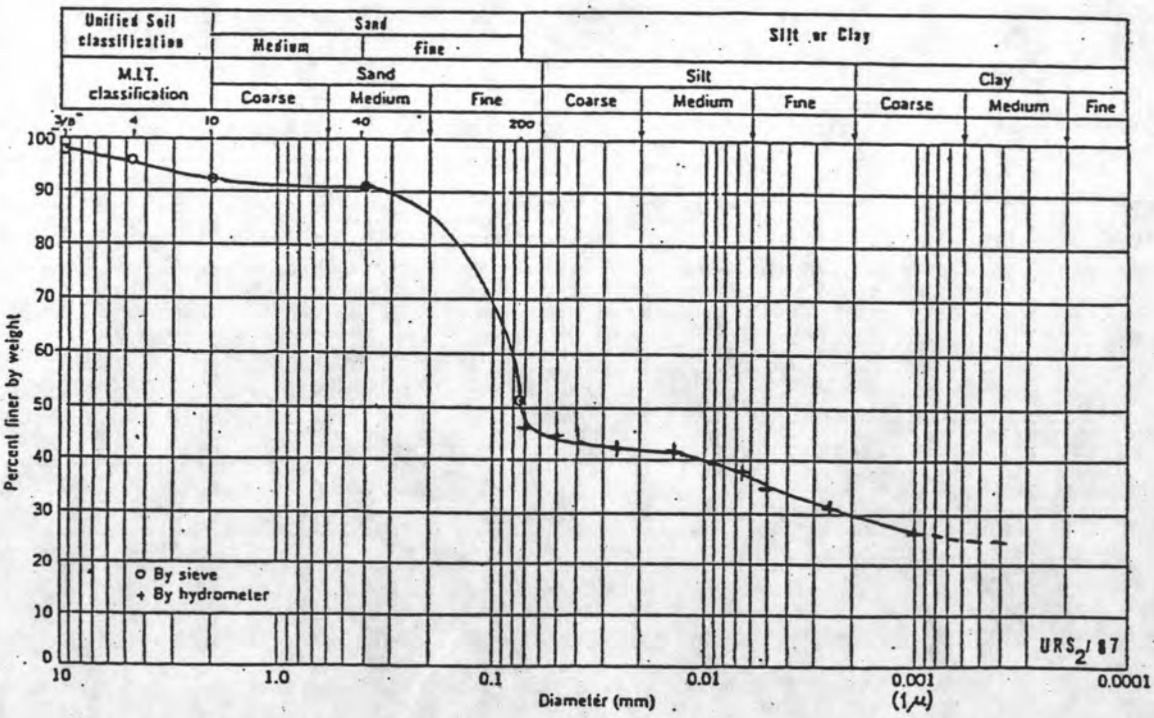
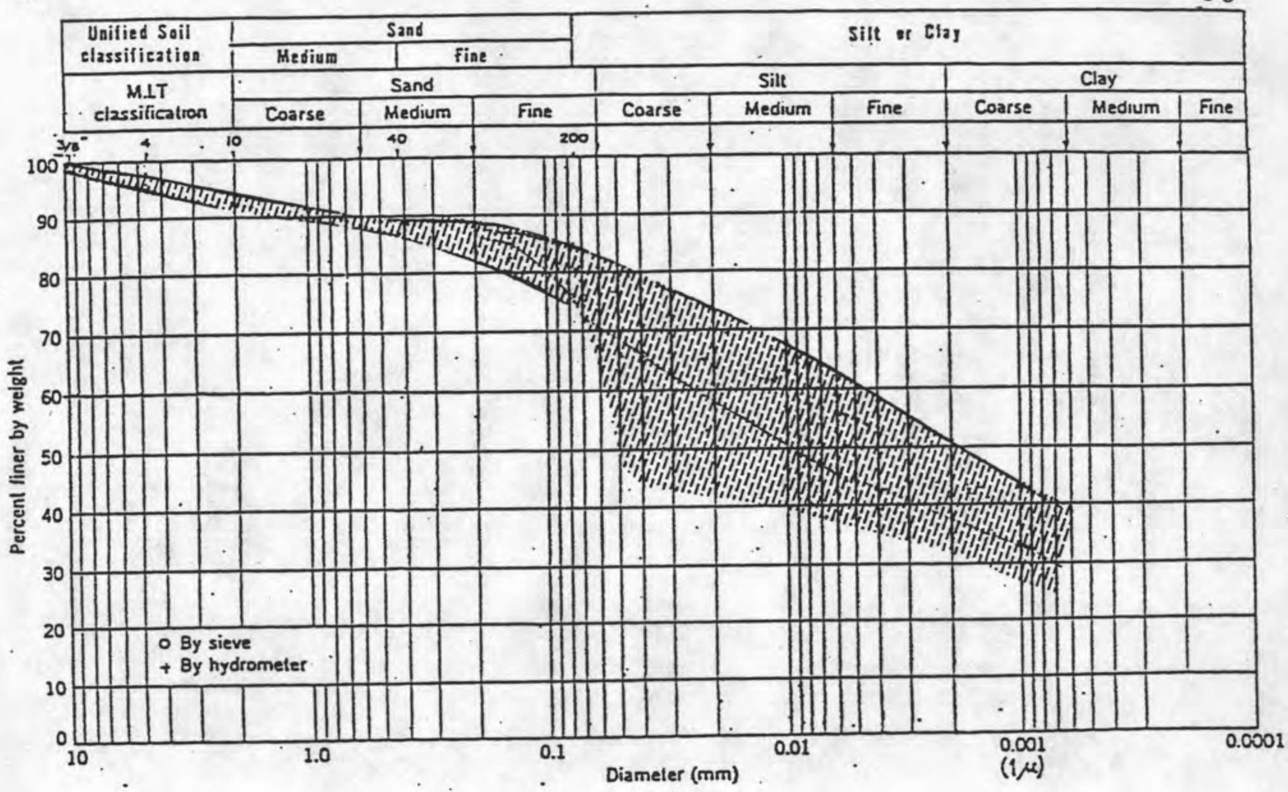


Figure 8.3 The grain - size distribution test results of the colluvial soil

a) The grain - size distribution ranges.

b) The representative grain - size distribution curve.

a)



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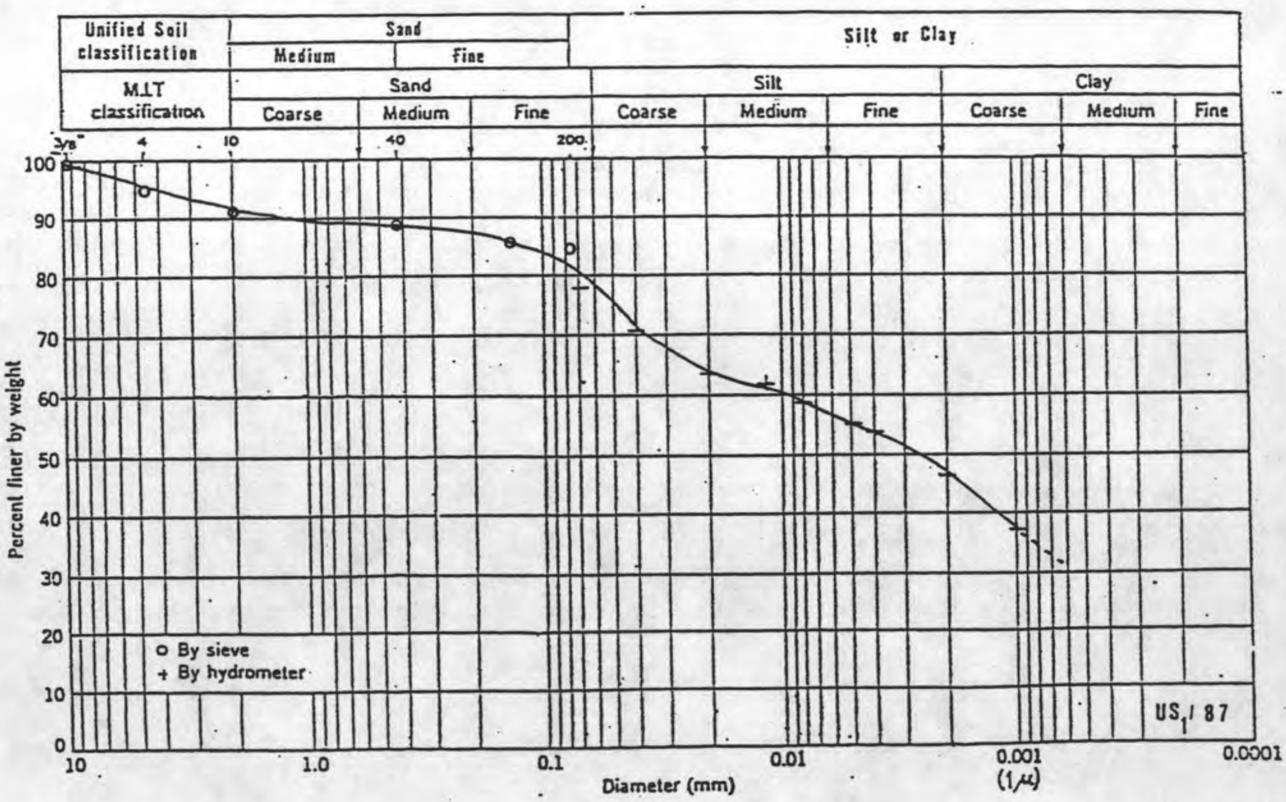


Figure 8.4 The grain - size distribution test results of shear zone soil.

a) The grain - size distribution ranges.

b) The representative grain - size distribution curve.

Standard compaction tests were performed on six remolded samples of filled materials of failures in 1983 and 1985-1986. The test procedure used in this experiment follows ASTM Standard Methods, Designation 698-78 (ASTM, 1981). The representative of compaction curves is presented in Figure 8.5.

The optimum water content and the maximum dry density for filled materials of failures in 1983 and 1985-1986 average 11.30%, 20.10 kN/m^3 and 9.28%, 19.41 kN/m^3, respectively.

According to the compaction characteristic of tested soils, 1983 and 1985-1986 filled materials are rating as fair to excellent and poor to good respectively, when regarding to the embankment performance (See Table D-2). However, with regard to values as an embankment material, these filled materials are classified as reasonably stable and reasonably stable when densed respectively (See Table D-3).

8.1.7 Direct shear test

In the study, undisturbed box samples were used to measure the shear strength parameters. Both peak strength and residual strength were determined using a drained direct shear test.

Direct shear test is widely used to determine the strength parameters in slope stability analysis, and is of particular value where "residual" strengths are required, since repeated reversals of the shear box can be employed. Santos (1985) reported that in an investigation of failed slopes, the residual strength of the undisturbed samples obtained by direct shear test has a better

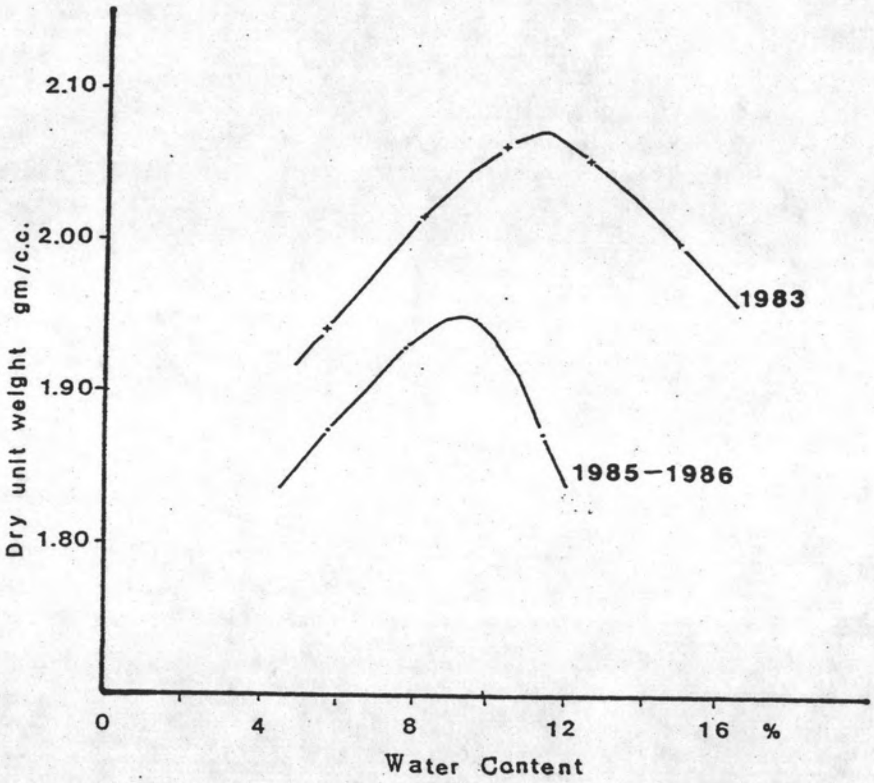


Figure 8.5 Standard compaction curves for filled materials of failure in 1983 and 1985-1986.

correlation with the real strength of the soil than those obtained from triaxial tests.

The shear strength of soils at very large strains is referred to as "residual shear strength or the ultimate shear strength", and it can be given by the equation.

$$S_r = \sigma' \tan \phi'_r$$

where, S_r = residual shear strength

σ' = effective normal stress

ϕ'_r = angle of residual shearing resistance in terms of effective stress

The residual condition termed as "critical state" by Roscoe et al.(1958) represents a deformation under condition of constant volume and constant fabric. Skempton (1964) and Bjerrum (1967) had emphasized the importance of this concept in long term stability analysis of natural slopes and cuts in over-consolidated clays and clay shale. Lupini, Skinner and Vaughan (1981) described the importance of drained residual strength that they play a part in the stability of old landslides, in the assessment of the engineering properties of soil deposits which contain existing shear surfaces, and in the assessment of the risk of progressive failure in stability problems in general. Therefore, it is importance to understand the concepts of residual strength.

8.1.7.1 Measurement of residual strength in laboratory.: Laboratory shear strength tests are frequently stopped once the peak strength has been passed, but it is now accepted for soils with brittle strength characteristics that a progressive failure in the field can lead to average mobilized shear strength much lower than the peak values and the complete stress-strain curve must be taken into account.

Residual strength generally determined from one or more of the following three types of test.

a) Reversing direct shear test.

b) Triaxial compression test.

c) Ring shear test.

a) Reversing direct shear.: Most residual strength has been done in 6 cm. square direct shear boxes, modified to permit reversal of the direction of shear. Skempton (1964) and Patton and Hendron, (1974) described the procedure that after completing the first shear, with the displacement of about 0.5 to 5 cm., the upper half of the shearing box was to be pushed back to its original position and then the process be repeated until the strength of clay has dropped to a steady value.

b) Triaxial Compression test.: Depending on the field evidence of natural water condition, drained or undrained test is conducted on a conventional triaxial apparatus to determine the residual strength. In the drained test, which is conducted in the case of long-term stability analysis, the sample is first consolidated under cell pressure and sheared sufficiently slowly so that the pore pressure developed during shearing is dissipated.

c) Ring shear test.: The ring shear apparatus offers an unambiguous method of measuring residual strength (Lupini et al., 1981). The advantage of using ring shear apparatus, is mainly because it allows large unidirectional displacement to be applied to the sample. A thin annular soil specimen is sheared by clamping it between two metal disc which are then rotated in opposite direction.

The complicacy of ring shear apparatus and time taken to carry out the experiment made the test technique unpopular. Triaxial tests is also unsuitable for obtaining residual values of shearing resistance because the available displacement is so limited and because it is not as convenient to test parallel to the critical surface as in a direct shear test. For most purposes, design values can be obtained from a simple reversed direct shear test device (Patton et al., 1974)

8.1.7.2 <u>Determination of residual and peak strength of soils for stability analysis</u>.: Kenney (1967) conducted a series of laboratorial drained direct-shear tests on natural soils, pure minerals and mineral mixtures. He concluded that $\emptyset'_r$ depends on the soil mineralogy. All clay mineral groups show a significant difference between $\emptyset'$ and $\emptyset'_r$ the largest difference was found in montmorillonite clays which have $\emptyset'_r$ below 10. The strain rate also has little influence on residual strength.

Rocks and soils composed mostly of clay minerals will have $\emptyset'_r$ values ranging from 4 to about 14 degrees. For most natural soils with various mixtures of clay, silt and sand, $\emptyset'_r$ will vary from 12 to 24 degrees. Angles of residual shearing resistance for fault gouge tend to be in an average of about 22 degrees but can range over wide values depending upon the mineralogy and grain size.

High values of $\emptyset'_r$ of 24 and 28 degrees or more are sometimes encountered for fault gouge probably due to the number of angular silt and sand fragments, being presented (Patton et al., 1974).

Kanji(1970) had given a preliminary estimation of the drained residual angle of shearing resistance for natural gouge materials. The relationship are given in Figure 8.6.

In the present study, consolidated drained direct shear box test was carried out on prepared 5 x 5 cm. and 24-hours soaked specimens to determined the peak strength of filled material and colluvial soil, and residual strength of shear zone soil.

The rates of shear were 0.05 in/min and 0.025 in/min for the tests to determine peak and residual strength respectively.

To determine the residual shear strength, the repeat cycle tests were applied. The direction of shear was maintained in the same direction as that found in the field. The specimens were sheared until the shear stress becomes essentially constant or until a shear deformation of 35-40 percent of original length had been reached. After completing the first shear, the upper half of the shearing box was pushed back to its original position and then the shear test repeated until the shear stress dropped to a steady value.

The shear stress versus shear displacement curves, maximum shear stress versus normal stress curves and the test results are presented in Figures 8.7 to 8.10 and summarized in Table 8.5.

A comparison of the angle of shearing resistance resulted from SPT and direct shear test reveals that SPT gives the

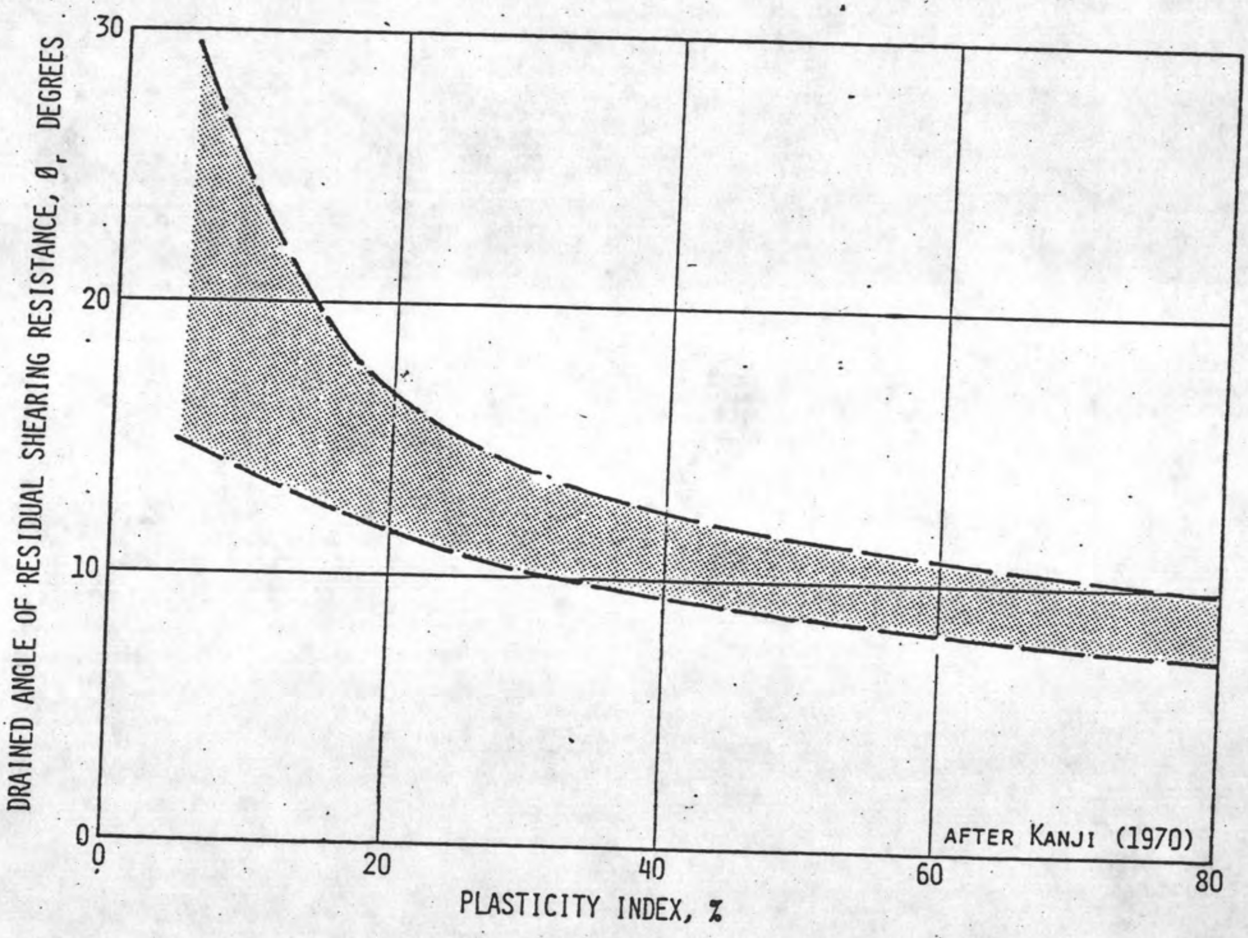
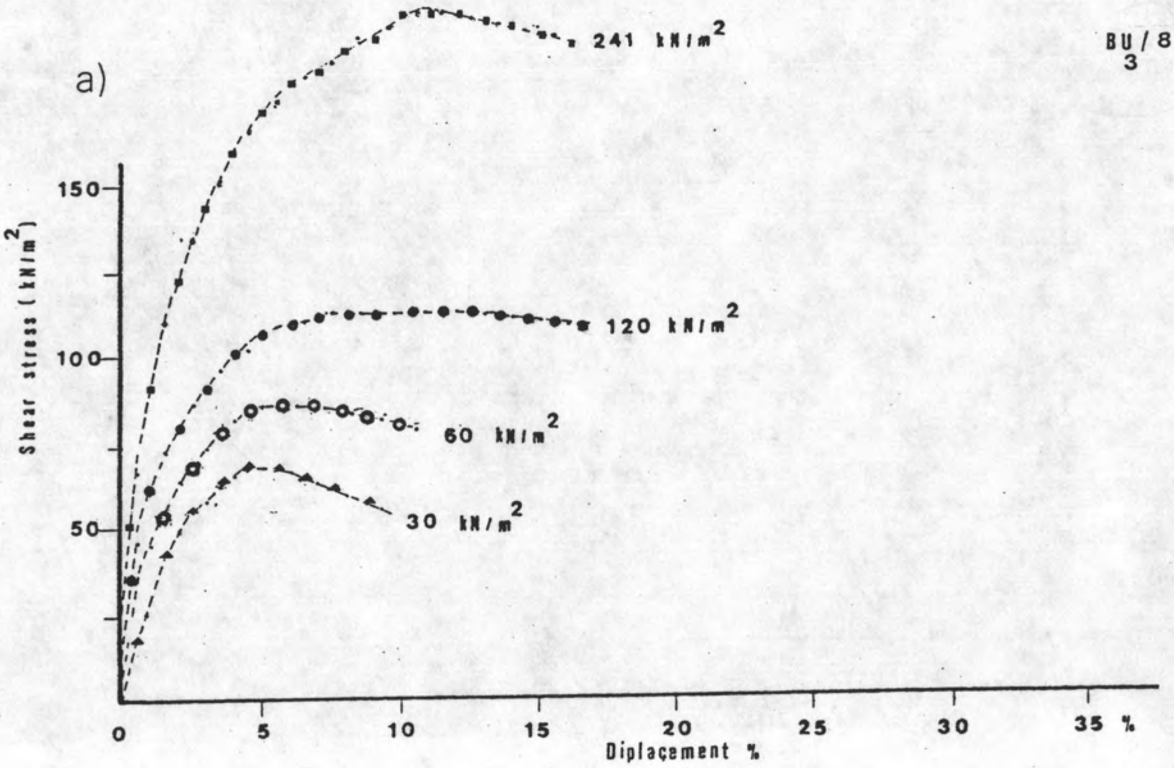


Figure 8.6 Approximate relationship between the drained angle of residual shearing resistance and the plasticity index.

BU/83
3

a)



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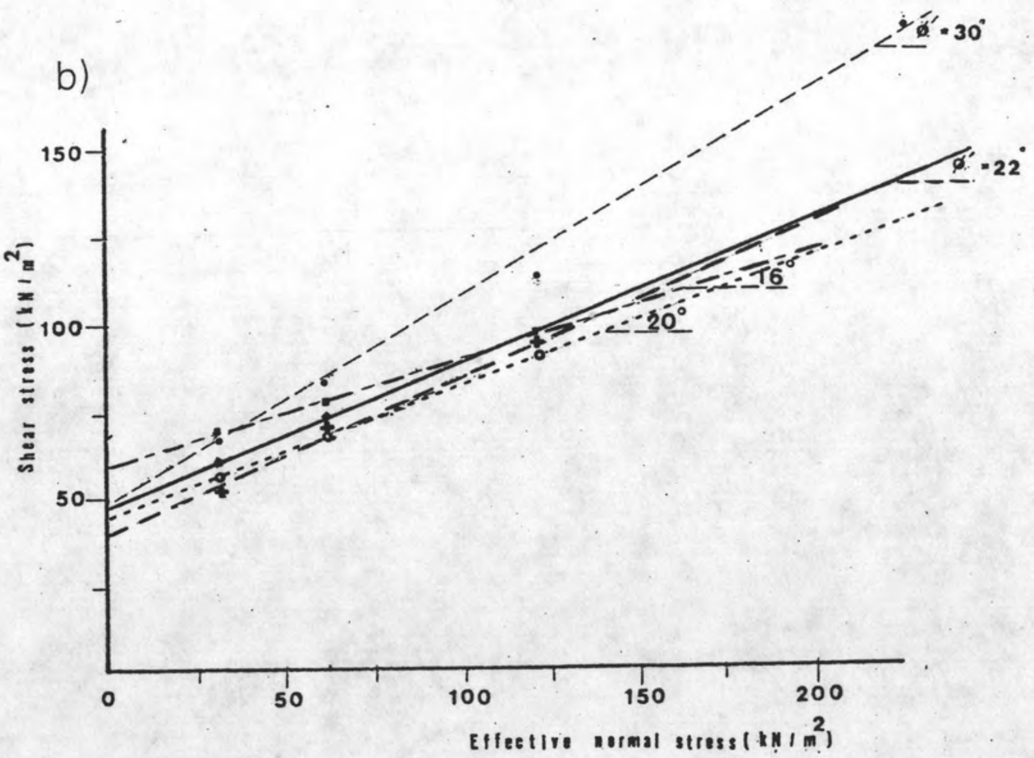
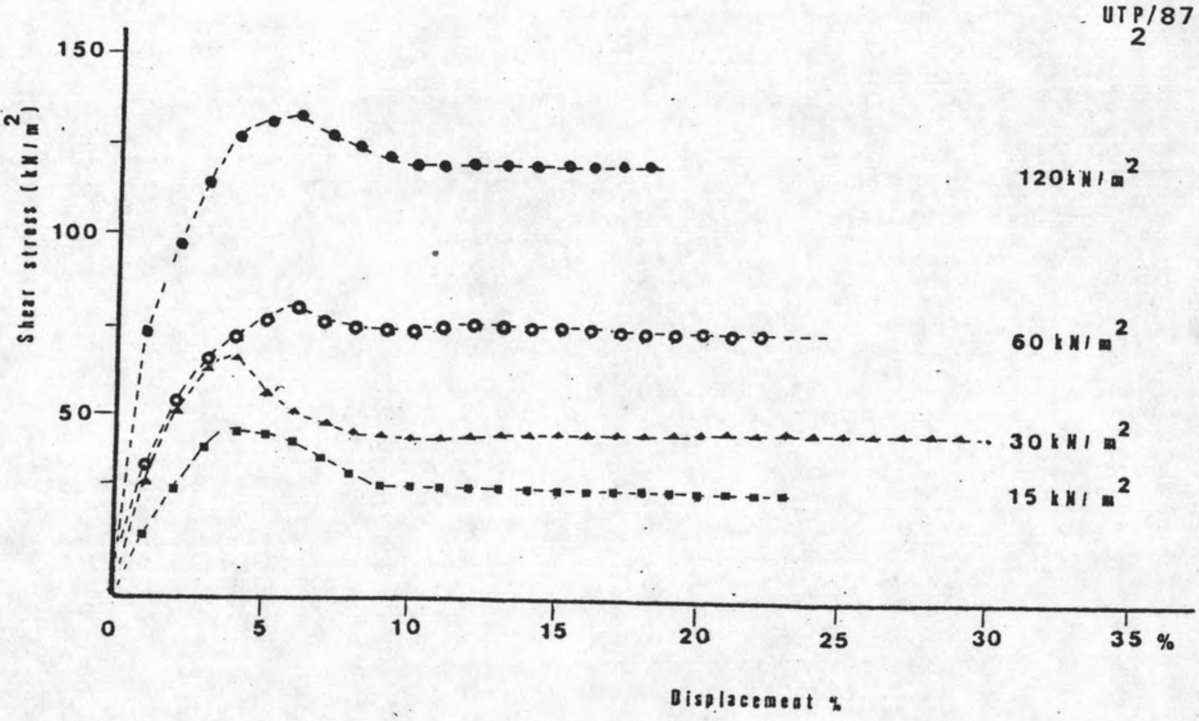


Figure 8.7 Direct shear test on 1983 filled material.

a) The representation of shear stress - displacement curves.

b) Maximum shear stress versus normal stress curves and the test results.

a)



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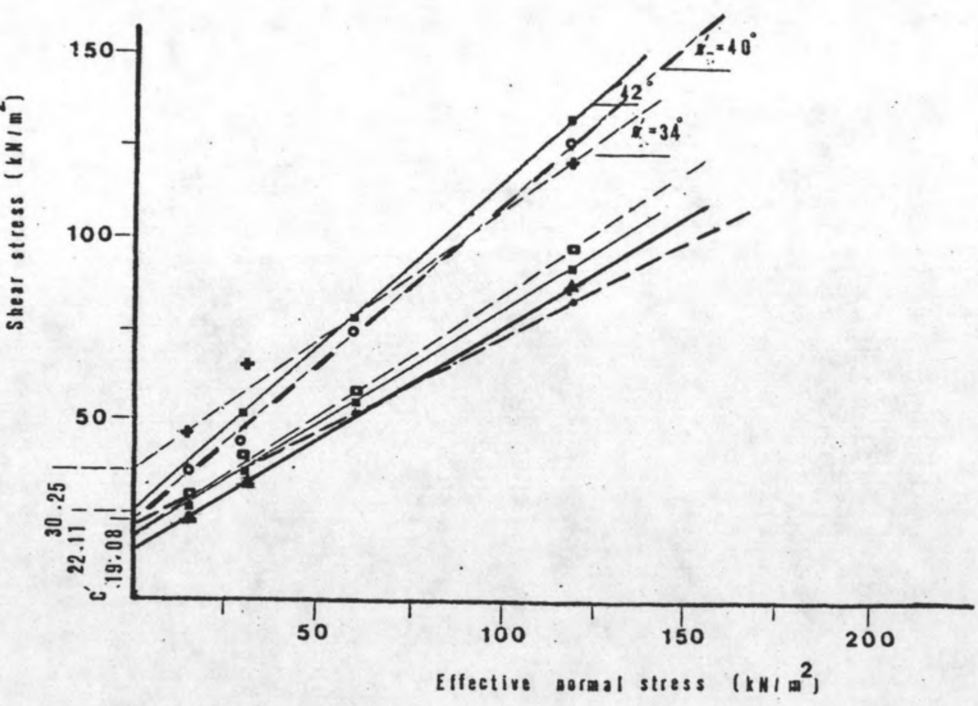
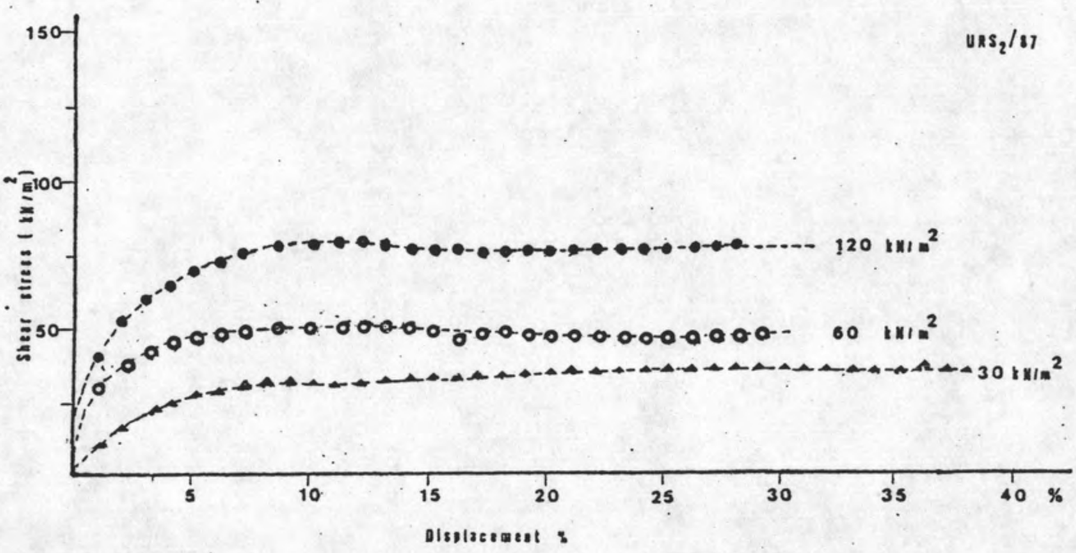


Figure 8.8 Direct shear test on 1985 - 1986 filled material

a) The representation of shear stress - displacement curves.

b) Maximum shear stress versus normal stress curves and the test results.

a)



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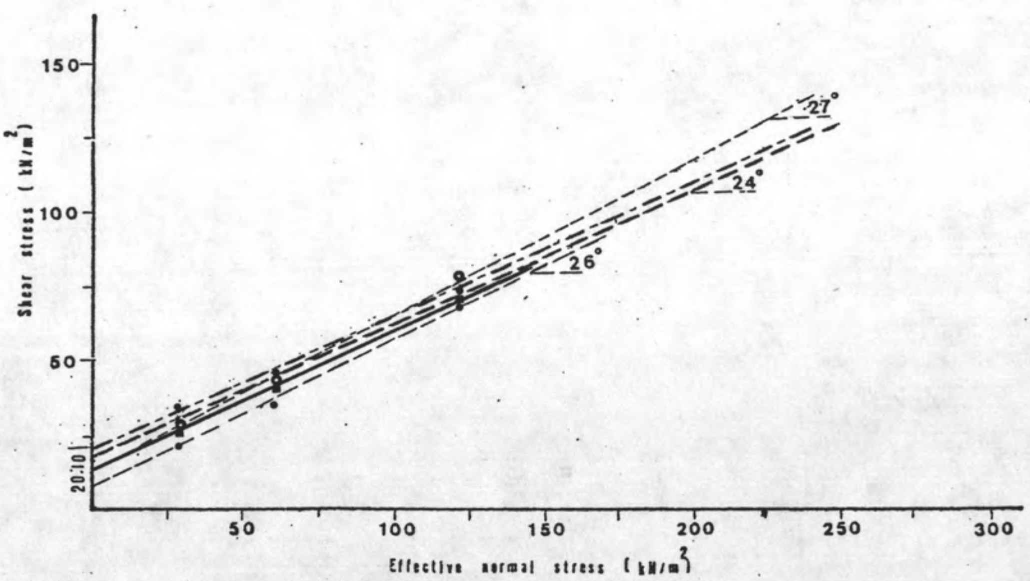
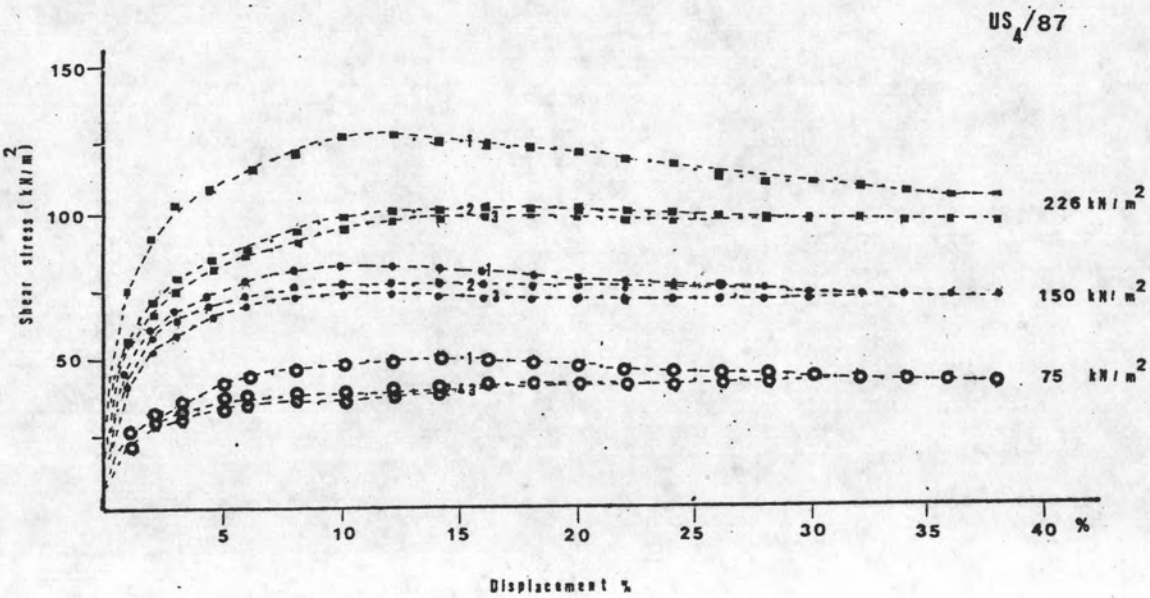


Figure 8.9 Direct shear test on colluvial soil

a) The representation of shear stress - displacement curves.

b) Maximum shear stress versus normal stress curves and the test results.

a)



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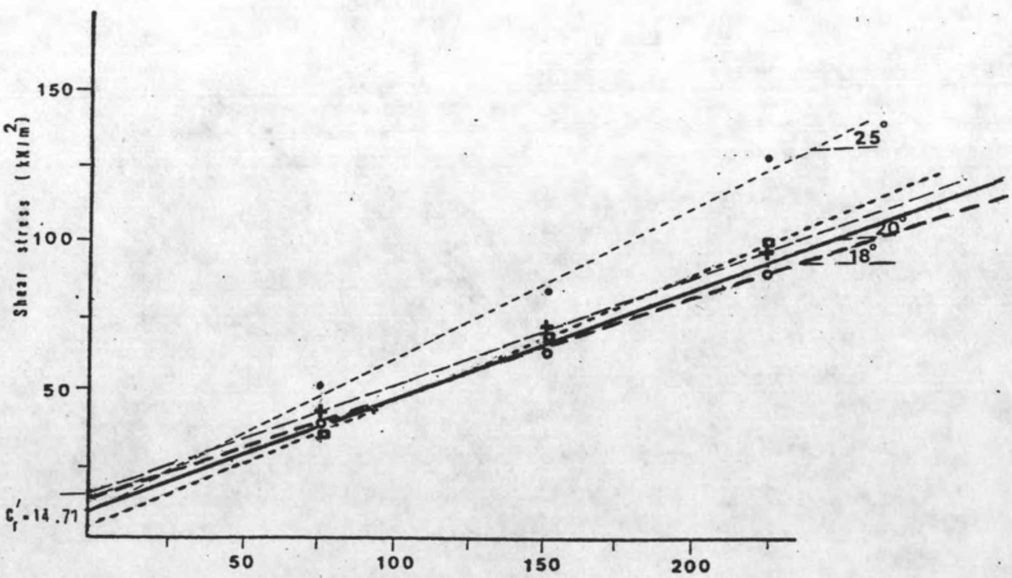


Figure 8.10 Repeat cycle direct shear test on undisturbed sample of shear zone soil.

a) The representation of shear stress - displacement curves.

b) Maximum shear stress versus normal stress curves and the test results.

Table 8.5 Summary of direct shear test results, showing soils strength parameters.

Soil Samples	Peak C' (kN/m^2)	Peak Ø' (degree)	Residual C'_r (kN/m^2)	Residual $Ø'_r$ (degree)	Average C'	Average Ø'
1983 filled material	49.02	30				
	46.08	22				
	40.17	23				
	60.34	16				
	42.18	20			47.56	22
1985-1986 filled material	22.11	42				
	19.08	40				
	30.25	34				
	17.28	31				
	12.31	30				
	20.12	26				
	16.14	32			19.61	33
Colluvium	20.10	24				
	18.07	24				
	12.29	27				
	12.16	25				
	6.23	26				
	7.07	26			12.6	25
Shear zone soil			5.88	25	C'	$Ø'_r$
			2.94	25		
			14.71	19		
			10.78	20		
			11.40	18		
			9.20	20	9.16	21

higher values than those obtained from direct shear test. The different results are probably caused by a high content of rock fragments in these soils which had been encountered in SPT but not in the direct shear test.

According to the test, the present writer hardly found peak strength developing on shear stress-displacement curves of colluvial and shear zone soils. These curves thus show the characteristics of shear stress-displacement curves of loose sand and failed materials.

The average values of angle of residual shearing resistance obtained from this study are in the same manner as the ones recommended by Patton et al. (1974) and Kanji (1970).

8.2 Engineering Classification of Soils.

For an engineering purpose, the soils were classified according to the AASHTO and Unified Soil Classification systems. The plots on plasticity chart of these soils for Unified Soil Classification System are shown in Figure 8.11. The results of soils classification are presented in Table 8.6.

Due to the extremely heterogeneous nature of colluvial soil, this material can be classified into 2 categories which are ML-CL and SC respectively.

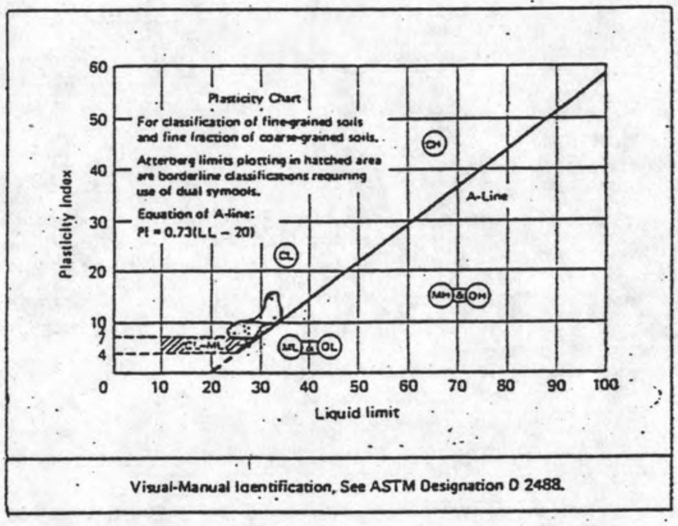
Table 8.6 Results of soils classification

Materials	Unified soil classification		AASHTO soil classification	
	Group classification	Typical name	Group classification	Constituent materials
1983 filled material	SC	Clayey fine sand	A-4, A-2-4	silty soils, clayey sand
1985-1986 filled	SM	silty sand	A - 4	silty soils
Colluvium	ML - CL, SC	Clayey fine sand	A - 4	silty soils
Shear zone soil	ML	rock flour	A - 7 - 5	clayey soils

8.3 Mineralogical Identification of Soils.

The X-ray diffractometry analysis had been employed to identify the minerals in soil from sheared material. From the study, muscovite and kaolinite were recorded without any expansive clay mineral in the sheared zone soil. The X-ray diffractogram is shown in Figure 8.12. The test results of soils done in this study are summarized and presented in Table 8.7.

a)



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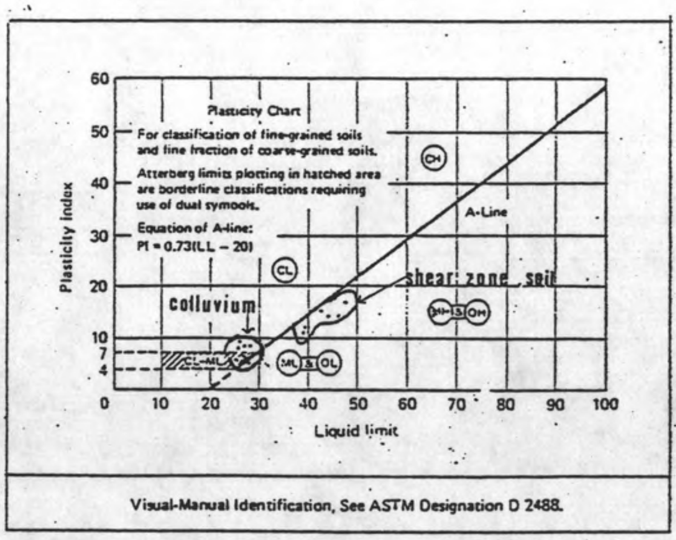

Figure 8.11 The plots of liquid limits versus plastic limit

a) 1983 filled material

b) Colluvial and shear zone soils.

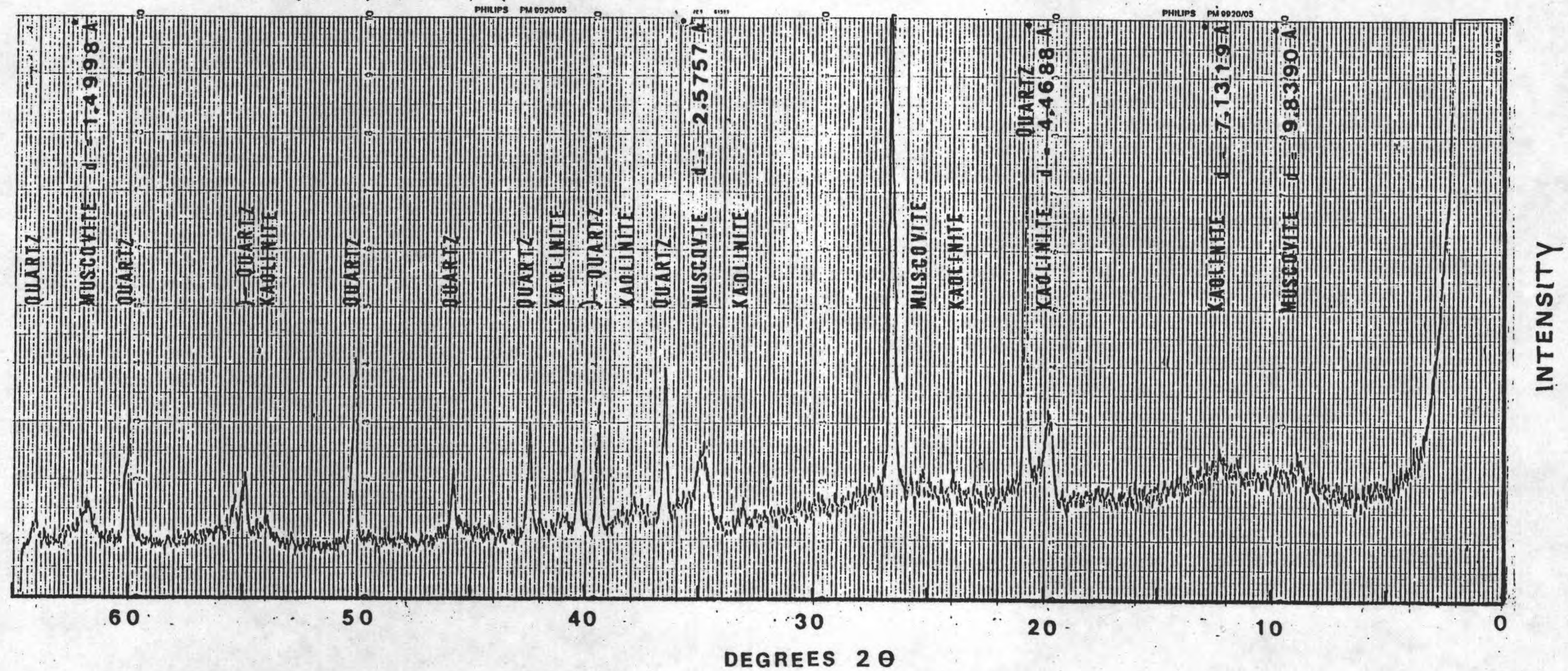

Figure 8.12 X-ray diffractogram of the shear zone sample Cu K_{α} radiation (λ=1.5405 Å, 30KV) was employed.

Table 8.7 Summary of the test results of soils

Tests	1983 filled material	1985 filled material	Colluvium	Shear zone soil
Unified Soil Classification	SC	SM	ML-CL, SC	ML
AASHTO Soil Classification	A-4, A-2-4	A-4	A-4	A-7-5
Saturated water content%	20.16	20.0	-	-
Natural water content%	18.3	17.85	18.25	-
Specific gravity	2.74	2.66	2.70	2.71
Bulk unit weight (kN/m^3)	19.97	20.02	21.34	19.32
Percentage passing No.200	35.55	45.12	47.64	76.78
Liquid Limit	27.78	-	27.75	46.62
Plastic Index	9.33	N.P.	7.32	14.38
CBR	-	6.6	-	-
Maximum dry density (kN/m^3) (Standard Proctor)	20.10	19.41	-	-
Optimum moisture content% (Standard Proctor)	11.30	9.28	-	-
Effective stress Parameters				
C' (kN/m^2)	47.56	19.61	12.60	-
ϕ' (degree)	22	33	25	-
C_r' (kN/m^2)	-	-	-	9.16
ϕ'_r (degree)	-	-	-	21