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AGENCY FOR INTERNATIONAL DEVELOPMENT
WASHINGTON, D.C. 20523

DATE: 7/29/88

MEMORANDUM

TO: AID/PPC/CDIE/DI, room 209 SA-18

FROM: AID/SCI, Victoria Ose

SUBJECT: Transmittal of AID/SCI Progress Report(s)

Attached for permanent retention/proper disposition is the following:

AID/SCI Progress Report No. 7.095
1st PR ending 6/30/88

Attachment

7.095

Asian Institute of Technology
Bangkok Thailand

WELDED-WIRE REINFORCED EARTH (MECHANICALLY STABILIZED) EMBANKMENTS WITH COHESIVE BACKFILL ON SOFT CLAY

First progress report for the
period ending June 30, 1988

Submitted by

Dr. Dennes T. Bergado, Principal Investigator
Mr. Casan L. Sampaco, Research Associate
Mr. Beni Lekhak, Research Associate

Division of Geotechnical and Transportation Engineering
Asian Institute of Technology
Bangkok, Thailand

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6 July 1988

WELDED-WIRE REINFORCED EARTH (MECHANICALLY STABILIZED) EMBANKMENTS WITH COHESIVE BACKFILL ON SOFT CLAY

SUMMARY

Existing data on the proposed backfill materials collected through extensive studies of past researches and project reports are included in this report. These backfill materials consist of weathered clay, lateritic residual soils, and clayey sand. Informations on the commercially available reinforcing materials, were likewise collected. These materials are the Tensar geogrids, welded wires, steel bars, and bamboo grids. Relevant engineering properties and characteristics of the above materials are presented. The site for test embankments was investigated for the subsoil properties based on in-situ and laboratory tests conducted in the past researches. The available subsoil data for the site reported by various investigators are also included in this report. Meanwhile, the laboratory and field testing spaces have been acquired and prepared. The containers for backfill materials have been made and filled with collected samples. The research personnel has been organized and presented in this report.

I GENERAL

For the period from September, 1987 to June, 1988 the following activities were undertaken:

(i) Hiring of personnel and designation of work.

(ii) Procurement of laboratory space and construction of pullout testing room.

(iii) Procurement and clearing of the field experimental land for test embankments.

(iv) Collection of existing data on the proposed backfill and reinforcing materials, and the subsoil properties of the test embankment site.

(v) Sampling, collection and storage of backfill materials for laboratory testing.

(vi) Planning and design of pullout test equipments.

II LABORATORY AND FIELD SITES

The laboratory site acquired for this project is shown in Figs. 1 and 2. The pullout testing machine will be constructed in this room. This room also serve as offices of the research personnel. The container for testing materials for backfill is

shown in Fig. 3. The site for field experiments are shown in Figs. 4 and 5. The full scale test embankment and full scale test fill will be constructed at this site. This site is located inside the AIT campus as shown in Fig. 56.

III BACKFILL MATERIAL SAMPLES

The representative samples collected for backfill materials for reinforced earth construction and for pullout test are shown in Figs. 6 and 7. These materials are now placed inside the container shown in Fig. 3.

IV RESEARCH PERSONNEL

The research personnel of the USAID Reinforced Earth Project comprises of 2 Research Associates, 1 Doctoral and 2 Masteral Students. The names are listed as follows:

1. Research Associates: Mr. Beni Lekhak
   Mr. Casan L. Sampaco

2. Doctoral Student : Mr. Shivasanker

3. Masteral Students : Mr. Ul Amin
   Ms. Chilvier Cisneros

The photos of the personnel involved in the project will be included in the next report.

V PLANS FOR THE NEXT REPORTING PERIOD

The laboratory tests for backfill materials are presently conducted at the Soil Engineering Laboratory of AIT. These tests aim to investigate the engineering properties of the backfill materials when used in the construction of the reinforced earth test embankments. The data obtained from these investigations will be reported next time. The backfill materials presently under test are: weathered Bangkok clay from AIT campus, reddish-brown clayey and gravelly lateritic soils from Saraburi, and light-brown clayey sand from Ayutthaya. Also, the longitudinal tensile strength tests of the reinforcing bars, will soon be conducted at the Structural Engineering Laboratory of AIT to verify the existing data collected. The test data will be included in the next report.

Structural design of the pullout testing cell is presently worked out. It is planned to complete the design and construction of this cell by the month of August, shortly after the arrival of the pullout testing equipments from the United States. The pullout testing apparatus and equipments, will be purchased by Dr. Bergado during his coming visit to the United States starting July 10, 1988. The two-week visit will be spent to do further research on the pullout testing procedures, and the current techniques on the processing and analysis of results will

likewise be studied. The visit will also be spent on actual observations of the various construction methods of reinforced earth embankments on difficult soil conditions in the United States. Meanwhile, laboratory test set-up and techniques for pullout testing are currently being reviewed from existing literatures. Laboratory pullout testing programs are also being planned and studied. These testing programs will be immediately implemented right after the completion of the pullout testing apparatus which is scheduled around September, 1988.

VI RESIDUAL AND LATERITIC SOILS IN THAILAND

Introduction

Residual and lateritic soils are abundant in Thailand where they are popularly used in road construction as well as building materials. These terminologies often indiscriminately applied to many red tropical and subtropical soils (LITTLE, 1969) including residual soils. Extensive studies and field observations by several researchers, however, reported that Thailand laterites are only iron-enriched and are not true laterites, and thus, considered to be lateritic soils in their initial stages of laterization.

It was believed that the average residual and lateritic soils present no problem in their evaluation using standard test procedures and classification criteria. However, field surveys of existing borrow pits in Thailand, revealed that lateritic soils are not only highly variable in-situ, but also highly susceptible to change in engineering properties when removed from the borrow pit and exposed to a new environment. Several observations show that this exposure of lateritic materials can result in either improved durability and strength characteristics or complete disintegration and loss in strength, depending on handling, method of preparation and environmental factors.

This report presents the review of existing data and observations obtained from several researches, project reports and field surveys of the laterite sources in Thailand. It is believed, that recognition of the problems associated with these types of materials, both in the laboratory and in the field, can somehow develop awareness and useful basis of the laboratory testing programs appropriate to the given condition being studied.

Origin and Formation

Residual and lateritic soil formations are found in great abundance in Thailand. In their studies and field survey of over 70 borrow pits in Thailand, VALLERGA & RANANANDANA (1969) concluded, that Thailand laterite is a hardened material formed by the primary weathering of nontransported soils, or by the secondary enrichment and cementation of transported or nontransported soils. In chemical and mineral composition,

laterite is rich in the sesquioxides of aluminum and iron. It is nearly void of bases and primary silicates, but may contain large amounts of quartz and kaolinite. In physical composition, it occurs as a hardened aggregate of consolidated or unconsolidated pisolithic masses, consolidated vesicular masses, or cemented pre-existing materials, such as quartz, in pisolithic or vesicular structures.

Lateritic soils, on the other hand, are those which contain hematite and have a red color, but which do not consist of hard laterite nodules or blocks in-place (SOIL AND PAVEMENT CONSULTANTS, 1968). Researches and field surveys revealed, that lateritic soils, rather than laterites, exist in great abundance in Thailand. Based on their method of formation, these lateritic soils can be divided into two groups, namely: primary and secondary lateritic soils.

1. Primary Lateritic Soils

A primary lateritic soil is an iron rich soil which forms in place over the parent bedrock and which derives its iron content from ferro magnesian minerals in the subjacent bedrock. A well-developed primary lateritic soil profile in Thailand usually contains the following gradational horizons from the surface to the bedrock (HONGSNOI, 1969):

i) surface soil;
ii) hard, dry lateritic nodules of coalescing masses of pea-sized hematite spheres together with a little clay;
iii) clay matrix with a high percentage of small, stiff, incipient laterite nodules of soft limonite (iron hydroxide);
iv) clay matrix with soft, moist, plastic iron oxide inclusions of all sizes;
v) grey clay containing limonite inclusions or stains in the fissures;
vi) decomposed bedrock (gravel, sand, silt and clay);
vii) fresh bedrock.

The Atterberg limits are generally lowest at the laterite horizon and increase with depth until the decomposed bedrock is reached.

2. Secondary Lateritic Soils

Secondary lateritic soils are defined as those which are transported from its bedrock source, then altered by the introduction of iron oxide-cementing agents which are carried into the soil by percolating groundwater. They are usually devoid of horizons and exhibit variable concentrations of red iron oxide, depending on the heterogeneity and on the variations in soil permeability in the particular deposit. The contact between secondary lateritic soils and the bedrock is usually quite

distinct. The Atterberg limits of secondary deposits are usually lower than those of primary deposits.

Physiographic Features

A field study made by soil engineers and engineering geologists in Thailand in 1967 to evaluate sources of lateritic materials, described the details of geologic setting, laterite formation, borrow pit profile and physical properties. Accordingly, as shown in Fig. 8, Thailand is divided into 5 physiographic regions, i.e. The Northwest Highlands, the Chao Phraya (Bangkok) Plain, the Khorat Plateau, the Southeast Coast, and the Peninsular Province.

The climate is an important factor in the laterite and residual soil formation, and Thailand has slightly different climate for each region. For instance, the highlands has a mild subtropical climate with moderate seasonal rainfall; the plateau has a monsoonal climate alternating between flood and wet seasons and parched dry seasons; the Southeast coast and peninsular areas have a tropical rain forest climate with no prolonged dry spells. Thus, the geology of Thailand is characterized by rocks of diverse geologic character covered with soils of variable drainage characteristics in the humid monsoonal environment. This leads to deep soil weathering and rapidly fluctuating ground water movement in most areas, prime factors in laterite formation. The distribution of existing lateritic soil sources throughout the country is given in Fig. 9.

1) The Northwest Highlands

The highlands is a high relief area covered by well-drained soils and underlain by folded complexes of limestone, granite, shales, sandstones, and thin metamorphic equivalents (VALLERGA & RANANANDANA, 1969). Laterites in this region are iron-stained decomposed granites. Some areas have medium-hard, pea-sized, black, hematite nodules mixed into a stiff, moist, clayey sand groundmass. They are permeable and is constantly wet with seepage. These laterite nodules are the materials of interest for road aggregate and quarries can be found in Sukothai and in Lamphoon provinces.

2) The Chao Phraya Plain

The region is covered by a thick sequence of wet, poorly-drained soils and underlain at great depth by shales and sandstones. Laterite materials in this region are of hard, angular nodules of hematite and occur to a depth of about 120 cm. In the groundmass of silty clay, the hard nodules are difficult to break with a geologic pick and had a random and imperfectly formed vesicular fabric (VALLERGA & RANANANDANA, 1969). HONGSNOI (1969) and MAZHAR (1969) reported that in Saraburi, lateritic soils contain silty and sand-sized particles and are iron-cemented with nodules. BOONSRI (1971) studied the shear strength

characteristics of a compacted lateritic soil obtained from a borrow pit, 5 km from Saraburi. From the visual inspection, the soil appeared to be a clayey gravel, largely composed of rock fragments with clay which was rather sticky with soft consistency. The soil was found to exist in a band at a depth of 1.5 - 2.0 meters from the surface and directly overlying the parent rock.

3) The Khorat Plateau

The region is a flat area covered by less than 6 meters of well-drained soils and underlain by sandstone, shale, salt beds and occasional extrusive igneous rocks such as rhyolite. Nodular laterites which are secondary or primary in variety, were found to exist in this region. Most of the lateritic soils in this region, however, are more predominant secondary lateritic soils (VALLERGA & RANANANDANA, 1969). BOONSENER (1977) found gravelly laterite, pisolithic laterite and massive bedded laterite in Khon Kaen. A poorly cemented gravelly laterite consisting of about 45% gravel, 40% sand, and 15% of silt and clay and occurring up to a depth of 3 m can be found in Ban Nonchai. In Khon Kaen University, pisolithic as well as massive bedded laterite were reported. CHAMNONGPIPATKUL (1980) stated that laterites in Yasothorn, Ubon Ratchathani, contain ironstone and iron-stained gravels in an old alluvium and can be related to a sandstone formation. The same types can be found in Roi Et and Kalasin. MOH and MAHMOOD (1969) and MAZHAR (1969) tested the sample obtained from a borrow pit near the town of Khorat. The sample was said to be in the initial stages of laterization, had soft limonitic cores and possessed only a thin skin of hard iron oxide.

4) The Southeast Coast

The laterite formations in this region are closely related to the low, planar landform. They are iron-stone gravels related to basalt parent rock, and plinthite in an old alluvium, which is related to shale as the parent rock. HONGSNOI (1969) and MAZHAR (1969) reported the presence of yellow clayey and red clayey lateritic soils in Chantaburi Province. YONG (1975) investigated the sample obtained from an active borrow pit which is about 8 km west of Chantaburi. The sample was essentially reddish-yellow and was described to have agglomerated at the surface, whereas the soil below was rather sticky and of soft consistency. He attributed the agglomeration to the presence of iron oxides which usually act as cementing agents and tend to agglomerate the clay particles when dry. SUWANASING (1974) found a nickeliferous laterite soil deposit in Prachinburi Province. The upper zone of laterite is dusky red to dark reddish brown and moderate brown with very plastic clay-like material. The middle zone is light brown with scattered limonite concretion in the upper part and earthy magnetite streaks in the lower part. The lowest zone varies from yellow brown to yellowish orange with some bands and seams of dark magnetite. These laterites occur

above the weathered serpentinite and serpentinite fresh rock. Similar features had been observed in the Nong Katum district. BOONTAN (1984) visited 14 locations in this region, including Prachinburi, Chachoeng Sao, Chantaburi and Chonburi provinces. The genesis and the type of laterites are studied from the profiles. He concluded that on the basis of morphological classification, laterites in this region can be divided into 3 main groups, namely: lateritic rocks, lateritic gravels, and gravelly and lateritic soils.

5) The Peninsular Province

The region has similar features with that of the Northwest Highlands, but with rain forest climate. A typical southern pit, and one of the largest deposits of primary laterite in this area is in the town of Trang. This pit has developed over limestone bedrock, the soil is typical of the classic terra rosa deposits that form over limestone, the iron oxides resulting from impurities weathered out of the limestone. In most of this area, the ferruginous enrichment is carried to a high degree by the high rainfall and good permeability of the soil. Where the soils are less permeable, laterization is accordingly less advanced and limonite is more common; but where the imperfectly laterized soils are exposed to air and surface-water flows, laterization is accelerated. HONGSNOI (1969) and MAZHAR (1969) reported the presence of coarse, dark-red lateritic soils in Hua Hin.

Engineering Properties

Considerable studies on the engineering properties of laterite and lateritic soils have been carried out by many authors (WINTERKORN & CHANDRASEKHARAN, 1951; NIXON & SKIPP, 1957; MOH & MAZHAR, 1969; BRAND & HONGSNOI, 1969; FERREIRA & MEIRELES, 1969; BERGADO, 1976; LAM, 1979; CHEN, 1980). These studies reported that residual and lateritic materials may not be reliably classified on the basis of standard particle size distribution and plasticity tests alone. Particle size distribution and plasticity tests do not yield reproducible results as they clearly are influenced by the method of handling and preparation during test. Also, the standard soil classification tests upon some laterite materials have shown them to be sensitive to degrees of drying, time of mixing and type of dispersion agent. The particle size distribution for residual lateritic soils from various parts of the world varies within wide limits. Studies believed, that there are two basic factors which underlie these inconsistencies, i.e. the vulnerability to degradation of some relatively weak nodular materials in coarse grained lateritic soils; and the cementing effect of sesquioxides which bind the natural clay and silt size fractions into coarser fractions.

The effect of the method of preparation on the engineering properties of lateritic soils in Thailand have been investigated by MOH & MAZHAR (1969), BRAND & HONGSNOI (1969) and BERGADO

(1976). There were eleven sources selected as being representative of the lateritic soils in Thailand, the selection of which, was based primarily on different geophysical and environmental conditions rather than on specific properties of the soils. The locations of these samples are shown in Fig. 10. Geophysically, the lateritic soils from Chanthaburi (sources £ 5,6 and 9) are representative of the soils of the Southeast Coast, while those from Hua Hin (sources £ 10 and 11) are representative of the soils of the Peninsular Province. The other soils are representative samples of the Northeast. The samples from eleven sites were prepared under the following three conditions:

(1) Natural Water Content (NMC) - no treatment

(2) Air-dried Samples (AD) - dried under 25°-30°C.

(3) Oven-dried Samples (OD) - dried under 110° 5°C in an electric oven.

BERGADO (1976) studied the effect of the degree of weathering on the mineralogical, chemical and physical properties of granite residual soil formations in Northern Thailand. It was concluded that these properties at any point of the deposit depends on the degree of weathering.

1) Mineralogical Compositions

MAZHAR (1969) reported that the major minerals present in the clay-size fraction (< 0.002 mm) of the soil samples were kaolinite, some illite and montmorillonite or a mixture of these three. Quartz was detected on some samples (sources £ 3,7,8 and 9). None of the eleven samples investigated, contained halloysite or meta-halloysite. Results of the analysis are summarized in Table 1.

2) Chemical Properties

a) pH

pH is a term used to express the hydrogen ion concentration, or more precisely, hydrogen ion activity. A pH value of less than 7 indicates an acidic substance. Table 2 summarizes the pH values of each of the 11 samples from Thailand as determined by MAZHAR (1969). The values indicate that all soils tested were acidic, which is one of the basic conditions of laterization in soils. It was also found out that oven drying of the samples yield lower pH values.

b) Soluble Salt Content

Table 2 also shows the values of the soluble salt content expressed in meq. NaCl/100 gm as determined on the portion passing U.S. standard No. 10 sieve. Different methods of preparation did not show any significant effect on the values obtained.

c) Organic Matter Content

Readily oxidizable organic matter (organic carbon) was determined and then converted to the total organic content. Table 2 summarizes the results of the organic matter tests. The report shows that organic content in all of the soils decreased due to oven-drying.

d) Free Iron Oxide Content

A large portion of the total iron in soils is frequently in the form of oxides which may exist as discrete particles, as coatings on soil minerals, and as cement between mineral particles. Quantitative determination of these free iron oxide content is of interest because of their importance in the laterization process. The free iron oxide content of each of the eleven lateritic soils are tabulated in Table 2.

e) Degree of Laterization

Laterization, as defined by the SOIL & PAVEMENT CONSULTANTS (1968), is the intense concentration of hematite in soils. Cementation in lateritic soils is due to the presence of free iron oxide, which may exist in three different forms, viz. hematite, limonite, and goethite. The process of laterization is dictated by the amount of free iron oxide coating, i.e. hematite, on the soil particles due to the higher cementing effect of hematite than the two other forms. They further concluded, that the laterites of Thailand are only iron-enriched soils and are not true laterites. Their hematite content and mechanical resistance can be further increased if they are excavated and exposed to air and flowing water. Available data on the chemical composition of some of the samples, suggest that the soils are non laterite according to the definition based on the silica/alumina ratio (SHUSTER, 1969). However, consideration of some other factors indicate that the eleven samples are lateritic soils in their initial stages of laterization (MAZHAR, 1969).

3) Physical Properties

a) Specific Gravity and Unit Weight

Specific gravity of laterites is generally higher than most of the nonlateritic materials and usually ranges between 2.5 and 3.6 and may vary with the size fraction. It increases with iron content, with age of formation, and with cementation, and may either increase or decrease with depth of the various levels in the deposit. Unit weight of laterites follow the same pattern as the specific gravity (VALLERGA & RANANANDANA, 1969). Ranges of the specific gravity values for the 11 samples tested are summarized in Table 3.

b) Particle Size Distribution

VALLERGA & RANANANDANA (1969) investigated about 57 sources of lateritic soils in Thailand and observed that the particle size distribution of laterites is extremely varied and erratic. Because the distribution covers all size fractions, from clay to large gravels, it is impossible to derive an acceptable generalization. On the other hand, MOH & MAZHAR (1969) reported that only 2 samples indicated a decrease of 4 to 5% in clay content due to oven-drying. Soils from sources 3,4,6,7 and 8 show exceptional cases by not indicating any effect of the pre-test preparation. Table 4 summarizes the results of the grain size analysis, while the gradation curves are plotted as shown in Fig. 11. Similar results were obtained by BERGADO (1976).

c) Atterberg Limits

The Atterberg limits, so commonly used to classify soils for engineering purposes, seem to be of limited assistance in classifying lateritic soils because of the extremely wide variations obtained by many investigators. VALLERGA & RANANANDANA (1969) reported a minimum liquid limit value of 18% and a maximum of 97% for the lateritic soils of Thailand. The variation of plasticity index ranges from no plasticity to a maximum of 51%. Table 5 gives the range of the Atterberg limit values of each of the samples from eleven sources in Thailand. It was reported that the liquid limit of all the soils decreased due to oven-drying. This could best be explained by the decrease in clay content upon drying. On the other hand, no significant effect of the pre-test preparation was observed on the values of the plastic and the shrinkage limits. However, BERGADO (1976) reported that the plastic and shrinkage limits of granite residual soils of Northern Thailand, are significantly affected by the pre-test preparation. Figure 12 shows the plasticity chart of the 11 soils tested. It can be seen that soils 3,7,9,10 and 11 fall at or above the "A" line, indicating the presence of montmorillonite as one of the clay minerals.

d) Engineering Classification

Reports from many researches suggest that the systems of classification available at present for the soils of temperate regions are not suitable for laterite soils formed in tropical regions. However, in the absence of a suitable classification system, these systems are usually employed for lateritic soils. Representative samples obtained from 11 sources in Thailand were classified by MAZHAR (1969) as tabulated in Table 6. As can be seen, some soils such as samples £ 1,2,5 and 6, were classified as MH or OH for the fine fractions using the Unified Soil Classification System. Description of the soil sources given in the study, did not mention that the samples have high organic contents. This demonstrates the limited applicability of the current soil classification systems for the case of lateritic

soils, which, as a result, not very reliable for the prediction of the engineering behaviour of these materials.

e) Compaction Characteristics

Moisture content - dry density curves are reported to be subjected to wide variations resulting from material degradation during compaction. For a given soil and using a given compactive effort, results of maximum dry densities may range from approximately, 1.36 t/m^3 to as high as 2.32 t/m^3. Optimum moisture contents cover a range of about 6 to 20 percent. MOH & MAHMOOD (1969) observed that optimum moisture content of a compacted lateritic soil is sensitive to moisture equilibration before compaction. To partially overcome the effects of degradation during testing, the majority of the investigators agree that new material must be used with every point determination. Table 7 presents the summary of the results obtained by HONGSNOI (1969) for the lateritic soils of Thailand. The test was carried out using the modified Proctor compaction test. Comparison of results for different pre-test conditions shows that drying caused an increase in the maximum dry density and a decrease in the optimum moisture content.

f) Strength Characteristics

Studies on the undisturbed, remoulded and compacted laterite and lateritic soils concluded that the shear strength characteristics of laterite and lateritic soils have been found to depend significantly on the parent rock, the degree of decomposition, laterization and dessication, as well as the pre-test preparation of the samples. The strength had been determined by unconfined compression, California Bearing Ratio (CBR), stabilometer tests and triaxial tests. Investigators reported that strength characteristics of lateritic soils are as varied as any of the other properties described previously. The greatest variation occurs generally in the CBR values that have been found to vary from 1 to 78.

HONGSNOI (1969) observed that the method of sample preparation had a significant effect on the strength and compaction characteristics of the 11 samples studied. The major factor involved was the change in the clay fraction brought about by pre-treatment, but this effect cannot be predicted on the basis of soil type alone, unless laboratory tests are run. MOH & MAHMOOD (1969) investigated the strength characteristics of compacted lateritic soils from Khorat, Northeastern Thailand by means of triaxial and stabilometer tests. Isotropically consolidated-undrained (CIU) tests with pore pressure measurements on 12 samples compacted, either at optimum or on the wet side of optimum, was carried out. The specimen were saturated at back pressures ranging from 550 to 690 kPa. They found out that at constant dry density, samples compacted wet of optimum had lower shear strengths and higher excess pore pressures. Table 8 and Figs. 13 and 14 show typical results obtained.

A study by BOONSRI (1971) on the shear strength of two lateritic soils from Thailand revealed that the compacted samples behaved just like heavily overconsolidated soils. In all of the tests, the maximum deviator stress has never reached a peak, but the rate of increase decreased greatly after about 10-12% axial strain. He also found out that the placement moisture content had little effect on the ϕ of a clayey sand (from Chantaburi), but the ϕ of clayey gravel (from Saraburi) did increase as the moisture content increased. Figures 15 and 16 show the plots of obliquity ratio (σ_1/σ_3) vs. axial strain (ε) for the two soil samples compacted at optimum and at the dry side of optimum (95% density). The total and effective stress paths and strength envelopes for (σ_1/σ_3)$_{max}$, are given in Figs. 17 and 18. The stress paths and strength envelopes at 12% strain for the two samples are also shown in Figs. 19 and 20. Table 9 summarizes the shear strength parameters in terms of the total and effective stresses, while the undrained strength parameters are given in Table 10.

A clayey, gravelly lateritic soil from Chantaburi, Thailand was investigated by YONG (1975) to study its shear strength characteristics at saturated and partially saturated conditions. He concluded that the samples tested without back pressuring, generally have higher undrained strength than those tested with back pressuring. It was found out that for the test series with back pressure to saturation, samples compacted at optimum moisture content exhibit highest undrained shear strength. For samples tested without back pressuring, those compacted at optimum moisture content tends to have higher undrained strength at low consolidation pressures, but as consolidation increases, samples compacted dry of optimum, indicated the highest strength. The strength envelopes of the samples tested without back pressuring tends to fall below those tested with back pressuring. It was also concluded that, saturated sample have higher ϕ but lower c than those of unsaturated samples. Table 11 shows the shear strength parameters in terms of effective stress and Table 12 summarizes the undrained strength parameters.

g) Other Properties

Other properties of lateritic soils such as permeability, swelling, etc. have also been investigated by some authors. Because of their granular characteristics, lateritic soils are considered to be permeable and can provide adequate drainage. It is reported, however, that the coefficient of permeability decreases as the material degrades and becomes more clayey and plastic. Usually, lateritic soils are generally of low permeability because of the high percentage of clayey fines. Swelling problems are not considered significant in laterites and lateritic soils because they generally contain clay minerals like kaolinite that have low swelling characteristics.

VII THE WEATHERED BANGKOK CLAY

Introduction

Embankments constructed on soft marine clays with frictional materials like sand and gravels cost a lot due to large amount of transportation expenses involved. Engineers, therefore, have been always trying to find ways to lower the costs maintaining the proper factor of safety. It could be achieved to a considerable extent if locally available soils are utilized. Due to its abundancy, the weathered clay of Bangkok Plain has been proposed as one of the economically sound backfill material for the welded wire earth reinforcement research project. Engineering properties of this weathered clay based on the existing data and observations, both in compacted and noncompacted form is discussed in this section.

The Weathered Crust

Subsoils in the Bangkok area consist of thick deposits of marine clay which have been laid down at the different times in the geologic history of the Chao Phraya Plain. The typical subsoil strata are given in Fig. 21. The weathered crust with yellowish-brown to dark gray color forms the topmost 2 to 4 m of the subsoil. Red mottling near the surface is followed by yellow mottling towards the bottom of the crust, which is an indication of oxidation process. Oxygen in the leaching flood waters has changed the iron compounds present in the soil to ferrous oxides (red mottling) and ferric oxides (yellow mottling). The depth and magnitude of oxidation increases with increasing distance from the coast line as the sediments become older and have been subjected to greater leaching effects away from the coast. The engineering characteristics of this crustal zone (weathered clay zone) change with the time due to weathering, oxidation, vegetation effects, etc.

Index Properties

Early researchers like KANJANOPHAS (1969) reported the presence of small root holes and oxidation holes accompanied by some non-homogeneity in weathered Bangkok clay. It was observed that the samples were rather easily cracked and broken along the cleavage planes of the fissures during extruding. It was found that the initial water content varied greatly in the order of 54% to 69%. Typical values of the physical and the chemical properties of weathered clay at a depth of 0.20-1.0 m, 1.0- 2.0 m and 2.0-3.0 m from AIT campus (PLANGPONGPUN, 1977; HAQUE, 1977; LEELASITHORN, 1978; NIEW, 1979; KANJANOPHAS, 1969; GULACHOL, 1970; and QURESHI & MOH, 1973) are presented in Tables 13(a) and 13(b). The general properties of weathered clay backfill used by BUKKANASUTA (1986) are shown in Table 14. In general, the index properties lie in the following range: liquid limit = 64±2 percent; plastic limit = 27 ± 1 percent ; particles finer than 2u = 45±3 percent; specific gravity = 2.71; compacted optimum

moisture content = 24.5±0.3 percent; and maximum dry density = 1.55 t/m^3. PLANGPONGPUN (1977) and LIEW (1979) investigated that the index properties of the sample tested with natural water content and air dried case are not the same (Table 15). In case of air dried sample, reduction in Atterberg limits takes place due to drying of samples followed by the increase in specific surface area in the soil. The plasticity chart and grain size distribution chart of weathered Bangkok clay after PLANGPONGPUN (1977) is presented in Fig. 22. It was classified as inorganic clay of high plasticity (CH).

Engineering Behaviour at Uncompacted State

1) Permeability

BRAND & DUANGKHAE (1971) were the first to measure the field permeability of Bangkok clay. For weathered clay the field permeability reported, was in the order of 10^{-4} cm/s. However, large differences between the laboratory measured permeability and field permeability were observed due to the presence of fissures and root holes in the weathered clay. DANISH (1975) found the vertical permeability of the order of 8×10^{-5} cm/s in the upper weathered clay and 6×10^{-8} cm/s in the lower weathered clay which further drops to 3×10^{-8} cm/s. On the other hand, the horizontal permeability is 6×10^{-5} cm/s in the upper weathered clay, 2×10^{-7} cm/s in the lower weathered clay, and 6×10^{-8} cm/s in the soft clay. It was observed that in general, the horizontal permeability is higher than vertical. However, the deviation from this general behaviour was found in upper two meters of weathered zone due to weathering effect.

2) Compressibility Characteristics

The one-dimensional compressibility characteristics of weathered Bangkok clay have been studied by KANJANOPHAS (1969) & THUMAPRUDTI (1974) for AIT campus and Nong Ngoo Hao weathered clays, respectively. It was observed that the compressibility of weathered Bangkok clay increases with depth (Fig. 23). It was also concluded that the weathered clay has an apparent preconsolidation which is due to the increase in the bond strength of the soil skeleton brought about by weathering process. However, general consolidation behaviour was reported similar to that of normally consolidated clay. The compressibility parameters of weathered Bangkok clay as compiled by YUAN (1982) are shown in Table 16.

3) Shear Strength Characteristics

Shear strength characteristics of undisturbed Bangkok Clay in the weathered zone were discussed by ARAYASIRI (1969), MOH et al (1969), GULACHOL (1970), TAVARAYUT (1971), WU (1973), WANG (1974), MING (1975), and UDDIN (1975). The shear strength characteristics of weathered Bangkok clay were investigated by ARAYASIRI (1969), using isotropically consolidated-undrained

triaxial tests. It was observed that at consolidation pressure less than a certain pressure, the soil behaved in some aspects like slightly overconsolidated clay. For the samples consolidated at pressures greater than this pressure the soil behaved like normally consolidated clay. This pressure was called the 'critical pressure' or 'apparent preconsolidation pressure' and it depended on the degree of weathering. It was also reported that the effect of weathering process on the shear strength parameters was to decrease the angle of shearing resistance ϕ or ϕ' and to increase the cohesion intercept c or c'. Shear strength parameters after ARAYASIRI (1969) are shown in Table 17. GULACHOL (1970) investigated the effect of anisotropic consolidation on the shear strength parameters using anisotropically consolidated-undrained triaxial tests with pore pressure measurements (Table 18). The effects of anisotropic consolidation on shear strength were: (i) cohesion, c, decreased much with depth, and (ii) angle of friction, ϕ, was increased by 4 to 7 degrees. A comparison of shear strength parameters from these studies is presented in Table 19. Undrained shear strength of the weathered Bangkok clay was also correlated in terms of consolidation pressure. For apparent over consolidated clay, it was given by:

$$S_u = 2.4 + \bar{\sigma}_{1c} \tan 16.2^o \quad \text{(depth 1 - 1.8 m)} \quad .....(1)$$

$$S_u = 1.4 + \bar{\sigma}_{1c} \tan 13.5^o \quad \text{(depth 3.3-4.0 m)} \quad .....(2)$$

where S_u is in terms of psi.

WU (1973) studied the variation of in-situ strength and pore pressure of Bangkok clay from 1.5 m (weathered zone) to a depth of 14 m using anisotropic consolidated-undrained triaxial tests with no lateral strain (Ko) condition. It was observed that in the weathered clay layer the undrained shear strength decreased with depth because the properties of the uppermost soil layer were already altered by weathering process. It was also noticed that the pore pressure parameter A, in the weathered zone tended to decrease with depth. This behaviour was explained in light of the fact that pore pressure parameter A, depended on the rigidity of soil skeleton which decreases with the over consolidation pressure.

Shear strength parameters of Bangkok clay determined from undrained triaxial shear tests after MOH et al (1969) are presented in Table 20. It was noticed that for the weathered clay ϕ, ϕ', c, and c' were very nearly equal as opposed to stiff clay behaviour for which c is greater than c'. Thus, although the Bangkok clay in the weathered zone exhibits most of its strength and compressibility characteristics of a weathered clay, the soil appeared to be different from the truly overconsolidated stiff clay. QURESHI & MOH (1973) investigated the shear strength characteristics of Bangkok clay samples from a depth of 2 to 7 m using anisotropically consolidated-undrained triaxial tests on

horizontal and vertical samples. Shear strength parameters obtained from this investigation are presented in Table 21. For weathered Bangkok clay, BUKKANASUTA (1986) obtained an undrained shear strength of 2.74 t/m^2 and angle of internal friction of 20 degrees (Table 14) using direct shear tests.

Engineering Behaviour at Compacted State

The engineering behaviour of compacted weathered Bangkok clay from the drying crust at a depth of 0.20 to 1.00 m, in the AIT campus has been studied by PLANGPONGPUN (1978), HAQUE (1977), LEELASITHORN (1978) and LIEW (1979). A compilation of these data has been made by SUWONO (1979). However, in connection with the settlement and stability of embankments, the soil parameters are: index properties, drained and undrained strength parameters, compressibility and swelling characteristics, and permeability and seepage characteristics.

1) Compressibility Characteristics

SUTABUTRA (1967) studied the effects of various methods of compaction on the compressibility of weathered Bangkok clay and found that the compressibility of compacted clay is highest if the sample is prepared by kneading compaction, moderate by dynamic compaction and the least by static compaction. LIEW (1979) further investigated the compressibility characteristics and observed that the compacted weathered clay samples behave as natural stiff clay (Fig. 24). The shape of compression-log pressure curves for stiff clay and compacted weathered clays appears to be very flat and showing incompressible nature. The values of compression ratio and modulus of consolidation were reported in the range of 0.08 to 0.12 and 800 to 1600 t/m^2, respectively.

2) Swelling Characteristics

PLANGPONGPUN (1977), LEELASITHORN (1978), SUWONO (1979), and LIEW (1979) have discussed swelling characteristics of compacted weathered Bangkok clay. Within the range of variation of dry density and molding water content applied in his investigation, PLANGPONGPUN (1977) observed that swelling and swell pressure were governed by the molding water content and dry density. Higher dry density and lower water content cause larger swelling and swell pressure as shown in Figs. 25 and 26. The time to reach maximum swelling and equilibrium swell pressure tends to increase with the decrease in molding water content and increase in dry density. PLANGPONGPUN (1977) suggested that for clay embankment construction areas where soaking might occur, the degree of compaction should be lowered due to the expected large swelling and high swell pressure. LEELASITHORN (1978) tested a number of Bangkok compacted clay subjected to alternate soaking and drying simulating the seasonal variations in moisture content and found that maximum swell remained constant. It was also found

that for specimens compacted at optimum water content, the volumetric swell decreases with an increase in compactive effort.

3) Permeability Characteristics

HAQUE (1977) measured the permeability of compacted AIT weathered clay in the order of 10^{-7} cm/s and found that structure is the most important variable influencing the permeability of compacted clay. Specimens compacted wet of optimum showed permeability value 2 or 3 times less than samples compacted dry of optimum, other conditions being almost equal.

4) Thixotropic Behaviour

Thixotropy is defined as the increase in the strength of soil with time. THIRAWAT (1968) investigated thixotropic characteristic of compacted weathered Bangkok clay and reported that it is thixotropic in nature where the thixotropic behaviour increases with increasing molding water content and energy of compaction. However, these effects decreased with the increase of strain.

5) Strength Characteristics

Various researchers like TAVARAYUT (1971), HAQUE (1977), PLANGPONGPUN (1977) and LEELASITHORN (1978) have studied the strength characteristics of compacted weathered Bangkok clay by means of undrained strength, effective strength, tensile strength and CBR strength. TAVARAYUT (1971) conducted Unconsolidated-Undrained (UU) triaxial tests to study the undrained triaxial strength of compacted weathered Bangkok clay (AIT campus) and found decreasing strength with an increasing molding water content at various confining pressures for a given strain of 0.054 in/min (Fig. 27). It was noticed the specimens compacted near and at optimum moisture content exhibited a higher strength increase than the specimens compacted at other water contents. PLANGPONGPUN (1977) concluded that CBR value and undrained shear strength (determined by means of unconfined compression tests) of compacted clay increased with both the dry density and molding water content. It was also observed that the increase in CBR value and undrained strength due to the increase in dry density of samples compacted on the dry side of optimum was greater than that of samples compacted on the wet side. Regarding the effects of soaking on undrained strength, PLANGPONGPUN (1977) and LEELASITHORN (1978) found that CBR values and undrained strengths decreased sharply due to soaking and the effect of soaking on these parameters increased with the degree of compaction (Figs. 28 and 29).

RUENKRAIRERGSA (1968) conducted consolidated-undrained triaxial tests with pore pressure measurement for three differently compacted (dynamic, static, and kneading) samples. The strength of specimen prepared at the same molding water content were in the descending order as static, dynamic, and

kneading (Fig. 30). The excess pore pressure developed were in the descending order as kneading, dynamic, and static (Fig. 31). For the same compaction method, specimens prepared dry of optimum generally yielded higher strength and lower excess pore pressure than those compacted at wet of optimum (Figs. 32 and 33).

HAQUE (1977) made a series of tests on compacted AIT clay, which include unconfined compression (UC), isotropically consolidated-undrained (CIU) and isotropically consolidated-drained (CID) tests. Unconfined compression and unconsolidated-undrained tests results showed similar variation in strength with density and molding water content. Like previous investigators the undrained strength from both UC and UU tests increased with dry density and decreased with molding water content. As shown in Fig. 34, for a UC tests the undrained strength increased from 10.4 t/m^2 to 19.8 t/m^2 corresponding to the increase of dry density from 1.4 t/m^3 to 1.55 t/m^3. Similarly, the undrained strength from UU tests increased from 12.1 t/m^2 to 15.0 t/m^2 (24% increase) corresponding to an increase in the dry density from 1.41 t/m^3 to 1.55 t/m^3 (Fig. 35). The variation of undrained strength with molding water content from UC and UU tests are presented in Figs. 36 and 37. These figures also illustrate that the soaked specimen had lower strength than unsoaked, but at higher water content the differences were quite small. The stress-strain behaviour of samples from UC and UU tests were reported to be different in nature. UC tests showed a more prominent brittle behaviour than UU tests. However, it was observed that when the molding water content was increased, this brittle behaviour disappeared and the stress-strain curves for the unsoaked and the soaked specimen became similar with the maximum shear stress mobilized at large strains ($\varepsilon_f \div 10\%$). This conclusion is shown in Figs. 38 and 39, where failure strain is related to molding water content and dry density. A comparison of UC and UU undrained strengths is shown in Fig. 40, where UU tests showed higher value than UC tests. The strength difference was most pronounced on the dry side and was almost negligible on the wet side.

Normalized excess pore pressure vs. axial strain relationship showed that it increases rapidly (to a maximum value at 4% strain) and decreasing gradually due to dilatancy of the specimen when approaching failure (Figs. 41a,b). Pore pressure parameter A vs. axial strain relationship also showed similar trend. The peak value of the pore pressure parameter were reported as 0.6 to 0.67. The pore pressure coefficent at failure, A_f, was found in the range of 0.3 to 0.4. Effective stress paths and stress envelopes are shown in Figs. 42 and 43 for unsoaked and soaked samples, respectively. The undrained stress paths are similar to those followed by lightly-overconsolidated clay. The drained strengths from CID tests was almost twice the undrained strength from CIU tests where the strength increased with the increase in consolidation pressure (Fig. 44). The values of the effective shear strength parameters from CIU and CID tests are presented in Table 22. For soaked samples the values of

cohesion, c' and friction angle, ϕ_d were 4.6 t/m^2 and 25.5 degrees, respectively.

6) Tensile Strength

LEELASITHORN (1978) used split (Brazilian test) test to investigate the tensile strength of compacted weathered Bangkok clay. A typical behaviour shown in Fig. 45 demonstrates its brittle behaviour. The increase in tensile strength with increasing tensile stress before the failure is very small. This brittle behaviour seemed to decrease with an increase in molding water content, a decrease in compactive effort on the dry side of optimum, and an increase in compactive effort on the wet side of optimum. It was found that in general the tensile strength of compacted clay increases with an increase in molding dry density on the dry side of optimum as shown in Fig. 46. However, the tensile strength on the wet side of optimum is low.

VIII REINFORCING MATERIALS FOR THE REINFORCED EARTH EMBANKMENTS

Introduction

Reinforcing elements in a reinforced earth structure can be in the form of strip, anchor, sheet or grid reinforcement. There exist a variety of materials which can be used as reinforcing materials, but the most commonly used are metals, geotextiles and geogrids.

Metals are commonly made in the form of strip reinforcement which has either plain or ribbed surface. The metals being used for reinforcing elements are stainless steel, galvanized steel or coated steel strips. Aluminum and copper are also employed.

Geotextiles are made from fabrics and polymers which constitutes the fibers. These fabrics may be woven, knitted, or nonwoven. The geotextiles also differ by the type of polymers. The most commonly used polymers are polyester, polypropylene, polyethylene and polyamide. All these polymers were reported to have high mechanical strength.

Geogrids consist of transverse and longitudinal members, where the transverse members run parallel to the face of structure, while longitudinal members run perpendicular to the face. This type of reinforcement was at first developed using steel mesh (CHANG et al, 1977), and later, polymer was utilized in the form of mesh reinforcement (JONES, 1985). Studies on the pullout strength of bamboo geogrids were also reported recently. Geogrids have a good interlocking characteristics due to the transverse members which behave as strip footings or anchorages. This transverse members interlocking, results in a better performance of grids than that of plain or strip reinforcement.

Tensar Geogrids

Tensar Geogrids are a range of very high tensile strength polymer grids which have been developed specifically for the reinforcement of soils. They are manufactured from specially selected polyolefins to produce consistent and chemically inert engineering materials, suitable for long-term reinforcement. The open grid structure permits the soil particles to interlock through the apertures, with each transverse rib acting as a discrete anchorage point. This enables very efficient mobilization of the tensile strength of the geogrid. The soil-grid interlock creates an integral reinforced structure with either granular or cohesive soils. This unique property, combined with the polymer's inherent resistance to chemical attack, has permitted their use in many innovative design solutions. Such designs have proved to be cheaper, quicker and safer alternatives in a number of different applications (NETLON, LTD.). Tensar geogrids may either be uniaxially or biaxially orientated. Uniaxially orientated grids, such as Tensar SR2, are manufactured by stretching the punched sheet in one direction under carefully controlled conditions. Biaxially orientated grids, such as Tensar SS2, are produced by stretching uniaxially orientated grids, again under carefully controlled conditions, in the direction normal to the uniaxial orientation. The nomenclature for uniaxiallly and biaxially orientated grids is illustrated in Fig. 47. The typical set of dimensions that describe the geometry of Tensar geogrids, are provided in their respective specification sheets shown in the Appendix.

1) Tensile Tests

The two types of tensile testing which have been developed for Tensar geogrids are: the Quality Control Tests (CHEETHAM, 1983) and the Index Tests (MCGOWN, 1982). Quality Control Tests are rapid tests used to monitor the variability in strength of the geogrids, while the Index Tests are designed to provide a means by which the properties of all soil reinforcing materials could be compared, under standard rate of strain test conditions. A typical load extension curve for a uniaxially orientated Tensar SR2 grids when tested under index test conditions, is shown in Fig. 48. Figure 49a,b also illustrate the typical load extension curves for the longitudinal and transverse roll directions of a biaxial product when tested at Index conditions.

2) Long-Term Creep Tests

All polymer products exhibit time-dependent load extension behaviour, i.e. they creep under the application of a constant load. The pre-stretching operation of the Tensar process minimizes this effect. This phenomenon is particularly important since designs for permanent works require that reinforcing elements may be subjected to constant loads for periods up to 120 years. A thorough examination of the long-term creep properties

of Tensar geogrids has been undertaken at the University of Strathclyde and in the laboratories of Netlon Limited. A creep sample is loaded and the extension with time is recorded. Typical creep curves for Tensar SR2 at 20 ^{o}C is shown in Fig. 50. The creep performance shows, that after the initial extension has taken place due to the application of the load, the creep strains are small. A comprehensive description of the creep test method is given by MCGOWN (1982).

Bamboo

Bamboo is a native plant of tropical and sub-tropical areas and is defined as a perennial grass belonging to the monocotyledoneae class. They usually consist of jointed hollow or rarely solid clums which are unlike tree trunk in its structure and method of growth. Most of the species are usually 1 to 5 inches in diameter and grow up to a height of 50 to 60 feet but some are as large in diameter as 12 inches and reach a height of 100 feet or even more. Their rate of growth is very rapid, which can sometimes elongate as much as 2 to 3 ft per day. The individual bamboo clum is divided into nodes and inter-nodes. Because the greatest amount of meristematic tissue for the elongation of internode is found just above the node, the upper portion is generally the weakest part of the clum. There are about 1250 species of bamboo reported which are known to exist in the world.

The tensile strength of bamboo was observed to be about four times of its compressive strength. COX & GEYMAYER (1969) found a tensile strength in the order of 6000 to 25000 psi and the modulus of elasticity of 1.26×10^{6} to 4.01×10^{6} psi. It was also concluded that the stress-strain relation was essentially linear and exhibited a brittle type of failure. An average compressive strength of 8000 psi and modulus of elasticity of 2.16×10^{6} psi was obtained by GLENN (1950). It was further concluded that the average extreme fiber stress for seasoned clum was higher than for green unseasoned clum and it depended upon the positioning of the outer perimeter in the case of split clums.

DURRANI (1975) studied the bond slip and bamboo stress relationship to decide the permissible bamboo stress required in the design of bamboo reinforcement for crack control in pavements. Pai Ruak variety of bamboo, known scientifically as Thyrsostachys Oliveri Gamble, a common variety in Thailand, was used in the investigation. The typical tensile strength obtained from the investigation is summarized in Table 23. A study on the pullout resistance of bamboo grids of the same variety, was carried out by BUKKANASUTTA (1986). It was observed that bamboo grids have higher pullout resistance per unit area than that of the Tensar SS2 geogrids. One major disadvantage of using bamboo is their low durability due to their susceptibility to decay.

Metals

The critical condition in an embankment on soft clays is the short term stability where only undrained shear strength of the soil is mobilised to resist the embankment loadings against lateral spreading or rotational failure. In such cases only metallic reinforcements which have high stiffness, low extensibility and high modulus in the form of grids have been found to be most effective against lateral spreading. Geosynthetic reinforcements meet most of the strength criterion, however, they fail to keep the strain range in the structure within tolerable limits due to their low modulus. Locally manufactured steel reinforcements in the form of mats have been widely used in Thailand for highway embankments. Steel bars of diameters ranging from 4mm to 28 mm manufactured by local Thai companies are abundantly available in the market. Tensile strength tests on these reinforcement bars belonging to various grade sizes and manufactured by different local companies have been made at AIT Structural Engineering and Construction Division. These results have been compiled and presented in Tables 24, 25, and 26. Specifications for these reinforcement bars as per Thai standards TIS 20-1984 and TIS 24-1984 are shown in Tables 27 and 28. Another clear possibility lies with the application of steel meshes which have ben popular in ferrocement structures. Common wire meshes have either hexagonal opening or square opening. Meshes with hexagonal opening are also referred as chicken wire mesh or aviary mesh. They are not structurally efficient as meshes with square openings because the wires are not oriented in the directions of the principal (maximum) stresses. Meshes with square openings are available in either in welded form or in woven form. Welded form meshes have a higher modulus and hence high stiffness. Woven wire meshes are more flexible and easier to work with than the welded mesh. In the ferrocement applications , these wire mesh reinforcements are usuallly designed on the basis of ASTM standard A-185 or equivalent. The recommended design values of yield strength and effective modulus for steel meshes are shown in Table 29. Similarly, typical stress-strain curves for steel meshes are presented in Fig. 51.

IX SUBSOIL PROPERTIES OF THE PROPOSED SITE FOR THE USAID REINFORCED EARTH TEST EMBANKMENTS

Introduction

The USAID Research Project on reinforced earth is planning to construct a full-scale, fully-instrumented reinforced earth embankment using the most efficient reinforcement as obtained from the laboratory pullout tests. The possible backfill materials are the weathered Bangkok clay from AIT Campus, and the cohesive residual lateritic soils from Saraburi. The proposed site is to be located at the campus of the Asian Institute of Technology (Rangsit Area), about 42 km north of Bangkok which is

within the physiography of the Chao Phraya Plain. It is thus required to collect the existing data of the subsoil properties at the site as outlined in Task 1 of this research project.

Several extensive investigations had already been made in the past to study the site which is directly adjacent to the proposed experimental site. OHTA & HO (1982) initiated the study of the AIT test embankment, which was constructed in an open field area, north of the Academic Building. A study of the case history of the embankment settlement and stability was made as it was being loaded into a soft clay ground. The study covers the duration, from October,1980 to March,1982.

Additional studies were carried out by MOY (1984), JIAN (1983), and CHEN (1984). DANZUKA (1985) performed the reliability analysis of this test embankment. Actual results reported by these authors, indicated some similarity in the trend, and compared values were found to vary only slightly.

The Bangkok Subsoils

The subsoils in the Chao Phraya Plain (Fig. 52) are alluvial marine deposits. The top layer comprises of weathered clay which varies in depth from 1.5 m to 4.0 m depending on the location. Under the weathered clay layer, a soft clay layer may extend up to 8 to 15 meters, followed by a medium to stiff clay layer. Subsequently, dense sand and gravel layers alternate with stiff clay layers to an indeterminate depth. During the period of sedimentation, the deposition occurred in the northern plain first, then in the city area of Bangkok. Thus, the subsoil in the northern plain is slightly older and the engineering properties may slightly vary from that of the Bangkok City area. The typical subsoil profiles from north to south part of the Chao Phraya Plain are shown in Figs. 53, 54 and 55.

Site Location

The site of the proposed test embankments is located at the campus of the Asian Institute of Technology (AIT) as shown in Fig. 56. The subsoil condition is within the physiography of the Chao Phraya Plain where the clay subsoils can also be classified as Bangkok clay. However, certain engineering properties such as the undrained shear strength and overconsolidation ratio (O.C.R.), are slightly different from the Bangkok area. As reported by OHTA & HO (1982), the O.C.R. value here showed a great scatter and the undrained shear strength is slightly higher than that observed in the Bangkok area. Although the Bangkok clay is normally consolidated, the maximum effective past pressure is generally greater than the effective overburden pressure by 50 to 70% due to aging (secondary consolidation).

The locations of the proposed site and the AIT test embankment studied by OHTA & HO (1982), are also shown in Fig. 56. As can be seen from the figure, the area studied is directly

adjacent to the proposed embankment site of the USAID research project. Two boreholes of 3 inches diameter and one of 10 inches in diameter, were made by OHTA & HO (1982) to collect undisturbed samples for subsoil properties at the site. These samples were taken continuously from the ground surface down to a depth of 8 meters. Later, MOY (1984) made another two boreholes at the north and south side of the test embankment.

Soil Profile

The soil profile in AIT Campus as shown in Fig. 57, varies a little from area to area. The thickness of the weathered clay varies from 1.5 m to 2.0 m at the crust, and sometimes, the weathering effects do extend to somewhat greater than the depth of 2 m as reported by LADD et al (1971). The weathered clay is of dark-brown color embedded with rust-colored clay, which becomes less as the soil depth increases till the soft clay layer. This weathered clay usually has cracks and fissures due to alternate cycle of wetting and drying. The water table is at 1.0 m to 1.5 m below the ground level, but this level may go up and down depending on the season.

The soft clay layer is medium dark gray in color. There are pockets of silt and sand lenses in this layer. Sometimes slickensides of vertical and horizontal orientation are found. The soft clay is reported to vary in thickness in this campus. OHTA & HO (1982) and MOY (1984) reported that, the soft clay layer extends to a depth of 7.5 m to 8.5 m at the site. Below this soft clay layer is the stiff clay layer.

Laboratory Tests

1) Index Properties

Table 30 and Fig. 57 show the general index properties obtained by MOY (1984) while Table 31 shows the general index properties reported by OHTA & HO (1982). A quite similar trend of results, except for a few points, can be seen in Tables 30 and 31. The observed higher values of the natural moisture content in the weathered clay zone was attributed to the difference in season when samples were collected, since the properties of of the weathered clay in the crust layer is very subjective to weather change. Although the previous results of OHTA & HO (1982) showed quite randomly scattered values with depth, and the total unit weight showed higher values than the result of MOY (1984), on the overall, the results of both studies showed a similar trend with those obtained by TSAI (1981).

2) Compressibility Characteristics

There were two series of oedometer tests carried out by MOY (1984), namely, the standard oedometer test employing a double load increment after 24 hours duration, and the special oedometer test, employing a small load increment (0.1 to 0.2

kg/cm^2) method which usually took more than 24 hours duration for one loading. From the oedometer tests, the compressibility characteristics of the subsoil are represented by the parameters such as the initial void ratio, maximum past pressure, coefficient of consolidation, compression index and swelling index.

Table 30 shows the summary of these parameters for both the standard and special oedometer tests. The results of OHTA & HO (1982) using the standard oedometer test is summarized in Table 31. It can be seen that both sets of results show that the compressibility and swelling indices were smaller in the weathered clay layer than in the soft clay layer, and so was the void ratio. The coefficient of consolidation, determined by both the Taylor's square root of time fitting method and the Casagrande's logarithm of time fitting method, was found to be higher for the weathered clay than for the soft clay.

The maximum past pressure was determined from the e-log p curve using Casagrande's method. Table 30 and Fig. 58 show the results of maximum past pressures from oedometer tests. Results obtained by various investigators in their study of the ATT subsoils, are also shown in Fig. 58, where a similar trend can be observed. In the weathered clay layer, the maximum past pressure increases with depth until 2 m while in the soft clay layer, the value varies between 5 to 10 t/m^2. The maximum past pressure of the weathered clay is largely dependent on the apparent overconsolidation caused by the cycles of dessication. There is no observed trend which indicates some changes in the maximum past pressure since the first research in 1969. KANJANOPHAS (1969) and TAESIRI (1976) reported that the small increment load ratio is the better determination method for maximum past pressure than the double increment load method which usually yields much lower values.

3) Strength Properties

a) Anisotropically K_o Consolidated Undrained Triaxial Tests

The consolidated anisotropic (K_o) undrained triaxial compression and extension tests with pore pressure measurement, were carried out by MOY (1984) to determine the undrained shear strength of the clay. TANTIKOM (1982) suggested a K_o value of 0.68 for the weathered clay and a K_o value of 0.60 for the soft clay. A shearing rate of 1% per hour as suggested by LADD & FOOTT (1974), was used in all the above tests. Table 32 shows the summary of the test results obtained. The effective stress paths of the triaxial compression and extension tests are shown in Fig. 59. It can be seen from the triaxial compression tests, that almost all of the soil samples converged to a similar K_f-line, but in the extension tests, the results were quite scattered.

It was also reported that the degree of weathering was nonhomogeneous in the weathered clay layer and pockets of sand and silt lenses were embedded in the soft clay layer. DANZUKA (1985) concluded that there are two domains of soft clay having different character of the presence of sand and silt lenses. Domain 1 at 2 to 7 m depth includes 0.23 percent density of the lenses, and domain 2 at 7 to 9 m depth contains 19.3 percent density of the lenses. Since the extension test is very sensitive, the scattered data obtained was attributed to this nonhomogeneity.

The normalized undrained shear strength of the triaxial tests is plotted with the overconsolidation ratio as shown in Fig. 60. The result is compared with the steeper curve obtained by LADD et al (1971), who used the SHANSEP method to determine the undrained shear strength of the soft Bangkok clay. The difference was attributed to the slightly different undrained shear strength properties in the AIT campus compared to the Bangkok area, as well as to the different test procedures employed.

The variation of anisotropy effect of the clay with the overconsolidation ratio is shown in Fig. 61. LADD et al (1971) reported, that the anisotropy K_s, increases with O.C.R., and stiff fissured clays may exhibit a pronounced anisotropy greater than unity for heavily overconsolidated clay. On the average, the ratio of the undrained shear strength obtained from extension tests to that of compression tests ($S_{u(ext)}/S_{u(comp)}$) of the soft clay was found to be 0.80 as shown in Table 32. The result agrees well with that obtained by EIDE & HOLMBERG (1972), who used the anisotropy vane tests on the soft Bangkok clay.

b) Isotropically Consolidated-Undrained (CIU) Tests

Two series of CIU tests using different consolidation pressures were carried out by OHTA & HO (1982). A back pressure of 200 kN/m^2 was applied for 24 hours in the first series, then the sample was isotropically consolidated to an effective overburden pressure for at least another 24 hours. In another test, consolidation pressures exceeding well above the maximum past pressure was applied for at least 48 hours in four steps (i.e. 0-20-40-80-150 kN/m^2). In both series, the samples were then subjected to shear stresses under strain-controlled conditions by increasing the axial strain at the rate of 0.2%, 0.5%, 1%, 2%, 5%, 10%, or 15% per minute. The normalized effective mean principal stress vs. normalized deviator stress are shown in Tables 33 and 34 and Figs. 62 and 63. The variation of the friction angle based on effective stresses is presented in Fig. 64 and Table 35.

c) Unconsolidated Undrained (UU) and Unconfined Compression (UC) Tests

BERGADO et al (1986) conducted a series of laboratory triaxial tests at various locations in the AIT campus. Values of the undrained shear strength obtained from these tests are compared with the other tests as shown in Fig. 68. OHTA & HO (1982) performed unconfined compression tests. Table 36 and Fig. 64 present the variations of q_u and E_{50} with depth.

Field Tests

1) Field Vane Shear Tests

MOY (1984) carried out six field vane shear tests in November, 1983 before the additional excavation of the trench on the south side of the AIT embankment. The other vane test was performed in early January, 1984 after the excavation to check the changes in the properties due to the excavation which caused large cracks in the embankment. The latter test, however, did not show any significant change in the strength of the subsoil.

The undrained shear strength obtained from the field vane shear tests was higher than that of the Bangkok area, and range from 1.7 t/m^2 to 3 t/m^2. The remoulded strength is 0.5 t/m^2 with a medium sensitivity of 2 to 4.5. The test results were similar to that obtained by BRAND et al (1972) which was also conducted at the AIT campus. The results obtained above were quite similar to the undrained shear strengths previously reported by OHTA & HO (1982). Although their tests were carried out both in the area affected by the embankment, there was no significant difference in the strength of the soft clay. BERGADO et al (1986) measured the undrained shear strength from four sites in the AIT campus and the average values obtained are illustrated in Fig. 65. It can be seen that in the soft clay zone, the strengths are slightly increasing with depth, ranging from 0.25 to 0.35 kg/cm^2. They also reported that at the embankment site (behind ET Building), the weathered clay layer extended to 2.0 to 2.5 m depth and the soft clay ended at 8 m depth.

2) Dutch Cone Penetration Tests

There were five Dutch cone tests carried out by KHALEQUE (1984) near the test embankment. The Dutch cone resistance with depth at five locations near the embankment are shown in Fig. 66. The Dutch cone resistance was very high in the weathered clay crust and reduced with increasing depth. On the contrary, the Dutch cone resistance was observed to slightly increase with depth for the soft clay layer. There were some scattered results of lower resistance in the weathered clay layer, and higher resistance at the depth of 3.5 to 5.5 m in the soft clay. The scattered points on the low strength side in the weathered clay layer, was attributed to the local failure or fissures in the clay. On the other hand, scattered points on the

high strength side in the soft clay layer, was attributed to the presence of sand and silt lenses. The average test results from the four sites in AIT campus obtained by BERGADO et al (1986) are also plotted in Fig. 66. Both data sets showed similar trend with depth.

3) Pressuremeter Test

BERGADO et al (1986) perfor ed a series of pressuremeter test at four locations in the AIT campus. At each site, the tests were done in three locations. Values of undrained shear strength are plotted in Fig. 67 as average values with depth for the weathered, soft, and stiff clay layers. The calculated values of the undrained Young's modulus E_{up} for the soft clay layer is given in Fig. 70. The modulus value was found to vary between 20 kg/cm^2 and 60 kg/cm^2.

4) Screw Plate and Plate Load Tests

BERGADO et al (1986) also conducted a series of screw plate and plate load tests from 3 locations in AIT campus. The calculated undrained shear strength from these tests were compared with the pressuremeter test result as illustrated in Fig. 69. Comparison of the undrained modulus values obtained from these three tests are also shown in Fig. 70. The field and laboratory test results for the undrained shear strength of the subsoils, down to 8 m depth are compared in Fig. 68.

REFERENCES

ARAYASIRI, M. and NELSON, J.D. (1970), Shear Strength Characteristics of Bangkok Clay in the Weathered Zone, Research Report No. 8, AIT, Bangkok.

BERGADO, D.T. (1976), Engineering Properties of Granite Residual Soil Along the Hod-Mae Sariang Highway, M.Eng. Thesis No.899, AIT, Bangkok.

BERGADO, D.T., KHALEQUE, M.A., NEEYAPAN,R. and CHANG,C.C. (1986), Correlations of In-Situ Tests in Bangkok Subsoils, Geotechnical Engineering, Vol.17.

BERGADO, D.T. and HUAN, N.M. (1987), Undrained Deformability and Strength Characteristics of Soft Bangkok Clay by the Screw Plate Test, Geotechnical Testing Journal, CTJODJ, Vol.10, No.3, pp.113-122.

BOONSENER, M. (1977), Engineering Geology of the town of Khon Kaen, Northeastern Thailand, M.Eng. Thesis, AIT, Bangkok.

BOONSRI, N. (1971), Shear Strength Characteristics of Compacted Lateritic Soils, M.Eng. Thesis No.399, AIT, Bangkok.

BOONTAN, A. (1984), Engineering Classification of Laterites and Lateritic Soils, M.Eng. Thesis No. GT-83-3, AIT, Bangkok.

BRAND, E.W. and HONGSNOI, M. (1969), Effects of Method of Preparation on Compaction and Strength Characteristics of Lateritic Soils, Proc. Specialty Session on Eng. Properties of Lat. Soils, VII ICSMFE, Mexico City, Vol.1, pp. 107-116.

BRAND, E.W. and DUANGKHAE, S. (1971), The Permeability of Bangkok Clay, Research Report No. 13, AIT, Bangkok.

BRAND, E.W., MUKTABHANT, C. and AKRAPONGPISAI, S. (1972), Comparative Study of Undrained Strength Measurements of a Soft Clay, Proc. 3rd SE Asian Conf. Soil Eng., AIT, Bangkok.

BUKKANASUTTA, A. (1986), Laboratory Pullout Tests Using Bamboo and Tensar Geogrids Including a Case Study, M.Eng. Thesis No. GT-85-25, AIT, Bangkok.

CHAMNONGPIPATKUL, P. (1980), Remote Sensing Studies of Some Ironstone Gravels and Plinthite in Thailand, Ph.D. Thesis, Cornell University, New York.

CHANG, J.C., HANNON, L.B. and FORSYTH, R.A. (1977), Pull Resistance and Interaction of Earthwork Reinforcement and Soil, Transportation Res. Rep. No.640, pp. 1-7.

CHEN, I.P. (1984), Settlement and Stability Analysis of Embankment on Bangkok Soft Clay in AIT Campus (Phase IV), M.Eng. Thesis No. GT-83-22, AIT, Bangkok.

CHEN, J.S. (1980), Geotechnical Parameters for the Design of Structures in Residual Soils, M.Eng. Thesis No. GT-80-1, AIT, Bangkok.

CHEETHAM, G. (1983), Quality Control Test Methods Used in the Production of Tensar SR2, Internal Report, Netlon Limited, England.

COX, F.B. and GEYMAYER, H.G. (1969), Expedient Reinforcement for Concrete for Use in Southeast Asia, Technical Report C-69-3, U.S. Army Engineers WES.

DANISH, J.A. (1975), In-Situ and Laboratory Permeability of Bangkok at Ngoo Hao and Rangsit, M. Eng. Thesis No. 765, AIT, Bangkok.

DANZUKA, M. (1985), Reliability Analysis of AIT Test Embankment, M.Eng. Thesis No. GT-84-10, AIT, Bangkok.

DURRANI, A.J. (1975), A Study of Bamboo as Reinforcement for Slabs on Grade, M.Eng. Thesis No.801, AIT, Bangkok.

EIDE, O. and HOLMBERG, S. (1972), Test Fills to Failure on Soft Bangkok Clay, Proc. Spec. Conf. on Performance of Earth and Earth-Supported Structures, ASCE, Purdue University, Vol.1, Part 1, pp.159-180.

FERREIRA, H.N. and MEIRELES, J.M.F. (1969), The Influence of Temperature of Humidification on the Geotechnical Properties of Lateritic Soils, Proc. Specialty Session on Engineering Properties of Lateritic Soils, VII ICSMFE, Mexico City, Vol.1, pp. 65-74.

GLENN, H.E. (1950), Bamboo Reinforcement in Portland Cement Concrete, Bulletin No.4, Clemson Agricultural College, Clemson.

GULACHOL, C. (1970), Shear Strength Characteristics of Bangkok Clay in the Weathered Zone, M.Eng. Thesis No. 363, AIT, Bangkok.

HAQUE, M.A. (1977), Some Engineering Properties of Compacted Rangsit Clay Evaluated on the Basis of Laboratory Tests, M.Eng. Thesis No. 1005, AIT, Bangkok.

HONGSNOI, M. (1969), Effects of Methods of Preparation on Compaction and Strength Characteristics of Lateritic Soils, M.Eng. Thesis No. 256, AIT, Bangkok.

JIAN, Y.C. (1983), Analysis of Settlement of an Embankment on Bangkok Clay in AIT Campus, M.Eng. Thesis No. GT-82-21, AIT, Bangkok.

JONES, C.J.F.P. (1985), Earth Reinforcement and Soil Structures, 1st ed., Butterworths and Co. Ltd., London, England.

KANJANOPHAS, S. (1969), Compressibility of Bangkok Clay in the Weathered Zone, M.Eng. Thesis No. 338, AIT, Bangkok.

KHALEQUE, M.A. (1984), Correlation of Pressuremeter, Vane and Dutch Cone Tests of Rangsit Clay, M.Eng. Thesis , AIT, Bangkok.

LADD, C.C., MOH, Z.C. and GIFFORD, D.G. (1971), Statistical Analysis of Undrained Strength of Soft Bangkok Clay, Proc. 4th Asian Conf. on Soil Mech. Found. Eng., pp.135-140.

LADD, C.C. and FOOT,R. (1974), New Design Procedure for Stability of Soft Clays, J. Geotech. Eng. Div., ASCE, Vol.100, GT7, pp.763-786.

LAM, K.S. (1979), Strength and Suction-Moisture Retention of a Compacted Residual Volcanic Soil, M.Eng. Thesis No. GT-79-13, AIT, Bangkok.

LEELASSITHORN, T. (1978), Strength Characteristics of Compacted Clay, M.Eng. Thesis No. 1286, AIT, Bangkok.

LIEW, Y. Y. (1979), Compressibility and Permeability Characteristics of Compacted Clay, M.Eng. Thesis No.GT-78-12, AIT, Bangkok.

LITTLE, A.L. (1969), The Engineering Classification of Residual Tropical Soils, Proc. Specialty Session on Eng. Properties of Lateritic Soils, 7th ICSMFE, Mexico City, Vol.1, pp.1-10.

MAZHAR, M.F. (1969), Effects of Methods of Preparation on Index Properties of Lateritic Soils, M.Eng. Thesis No. 264, AIT, Bangkok.

MCGOWN, A. (1982), Progress Report, Internal Report Presented to the SERC/Netlon Co-operative Award Steering Committee.

MING, H.Z. (1975), Stress-Strain Behaviour and Strength Characteristics of Weathered Nong Ngoo Hao Clay, M.Eng. Thesis No. 769, AIT, Bangkok.

MOH, Z.C. and MAHMOOD, A. (1969), Triaxial and Stabilometer Properties of Lateritic Soils, Proc. Specialty Session on Eng. Properties of Lateritic Soils, VII ICSMFE, Mexico City, Vol.2, pp.110-121.

MOH, Z.C. and MAZHAR, M.F.(1969), Effect of Method of Preparation on Index Properties of Lateritic Soils, Proc. Specialty Session on Eng. Properties of Lateritic Soils, VII ICSMFE, Mexico City, Vol.1, pp.23-35.

MOH, Z.C., NELSON, J.D. and BRAND, E.W. (1969), Strength and Deformation Behaviour of Bangkok Clay, Proc. 7th ICSMFE, Mexico City, Vol.1, pp.287-296.

MOY, W.Y. (1984), Properties of Subsoil Related to the Stability and Settlement of AIT Test Embankment , M.Eng. Thesis No. GT-83-25, AIT, Bangkok.

NETLON LTD. (1985), Test Methods and Physical Properties of Tensar Geogrids, Publication, Netlon Limited, England.

NIXON, I.K. and SKIPP, B.O. (1957), Airfield Construction on Overseas Soils, Proc. Inst. Civil Engineers, Vol. 8, pp.253-292.

OHTA, H. and HO, Y.C. (1982), A Trial Embankment on Soft Bangkok Clay (Phase I-III), Res. Rep. No. RR-146, AIT, Bangkok.

PLANGPONGPUN, S. (1977), Strength Characteristics of Bangkok Clay in Relation to Pavement Design, M.Eng. Thesis No. 1014, AIT, Bangkok.

QURESHI, M.A. and MOH, Z.C. (1973), Anisotropy of Strength Characteristics of Bangkok Clay, Research Report No. 44, AIT, Bangkok.

RUEKRAIRERGSA, T. (1968), Effects of Compaction Method on Pore Pressure Development, M.Eng. Thesis No. 194, AIT, Bangkok.

SHUSTER, J. (1969), Personal Communications, Bangkok, Thailand.

SOIL AND PAVEMENT CONSULTANTS (1969), Engineering Study of Laterite and Lateritic Soils in Connection with Construction of Roads, Highways, and Airfields, Phase I Southeast Asia (Thailand), Final Report to USAID, Oakland, California.

SUTABUTRA, P. (1967), Compressibility of Compacted Bangkok Clay, M.Eng. Thesis No. 187, AIT, Bangkok.

SUWANASING, A. (1974), Geology and Nickeliferous Laterite Deposits of Ban Tha Kradan Nok, Prachinburi, Thailand, Econ. Geol. Bull., No.3, pp.82-106.

SUWONO, J.I. (1979), Engineering Properties of Compacted Clay, M.Eng. Thesis No. GT-78-17, AIT, Bangkok.

TAESIRI, Y. (1976), Consolidation Characteristics of Rangsit Clay, M.Eng. Thesis No. 918, AIT, Bangkok.

TANTIKOM, S. (1982), Ko-Measurements on Bangkok Clay, M.Eng. Thesis No. GT-81-13, AIT, Bangkok.

TAVARAYUT, P. (1971), Shear Strength of Compacted Clay under Rapid Transient Loading, M.Eng. Thesis No. 406, AIT, Bangkok.

THIRAWAT, S. (1968), Thixotropic Characteristics of Compacted Bangkok Clay, M.Eng. Thesis No. 211, AIT, Bangkok.

THUMAPRUDTI, A. (1974), One Dimensional Characteristics of Weathered Nong Ngoo Hao Clay, M.Eng. Thesis No. 705, AIT, Bangkok.

TIS 20-1984, Standard for Steel Bars for Reinforced Concrete: Round Bars, G.S. Steel Co. Ltd., Bangkok.

TIS 24-1984, Standard for Steel Bars for Reinforced Concrete: Deformed Bars, G.S. steel Co. Ltd., Bangkok.

TSAI, C.Y. (1982), A Monograph on the Engineering Properties of Bangkok Subsoil, M.Eng. Thesis No. GT-81-15, AIT, Bangkok.

UDDIN, W. (1975), Stress Strain Behaviour and Shear Strength Characteristics of Weathered Bangkok Clay under Extension Conditions, M.Eng. Thesis No. 774, AIT, Bangkok.

VALLERGA, B.A. and RANANANDANA, N. (1969), Characteristics of Lateritic Soils used in Thailand Road Construction, Highway Res. Record No.284, Highway Res. Board, Washington, D.C., pp. 86-103.

WANG, C.H. (1974), Undrained Shear Strength Characteristics of Nong Ngoo Hao Weathered Clay under Ko Conditions, M.Eng. Thesis No. 706, AIT, Bangkok.

WINTERKORN, H.F. and CHANDRASEKHARAN, E.C. (1951), Laterite Soils and their Stabilization, Highway Res. Bull. No. 44, pp.16-29.

WU, Y.M. (1973), Variation of In-Situ Strength and Pore Pressures of Bangkok Clay with Depth, M.Eng. Thesis No. 517, AIT, Bangkok.

YONG, F.O. (1975), The Effective Stress Strength Parameters of a Compacted Lateritic Soil, M.Eng. Thesis No. 777, AIT, Bangkok.

TABLES

Table 1 Mineralogical Composition of Thailand Lateritic Soils (after MAZHAR, 1969).

Soil Source No.	Method of Preparation	Mineralogical Composition				
		Differential Thermal Analysis				X-ray Diffraction[3]
		Kaolinite,%[2]		Other Minerals,%[2]		
		Sample 1	Sample 2	Sample 1	Sample 2	
1	NMC OD	15.0 9.0	11.0 10.0	7.0 9.0	11.0 8.0	Kaolinite, interstratified clay numerals-consisting of montmorillonite and hydrous mica in random stacking
2	NMC OD	7.0 5.0	5.0 -	5.0 4.0	7.0 -	Kaolinite
3	NMC OD	2.0 3.0	2.0 3.0	4.0 3.0	4.0 3.0	Kaolinite> montmorillonite > illite, some quartz
4	NMC OD	10.0 12.0	- -	4.0 4.0	- -	Kaolinite
5	NMC OD	9.0 9.0	- -	10.0 10.0	- -	Mainly kaolinite small amount of illite
6	NMC OD	19.0 19.0	- -	8.0 8.0	- -	Kaolinite + traces of illite
7	NMC OD	0.4 0.6	0.2 0.3	1.0 0.7	1.2 1.0	Mixed clay kaolinite> illite >mont. some quartz
8	NMC OD	1.1 1.1	1.1 0.9	1.2 0.9	1.2 1.1	Kaolinite, some illite and quartz
9	NMC OD	6.0 4.0	- -	9.0 10.0	- -	Kaolinite, some illite and montmorillonite appreciably amount of quartz
10	NMC OD	3.2 2.6	- -	4.5 4.6	- -	Kaolinite, some illite small amount of mont.
11	NMC OD	2.6 2.5	- 2.6	3.1 2.2	- 2.1	Kaolinite, some illite

1 Analyses were carried out on clay size fraction (< 0.002 mm)
2 Percent based on the total sample
3 Results are for all three methods of preparation

Table 2 Chemical Properties of Thailand Lateritic Soils (after MAZHAR, 1969).

Soil Source No.	pH	Soluble Salt Content	Organic Matter Content (%)	Free Iron Oxide Content
1	3.85-4.19	0.13-0.24	0.56-0.67	7.76-8.21
2	4.42-4.63	0.13-0.28	0.18-0.25	9.49-11.6
3	4.45-4.81	0.10-0.15	0.20-0.24	3.50-3.65
4	4.21-4.86	0.19	0.23-0.41	3.45-3.60
5	4.42-4.48	0.18-0.22	0.62-0.76	4.88-6.11
6	4.38-4.46	0.11-0.18	0.92-0.94	5.29-5 43
7	4.58-4.63	0.12-0.14	0.45-0.46	2.47
8	4.55-4.94	0.12-0.17	0.19-0.39	1.53-2.17
9	4.04-4.20	0.14-0.23	0.21-0.34	3.97-4.44
10	5.77-6.30	0.24-0.47	0.62-0.76	7.08-9.22
11	5.21-5.68	0.33-0.53	0.22-0.26	6.27-8.57

Table 3 Values of Specific Gravity for the Lateritic Soils of Thailand (after MAZHAR, 1969).

Soil Source No.	Method of Preparation		
	NMC	AD	OD
1	2.94	2.86	2.91
2	2.88	2.86	2.88
3	2.66	2.68	2.72
4	2.66	2.66	2.69
5	2.65	2.65	2.66
6	2.63	2.63	2.63
7	2.76	2.75	2.75
8	2.69	2.66	2.69
9	2.74	2.72	2.70
10	2.67	2.70	2.68
11	2.67	2.71	2.72

1 Determined on the fraction passing U.S. No. 10 sieve (2.00 mm)

Table 4 Grain Size Distribution of Thailand Lateritic Soils (after MAZHAR, 1969).

Soil Source No.	Method of Preparation	Grain Size Distribution, %				
		Gravel (>2.00 mm)	Sand (2.00 - 0.074 mm)	Silt (0.074 - 0.005 mm)	Clay (<0.005 mm)	Clay (<0.002 mm)
1	NMC[2]	58.3	6.4	11.0	24.3	22.0
	OD[3]	58.8	6.3	13.5	21.4	18.0
2	NMC	66.4	9.8	9.3	14.5	12.5
	OD	67.2	10.2	12.6	10.0	8.0
3	NMC	57.1	20.4	16.1	6.4	5.7
	OD	56.6	20.4	16.1	6.9	6.1
4	NMC	62.6	13.8	7.8	15.8	14.5
	OD	61.4	14.1	7.6	16.9	15.7
5	NMC	18.1	51.3	11.0	19.6	16.3
	OD	17.1	51.2	13.2	18.5	15.0
6	NMC	23.2	28.7	14.9	33.2	27.0
	OD	22.8	28.8	15.8	32.6	27.0
7	NMC	76.3	12.9	9.2	1.6	1.4
	OD	75.9	12.6	10.0	1.5	1.3
8	NMC	50.2	28.3	18.5	3.0	2.3
	OD	49.9	27.5	19.4	3.2	2.0
9	NMC	66.4	7.4	7.0	19.2	15.2
	OD	65.5	8.2	5.8	20.5	13.9
10	NMC	61.6	12.9	9.9	9.0	7.7
	OD	61.4	20.7	9.4	8.5	7.2
11	NMC	56.7	22.5	13.9	6.9	5.7
	OD	54.7	22.9	16.3	6.1	4.7

[1] Trisodium phosphate was used as dispersant in hydrometer analysis.
[2] Natural moisture content [3] Oven-dried sample

Table 5 Ranges of Atterberg Limit Values for Thailand Lateritic Soils.

Soil Source No.	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index (%)	Shrinkage Limit (%)	Activity
1	36 - 43	68 - 83	32 - 40	21	0.68
2	43 - 45	51 - 56	8 - 11	22	0.30
3	14 - 16	24 - 37	10 - 21	12	0.98
4	24 - 27	35 - 43	11 - 16	18	0.33
5	30 - 38	40 - 52	10 - 14	23	0.38
6	34 - 39	57 - 66	23 - 27	25	0.51
7	18 - 19	20 - 22	2 - 3	15	0.34
8	15 - 16	16 - 19	1 - 3	13	0.42
9	19 - 25	29 - 40	10 - 15	17	0.24
10	24 - 28	38 - 48	14 - 20	21	0.59
11	22 - 26	30 - 37	8 - 11	14	0.66

Table 6 Engineering Classification of Lateritic Soils in Thailand. (after MAZHAR, 1969)

Soil Source No.	Method of Preparation	Soil Classification			
		Textural (Bureau of Soils)	BPR (AASHO)	United (Casagrande)	CAA
1	NMC	gravelly clay	A-2-7(4)	GC-OH	E-12
	OD	gravelly clay	A-2-7(4)	GC-OH	E-10
2	NMC	gravelly clay	A-2-7(0)	GC-MH	E-9
	OD	gravelly clay	A-2-5(0)	GC-MH	E-9
3	NMC	gravelly clay	A-2-6(1)	GC-CL	E-7
	OD	gravelly clay	A-2-4(0)	GC-CL	E-5
4	NMC	gravelly clay	A-2-7(1)	GC-OL	E-7
	OD	gravelly clay	A-2-6(0)	GC-OL	E-7
5	NMC	gravelly sandy clay loam	A-2-7(1)	SC-OH	E-9
	OD	gravelly sandy clay loam	A-2-4(0)	SC-OL	E-7
6	NMC	gravelly clay	A-7-5(10)	SC-OH	E-10
	OD	gravelly clay	A-7-5(8)	SC-OH	E-8
7	NMC	gravelly sandy loam	A-1-a(0)	GM-ML	E-1
	OD	gravelly sandy loam	A-1-a(0)	GM-ML	E-1
8	NMC	gravelly sandy loam	A-1-b(0)	SM-ML	E-1
	OD	gravelly sandy loam	A-1-b(0)	SM-ML	E-1
9	NMC	gravelly clay	A-2-6(1)	GC-CL	E-5
	OD	gravelly clay	A-2-4(0)	GC-CL	E-4
10	NMC	gravelly sandy clay loam	A-2-7(0)	SC-OL	E-7
	OD	gravelly sandy clay loam	A-2-6(0)	SC-CL	E-5
11	NMC	gravelly sandy loam	A-2-6(0)	GC-OL	E-5
	OD	gravelly sandy loam	A-2-4(0)	GC-CL	E-4

Table 7 Compaction Properties of Lateritic Soils in Thailand (after HONGSNOI, 1969).

Source No.	Test Condition	Classification (AASHO)	Maximum Dry Density lb/cu ft	Optimum Moisture Content (OMC), %	CBR: % at γ_{max} and OMC	
					Unsoaked	Soaked
1	NMC	A-2-7(4)	119	16	89	5
	OD	A-2-7(4)	126	15	87	4
2	NMC	A-2-7(2)	109	20	54	39
	AD	-	112	20	36	56
	OD	A-2-7(0)	112	19	36	42
3	NMC	A-2-6(1)	142	8	23	37
	OD	A-2-4(0)	142	8	24	17
4	NMC	A-2-7(1)	122	14	45	40
	AD		124	13	28	35
	OD	A-2-6(0)	124	13	82	71
5	NMC	A-2-7(1)	111	16	34	41
	OD	A-2-4(1)	112	15	64	76
6	NMC	A-7-5(10)	111	17	43	30
	AD	-	111	17	35	34
	OD	A-7-5(8)	112	17	35	36
7	NMC	A-1-a(0)	142	12	35	63
	OD	A-1-a(0)	142	10	34	57
8	NMC	A-1-b(0)	137	10	40	37
	OD	A-1-b(0)	138	7	20	27
9	NMC	A-2-6(1)	128	13	40	50
	OD	A-2-4(0)	129	12	27	29
10	NMC	A-2-7(0)	131	9	81	72
	OD	A-2-6(0)	133	8	40	52
11	NMC	A-2-6(0)	135	10	18	26
	OD	A-2-4(0)	135	8	68	84

Table 8 Typical Values of c, ϕ, $\bar{c}$, $\bar{\phi}$ for Lateritic Soils from Khorat, Northeastern Thailand (after MOH & MAHMOOD, 1969)

Compaction water content, %	Moisture equilibriation time, hours	Total stress c (kg/cm^2)	Total stress ϕ (o)	Effective stress $\bar{c}$ (kg/cm^2)	Effective stress $\bar{\phi}$ (o)
19 (W_{opt})	24	0.385	25.1	0.49	32.9
21	0	0.33	23.2	0.34	35.3
21	24	0.014	24.1	0.10	37.9

Table 9 Shear Strength Parameters in terms of Total and Effective Stresses for the Clayey Sand from Chantaburi and Clayey Gravel from Saraburi (after BOONSRI, 1971).

Soil Number	Test Series	At $(\bar{\sigma}_1/\bar{\sigma}_3)$ max.								At 12% Strain							
		Total Stress				Effective Stress				Total Stress				Effective Stress			
		Strength Parameter		Mohr-Coulomb Parameter		Strength Parameter		Mohr-Coulomb Parameter		Strength Parameter		Mohr-Coulomb Parameter		Strength Parameter		Mohr-Coulomb Parameter	
		a psi	α deg.	c psi	ϕ deg.	$\bar{a}$ psi	$\bar{\alpha}$ deg.	$\bar{c}$ psi	$\bar{\phi}$ deg.	a psi	α deg.	c psi	ϕ deg.	$\bar{a}$ psi	$\bar{\alpha}$ deg.	$\bar{c}$ psi	$\bar{\phi}$ deg.
Soil No.1 Chantaburi	D1	3.4	20.6	3.7	22.1	2.0	31.0	2.5	37.0	10.0	21.7	10.9	23.4	0.0	29.7	0.0	34.7
	O1	3.1	28.1	3.7	32.2	5.0	31.6	6.3	38.0	32.0	27.0	37.2	30.6	0.0	30.2	0.0	35.6
	W1	4.5	21.0	4.9	22.6	3.0	32.4	3.9	39.4	40.0	20.0	42.9	21.4	6.0	29.4	7.2	34.3
Soil No.2 Saraburi	D2	5.0	18.8	5.3	19.9	0.0	36.1	0.0	36.1	10.0	17.9	10.6	18.8	0.0	29.5	0.0	34.5
	O2	2.2	25.5	2.5	28.5	1.4	31.6	1.8	38.0	12.0	31.7	15.2	38.2	8.0	27.4	9.3	31.2
	W2	5.2	16.7	5.5	17.5	7.0	26.5	8.1	29.9	16.6	16.6	17.4	17.3	0.0	30.1	0.0	35.4

Table 10 Undrained Strength Parameters of the Lateritic Soils from Chantaburi and Saraburi (after BOONSRI, 1971).

Series	At $\bar{\sigma}_1 / \bar{\sigma}_3$		At 12% Strain	
	C_u psi	ϕ_u Degree	C_u psi	ϕ_u Degree
D1	7.0	31.0	15.0*, 46.0**	36.3*, 11.3**
O1	14.6	45.0	56.0*, 110.0**	56.0*, 29.2**
W1	7.8	32.0	62.0	29.6
D2	9.0	25.0	16.0	26.1
O2	8.0	42.0	62.0	41.5
W2	8.0	23.9	24.0	22.6

* Consolidation Pressure Range from 0 to 60 psi

* * Consolidation Pressure Range from 60 to 100 psi

<u>Note</u>: $Su = C_u + \bar{\sigma}_c \tan \phi_u$

Table 11 Shear Strength Parameters in terms of Effective Stresses of the Clayey Gravel from Chantaburi (after YONG, 1975).

Test Series Compacted	At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$				At 12.5 % Strain			
	Saturated		Unsaturated		Saturated		Unsaturated	
	$\bar{c}$ lb/in^2	$\bar{\phi}$ deg.	$\bar{c}$ lb/in^2	$\bar{\phi}$ deg.	$\bar{c}$ lb/in^2	$\bar{\phi}$ deg.	$\bar{c}$ lb/in^2	$\bar{\phi}$ deg
Dry of opt.	1.3	37.6	3.5	30.9	1.4	36.9	4.2	30.6
At opt.	0.3	38.7	3.2	33.7	0.7	36.6	3.2	33.7
Wet of opt.	2.7	35.1	2.6	32.3	3.2	33.7	2.6	32.3

Table 12 Undrained Strength Parameters of the Clayey Gravel from Chantaburi. (after YONG, 1975)

Test Series Compacted	At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$				At 12.5 % Strain			
	Saturated		Unsaturated		Saturated		Unsaturated	
	Cu lb/in^2	ϕu deg.	Cu lb/in^2	ϕu deg.	Cu lb/in^2	ϕu deg.	Cu lb/in^2	ϕu deg.
Dry of Opt.	8.0	19.0	8.9	38.0	12.9	20.0	11.5	37.9
At Opt.	10.9	22.2	14.1	29.2	28.8	20.0	18.3	29.8
Wet of Opt.	4.0*	24.5*	5.0*	25.0*	8.0*	23.0*	7.5*	23.5*
	19.0**	11.0**	18.8**	13.0**	41.0**	2.5*	36.0**	5.1**

* Consolidation pressure before 60 lb/in^2

** Consolidation Pressure after 60 lb/in^2

Table 13(a) Index Properties of Weathered Bangkok Clay (0.20 m - 1.00 m)

Property	Piangpongpun (1977)	Haque (1977)	Lellasithorn (1978)	Liew (1979)
Physical Properties:				
Colour	grey & reddish	grey	reddish grey	grey
Consistency	stiff	stiff	stiff	stiff
In-situ water content, %	30	34	21	30
Grain Size Distribution				
Sand %	8±3	5±1	4	17±2
Silt %	47±3	46±1	36	48±2
Clay %	45±3	49±2	60	35±2
Specific Gravity	2.71	2.68	2.71	2.70
Liquid Limit %	66±1	62±1	62	60±1
Plastic Limit %	28±1	27±1	27	24±1
Plasticity Index %	38±1	35±1	35	35±1
Liquidity Index %	0.05	0.2	0.17	0.17
Activity	-	0.6	-	-
Chemical Properties:				
pH	3.66	3.88	5.05	-
Total Soluble Salt Content g/litre	3.53	3.39	3.76	-
Organic Carbon; %	0.68	0.84	0.68	-
Organic Matter; %	1.17	1.44	1.17	-
Standard Compaction Results:				
Optimum Water Content %	24.7	24.1	24.8	23.8
Optimum Dry Density t/m³	1.55	1.55	1.55	1.55

Table 13(b) Index Properties of Weathered Bangkok Clay (1.00 m - 3.00 m)

Property	Depth 1.00 - 2.00 m		Depth 2.00 - 3.00 m	
	Gulachol (1970)	Kanjanophas (1969)	Kanjanophas (1969)	Qureshi and Moh (1973)
Physical Properties:				
Colour	Dark Brown	Brownish Grey	Brownish Grey	Brownish Grey
Consistency	stiff	stiff	stiff	stiff
In-situ Water Content	51.2±6.9	54.3	62.9±4.2	79.7±7.4
Degree of Saturation	98.2±1.8	-	-	97.2±2
Natural Void Ratio	1.37±0.33	-	-	2.07±0.47
Grain Size Distribution;				
Sand %	2.5±0.5	3	2	1.5±0.5
Silt %	30.0±1.0	29	33	44.0±1.0
Clay %	67.5±1.5	68	65	54.5±0.5
Specific Gravity	2.71±0.1	2.69	2.69	2.67±0.1
Liquid Limit %	77.5±2.1	84.0	89.6±5.1	84.5±3.1
Plastic Limit %	33.9±0.5	39.6	38.6±2.4	33.2±6.4
Plasticity Index %	43.6±2.6	44.4	51.0±7.5	51.5±9.6
Liquidity Index	0.40	0.33	0.47	0.74±0.20
Activity	0.65	-	-	0.85±0.09
Chemical Properties:				
pH	3.4±0.2	3.35	6.40	-
Total Soluble Salt Content g/litre	6.6±0.1	6.57	7.23	-
Organic Matter %	1.2±0.3	1.23	7.18	-

Table 14 General Properties of Weathered Bangkok Clay
(After BUKKANASUTA, 1986)

Sample no.	Soil description	w_n (%)	G	γ_T kN/m	%passing no. 200	PL (%)	PI (%)	$\gamma_{d,max}$ kN/m	$w_{opt.}$ (%)
1	Light-brown, clayey sand (SC)	15.65	2.70	1.82	36.01	12.2	18.3	1.88	15.0
2	Reddish-brown, weathered clay (CH)	42.28	2.72	1.72	96.99	25.2	35.5	1.57	23.5

Table 15 Effect of Method of Preparation on Weathered Clay
(After PLANGPONGPUN, 1977)

Test no.	sample	w_L %	w_P %	PI %	w_S %	w_n %
1	Moist	67	29	38	16	30
2	sample	66	28	38	16	30
3		65	27	38	16	29
average		66	28	38	16	30
4	Air	62	26	36	16	6
5	dried	63	26	37	16	5
6	sample	63	26	37	16	6
average		63	26	37	16	6
7	Oven	61	26	35	15	-
8	dried	62	26	36	16	-
9	sample	63	27	36	16	-
average		62	26	36	16	-

Table 16 Compressiblity Parameter of Weathered Bangkok Clay (After YUAN, 1982)

a) Compressibility characteristics of AIT Campus

depth	eo			Cc			Cs			Pc (t/m²)		
(m)	n	μ	σ	n	μ	σ	n	μ	σ	n	μ	σ
1.0	5	1.21	0.11	4	0.52	0.23	4	0.10	0.02	5	22.6	2.8
1.5	7	1.73	0.20	2	0.88	0.64	1	0.08	-	2	17.8	10.3
2.0	4	1.98	0.25	18	0.90	0.24	17	0.16	0.03	23	16.9	8.2
2.5	5	2.15	0.15	4	1.06	0.38	3	0.14	0.05	7	10.9	3.8
3.0	3	2.28	0.22	7	1.14	0.25	3	0.19	0.05	7	9.4	2.1
3.5	4	2.36	0.14	4	1.26	0.27	3	0.19	0.04	5	8.2	1.4
4.0	5	2.28	0.17	12	1.17	0.20	6	0.19	0.03	11	10.0	3.6
4.5	11	2.26	0.16	12	1.31	0.21	10	0.20	0.03	12	9.0	1.8
5.0	14	2.30	0.15	18	1.22	0.26	10	0.18	0.05	18	9.4	1.9

b) Coefficient of primary consolidation Cv (10^{-4} cm²/sec.)

depth	AIT Campus			Nong Ngoo Hao			Pathum Wan			Pom Prachul		
(m)	n	μ	σ	n	μ	σ	n	μ	σ	n	μ	σ
0-1	2	2.5	-	-	-	-	-	-	-	1	1.0	-
1-2	18	1.9	1.6	2	2.0	-	-	-	-	5	3.3	2.1
2-3	18	1.3	0.7	4	1.5	1.0	-	-	-	3	3.5	4.3
3-4	5	1.7	0.7	-		-	1	2.0	-	-	-	-
4-5	16	1.1	0.5	1	1.0	-	-	-	-	1	2.5	-

n : sample size
μ : mean value
σ : standard deviation

Table 17 - **Undrained Strength Parameters**
(after ARAYASIRI, 1969)

Depth ft.	Apparent Overconsolidated		Normally Consolidated		Overconsolidated	
	c_u, lb/in^2	ϕ_u degree	c_u, lb/in^2	ϕ_u degree	c_u, lb/in^2	ϕ_u degree
4.75*	3.5	9	0	14	-	-
4.0-6.0	2.4	16.2	0	18.3	2.4	16.2
13.0*	2.8	9.5	0	15	-	-
11.0-13.0	1.4	13.5	0	19.3	1.8	13.5

* Isotropic consolidation reported by ARAYASIRI (1969)

Table 18 - **Undrained Shear Strength Parameters of the Weathered Bangkok Clay, (K_o Anisotropic Consolidation)**
(after GULYCHOL, 1970)

Soil	Depth, ft	Total Stress				Effective Stress			
		a	α	c	ϕ	$\bar{a}$	$\bar{\alpha}$	$\bar{c}$	$\bar{\phi}$
AOCC	4.0-6.0	1.5	18.2	1.6	19.2	1.6	20.8	1.7	22.3
	11.0-13.0	0.5	18.2	0.6	19.2	1.0	20.5	1.1	22.0
NCC	4.0-6.0	0	21.8	0	23.6	0	22.8	0	24.8
	11.0-13.0	0	18.9	0	20.0	0	24.4	0	27.0
OCC	4.0-6.0	0.5	19.3	0.6	20.5	1.6	21.8	1.7	23.6
	11.0-13.0	0.2	18.0	0.3	19.0	1.0	23.6	1.1	25.9

a, $\bar{a}$, c, $\bar{c}$ in lb/in^2

α, $\bar{\alpha}$, ϕ, $\bar{\phi}$ in degree

Table 19 Shear Strength Parameters in Terms of Total and Effective Stresses, Comparison Between Isotropic and Anisotropic Consolidation (after GULACHOL, 1970).

Test Series	Depth ft	Total / Stress				Effective Stress							
		At $(\sigma_1-\sigma_3)$ max				At $(\sigma_1-\sigma_3)$ max				At $(\bar{\sigma}_1/\bar{\sigma}_3)$ max			
		Strength Parameters		Mohr-Coulomb Parameters		Strength Parameters		Mohr-Coulomb Parameters		Strength Parameters		Mohr-Coulomb Parameters	
		a lb/in^2	α degree	c lb/in^2	ϕ degree	$\bar{a}$ lb/in^2	$\bar{\alpha}$ degree	$\bar{c}$ lb/in^2	$\bar{\phi}$ degree	$\bar{a}$ lb/in^2	$\bar{\alpha}$ degree	$\bar{c}$ lb/in^2	$\bar{\phi}$ degree
$\overline{CIAU}$ *	4.75	2.8	8.0	2.8	8.1	2.5	12.6	2.6	12.9	2.5	12.6	2.6	12.9
$\overline{CAAU}$	4.0 to 6.0	1.5	18.2	1.6	19.2	1.6	20.8	1.7	22.3	1.6	20.8	1.7	22.3
$\overline{CIU}$ *	4.75	0	11.0	0	11.2	0	16.5	0	17.2	0	16.5	0	17.2
$\overline{CAU}$	4.0 to 6.0	0	21.8	0	23.6	0	22.8	0	24.8	0	23.8	0	26.2
$\overline{CIAU}$ *	13.0	2.2	8.0	2.2	8.1	2.3	13.0	2.4	13.4	2.3	13.0	2.4	13.4
$\overline{CAAU}$	11.0 to 13.0	0.5	18.2	0.6	19.2	1.0	20.5	1.1	22.0	1.0	20.5	1.1	22.0
$\overline{CIU}$ *	13.0	0	12.0	0	12.3	0	19.0	0	20.2	0	19.0	0	20.2
$\overline{CAU}$	11.0 to 13.0	0	18.9	0	20.0	0	24.4	0	27.0	0	25.0	0	27.8
$\overline{CAOU}$	4.0 to 6.0	0.5	19.3	0.6	20.5	1.6	21.8	1.7	23.6	1.6	21.8	1.7	23.6
$\overline{CAOU}$	11.0 to 13.0	0.2	18.0	0.3	19.0	1.0	23.6	1.1	25.9	1.0	24.2	1.1	26.7

* Isotropic Consolidation (ARAYASIRI, 1969)

Table 20 Undrained Shear Strength Parameters of Weathered Bangkok Clay (After HOH et al, 1969)

Depth m	γ_m ton/m^2	Total Stress			Effective Stress		
		c ton/m^2	ϕ_n * degree	ϕ_o ** degree	$\bar{c}$ ton/m^2	$\bar{\phi}_n$ degree	$\bar{\phi}_o$ degree
1.1	27.7	0.5	13	-	0.5	20	-
1.5	25.6	2.0	11	9	1.8	18	13
2.5	24.6	0.8	11	10	0.7	20	19
4.0	17.5	1.4	13	9	1.5	21	16
5.3	0	0	11	-	0	20	-
10.5-11.7	6.1	6.1	15	10	3.7	24	20

* ϕ_n for normally consolidated soil

** ϕ for over consolidated soil

Table 21 Shear Strength Parameters of Bangkok Clay (After QURESHI and MOH, 1973)

Test Series	History	Type of Test	Depth m	Total Stresses				Effective Stresses							
				At $(\sigma_a - \sigma_r)_{max}$				At $(\sigma_a - \sigma_r)_{max}$				At $(\bar{\sigma}_a/\bar{\sigma}_r)_{max}$			
				Strength Parameters		Mohr-Coulomb Parameters		Strength Parameters		Mohr-Coulomb Parameters		Strength Parameters		Mohr-Coulomb Parameters	
				a	α	c	ϕ	$\bar{a}$	$\bar{\alpha}$	$\bar{c}$	$\bar{\phi}$	$\bar{a}$	$\bar{\alpha}$	$\bar{c}$	$\bar{\phi}$
				lb/in^2	degree	lb/in^2	degree	lb/in^2	degree	lb/in^2	degree	lb/in^2	degree	lb/in^2	degree
$\overline{CAU}$-I	AOCC	$\overline{CAU}$-V	2.0-2.5	1.0	17.0	1.0	17.8	2.0	16.0	2.1	16.7	2.0	16.5	2.1	17.2
	AOCC	$\overline{CAU}$-H	2.0-2.5	0.4	12.5	0.4	12.8	0.8	24.0	0.9	26.4	0.8	24.0	0.9	26.4
*	AOCC	$\overline{CAU}$-V	1.8	1.5	18.2	1.6	19.2	1.6	20.8	1.7	22.3	1.6	20.8	1.7	22.3
**	AOCC	$\overline{CIU}$-V	2.5	0.8	10.5	0.8	10.7	1.2	19.5	1.3	20.7	1.2	19.5	1.3	20.7
+ $\overline{CAU}$-II	AOCC	$\overline{CAU}$-V	5.0-5.5	4.0	7.5	4.0	7.6	3.7	10.0	3.8	10.2	3.7	10.0	3.8	10.2
	AOCC	$\overline{CAU}$-H	5.0-5.5	4.0	3.0	4.0	3.0	3.7	5.0	3.7	5.0	3.7	5.0	3.7	5.0
	NCC	$\overline{CAU}$-V	5.0-5.5	0.0	17.0	0.0	17.8	0.0	21.5	0.0	23.2	0.0	22.5	0.0	24.4
	NCC	$\overline{CAU}$-H	5.0-5.5	0.0	11.0	0.0	11.2	0.0	19.0	0.0	20.0	0.0	19.1	0.0	20.2
**	NCC	$\overline{CIU}$-V	5.3	0.0	11.0	0.0	11.2	0.0	20.0	0.0	21.4	0.0	21.5	0.0	23.2
+ $\overline{CAU}$-III	NCC	$\overline{CAU}$-V	7.0-7.5	0.0	20.0	0.0	21.4	2.9	19.0	3.1	20.2	2.9	19.0	3.1	20.2
	NCC	$\overline{CAU}$-H	7.0-7.5	0.0	14.0	0.0	14.4	3.3	16.0	3.4	16.7	3.3	16.0	3.4	16.7

* GULACHOL(1970)
** ARAYASIRI(1970)
\+ The strength parameters were based on the average envelope.

Table 22 Summary of the Strength Parameters and Strength Parameters from CIU and CID Triaxial Tests, Samples Compacted at Optimum Water Content (w = 24.1%) with 56 Blows per Layer (After HAQUE, 1977)

Type of Test	Effective Cosolidation Pressure t/m.²	Strength		Strength Parameters			
		Unsoaked	Soaked	Unsoaked		Soaked	
		t/m.²	t/m.²	$\bar{c}$ t/m.²	$\bar{\phi}$	$\bar{c}$ t/m.²	$\bar{\phi}$
CIU	15	9.9	9.9	At $(\sigma_1 - \sigma_3)$ max.			
	25	15.4	13.2	2.8	24.9°	0.8	27.1°
	35	20.5	19.9	At $(\bar{\sigma}_1 / \bar{\sigma}_3)$ max.			
				2.2	25.9°	1.4	26.4°
CID	15	-	18.9	At $(\sigma_1 - \sigma_3)$ max.			
	25	-	25.8	or $(\bar{\sigma}_1 / \bar{\sigma}_3)$ max.			
	40	-	38.2	-	-	4.6	25.5°

Table 23 Typical Tensile Strength of Bamboo (after DURRANI, 1975).

Specimen	Ultimate Tensile Stress (psi)	Modulus of Elasticity (psi) x 10^6	Remarks
Without node	18,800	2.0	Fractured at the centre
"	30,700	2.3	"
"	21,600	2.2	"
"	20,300	2.1	"
"	29,570	2.3	"
"	23,800	2.2	"
With node	18,300	2.0	Failure at node
"	19,850	1.9	"
"	24,100	2.1	"
"	12,200	1.9	"
"	19,900	1.9	"
"	17,900	2.0	"

Average Tensile Strength at nodes = 18,700 psi

Average Tensile Strength of internodes = 24,130 psi

Average modulus of elasticity = 2.11×10^6 psi

Table 24 Tensile Strength Test Results of Various Locally Manufactured Steel Bars
After AIT Structural Engineering and Construction Division, 1989

Diameter (mm)	Type or Grade	Unit Weight kg/m	Yield Stress kgf/mm²	Tensile Strength kgf/mm²	Elongation (%)	Local Manufacturer in Thailand	Remarks
[illegible]	-	[illegible]	[illegible]	[illegible]	[illegible]	-	
4		[illegible]	[illegible]	[illegible]	[illegible]	Thai Special Wire Co.,Ltd.	
4	-	0.10	50.21	62.76	4	-	Spiral Wire
4	-	0.10	44.35-44.70	51.88-54.39	4-5	-	Spiral Wire
5	SR24	0.152-0.153	61.10-62.37	63.65-64.65	4.7-9.4	SIS Factory	
5	WIRES	0.15	174.13-[illegible]	195.35-196.35	7.0	SSS Factory	
5	PCWIRE	0.154	168.02-169.04	179.74-180.75	6	Thai Special Wire Co.,Ltd.	
5	PCWIRE	0.15	166.50-170.57	181.26-182.79	5.5-6.4	TSW Factory	
6	SR24	0.219-0.229	35.90-39.52	45.23-55.80	16-27	BISW Factory	
6	SR24	0.52	39.92-40.55	52.81-59.42	29-[illegible]	GSS Factory	
6	SD40	0.23	33.60-35.37	53.06-55.18	25	GSS Factory	
6	SR24	0.22	29.87-34.78	39.35-44.52	30-33	OSS Factory	
6	SR24	0.205-0.220	24.76-39.62	42.30-48.11	22-32	SIS Factory	
6	SR24	0.22	32.90-34.67	48.11-49.52	20-25	SSS Factory	
6	SR24	0.16-0.17	35.53-36.49	46.09-48.97	17-27	WI Factory	
6	SR24	0.207-0.212	57.91-63.59	83.27-87.06	10-13	-	
7	PCWIRE	0.30	158.42-159.56	174.12-176.22	6.0-6.5	BSI Factory	
7	PCWIRE	0.302	161.12-162.68	178.02-179.05	6-7	Thai Special Wire Co.,Ltd.	
7	PCWIRE	0.30	153.30-159.80	171.23-176.17	5-7	-	
7	PCWIRE	0.30	157.22-159.56	172.56-174.38	6-7	-	
9	SR24	0.47-0.50	38.04-44.04	52.50-59.48	16-33	BISW Factory	
9	SR24	0.48-0.49	35.05-37.41	48.10-49.36	33-40	BSI Factory	
9	SR24	0.518-0.525	37.92-38.53	52.38-53.28	28-36	General Steel Co.,Ltd.	
9	SR24	0.470-0.499	33.01-40.24	46.53-55.49	24-38	GS Factory	
9	SR24	0.422-0.502	33.21-55.63	45.67-66.82	31-40	MSI Factory	
9	SR24	0.51-0.52	38.44-40.28	48.89-49.20	33-38	OSS Factory	
9	SR24	0.51	37.10-37.41	50.30-51.08	24-29	SIS Factory	
9	-	0.35-0.36	31.75-33.07	44.31-45.41	31-42	SM Factory	PC Seven Wire

Diameter (mm)	Type or Grade	Unit Weight kg/m	Yield Stress kgf/mm²	Tensile Strength kgf/mm²	Elongation (%)	Local Manufacturer in Thailand	Remarks
9	SR24	0.47-0.48	43.08-50.92	48.38-56.90	36	SSW Factory	
9	SR24	0.494-0.495	33.05-34.39	14.90-15.27	37-38	-	
9.62	270K	0.43	182.10-183.90	182.10-183.91	4.9-5.08	SSS Factory	
9.62	250K	0.40	168.92-169.88	168.92-169.88	4.9-5.2	SSS Factory	
9.62	270K	0.44	191.65-197.35	204.31-205.77	5.33-6.03	-	
9.62	270K	0.437-0.439	194.92	203.95-205.03	6.97-7.21	-	
9.62	270K	0.44	195.69-196.05	201.79-202.51	5.90-6.89	-	PC Seven Wire
9.62	270K	0.44	185.82-188.51	201.79-202.87	6.15-7.54	-	PC Seven Wire
9.62	270K	0.43	179.53	197.49	7.1-7.9	-	PC Seven Wire
9.62	270K	0.44	181.69-183.94	201.79-202.51	6.8-7.5	-	
10	SD30	0.60-0.61	37.68-39.38	54.43-55.08	26-32	BISW Factory	
10	SD40	0.62	42.80-43.81	61.53-54.96	28-30	BSI Factory	
10	SD30	0.64-0.66	38.53-39.74	55.12-55.56	30-31	GSS Factory	
10	SD40	0.64-0.67	40.68-47.81	64.76-73.74	20-29	GSS Factory	
10	-	0.55	39.68-41.39	55.38-56.09	30-34	NTS Factory	
12	SD30	0.87-0.90	35.28-45.86	51.16-67.03	27-37	BISW Factory	
12	SD30	0.81-0.88	36.06-37.70	52.27-57.69	26-30	BSI Factory	
12	SD40	0.902-0.917	40.06-40.37	58.56-58.89	27-29	BSI Factory	
12	SD30	0.88-0.89	39.96-40.04	57.27-57.33	27-28	GSS Factory	
12	SD40	0.87-0.92	45.08-48.72	66.65-76.09	20-25	GSS Factory	
12	SD40	0.829-0.872	43.44-46.79	65.44-71.30	20-27	Hanbu Steel Ind. Co.,Ltd.	
12	SD40	0.89	46.92-47.10	79.26	23-25	SSS Factory	
12	SD30	0.90	40.99-45.36	56.61-65.42	28	TICO Factory	
12.4	250K	0.73	172.54	188.29	6.8-7.1	BSI Factory	
12.4	270K	0.79	178.06-179.06	197.06-197.35	7.2-7.5	SSS Factory	
12.4	250K	0.73	177.94-179.01	189.80-190.23	7.5-8.4	SSS Factory	
12.4	270K	0.78	182.32-184.02	204.64-204.83	5.4-6.2	-	
12.85	270K	0.79	187.05-187.06	200.06-203.66	6.07-7.38	-	PC Seven Wire
12.85	270K	0.78	185.86-186.06	195.06-196.06	7.60-8.69	-	
16	SD30	1.52-1.59	36.43-40.27	53.16-57.70	25-35	BISW Factory	
16	SD40	1.570-1.580	43.79-44.64	66.58-69.02	20-25	BSI Factory	
16	SD30	2.46	36.06-38.93	53.61-57.44	28-30	BSI Factory	

Dia-meter (mm)	Type or Grade	Unit Weight kg/m	Yield Stress kgf/mm²	Tensile Strength kgf/mm²	Elongation (%)	Local Manufacturer in Thailand	Remarks
16	SD30	1.56-1.60	34.34-36.73	48.67-52.84	29-35	GSS Factory	
16	SD40	1.57-1.69	41.27-50.04	64.50-74.96	20-28	GSS Factory	
16	SD30	1.51-1.56	9.01-38.99	26.39-56.03	31.0-57.5	NTS Factory	PC Seven Wire
16	SD30	1.59-1.61	35.33-39.69	53.18-56.56	28-29	TICO Factory	
19	SR24	0.505-0.526	47.15-49.04	67.79-69.16	22-31	BSI Factory	
19	SR24	2.20	31.32-32.45	46.56-46.84	36	TPS Factory	
20	SD30	2.40-2.53	37.29-40.04	53.97-58.73	26-33	BISW Factory	
20	SD40	2.450-2.464	41.33-41.44	62.16-63.40	25-27	BSI Factory	
20	SD30	1.56-1.58	35.77-39.00	50.88-57.16	28-34	BSI Factory	
20	SD30	2.47-2.49	34.32-34.68	50.76-50.85	32-33	GSS Factory	
20	SD40	2.47-2.54	40.50-43.61	60.12-68.69	22-28	GSS Factory	
20	SD40	2.41-2.53	40.58-63.82	57.18-88.97	16-31	NTS Factory	
20	SD30	2.45-2.46	34.46-35.50	50.80-51.22	30-37	NTS Factory	
20	SD30	2.48-2.50	34.98-35.33	53.07-54.13	26-27	TICO Factory	
20	SD30	2.40	36.63-38.60	55.34-56.13	28-29	TPS Factory	
25	SD30	3.72-3.98	34.73-44.46	51.99-68.48	23-30	BISW Factory	
25	SD40	3.77-3.82	42.25-43.51	63.38-68.29	23-26	BISW Factory	
25	SD30	0.875-0.878	40.15-40.23	57.60-57.93	30-32	BSI Factory	
25	SD40	3.802-3.832	41.38-46.55	67.60-72.44	18-23	BSI Factory	
25	SD30	3.83-3.92	37.05-39.76	56.47-59.44	23-32	GSS Factory	
25	SD40	3.81-3.97	41.29-46.69	65.62-73.63	20-24	GSS Factory	
25	SD30	3.72-3.73	34.10-34.82	52.41-52.96	24-26	NTS Factory	
25	SD30	3.82-3.83	38.22-38.33	57.76-58.00	29-30	SIS Factory	
25	SD30	3.77-3.88	33.69-34.57	51.09-53.62	26-28	TICO Factory	
28	SD40	4.740-4.835	40.14-42.13	62.82-65.59	21-24	BSI Factory	
28	SD40	4.69-4.87	40.83-46.08	64.19-73.05	19-24	GSS Factory	
28	SD30	4.79	38.35	55.88-55.97	27-28	GSS Factory	
28	SD40	4.81-4.83	43.25-43.56	65.74-65.85	24-26	GSS Factory	
32	SD50	6.448-6.572	53.08-56.87	73.29-77.18	18-23	SIS Factory	

Table 25 Tensile Strength Test of Steel Welding Joint

S.No.	Weld Width (mm)	Weld Thickness (mm)	Bearing Load kgf	Tensile Strength kgf/mm²	Remarks
	37.6	10	18,240	4,851	Fracture occured outside
	38.0	10	18,280	4,811	Welding joint outside
	37.9	10	18,000	4,749	Welding joint outside
	38.0	10	17,480	4,600	Welding joint outside
	37.5	10	17,600	4,693	Welding joint outside

Table 26 Tensile Strength Test of Welded Steel Bars
(After AIT, Structural Engineering and Construction Division)

S.No.	Dia-meter (mm)	Unit Weight kg/m	Yield Stress kgf/mm²	Tensile Strength kgf/mm²	Elongation (%)	Modulus of Elasticity kgf/mm²	Remarks
1	25	3.85	43.92-45.67	60.69-66.40	20-24	20,370-28,400	

Table 27 Specifations for Deformed Steel Bars

(after TIS. 24- 1984)

Nominal Size	Nominal Diameter (mm.)	Unight Weight kg/m	Nominal X- Area (mm²)	Maximum Length Allowance (mm)		Weight Tolerances			Max. Chemical Composition (% by weight)						Mechanical Properties			Min. Elongation (%)
				10 m. and under	over 10 m.	for Average %		per piece %	Grade	C	Mn	P	S	C+ Mn/6	Grade	Min. Yield Stress kgf/mm²	Min.Tensile Stress kgf/mm²	
DB10	10	0.617	78	+55 mm. -0 mm.	Add 5 mm to the above for every additi-onal 1m. or fraction there of not to exc-eed a max. va-lue of 120 mm.	DB10	+3.5	+ 6.0	SD30	.28	-	.058	.058	.45	SD30	30	49	17
DB12	12	0.888	113			DB28	-		SD40	-	1.8	.058	.058	.55	SD40	40	57	15
DB16	16	1.578	201															
DB20	20	2.466	314															
DB22	22	2.984	380															
DB25	25	3.853	491															
DB28	28	4.834	616															

Table 28 Specifications for Round Steel Bars

Nominal Size	Nominal Diameter (mm)	Circumference (mm)	Nominal X-Area (mm²)	Unit Weight (kg/m)	Max. Length Allowance (mm)		Allowances for Weight			Max. Chemical Composition (% by weight)				Physical Properties			Min. Elongation %
					10m. and under	Over 10m.	for	Average %	per piece %	C	S	P		Grade	Min.Yield Stress kgf/mm²	Min.Tensile Str. kgf/mm²	
RB6	6	18.85	28.3	0.222	+55 mm. -0 mm.	Add 5 mm. to to the above for every additional 1m. or fraction there of next to exceed a maximum value of 120 mm.	RB6 RB9-RB28	±5.0 ±3.5	±10.0 ±16.0	.28	.058	.058		SR 24	24	39	21
RB9	9	28.27	63.6	0.499													
RB12	12	37.70	113.1	0.888													
RB15	15	47.12	176.7	1.387													
RB19	19	59.69	283.5	2.226													
RB22	22	69.12	380.1	2.984													
RB25	25	78.54	490.9	3.853													
RB28	28	87.96	615.8	4.834													

Table 29 Recommended Design Values of Yield Strength and Effective Modulus for Steel Meshes and Bars

		Woven Square Mesh	Welded Square Mesh	Hexagonal Mesh	Expanded Metal Lath	Longi u-dinal Bars
Yield Strength	f_y, ksi (MPa)	65 (450)	65 (450)	45 (310)	55 (380)	60 (414)
Effective Modulus	$(E_r)_{long.}$, 10^3ksi (10^3 MPa)	20 (138)	29 (200)	15 (104)	20 (138)	29 (200)
	$(E_r)_{tran.}$, 10^3ksi (10^3 MPa)	24 (165)	29 (200)	10 (69)	10 (69)	- -

Table 30 General Subsoil Properties at the Site (after MOY, 1984).

DEPTH (m)	W_n (%)	PL (%)	LL (%)	PI (%)	G_s	γ_t (t/m³)	e_o	S (%)	P'_o (t/m²)	P'_{max} (t/m²)	C_v x 10^{-4} cm²/sec ($\sqrt{t}$)	C_v x 10^{-4} cm²/sec (log-t)	C_c	C_s
0.5	24.7	23.7	50.4	26.7	2.70	1.87	0.8004	83.2	0.94	7.30	16.5	17.0	0.2226	0.0474
* 0.75	25.5					1.86	0.8393	83.6	1.40	13.0	5.0	1.0	0.2002	0.0402
1.5	43.7	36.0	78.8	42.8	2.67	1.71	1.2716	91.8	2.22	27.0	1.7	1.5	0.5970	0.1403
* 1.75	39.4					1.80	1.1034	95.3	2.40	32.0	1.5	0.4	0.3717	0.1157
2.5	77.0	39.5	83.8	44.3	2.63	1.48	2.3962	84.6	2.82	7.0	3.6	1.2	1.7575	-
* 2.75	88.1					1.50	2.3947	96.8	2.94	6.4	1.7	0.2	1.4920	0.1788
3.5	90.3	42.8	90.4	47.6	2.62	1.48	2.4451	96.8	3.30	10.2	1.1	1.0	1.5042	0.1329
* 3.75	89.7					1.48	2.4980	94.1	3.42	9.4	2.0	1.0	1.7154	0.1778
4.5	97.8	39.3	109.0	69.7	2.62	1.46	2.5281	101.4	3.77	3.8	1.2	1.0	0.9770	0.1871
* 4.75	90.5					1.45	2.7066	87.6	3.89	4.7	1.0	0.7	1.5329	0.1982
5.5	84.2	37.0	101.5	64.5	2.63	1.48	2.4121	91.4	4.24	8.5	0.9	0.8	1.4164	0.2200
* 5.75	86.6					1.50	2.1350	86.6	4.36	7.8	1.0	0.6	1.2497	0.1828
6.5	60.5	33.3	75.8	42.5	2.64	1.65	1.6174	98.8	4.81	8.6	2.0	1.5	1.0283	0.2624
* 6.75	60.8					1.61	1.6180	99.2	4.97	10.2	1.0	0.5	0.9367	0.1230

(* from Special Oedometer Test)

Table 31 General Subsoil Properties at the AIT Campus (after OHTA & HO, 1982)

DEPTH (m)	W_n (%)	PL (%)	LL (%)	PI (%)	G_s	γ_t (t/m³)	e_o	S (%)	P_o' (t/m²)	P_{max}' (t/m²)	C_v x10⁻⁴ cm²/sec ($\sqrt{t}$)	C_v x10⁻⁴ cm²/sec (log t)	C_c	C_s
0.5	30.0	20.9	44.0	23.0	2.62	1.93	0.86	91.5	0.97	2.7	26.0	12.0	0.146	0.020
1.4	62.6	31.3	83.2	52.0	2.67	1.96	1.595	104.5	1.36	4.6	26.0	10.0	0.407	0.103
1.75	78.2	38.2	104.5	66.0	2.71	1.62	2.283	92.9	1.58	10.0	50.0	4.0	1.684	0.168
2.4	83.5	33.9	88.5	54.0	2.65	1.64	3.170	69.8	1.99	5.6	2.0	1.0	1.630	0.240
2.75	95.3	38.2	107.5	69.0	2.63	1.55	2.570	97.3	2.18	5.0	3.0	2.0	1.420	0.210
3.35	90.2	36.8	109.0	72.0	2.70	1.56	2.659	91.5	2.52	6.5	2.0	1.0	1.580	0.238
3.9	84.7	34.8	91.4	56.0	2.66	1.61	2.317	97.3	2.85	9.0	3.0	2.0	1.590	0.185
4.25	95.2	38.8	114.5	76.0	2.68	1.57	2.627	97.3	3.05	7.2	0.1	0.1	1.580	0.250
5.3	90.8	36.5	103.1	67.0	2.65	1.58	2.444	98.6	3.66	9.5	1.0	1.0	1.104	0.154
5.8	80.5	37.0	100.5	63.0	2.69	1.69	2.280	94.8	4.00	6.4	1.5	0.5	0.780	0.220
6.4	66.8	[illegible]	104.3	67.0	2.63	1.70	1.414	124.2	4.42	12.3	2.0	1.0	1.530	0.220
6.8	67.3	32.1	68.4	36.0	2.71	1.68	1.683	108.4	4.69	9.2	1.0	1.0	0.827	0.172
7.4	63.2	30.8	73.4	43.0	2.66	1.71	2.202	76.3	5.12	3.1	4.0	0.5	1.000	0.175
7.8	35.3	33.6	83.7	50.0	2.67	1.92	1.109	85.0	5.49	8.0	7.0	5.0	0.400	0.101

Table 32 Summary of Triaxial Test Results (Anisotropically K_o Consolidated Undrained Triaxial Compression and Extension Tests) (after MOY, 1984).

No.	Depth (m)	$\sigma_1' = p_o'$ (t/m²)	$\sigma_3' = k_o p_o'$ (t/m²)	$\overline{CK_oU}$-comp.			$\overline{CK_oU}$-ext.			S_{uH}/S_{uV}
				S_u (t/m²)	S_u/p_o'	A_f	S_u (t/m²)	S_u/p_o'	A_f	
1	1.0	1.87	1.27	1.71	0.91	0.05	1.63	0.87	0.32	0.95
2	1.75	2.22	1.51	1.71	0.71	0.15	2.40	1.00	0.38	1.42
3	2.75	2.94	1.76	2.07	0.70	0.17	1.69	0.57	0.43	0.82
4	3.75	3.30	1.98	2.06	0.60	0.49	2.16	0.63	0.59	1.05
5	4.75	3.89	2.34	1.86	0.47	0.61	1.44	0.37	0.66	0.77
6	5.75	4.36	2.62	2.36	0.54	0.46	1.96	0.45	0.62	0.83
7	6.75	4.97	2.98	2.35	0.47	0.63	1.94	0.39	0.77	0.79

Table 33 Summary of Triaxial Test Results (Isotropically Consolidated Undrained Triaxial Tests) (after OHTA & HO, 1982).

Depth (m)	σ'_{vo} (kN/m^2)	p (kN/m^2)	Axial Strain												
			0%	0.2		0.5		1		2		5		10	
			p/σ'_{vo}	q/σ'_{vo}	p/σ'_{vo}	q/σ'_{vo}	p/σ'_{vo}	q/σ'_{vo}	p/σ'_{vo}	p/σ'_{vo}	q/σ'_{vo}	p/σ'_{vo}	q/σ'_{vo}	p/σ'_{vo}	q/σ'_{vo}
0.50	32	15	0.47	0.19	0.48	0.38	0.52	0.44	0.52	0.64	0.58	0.71	0.62	0.79	0.69
1.40	46	27	0.59	0.39	0.96	0.62	0.90	0.80	0.85	0.98	0.87	0.83	0.96	0.83	1.01
1.75	100	31	0.31	0.18	0.34	0.51	0.39	0.66	0.40	0.42	0.34	0.33	0.39	0.33	0.34
2.40	56	37	0.66	0.20	0.70	0.21	0.66	0.38	0.66	0.52	0.64	0.48	0.61	0.34	0.52
2.75	50	40	0.80	0.12	0.82	0.14	0.76	0.32	0.74	0.32	0.62	0.14	0.52	0.12	0.50
3.35	65	45	0.69	0.20	0.69	0.37	0.66	0.55	0.65	0.68	0.58	0.55	0.52	0.49	0.49
4.25	72	52	0.72	0.10	0.71	0.24	0.68	0.29	0.64	0.26	0.60	0.25	0.58	0.24	0.54
5.30	95	60	0.63	0.15	0.60	0.29	0.59	0.38	0.53	0.46	0.45	0.45	0.33	0.35	0.28
5.80	64	64	1	0.23	1.03	0.39	1.00	0.56	0.97	0.67	0.89	0.56	0.81	0.47	0.77
6.80	92	72	0.78	0.22	0.80	0.37	0.79	0.51	0.77	0.67	0.72	0.70	0.63	0.55	0.58
7.40	(31)	78	1	0.26	1.10	0.44	1.05	0.63	0.99	0.68	0.86	0.56	0.74	0.44	0.69
7.80	(80)	81	1	0.21	0.95	0.44	0.93	0.45	0.85	0.53	0.79	0.50	0.73	0.39	0.65

Table 34 Summary of Triaxial Test Results (Isotropically Consolidated Undrained Triaxial Tests) (after OHTA & HO, 1982).

Depth (m)	σ'_{vo} (kN/m^2)	p (kN/m^2)	Axial Strain															
			0%	0.2%		0.5%		1%		2%		5%		10%		15%	
			p/σ'_{vo}	q/σ'_{vo}	p/σ'_{vo}	q/σ'_{vo}	p/σ'_{vo}	q/σ'_{vo}	p/σ'_{vo}	q/σ'_{vo}	p/σ'_{vo}	q/σ'_{vo}	p/σ'_{vo}	q/σ'_{vo}	p/σ'_{vo}	q/σ'_{vo}	p/σ'_{vo}
0.50	150	150	1	0.21	0.91	0.38	0.99	0.56	0.83	0.67	0.69	0.76	0.59	0.78	0.55	0.87	0.60
1.75	150	150	1	0.05	0.90	0.22	0.80	0.29	0.74	0.32	0.65	0.27	0.50	0.24	0.46	0.24	0.43
2.40	150	150	1	0	0.57	0.07	0.52	0.15	0.14	0.21	0.37	0.23	0.25	0.20	0.60		
2.75	150	150	1	0	0.91	0.08	0.83	0.19	0.79	0.30	0.73	0.32	0.60	0.31	0.51	0.32	0.47
3.35	150	150	1	0.11	1.03	0.18	0.97	0.26	0.89	0.29	0.75	0.32	0.63	0.33	0.59		
3.90	150	150	1	0.04	0.95	0.15	0.81	0.23	0.75	0.30	0.69	0.35	0.55	0.29	0.50		
4.25	150	150	1	0.09	0.84	0.17	0.71	0.25	0.65	0.30	0.56	0.33	0.40	0.32	0.29	0.30	0.22
5.30	150	150	1	0.07	0.89	0.17	0.78	0.23	0.71	0.29	0.66	0.29	0.51	0.33	0.45		
5.80	150	150	1	0.07	0.94	0.16	0.88	0.24	0.80	0.33	0.67	0.38	0.54	0.39	0.42		
5.80	150	150	1	0.14	0.04	0.21	0.91	0.27	0.85	0.40	0.80	0.45	0.66	0.51	0.57	0.43	0.49
6.80	150	150	1	0.14	0.97	0.29	0.96	0.49	0.95	0.55	0.92	0.47	0.82	0.61	0.83	0.45	0.76
7.40	150	150	1	0.16	0.96	0.16	0.87	0.25	0.81	0.30	0.67	0.41	0.57	0.33	0.45		
7.80	150	150	1	0.03	1.03	0.21	0.99	0.57	0.78	0.64	0.58	0.63	0.45	0.63	0.29		

Table 35 Variation of the Friction Angle Based on Effective Stresses with Depth (after OHTA & HO, 1982).

OHTA & HO (1982)			Past Theses	
Depth (m)	ϕ' (degree)	ϕ' (degree)	Depth (m)	ϕ' (degree)
0.5	34.0	35.8	-	-
1.4	21.1		1.25	19.8
1.75	16.4	15.0	-	-
2.40	17.1	4.4	2.25	21.0
2.75	7.3	4.8	-	-
3.35	30.5	14.7	-	-
3.90	5.7	15.5	-	-
4.25	11.8	33.7	4.25	2.30
5.30	37.1	19.7	-	-
5.75	16.4	15.3	5.5	22.7
6.40	21.1	15.8	6.5	23.2
6.80	24.1		-	-
7.40	16.6	19.6	-	-
7.80	16.1		-	-
			8.25	21.5

Table 36 Variations of the Unconfined Compressive Strength ($q_u/2$) and E_{50} with Depth (after OHTA & HO, 1982).

Depth (m)	qu (kN/m^2)	qu/2 (kN/m^2)	ϵ	E_{50} (kN/m^2)
0.50	64.0	32.0	0.0185	1730
1.40	40.0	20.0	0.0050	4000
1.75	53.0	26.5	0.0065	4077
2.35	36.8	18.4	0.0035	5257
2.75	30.0	15.0	0.0070	2143
3.35	51.0	25.5	0.0075	3643
3.75	67.4	33.7	0.0060	5617
4.25	28.6	14.3	0.0076	1882
5.35	54.0	27.0	0.0060	4500
5.75	63.0	31.5	0.0082	3841
6.40	86.5	43.2	0.0100	4320
6.80	49.0	24.5	0.0095	2579
7.40	54.5	27.2	0.0080	3400
7.80	57.0	28.5	0.0080	3563

FIGURES

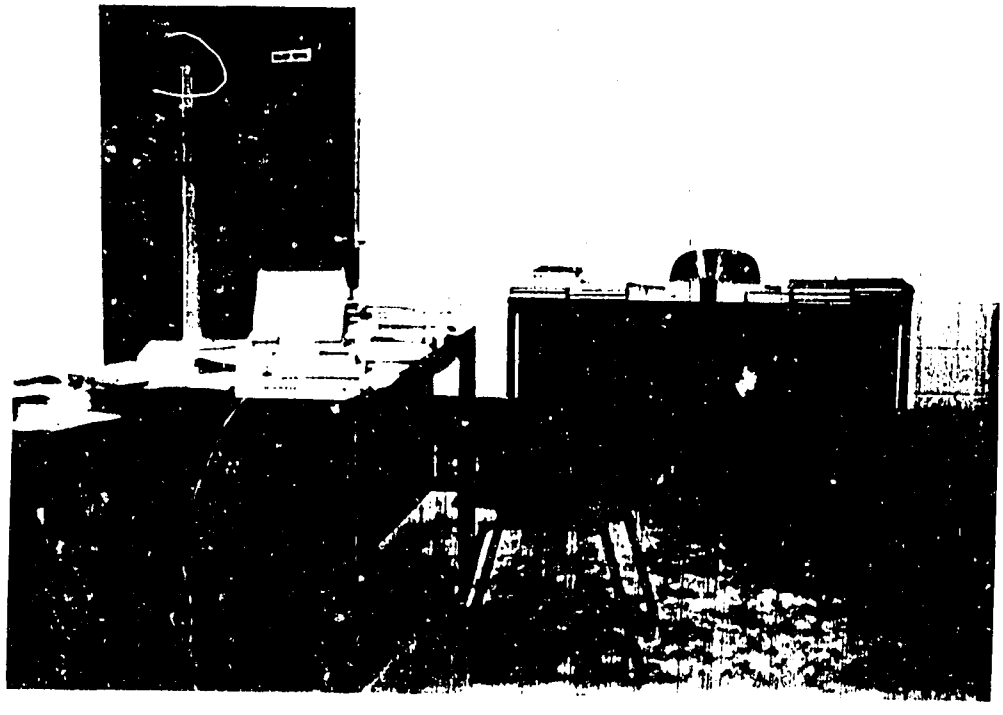


Fig.1 The Pullout Testing Room Showing Tables of Research Associates.

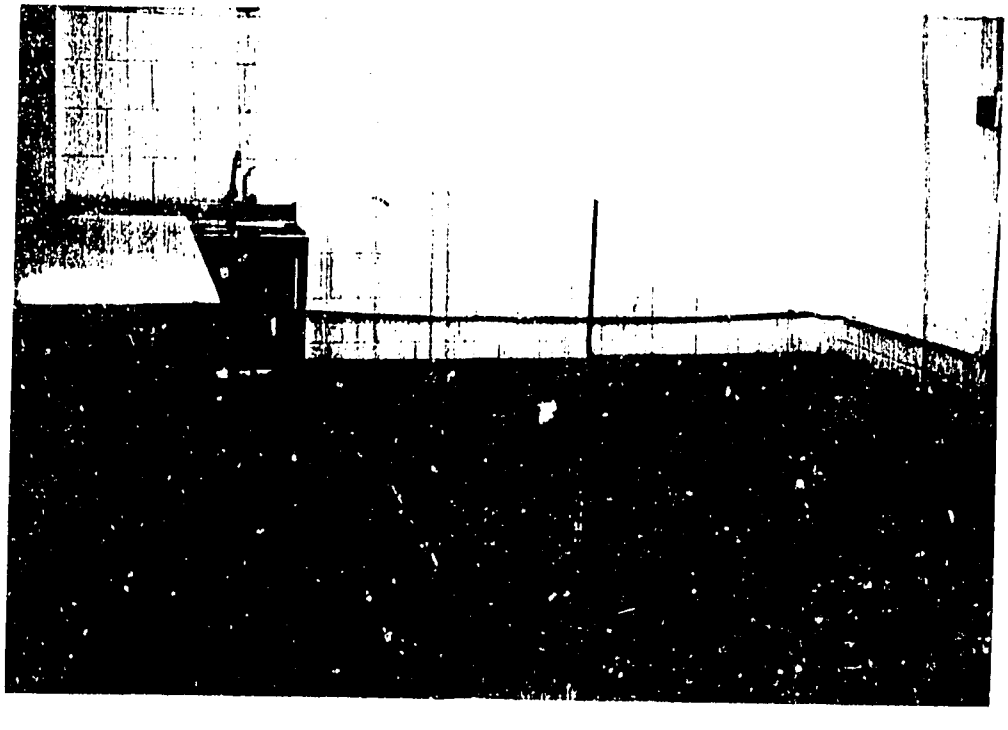


Fig.2 The Pullout Testing Room Showing the Future Location of the Pullout Apparatus.

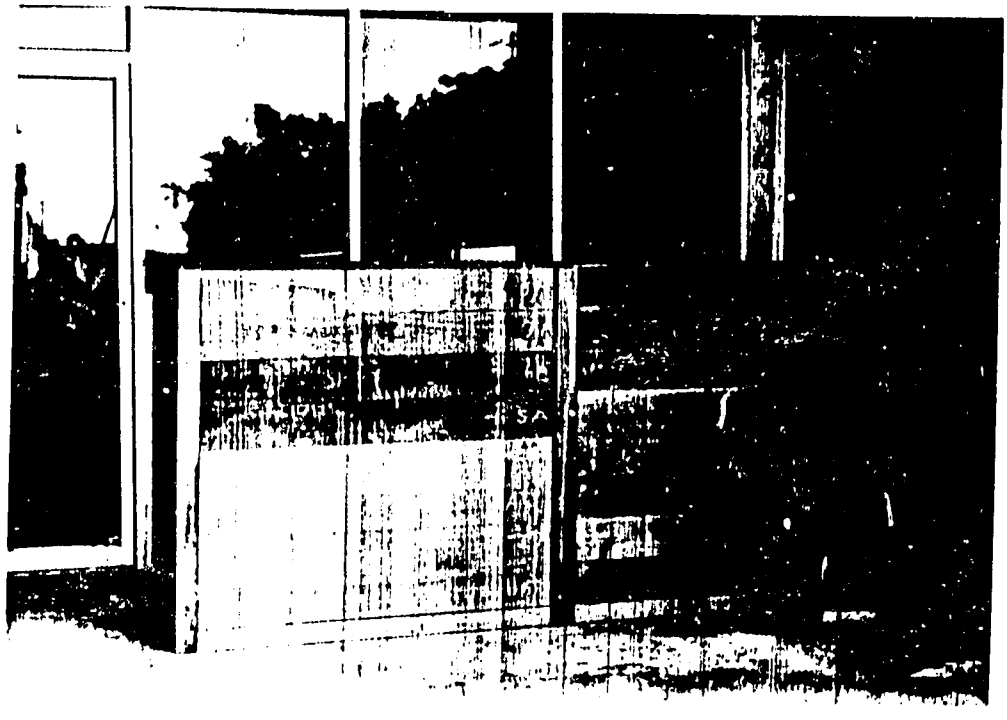


Fig.3 Container for Backfill Material Samples.

Fig.4 Site of Field Tests.

Fig.5 Another View of the Field Testing Site.

Fig.6 Lateritic Soil Samples for Backfill Materials.

Fig.7 Clayey Sand Samples for Backfill Materials.

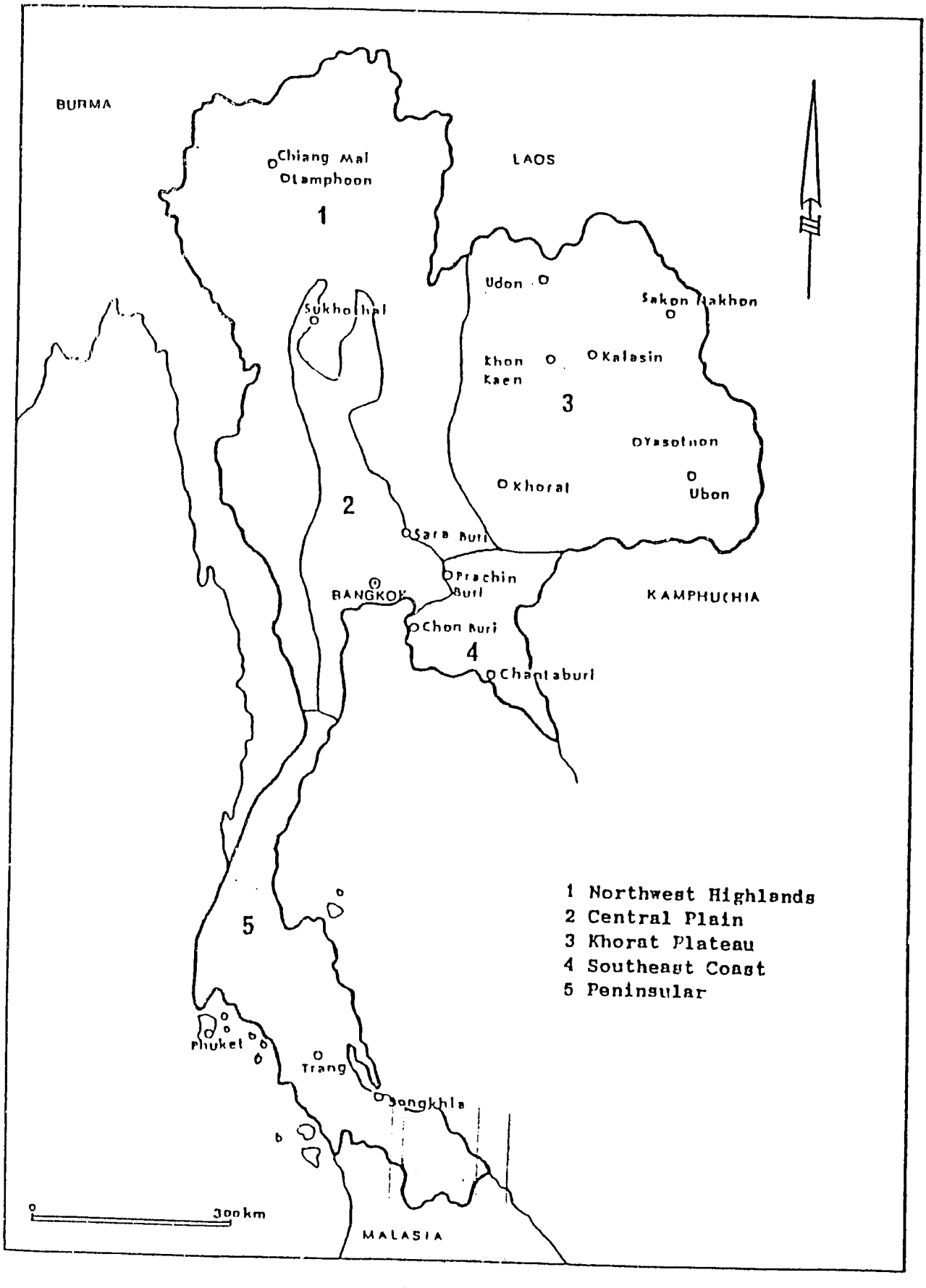


Fig.8 Physiographic Features of Thailand (after BOONTAN, 1984).

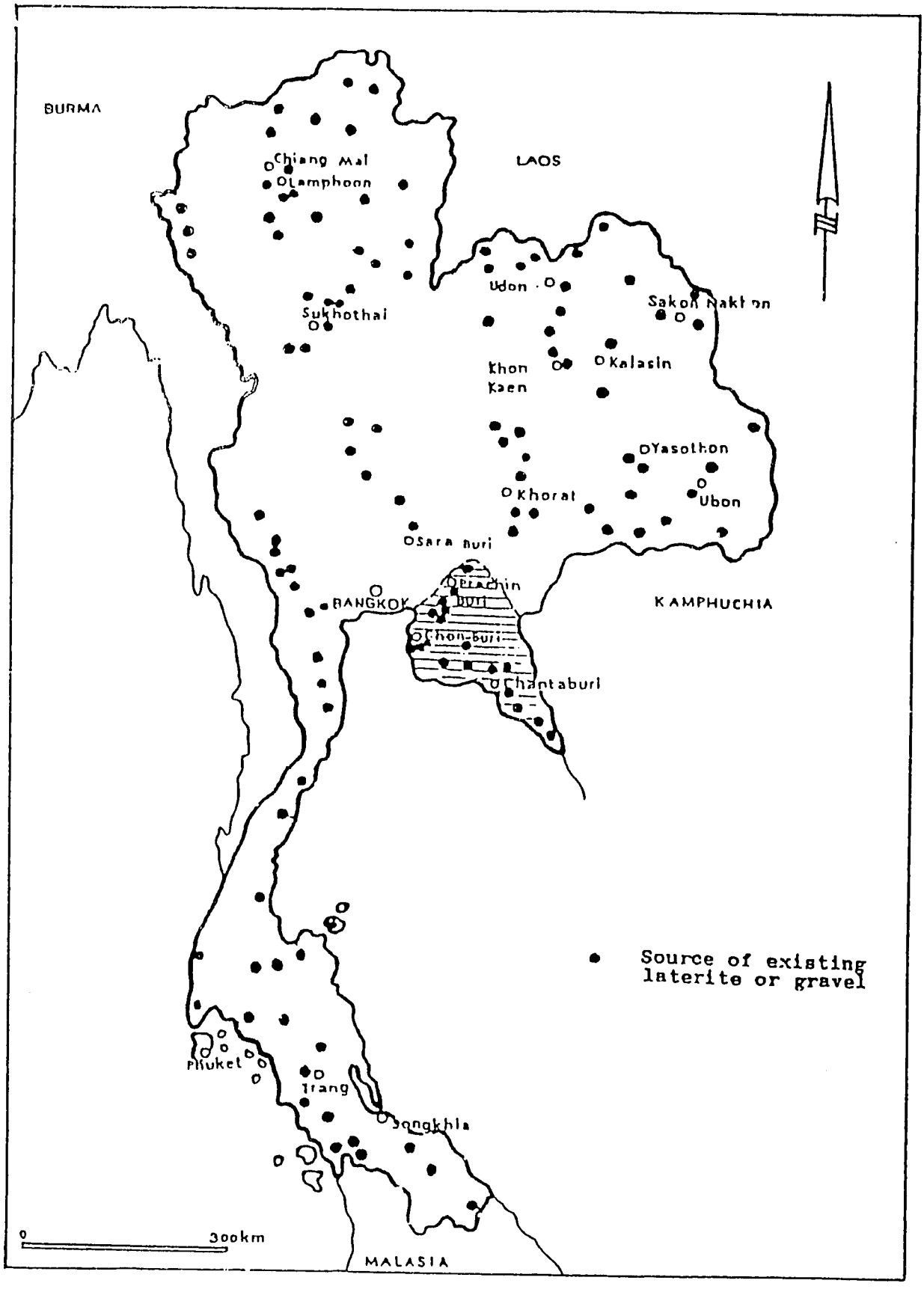


Fig.9 Distribution of Existing Laterites and Lateritic Soils in Thailand.

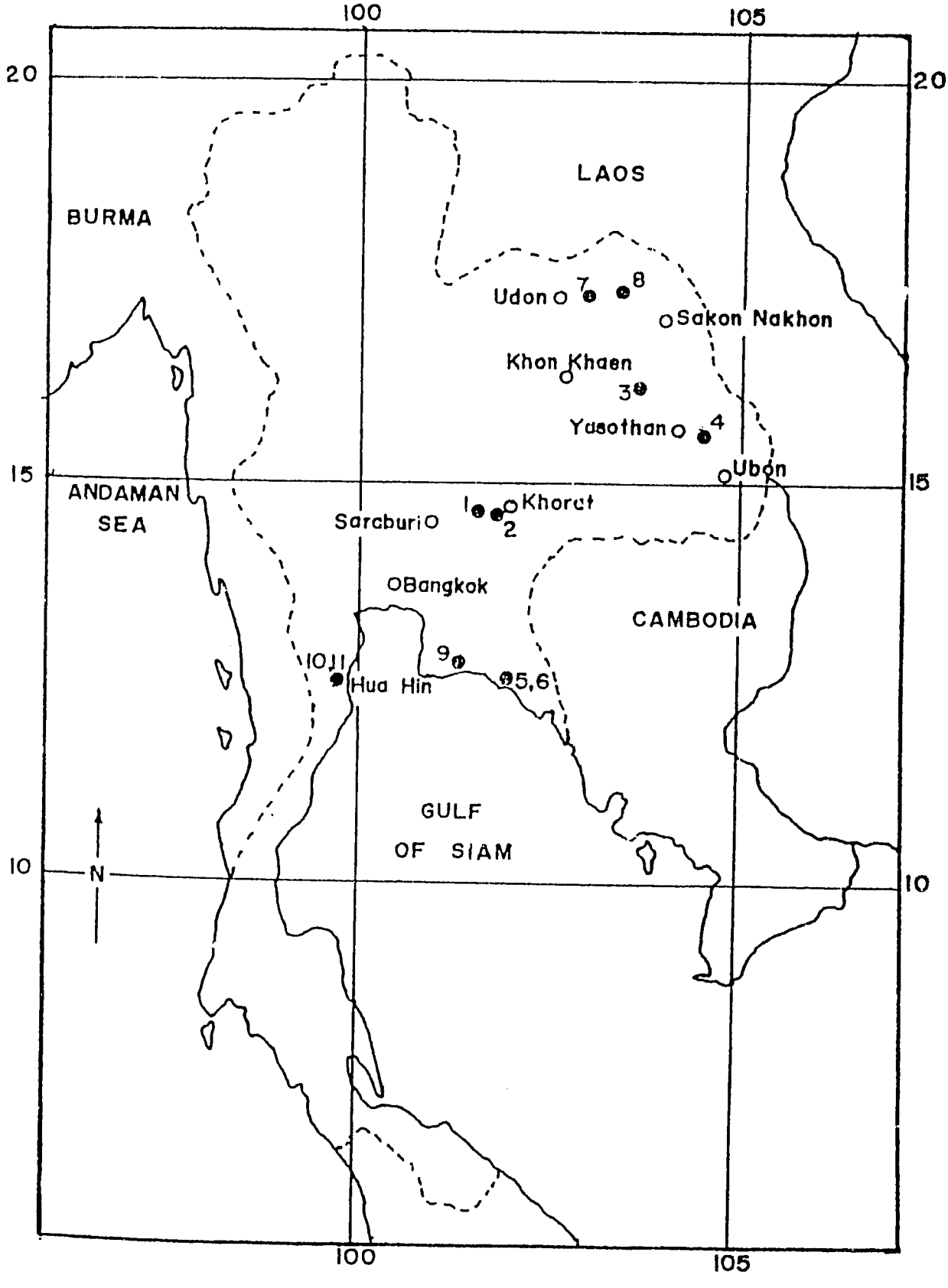


Fig.10 Map of Thailand Showing the Locations of Representative Lateritic Soils (after MAZHAR, 1969).

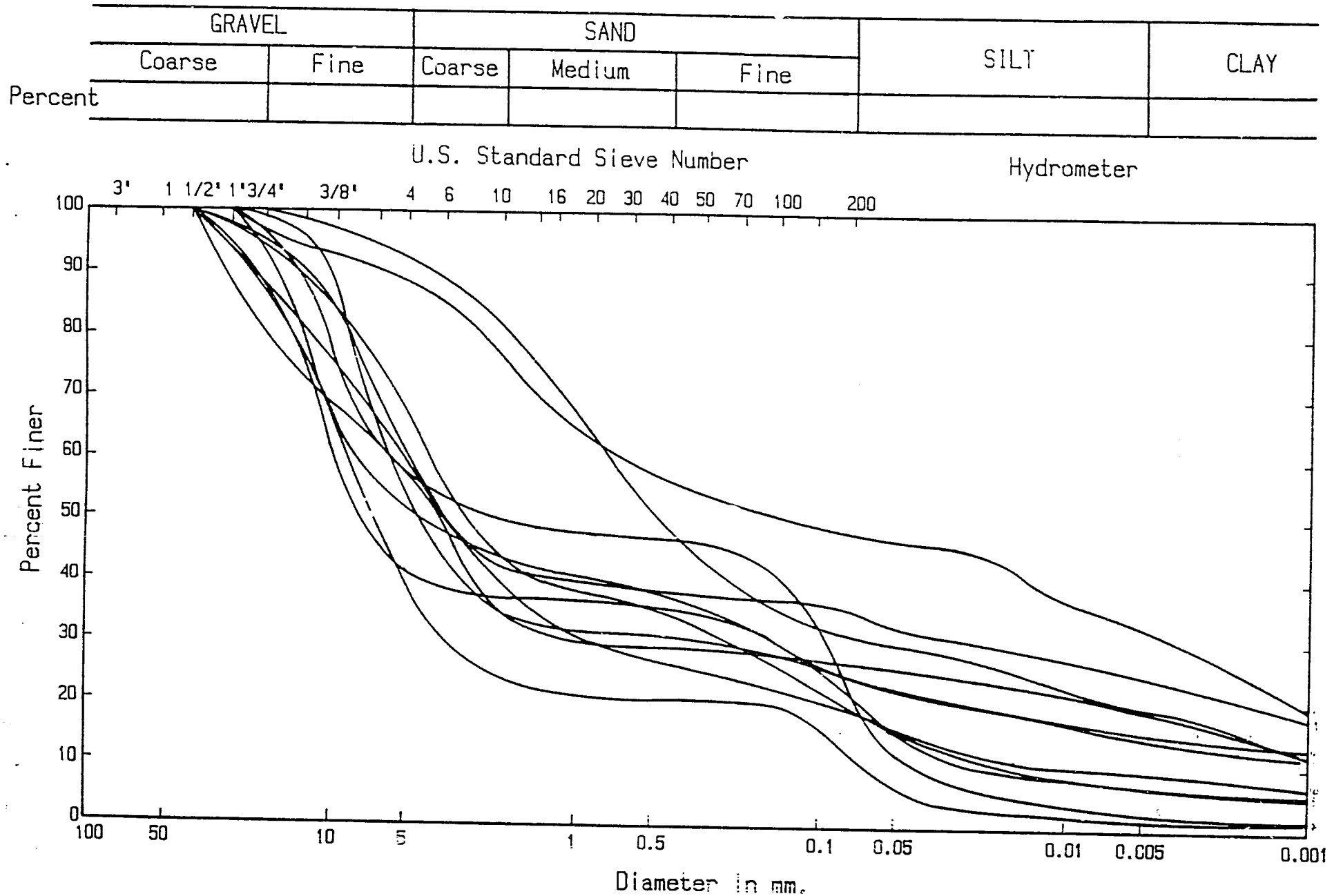


Fig.11 Particle Size Distributions of Representative Lateritic Soils of Thailand.

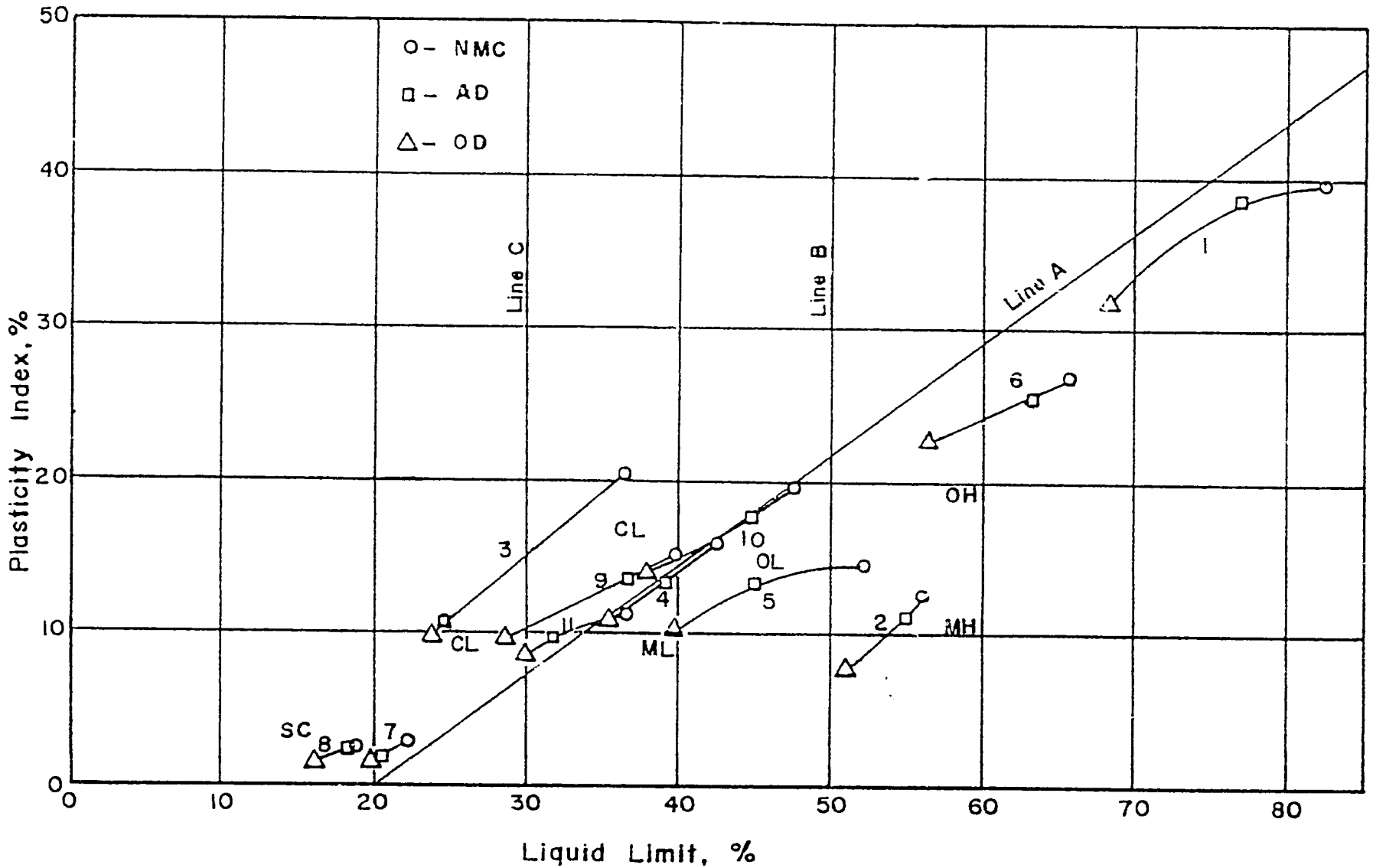


Fig.12 Plasticity Chart of Thailand Lateritic Soils (after MAZHAR, 1969).

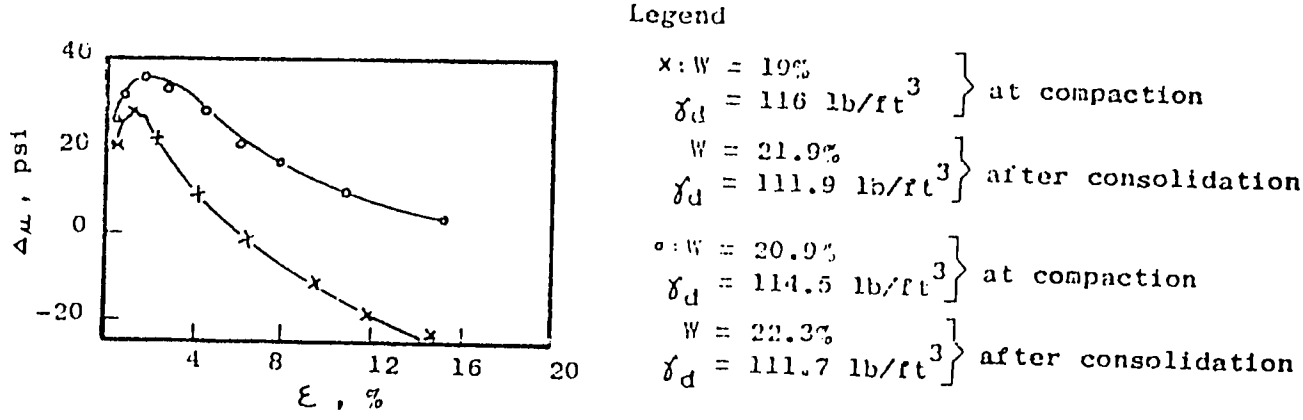


Fig.13 Relationship between Excess Pore Pressure (Δu) and Axial Strain (ε) (after MOH & MAHMOOD, 1969).

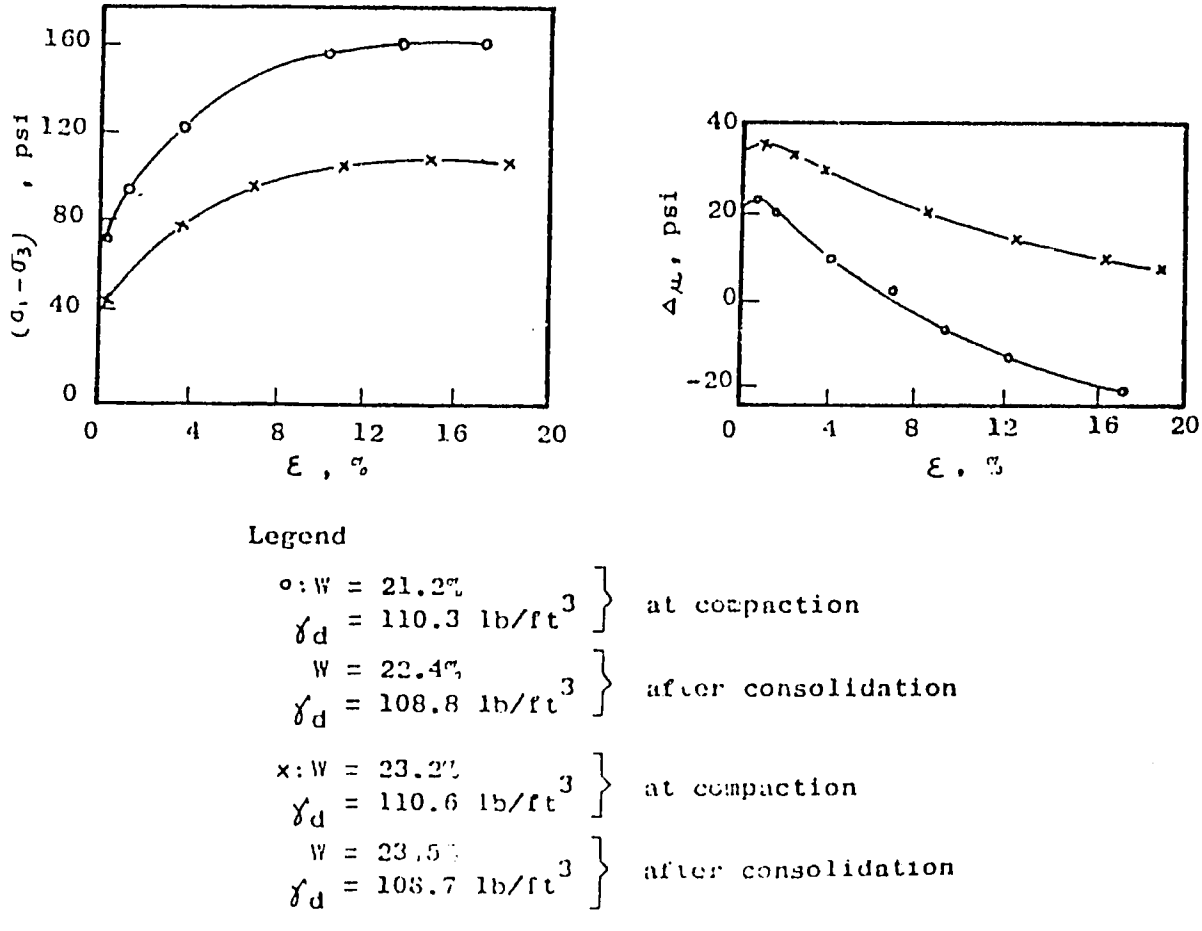
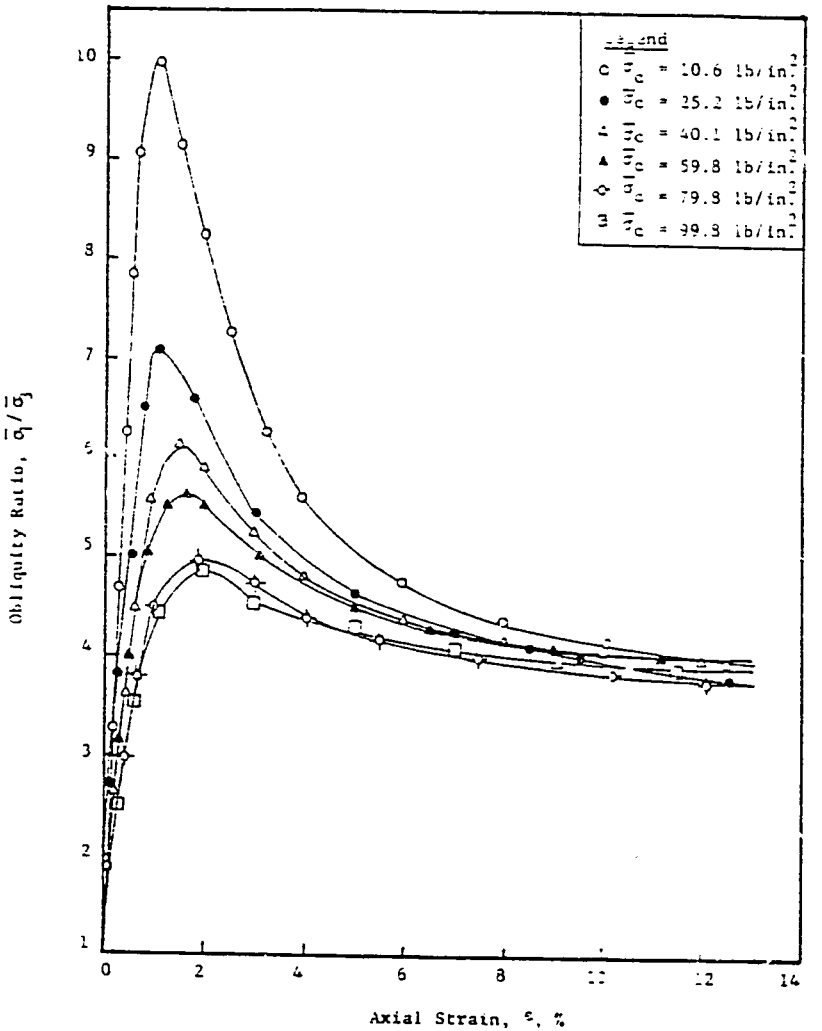
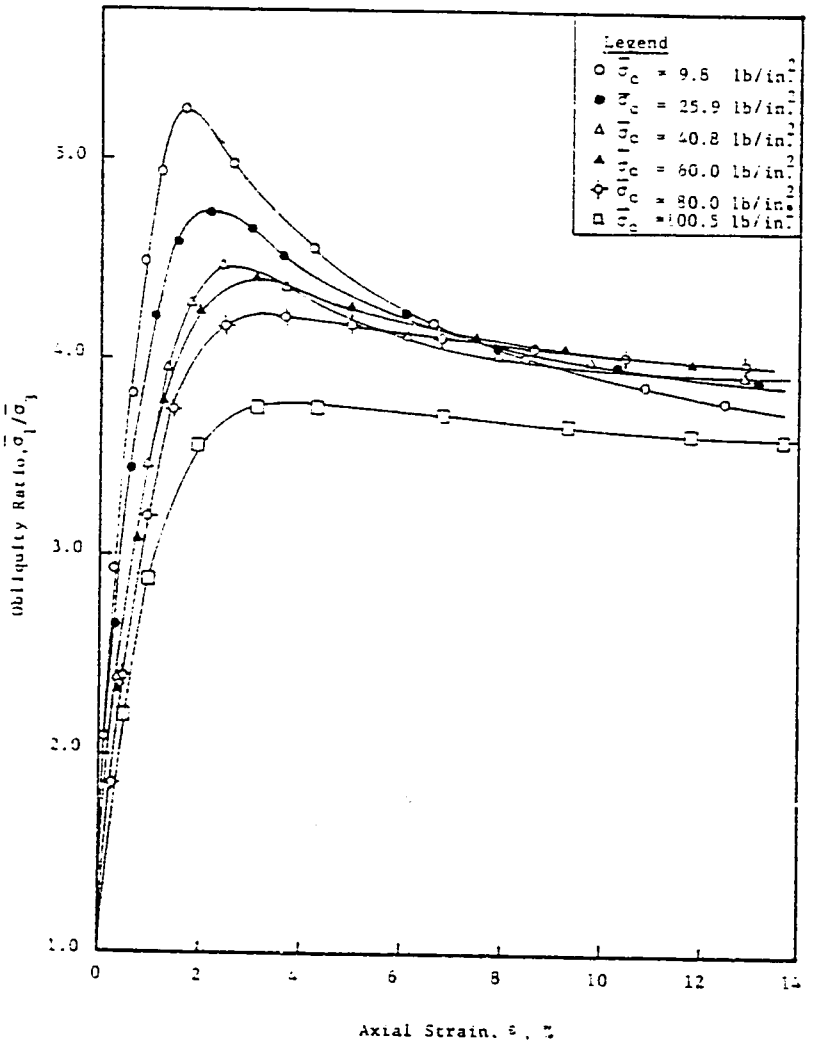


Fig.14 Plots of Deviator Stress and Excess Pore Pressure versus Axial Strain for the Lateritic Soil of Khorat, Northeastern Thailand (after MOH & MAHMOOD, 1969).

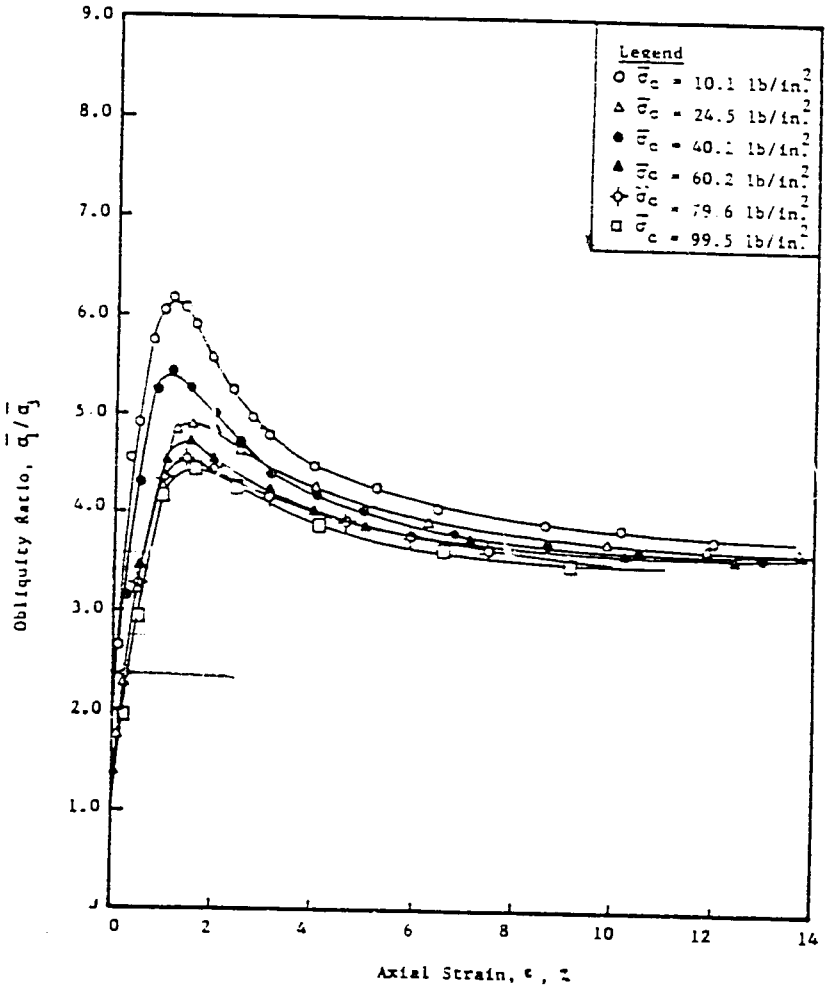


(a) Sample compacted at optimum

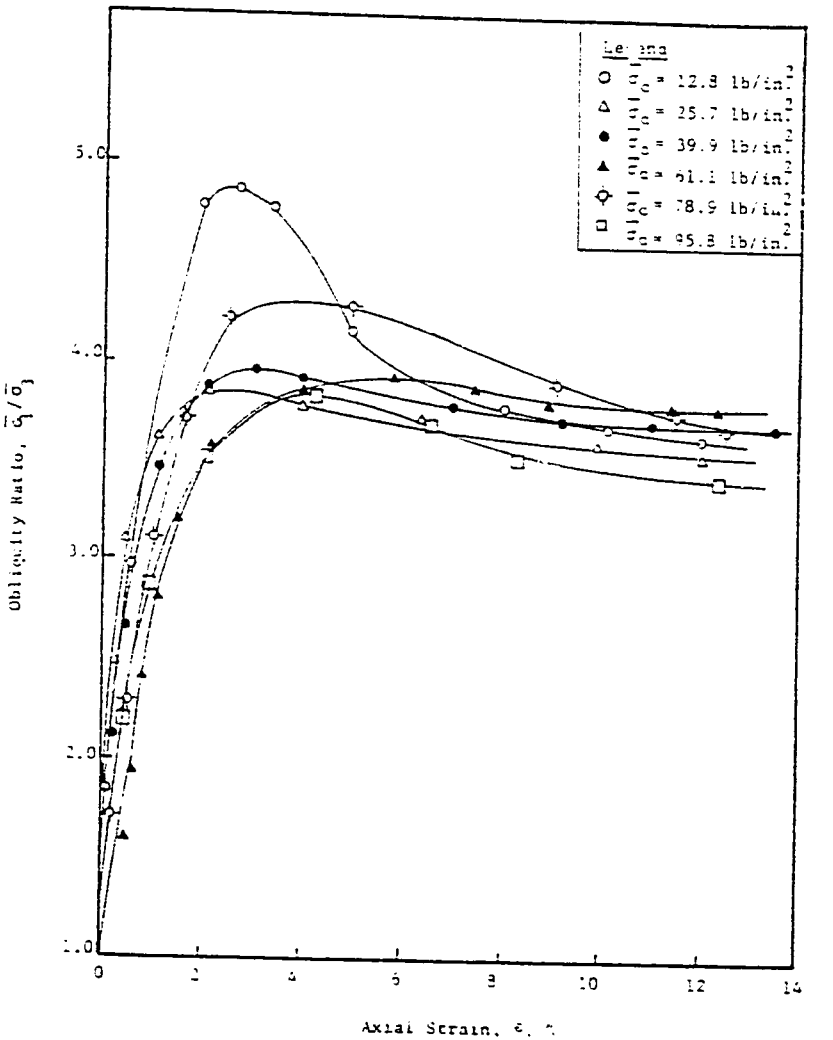


(b) Sample compacted dry of optimum (95% density)

Fig.15 Obliquity Ratio vs. Axial Strain Relationship for the Clayey Sand of Chantaburi (after BOONSRI, 1971).

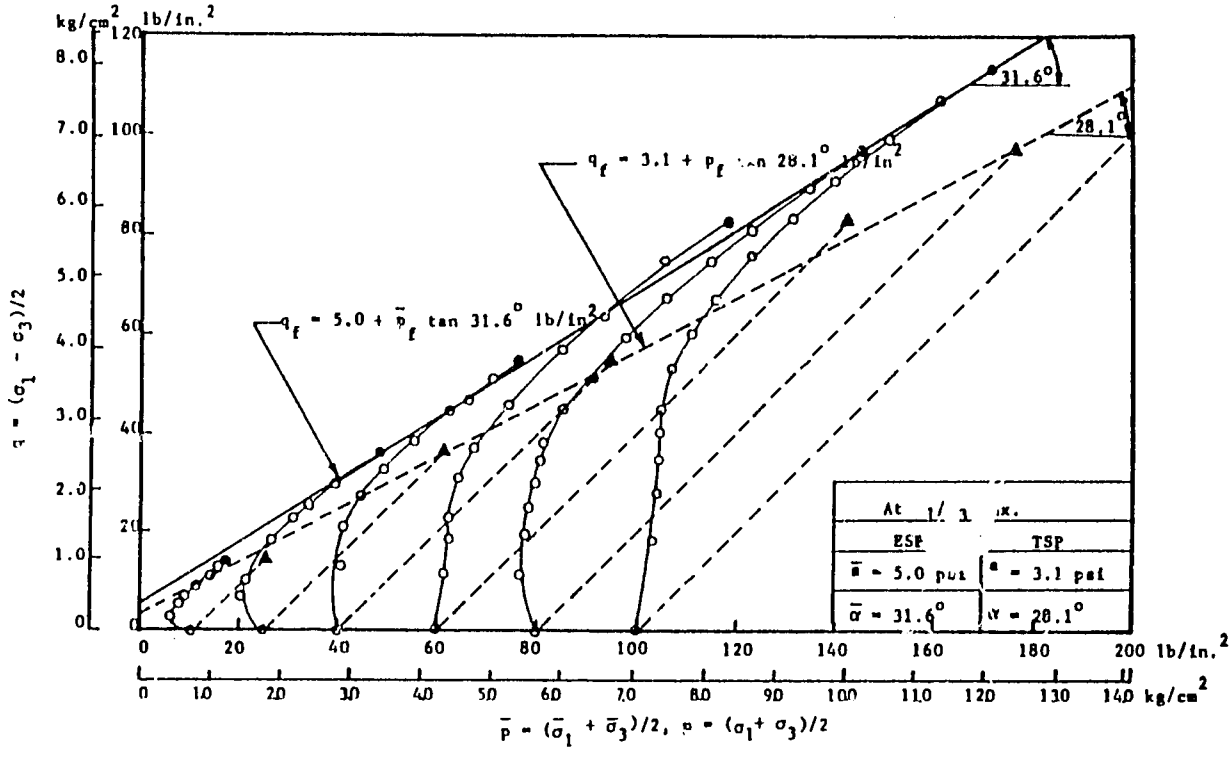


(a) Sample compacted at optimum

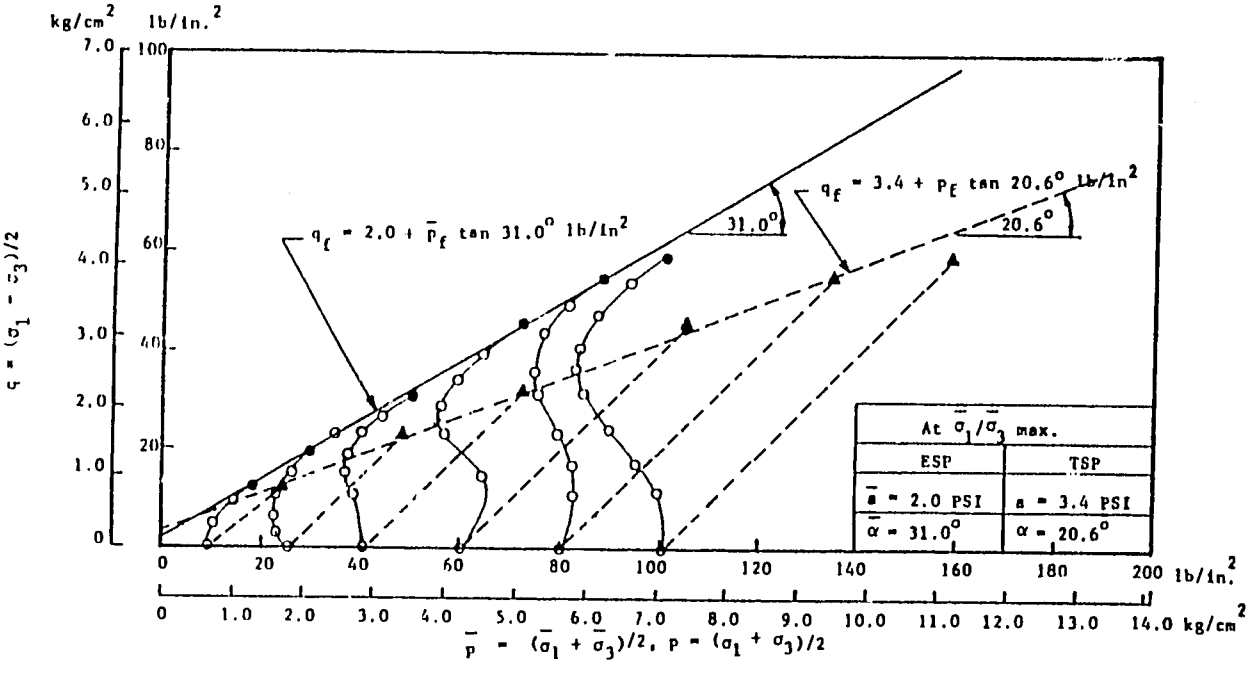


(b) Sample compacted dry of optimum (95% density)

Fig.16 Obliquity Ratio vs. Axial Strain Relationship for the Clayey Gravel of Saraburi (after BOONSRI, 1971).

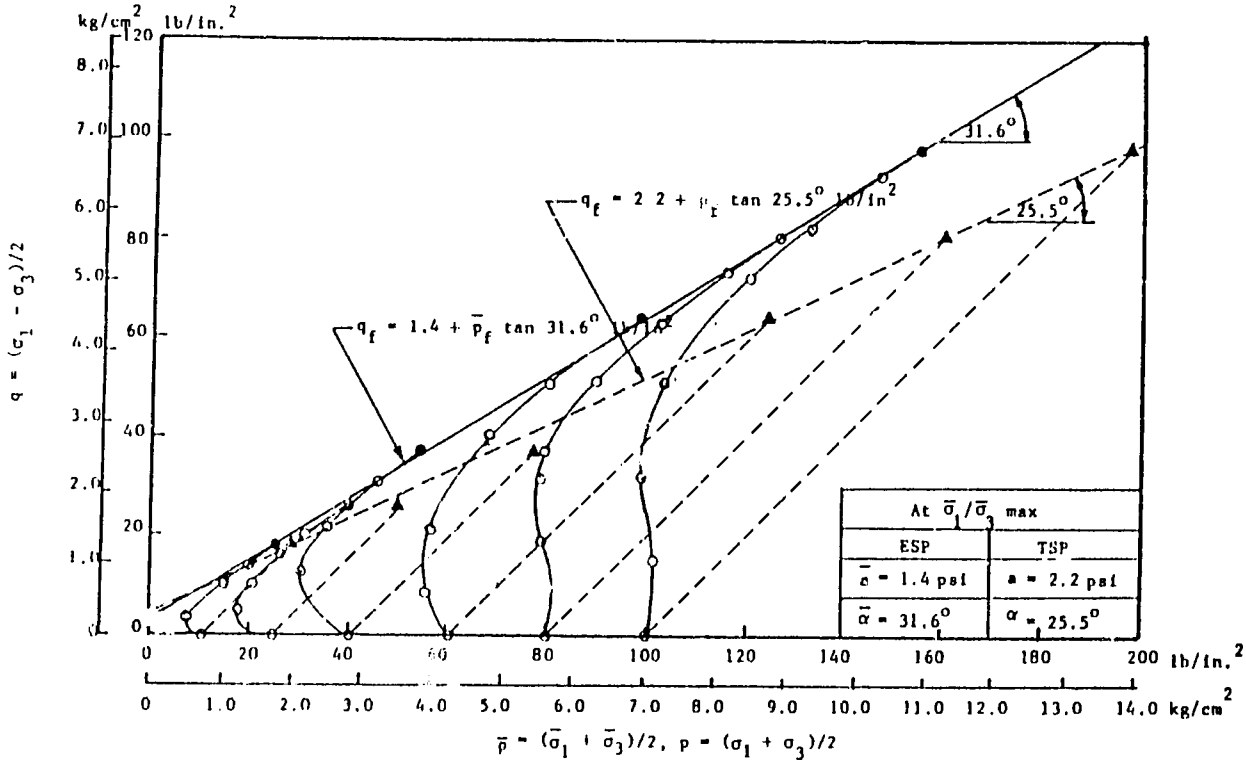


(a) Sample compacted at optimum

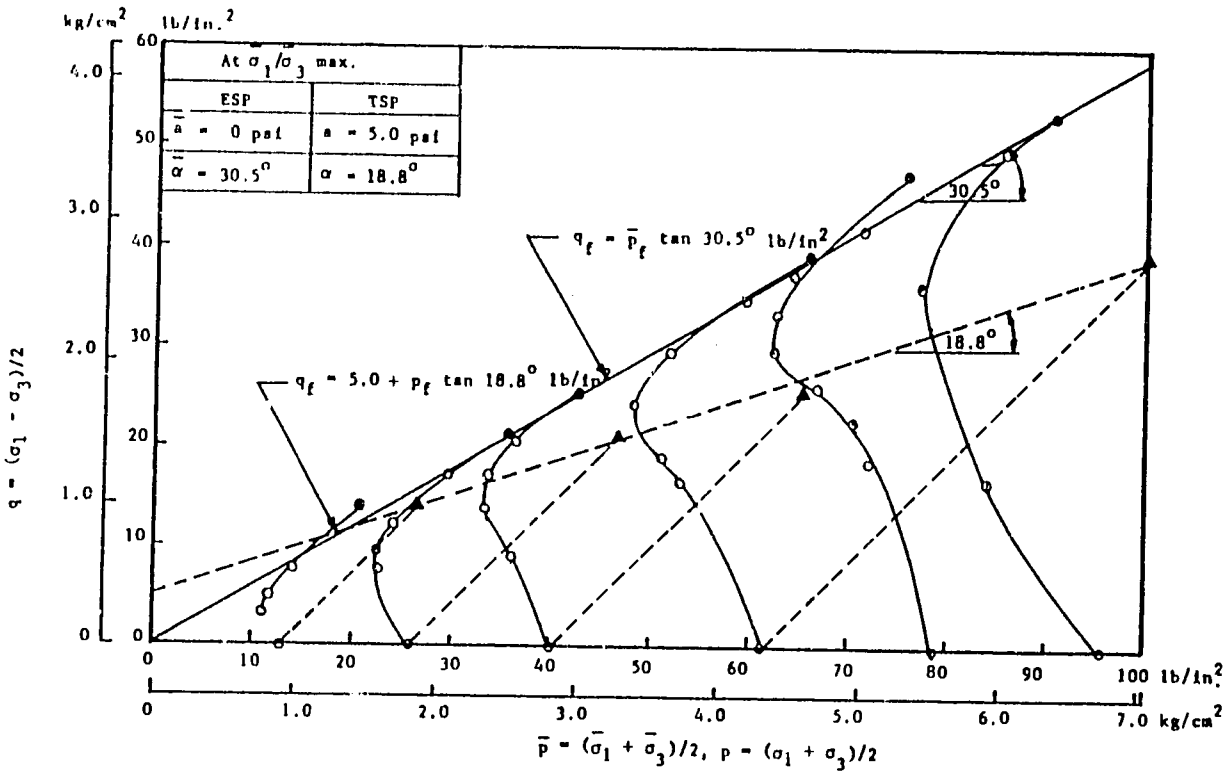


(b) Sample compacted dry of optimum (95% density)

Fig.17 Stress Paths and Strength Envelopes of the Clayey Sand from Chantaburi at [illegible] (after BOONSRI, 1971).

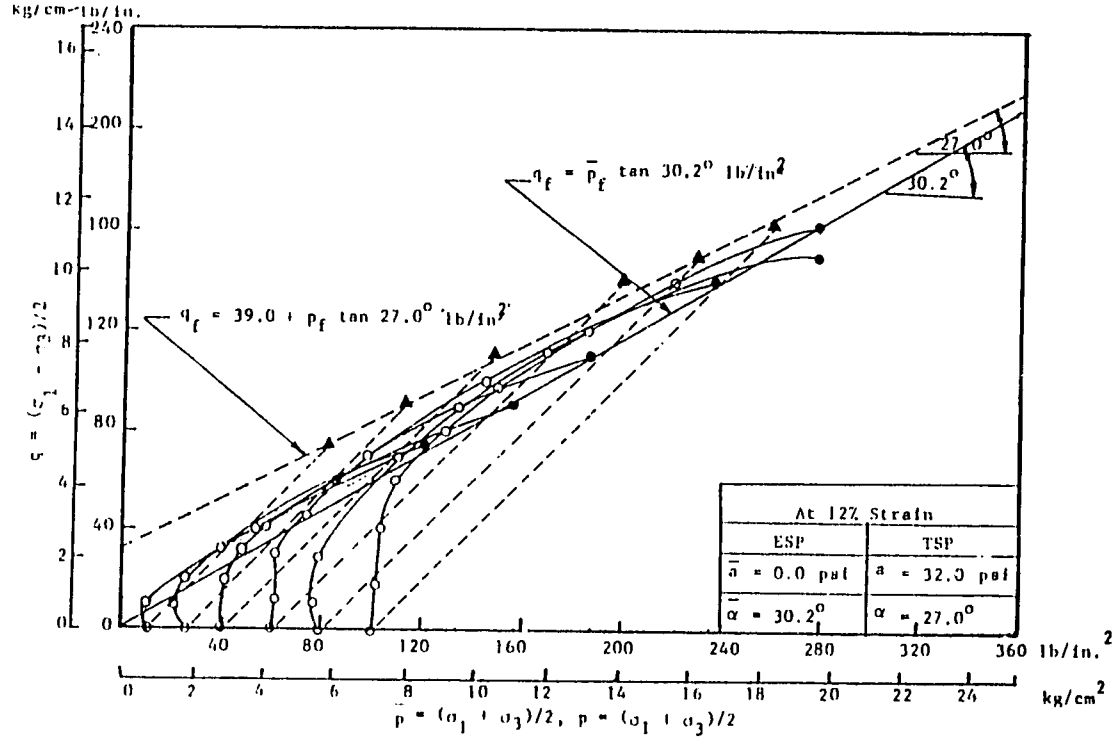


(a) Sample compacted at optimum;

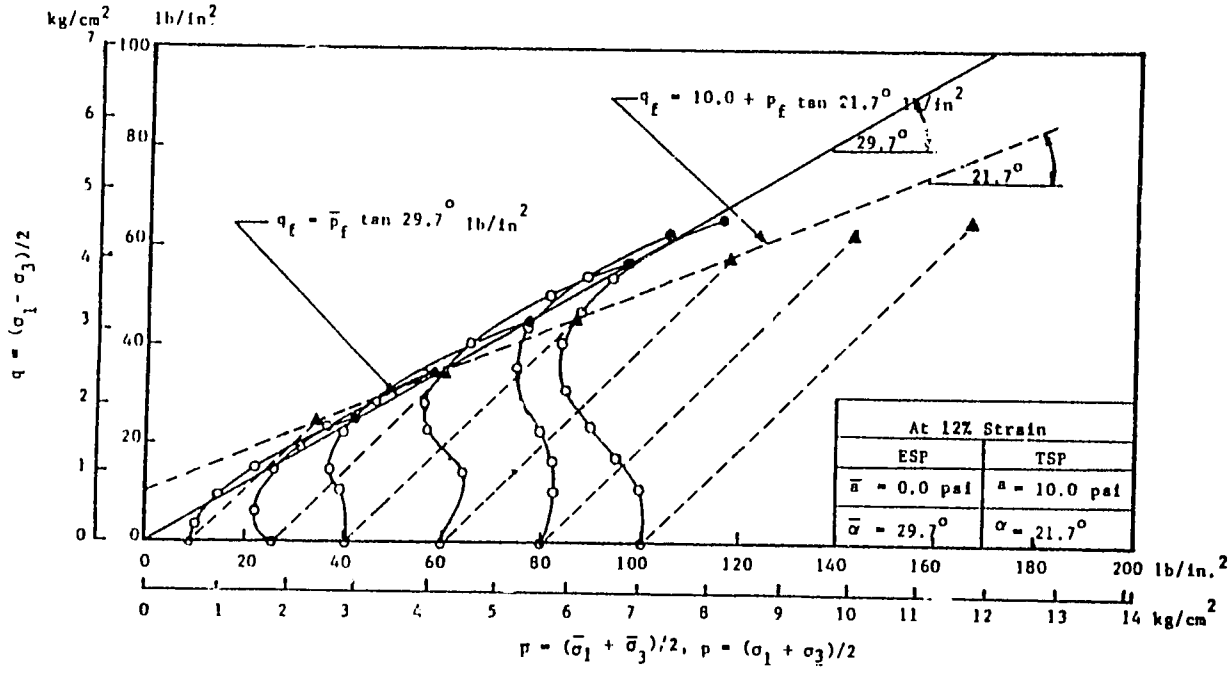


(b) Sample compacted dry side of optimum (95% density)

Fig.18 Stress Paths and Strength Envelopes of the Clayey Gravel from Saraburi at [illegible] (after BOONSRI, 1971).

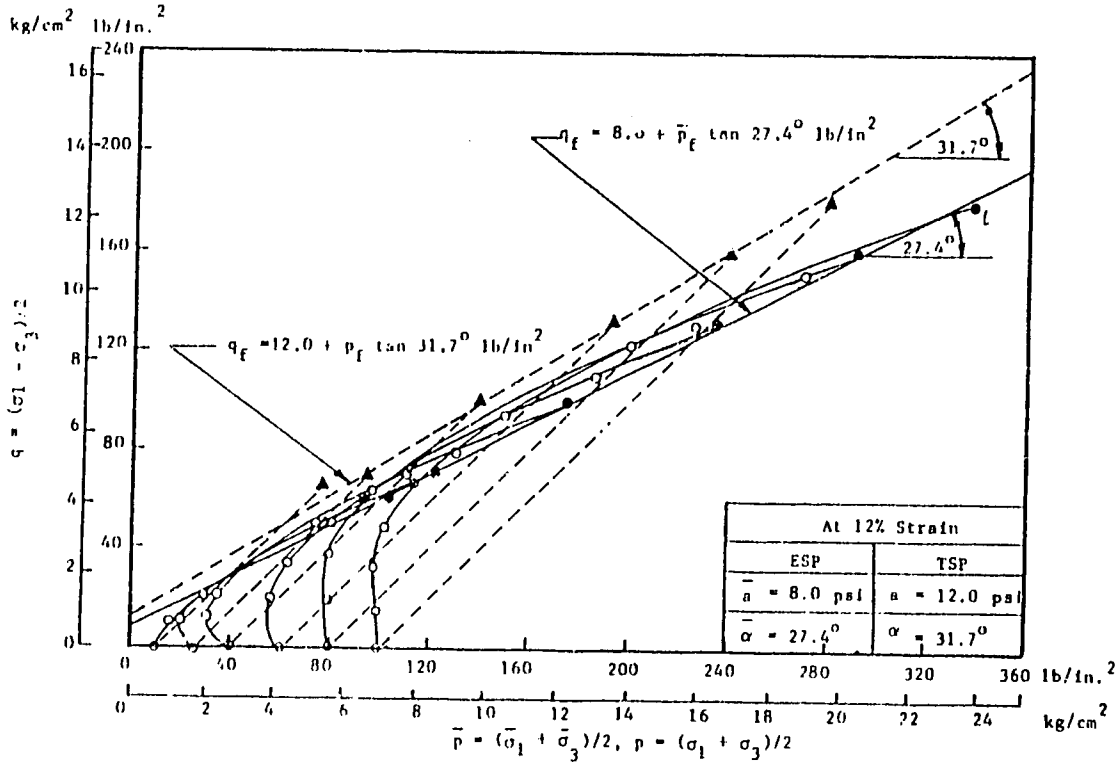


(a) Sample compacted at optimum

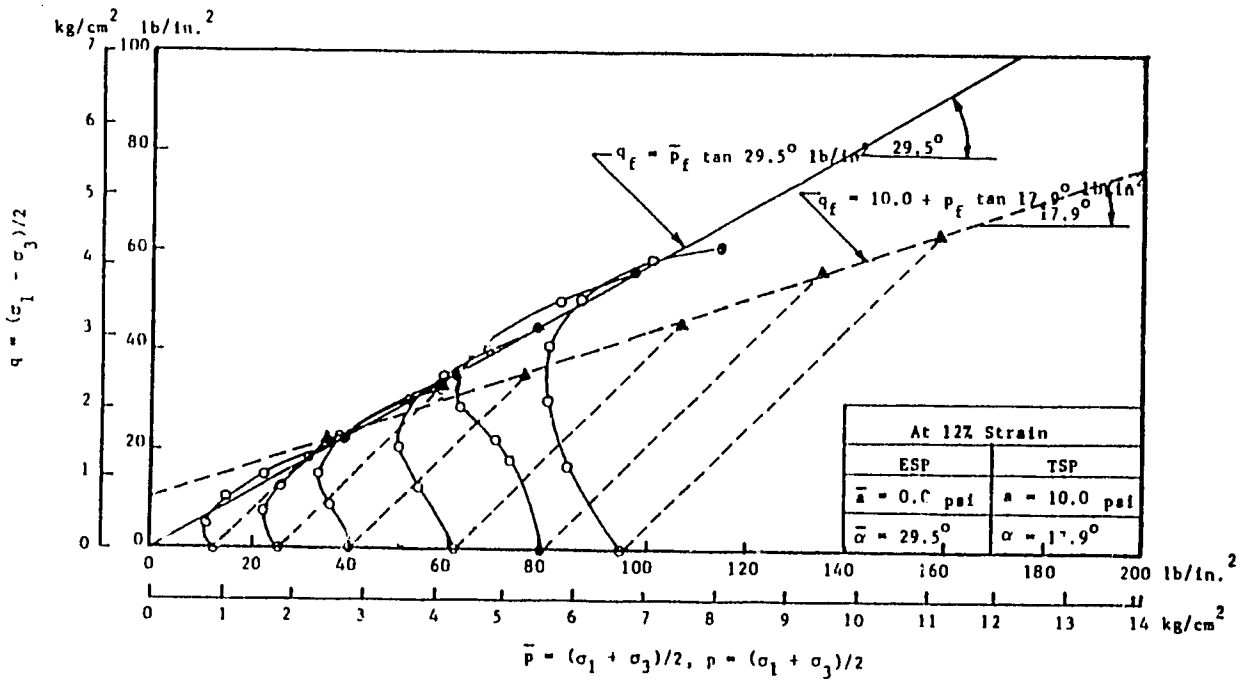


(b) Sample compacted dry of optimum (95% density)

Fig.19 Stress Paths and Strength Envelopes of the Clayey Sand from Chantaburi at 12% Strain (after BOONSRI, 1971).



(a) Sample compacted at optimum



(b) Sample compacted dry of optimum (95% density)

Fig.20 Stress Paths and Strength Envelopes of the Clayey Gravel from Saraburi at 12% Strain (after BOONSRI, 1971).

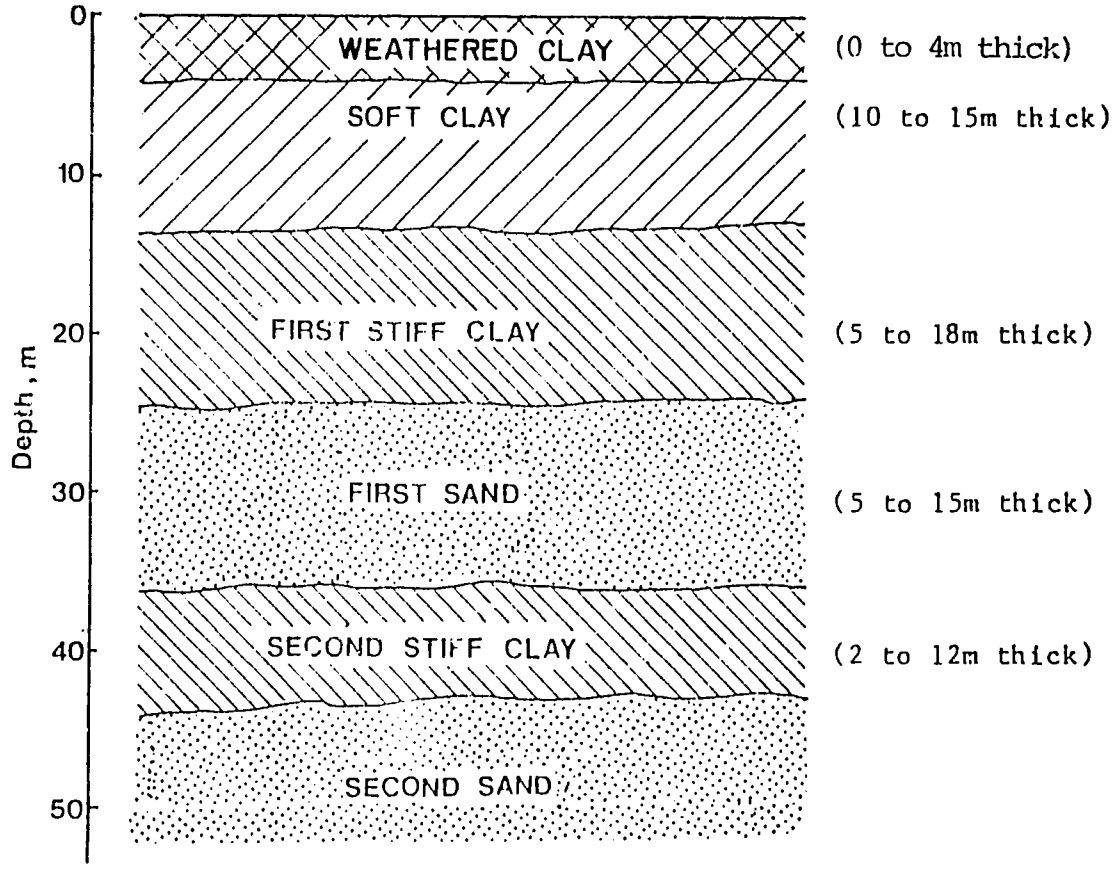
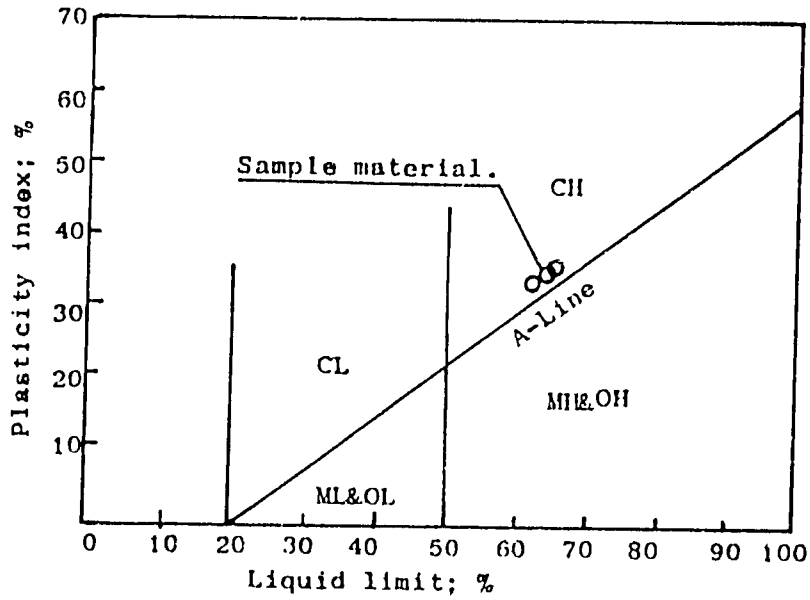


Fig.21 Typical Soil Profile of Bangkok Clay



Plasticity chart

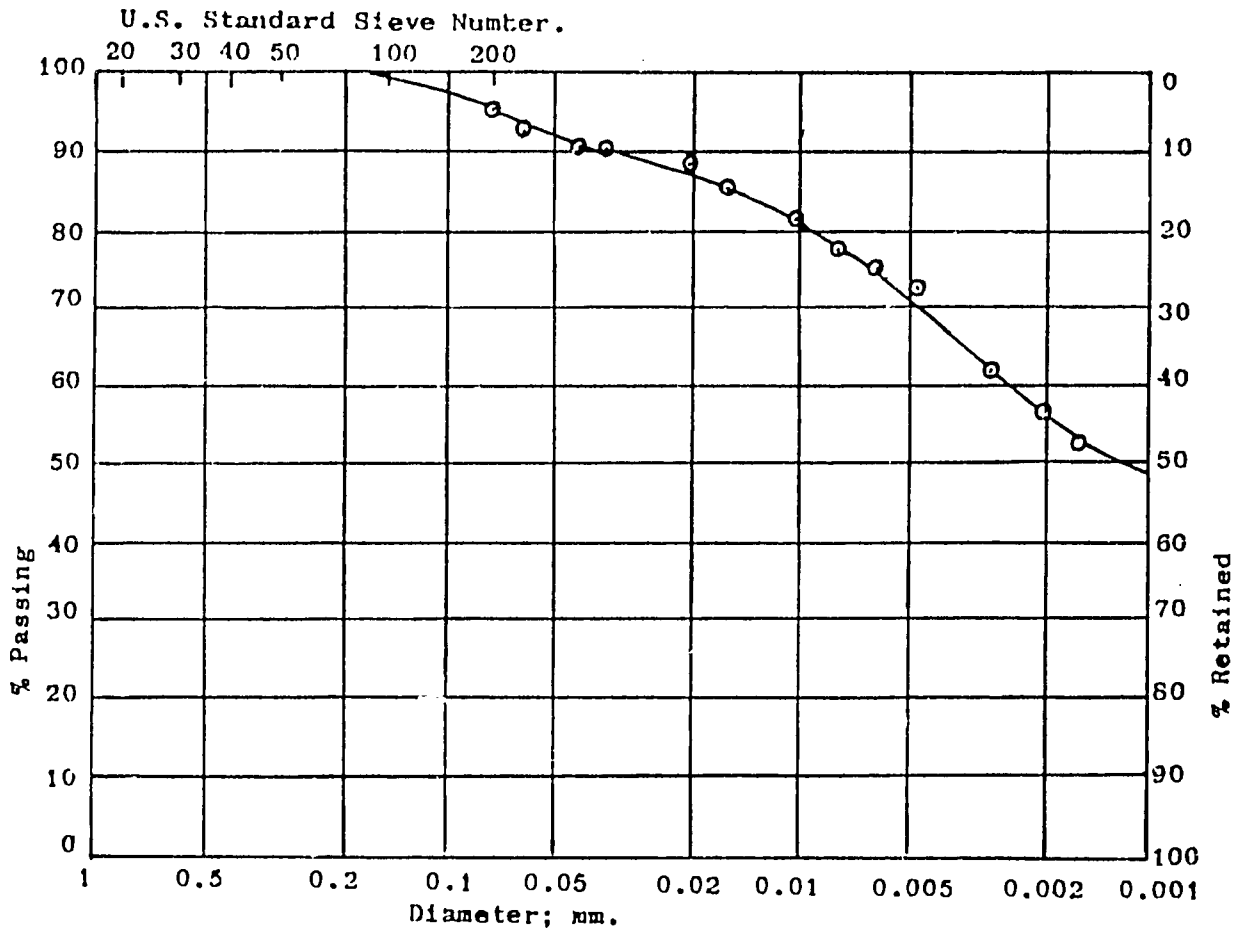


Fig.22 Plasticity Chart and Particles Size Distribution of Sample Weathered Clay (After PLANGPONGPUN, 1977)

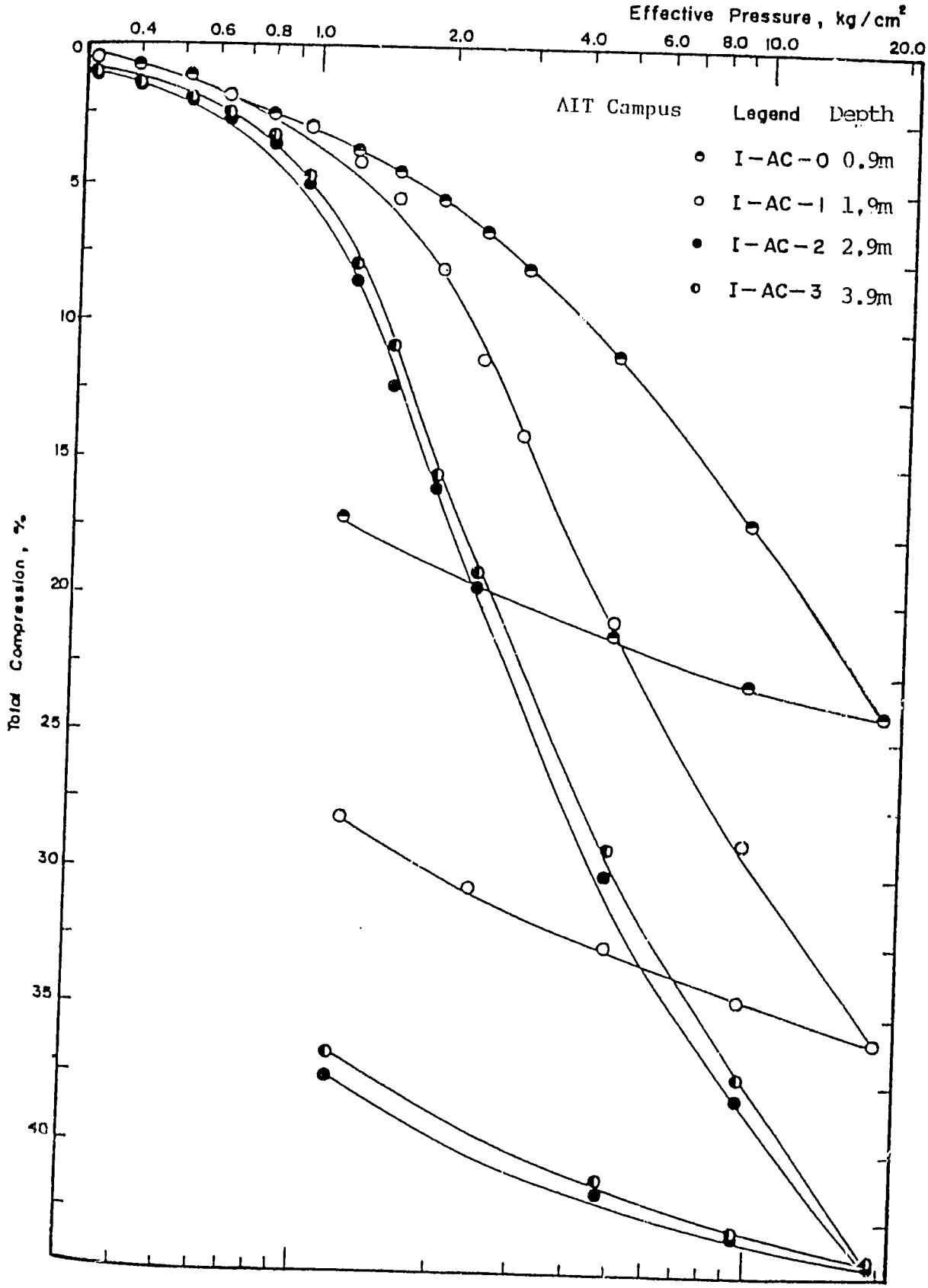


Fig.23 Effect of Depth on Compression - Log Pressure Relationship of Weathered Bangkok Clay (After KANJANOPHAS, 1969)

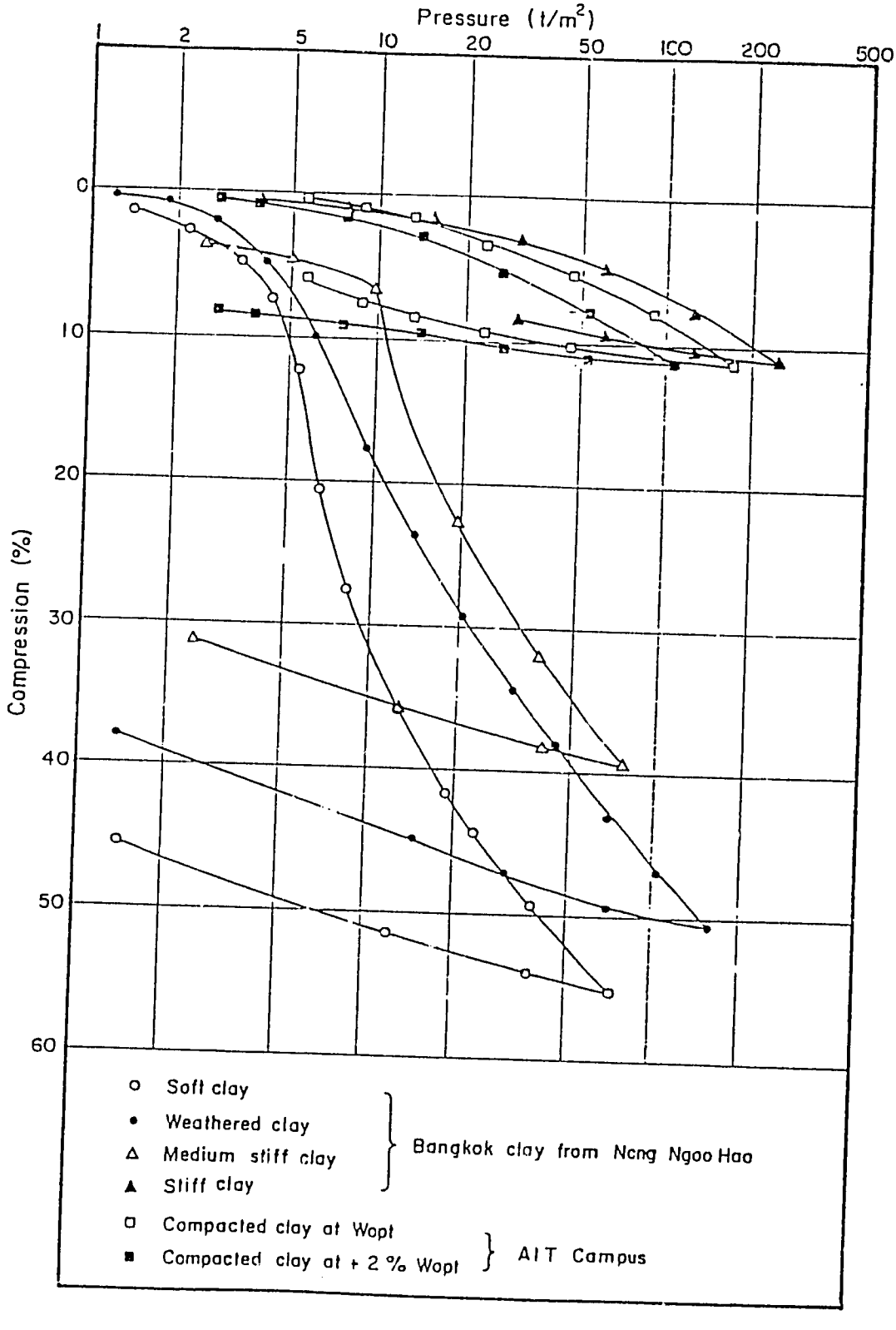


Fig.24 Compression vs Log Pressure, a Comparison between Compacted Clay and Natural Clay (After LIEW, 1979)

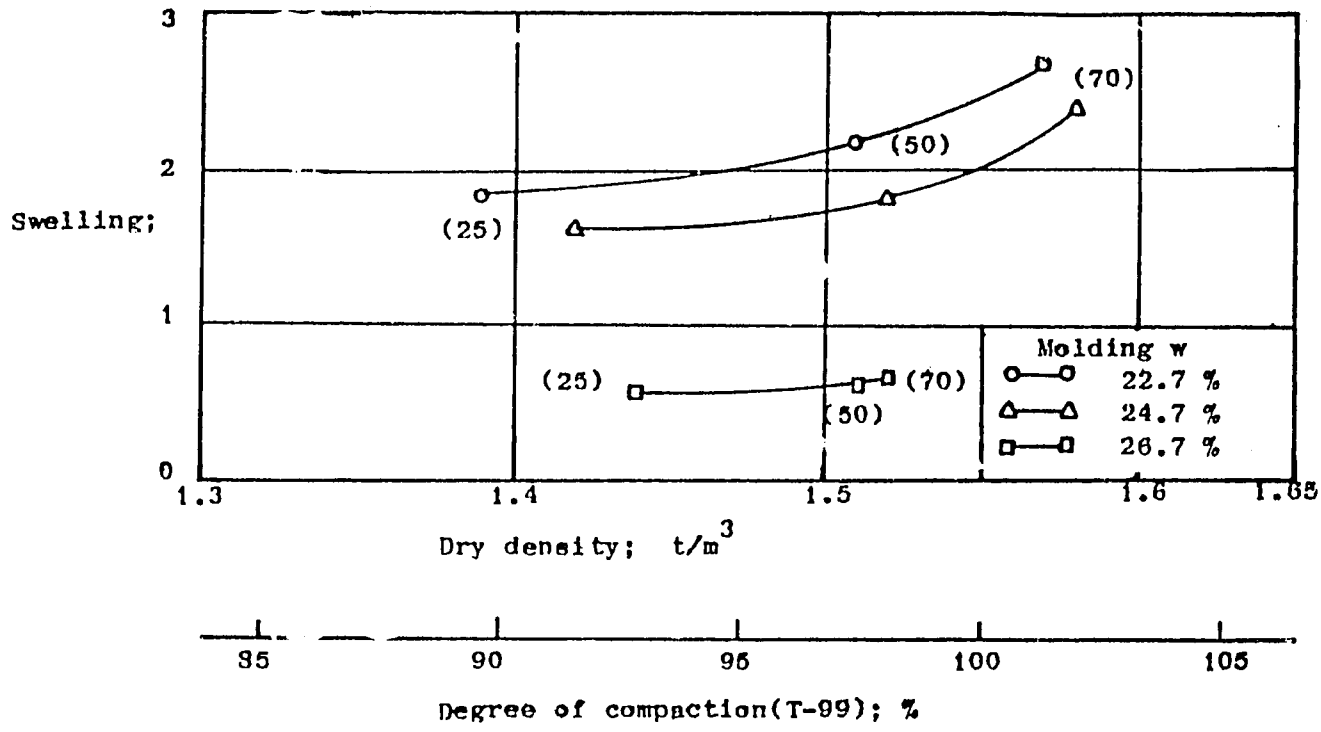


Fig.25 Percentage Swell vs Dry Density Relationship for Samples Compacted by 5.5 lbs hammer, 12 in. Drop Height (After PLANGPONGPUN, 1977)

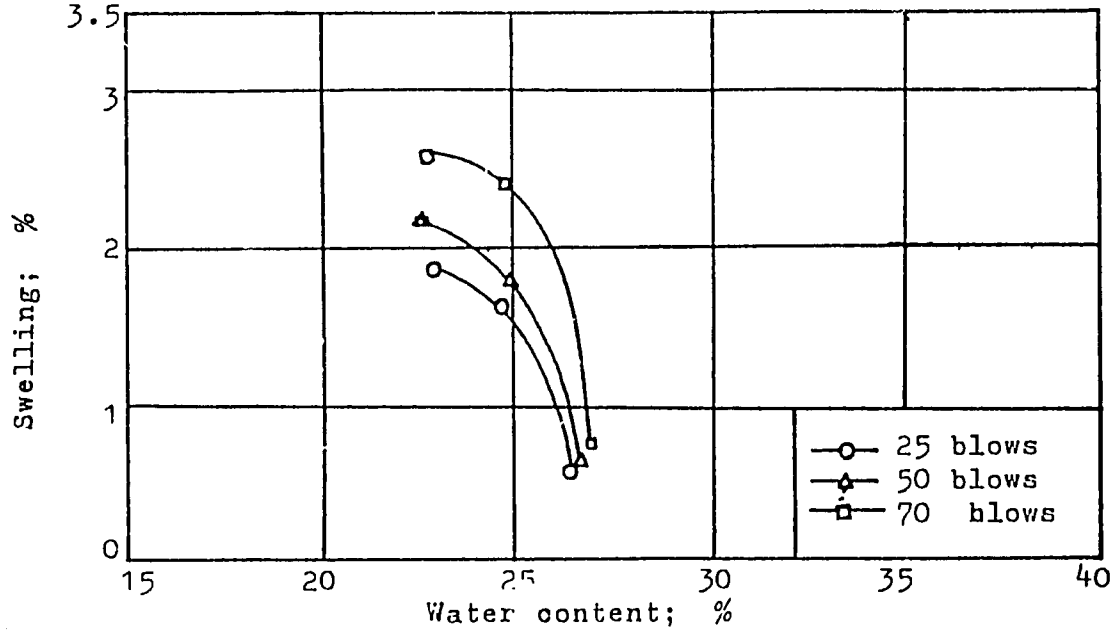


Fig.26 Percentage Swell vs Water Content Relationship for Samples Compacted by 5.5 lbs hammer, 12 in Drop Height (After PLANGPONGPUN, 1977)

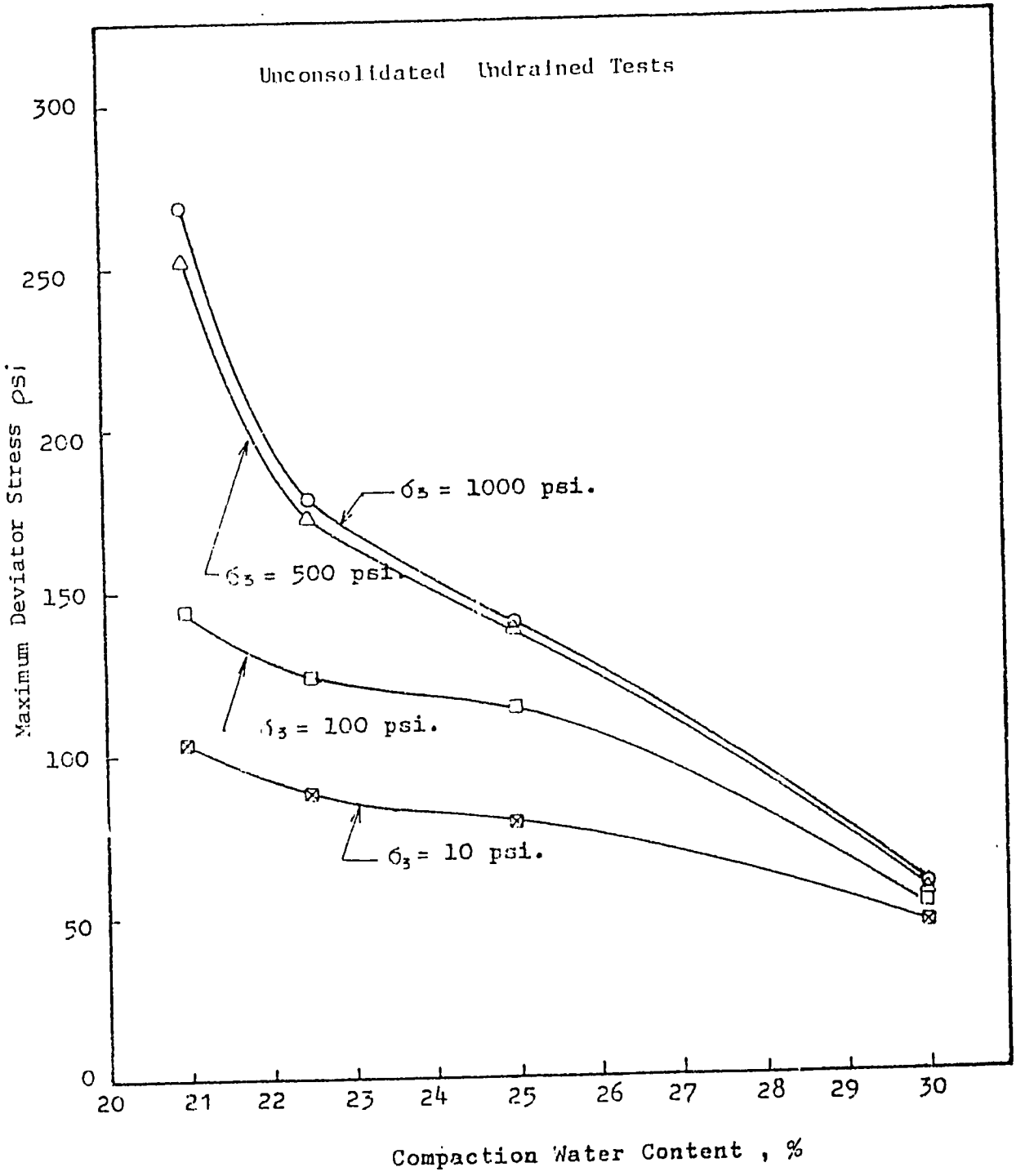


Fig.27 Influence of Molding Water Content on the Static Strength (After TAVARAYUT, 1971)

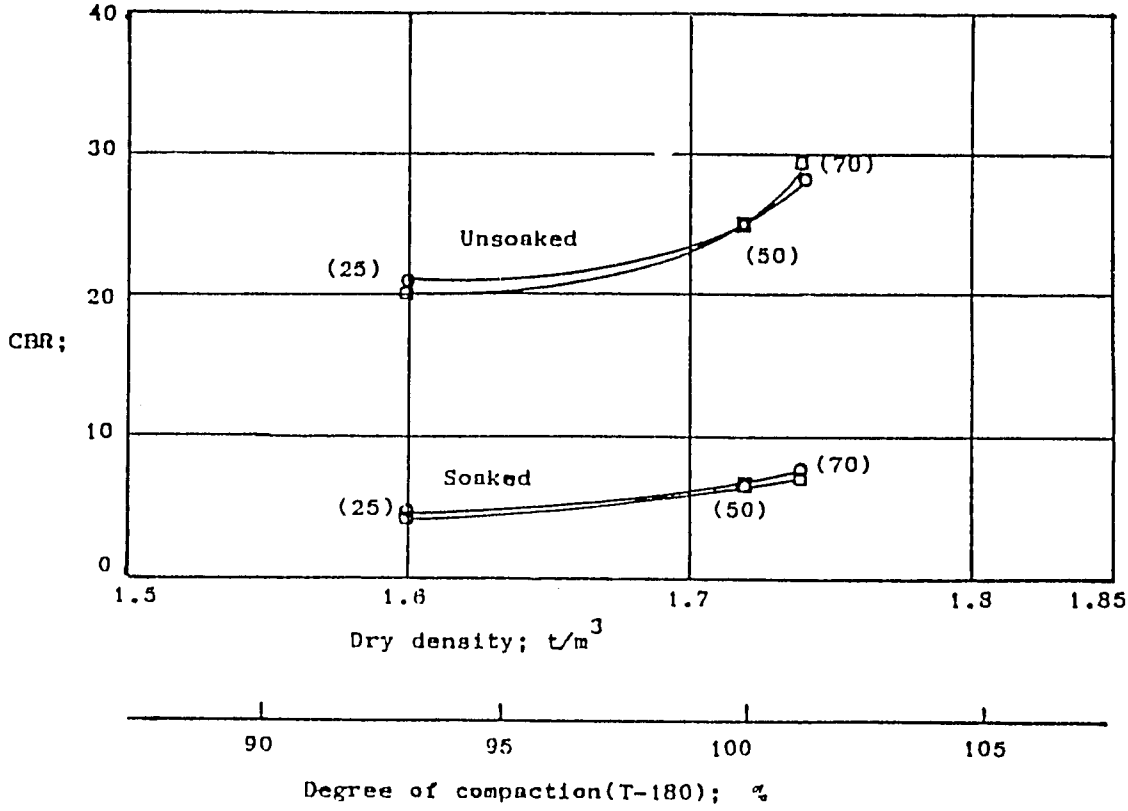


Fig.28 Relationship between CBR Values & Dry Density of the Samples Compacted at wopt (17.0 %) of Modified Compaction (After PLANGPONGPUN, 1977)

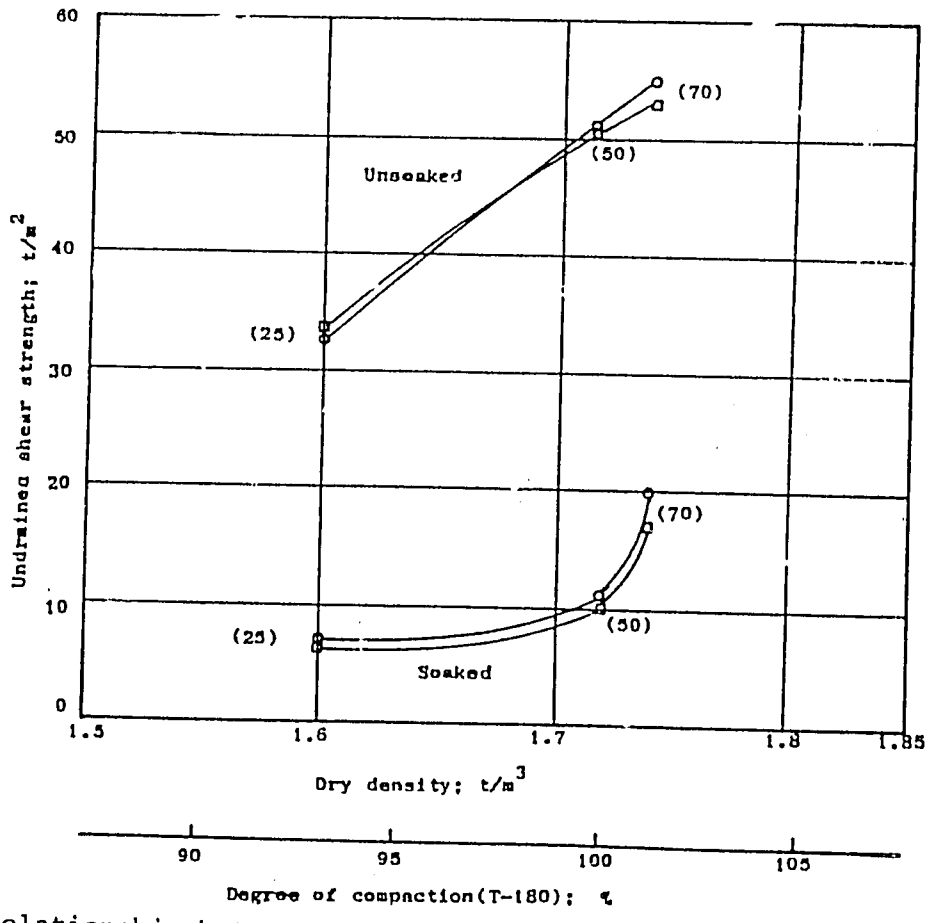


Fig.29 Relationship between Undrained Shear Strength and Dry Density of Samples Compacted at wopt (17.0 %) of Modified Compaction (After PLANGPONGPUN, 1977)

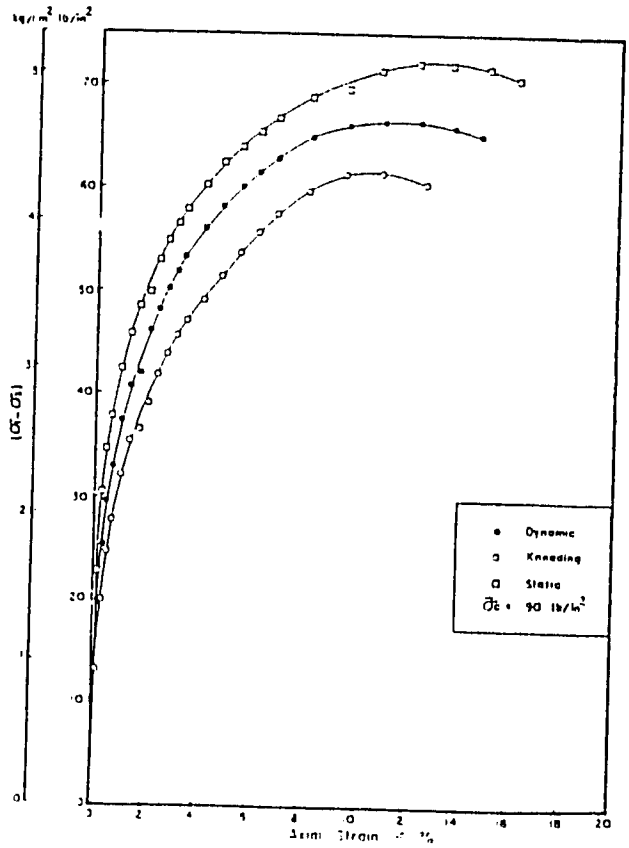


Fig.30 Stress-Strain Relationship of Specimens Compacted at Optimum Water Content and Consolidated under 90 lb/in² (After REUENKRAIRERGSA, 1968)

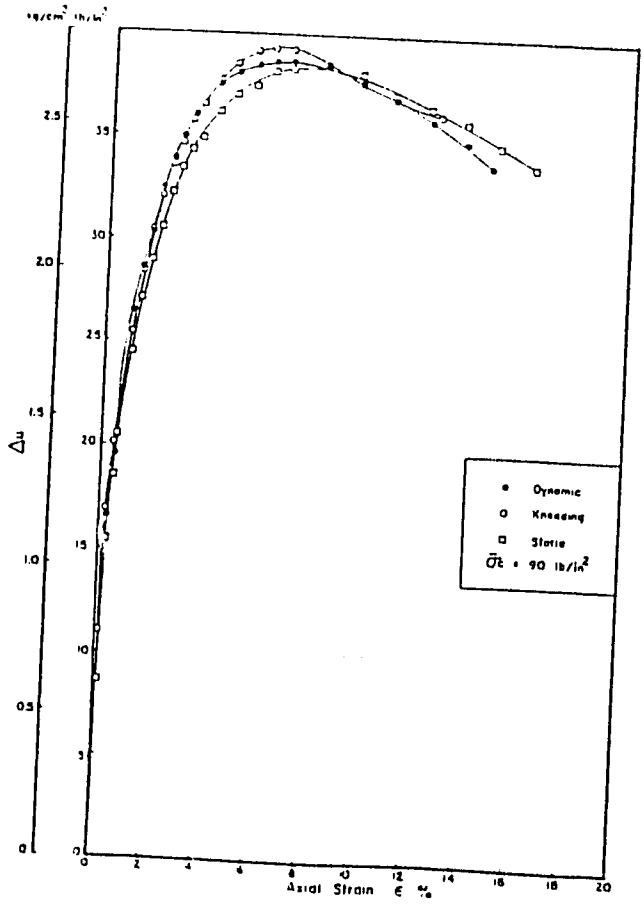


Fig.31 Excess Pore Pressure vs Axial Strain Relationship of Specimens Compacted at Optimum Water Content and Consolidation Under 90 lb/in² (After REUENKRAIRERGSA, 1968)

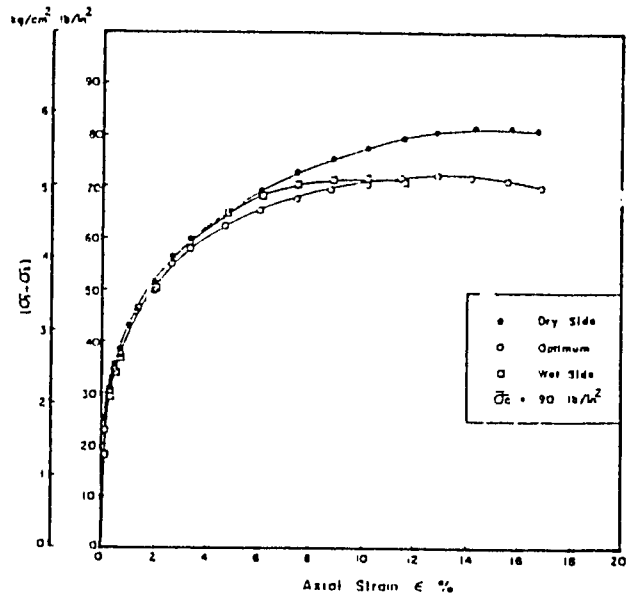


Fig.32 Stress-Strain Relationship of Specimens Compacted by Static Method and consolidated Under 90 lb/in² (After REUENKRAIRERGSA, 1968)

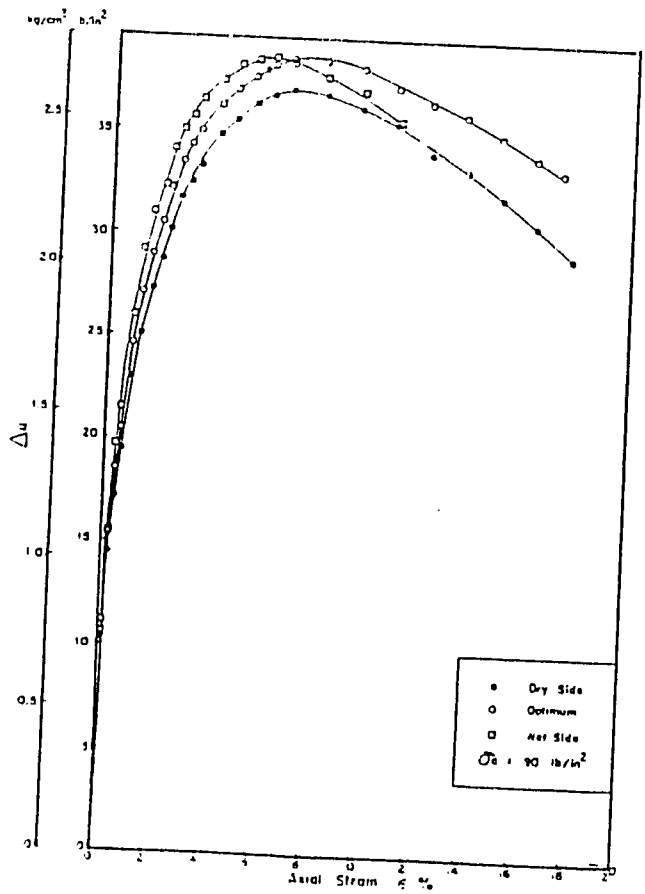


Fig.33 Excess Pore Pressure vs Axial Strain Relationship of Specimens Compacted by Static Method and Consolidated Under 90 lb/in² (After REUENKRAIRERGSA, 1968)

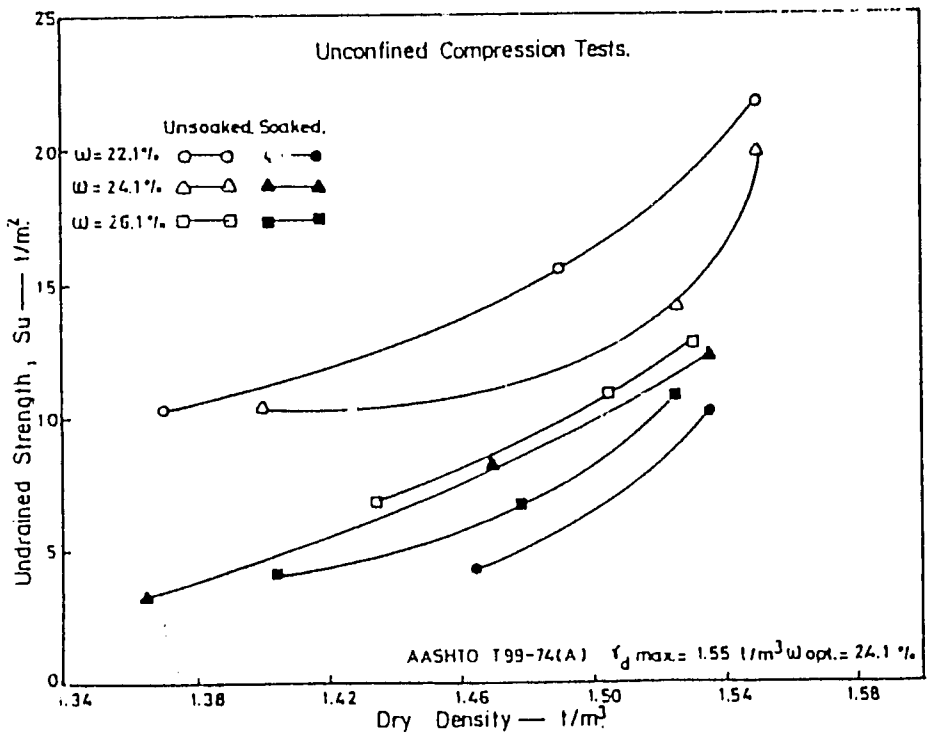


Fig.34 Effect of Dry Density on Undrained Strength in UC Tests (After HAQUE, 1977)

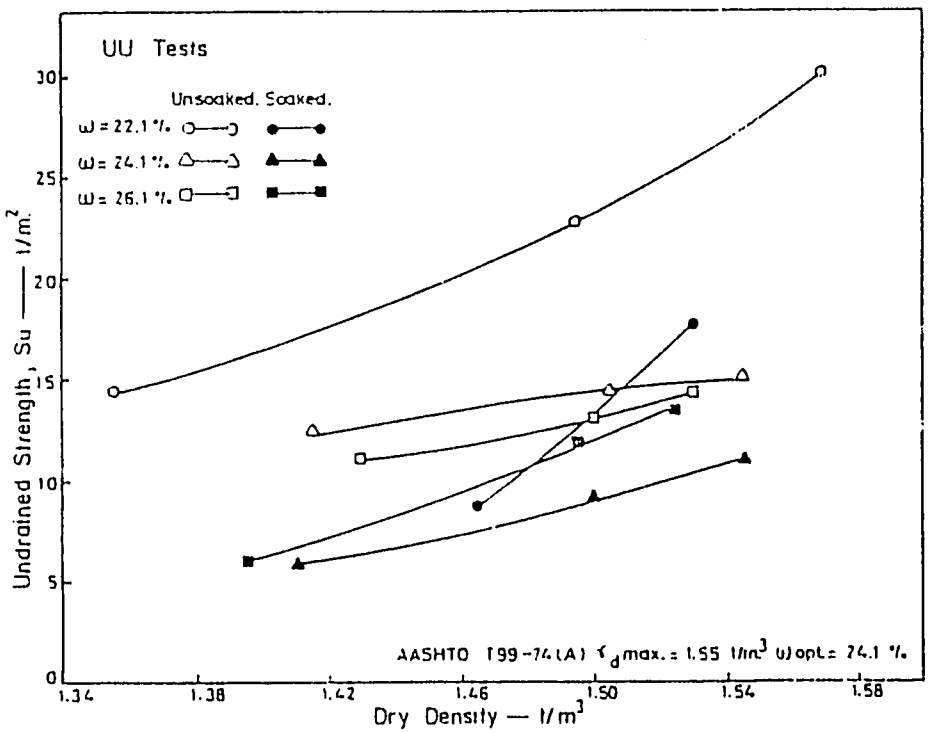


Fig.35 Effect of Dry Density on Undrained Strength-UU Tests (After HAQUE, 1977)

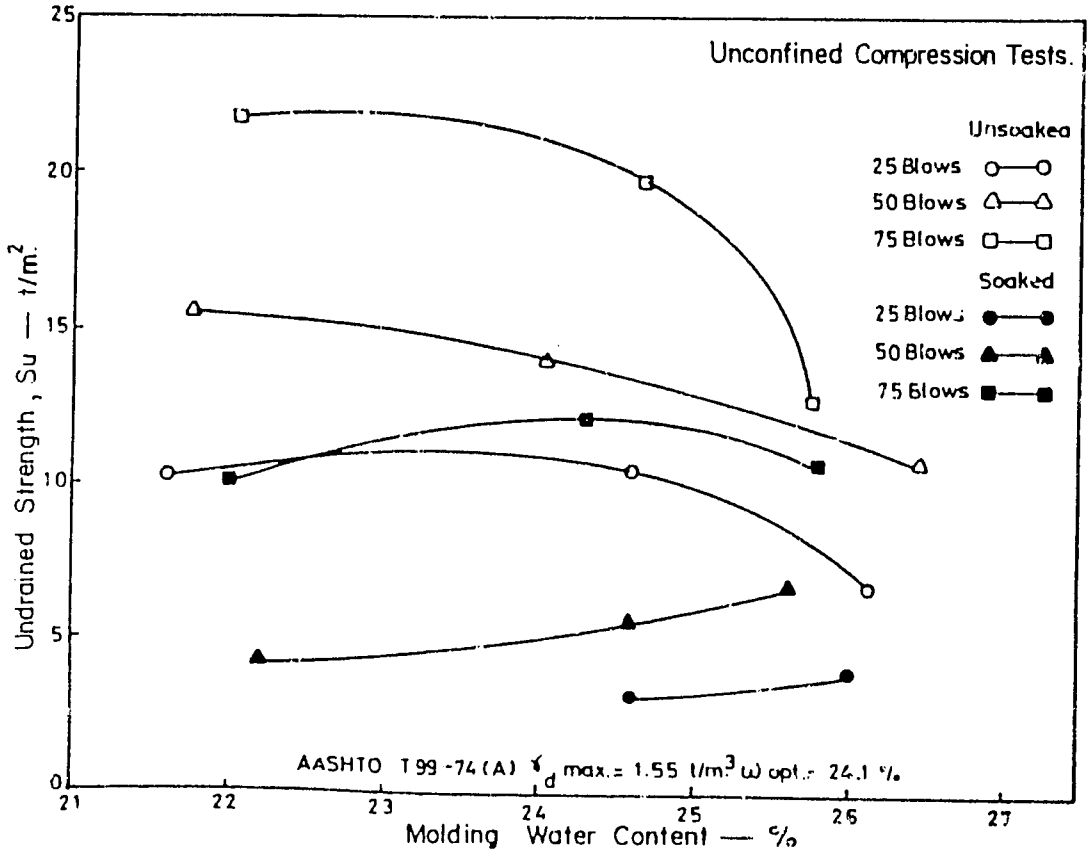


Fig.36 Effect of Molding Water Content on Undrained Strength UC Tests (After HAQUE, 1977)

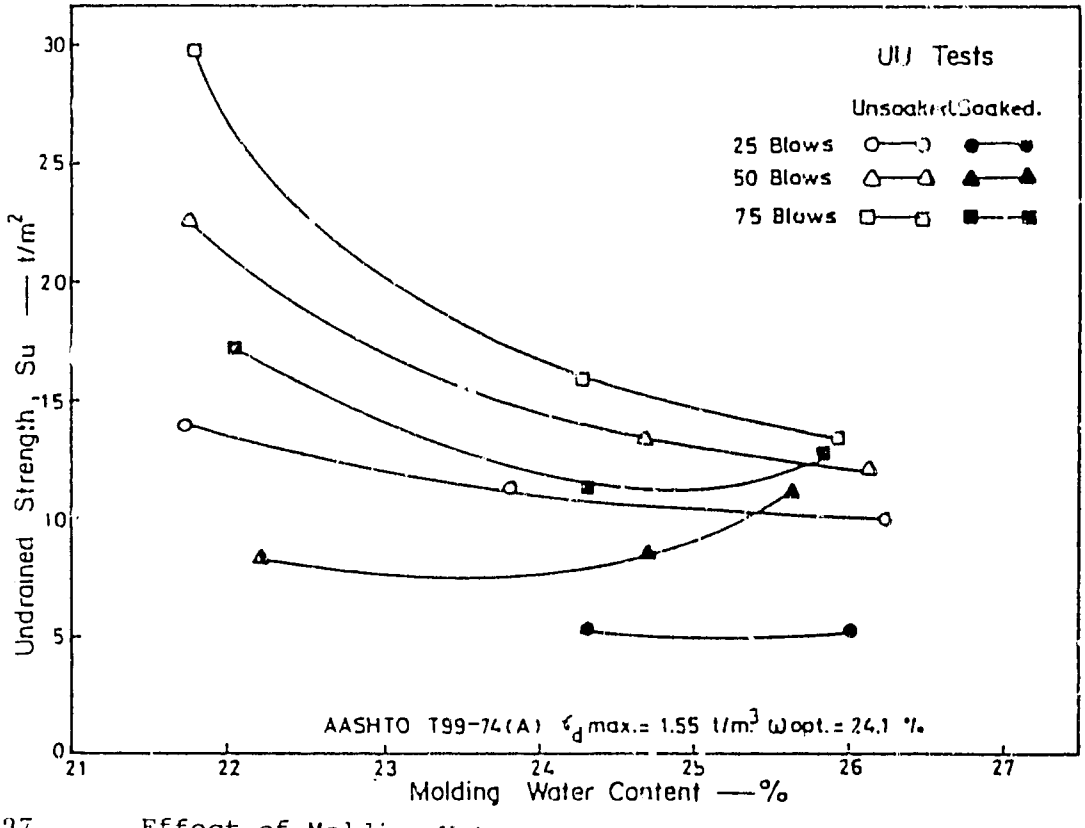


Fig.37 Effect of Molding Water Content on Undrained Strength-UU Tests (After HAQUE, 1977)

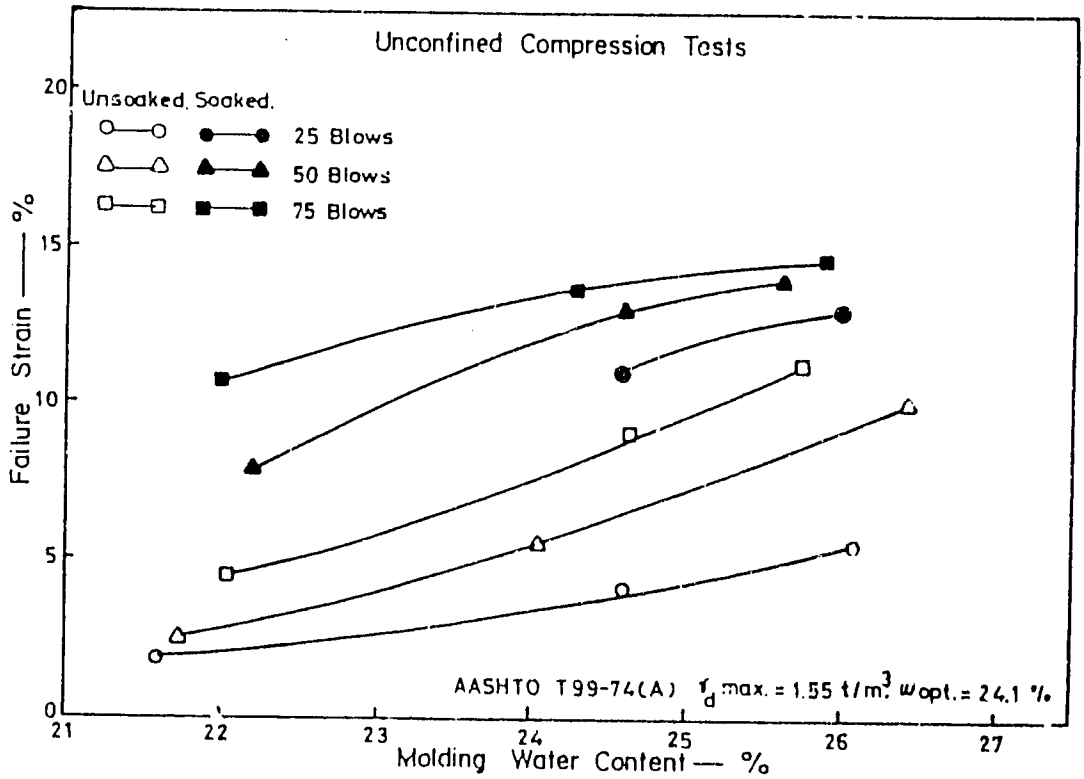


Fig.38 Molding Water Content vs Failure Strain Relationship in UC Tests (After HAQUE, 1977)

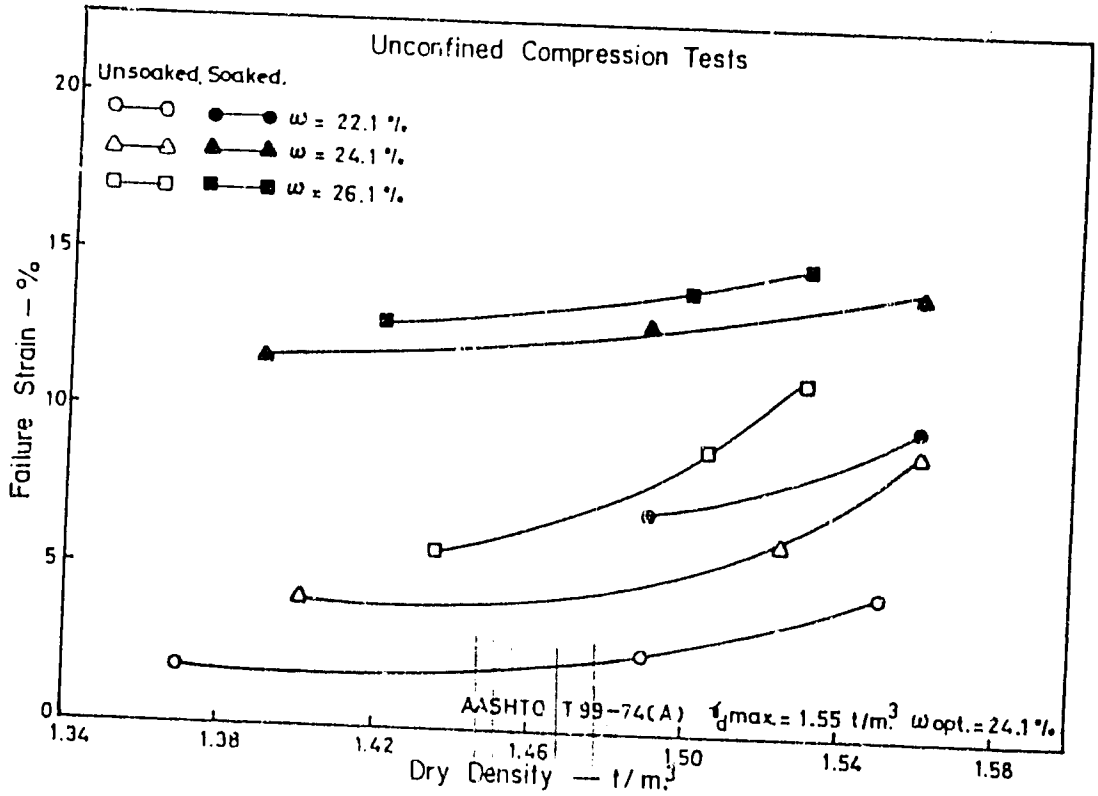


Fig.39 Dry Density vs Failure Strain Relationship in UC Tests (After HAQUE, 1977)

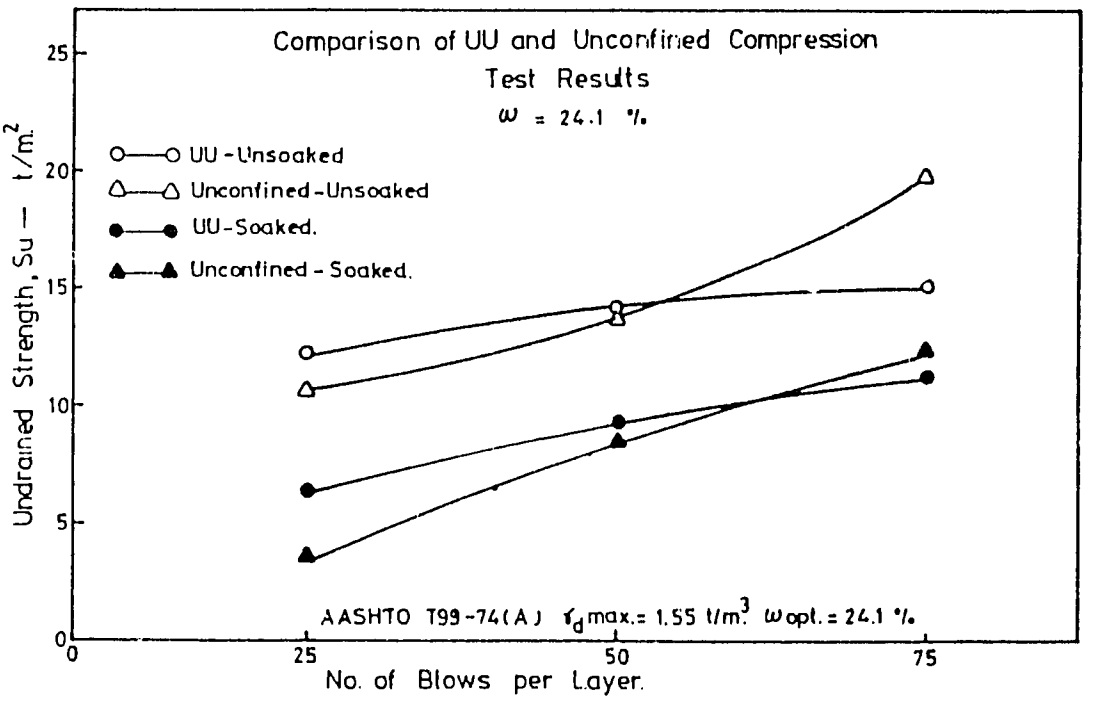
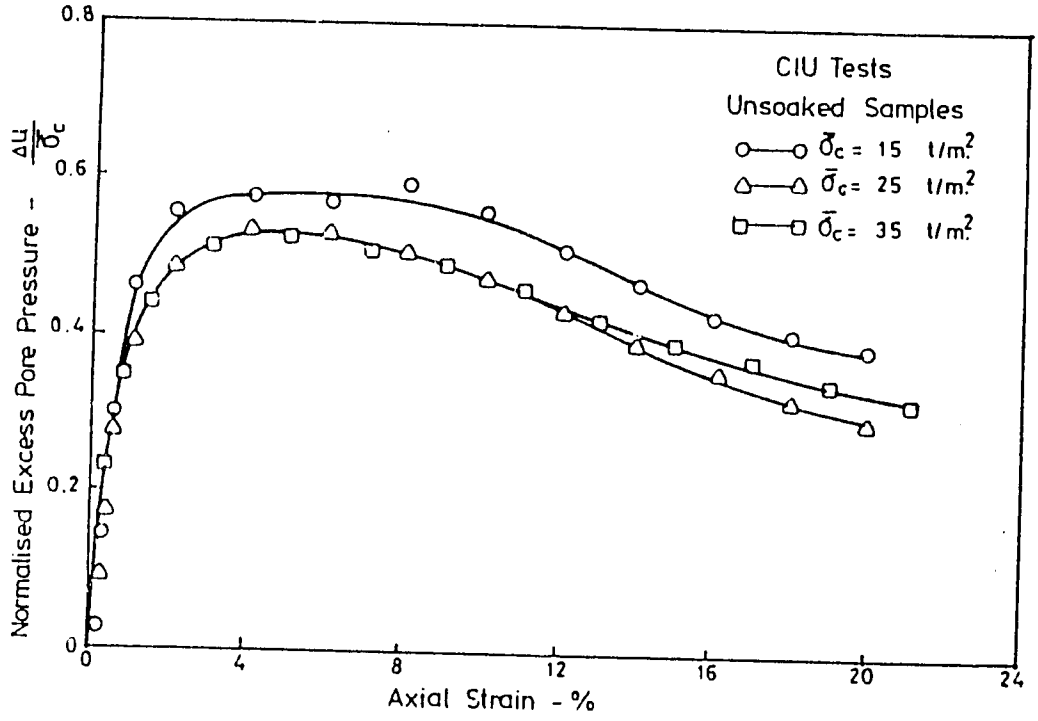


Fig.40 Comparison of Undrained Strength from UU and UC Tests for Samples Compacted at Optimum Water Content of 24.1 % (After HAQUE, 1977)

(a)



(b)

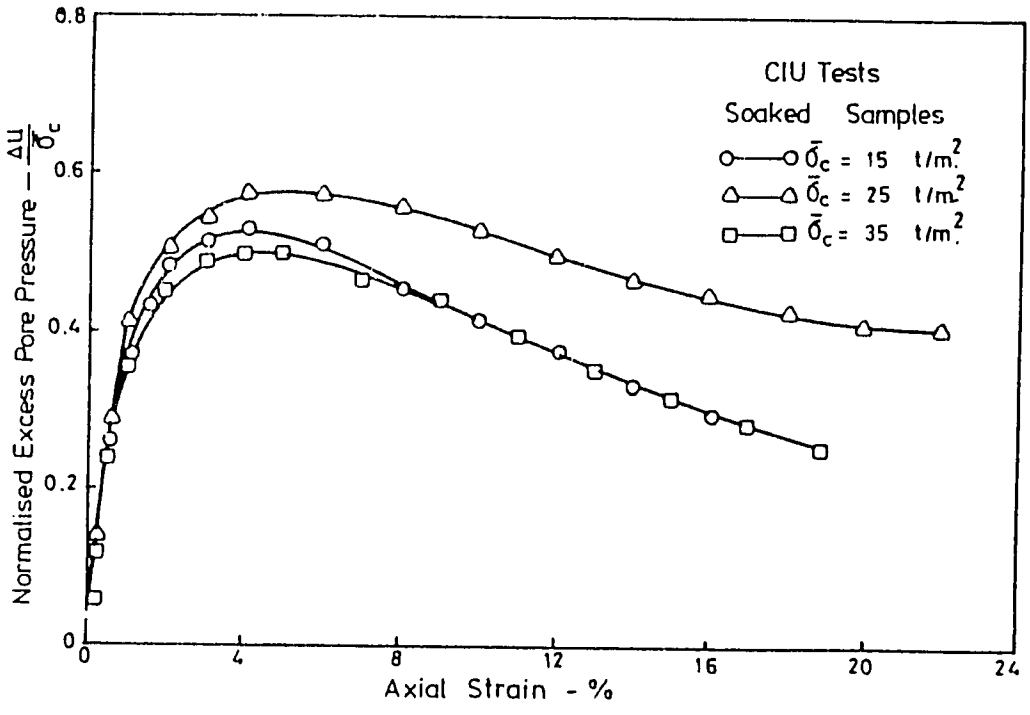


Fig.41 Normalised Excess Pore Pressure vs Axial Strain Relationship for Unsoaked and Soaked Samples - CIU Tests (After HAQUE, 1977)

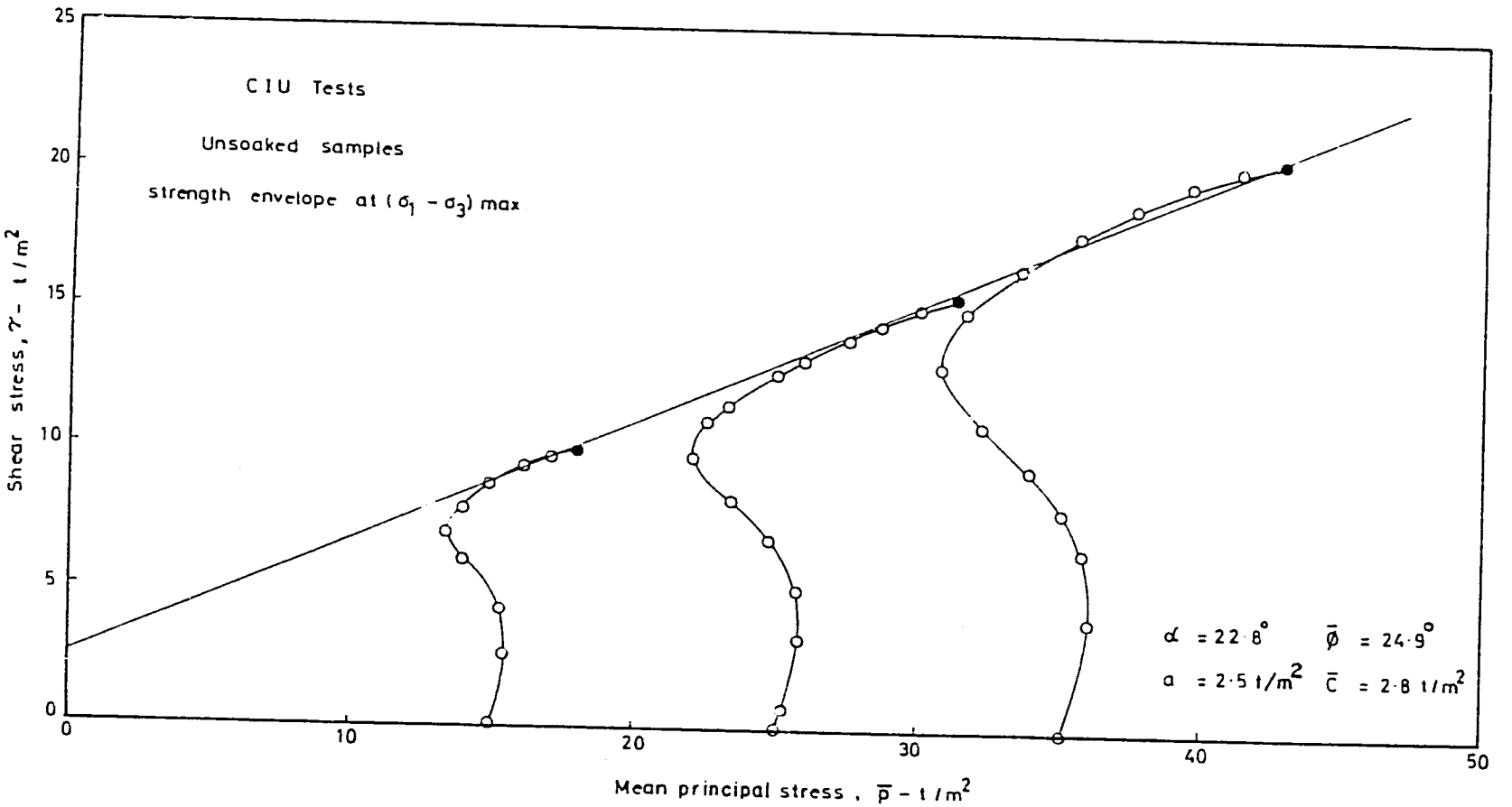


Fig.42 Effective Stress Path and Strength Envelope at $(\sigma_1 - \sigma_3)_{max}$ - CIU Tests, Unsoaked Samples (After HAQUE, 1977)

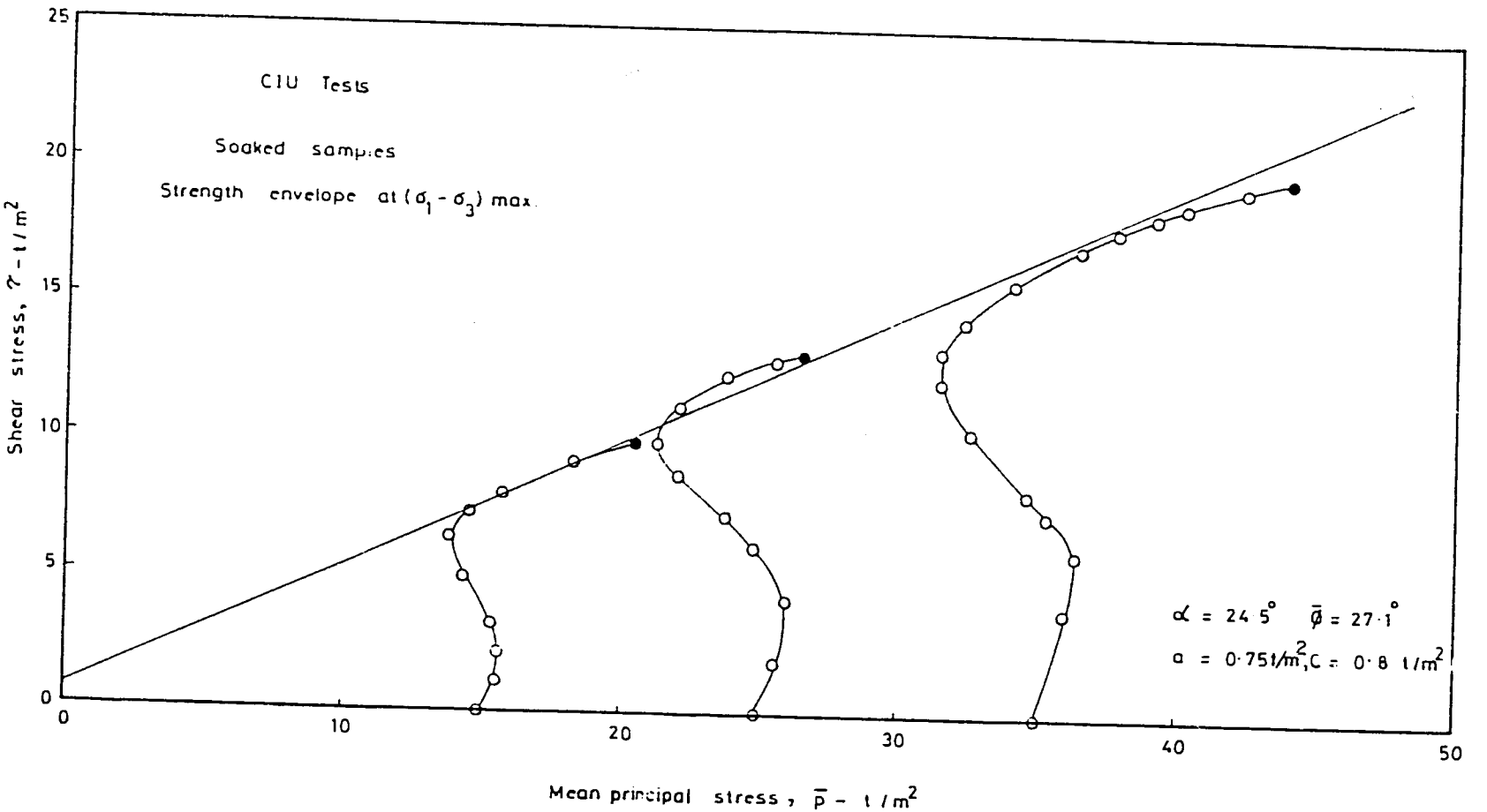


Fig.43 Effective Stress Path and Strength Envelope at $(\sigma_1 - \sigma_3)_{max}$ - CIU Tests, soaked Samples (After HAQUE, 1977)

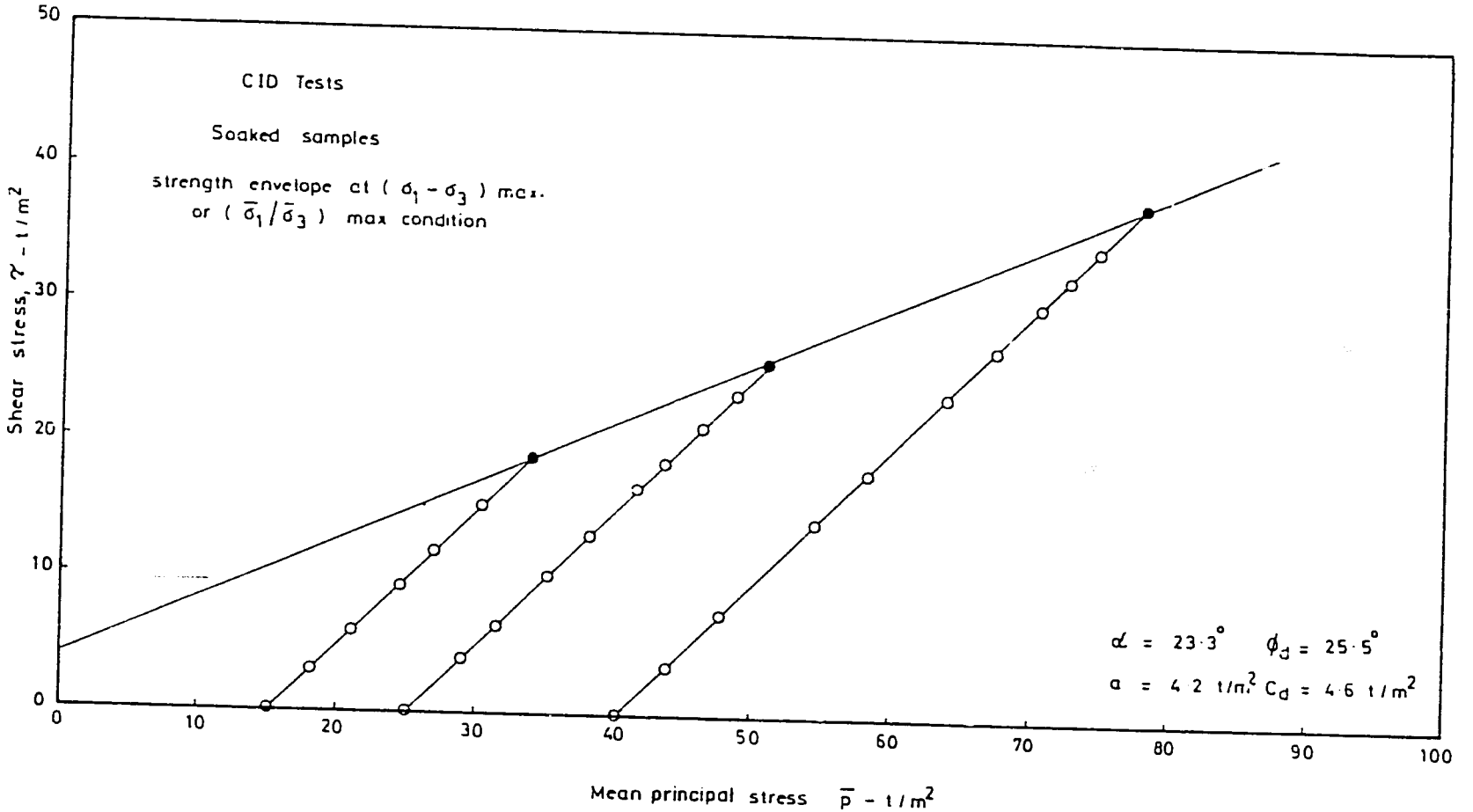


Fig.44 Effective Stress Path and Strength Envelope at $(\sigma_1 - \sigma_3)_{max}$ or, $\frac{\sigma_1}{\sigma_3}$ max - CID Tests, Soaked Samples (After HAQUE, 1977)

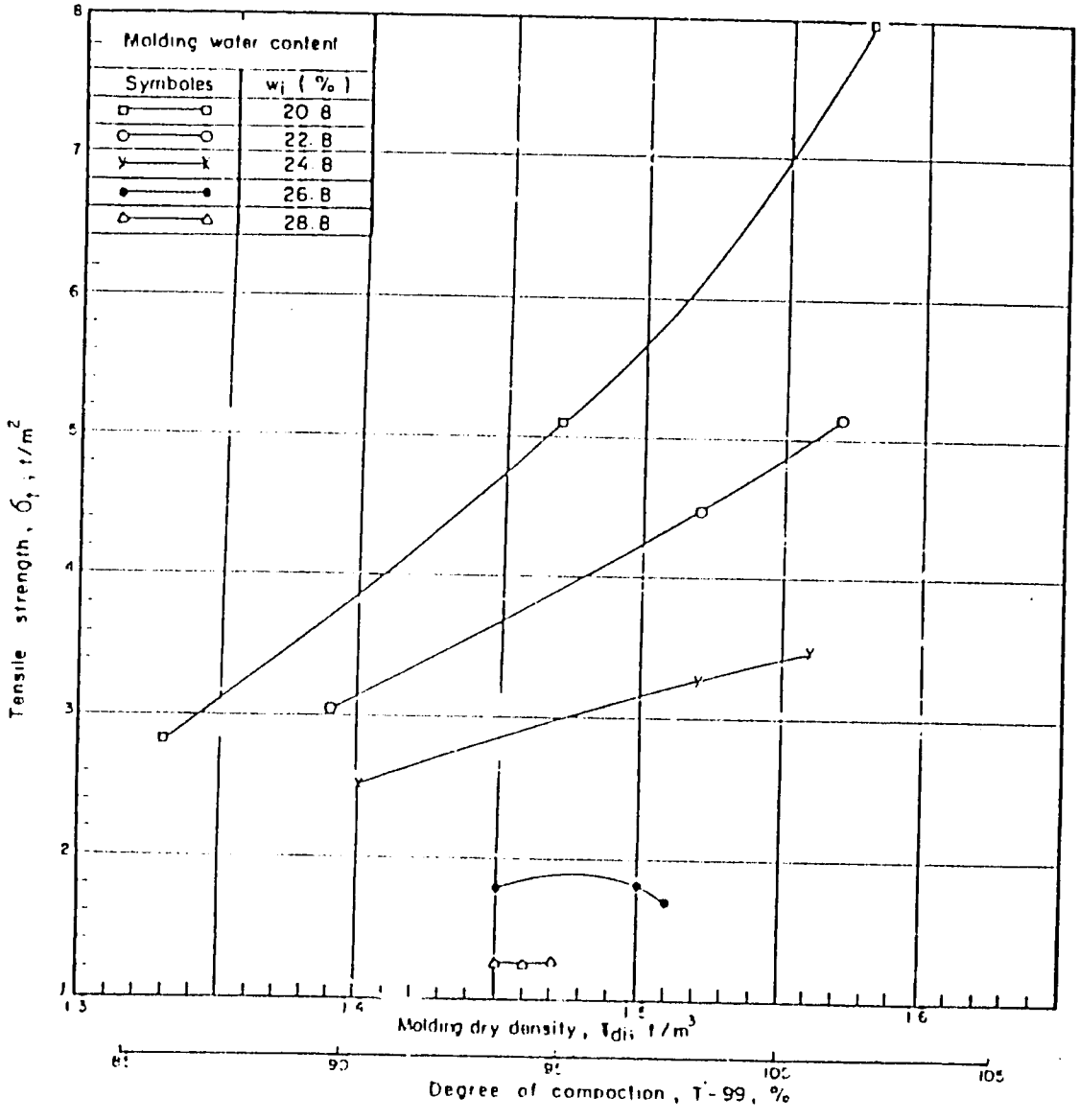


Fig.45 Relationship between the Tensile Strength and the Molding Dry Density (After LEELASITHORN, 1978)

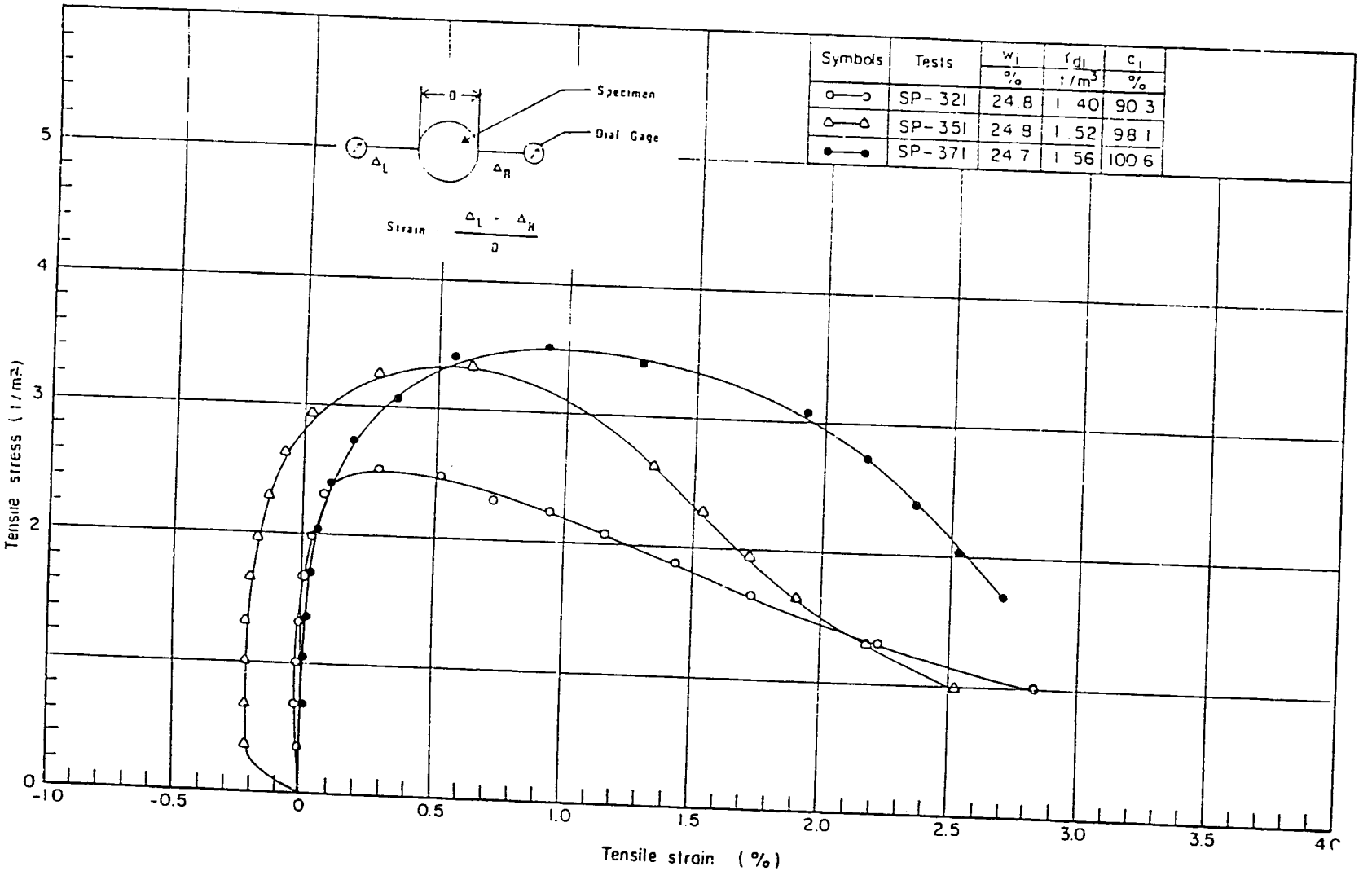
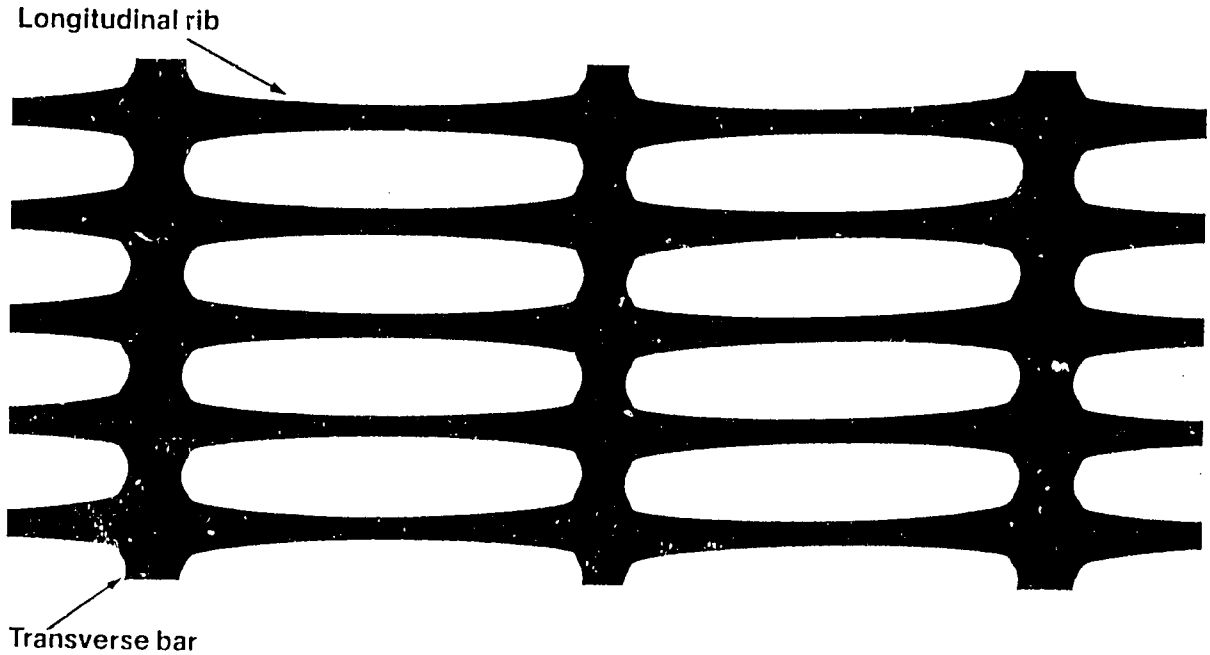
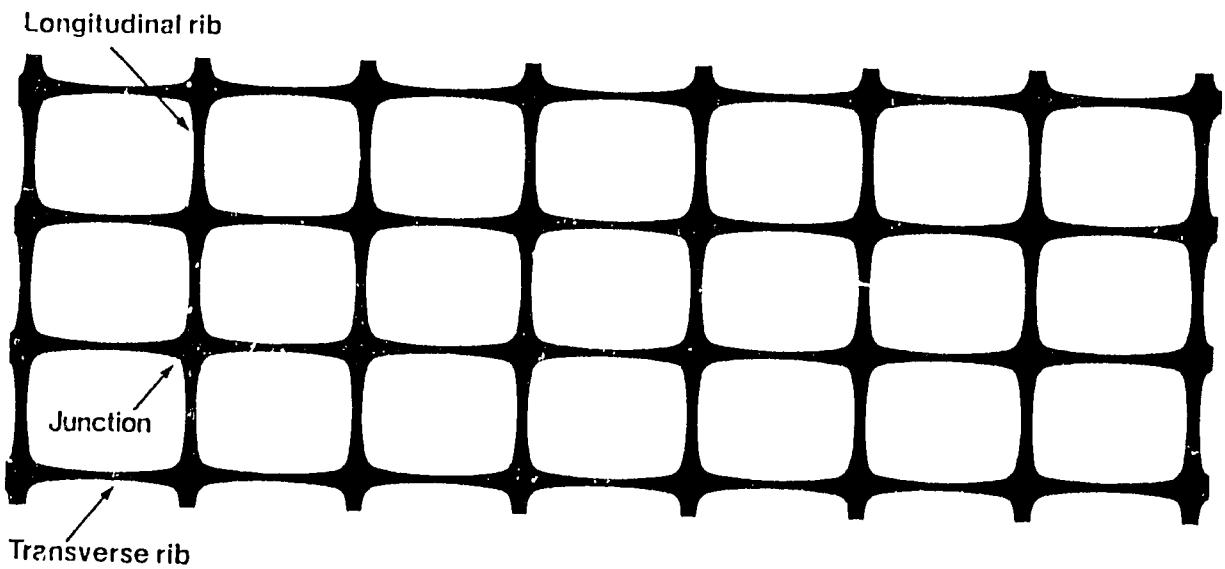


Fig.46 Tensile Stress-Strain Relationship of Specimens Compacted at W_i = 24.8 %
(After LEELASITHORN, 1978)



(a) Uniaxially Orientated Grid



(b) Biaxially Orientated Grid

Fig.47 Uniaxially and Biaxially Orientated Tensar Grids (after NETLON, 1986).

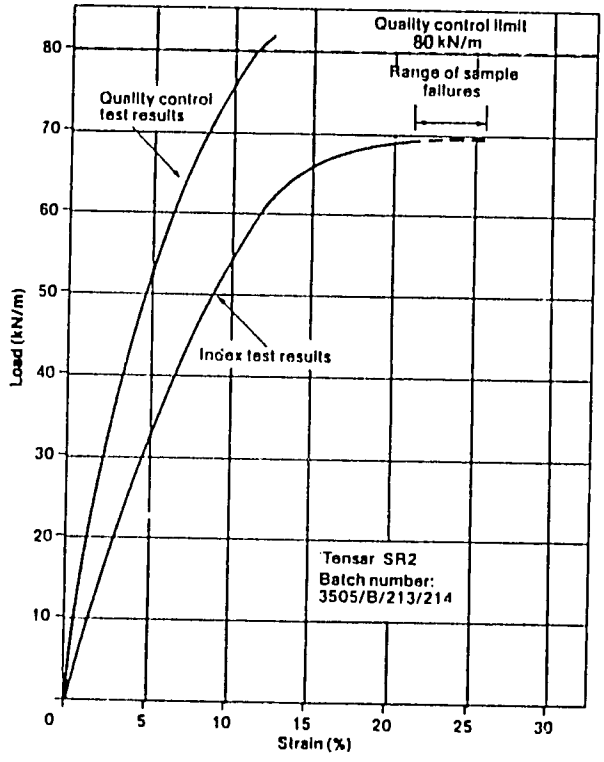
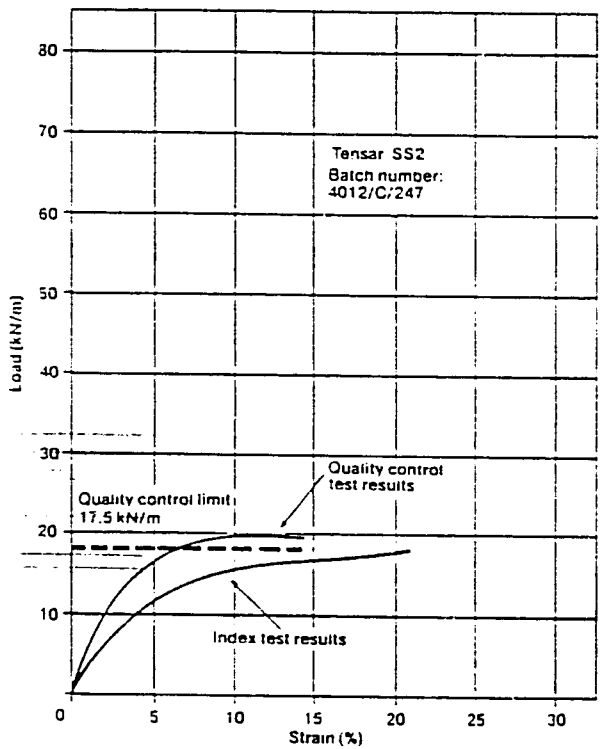
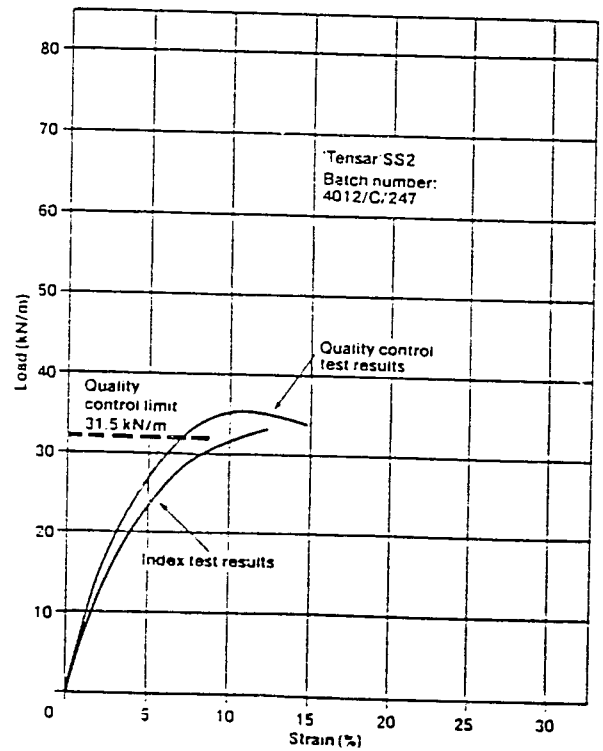


Fig.48 Typical Tensile Test Results for a Uniaxially Orientated Grid (Tensar SR2) (after NETLON, 1986).



(a) Longitudinal



(b) Transverse

Fig.49 Typical Tensile Test Results for a Biaxially Orientated Grid (Tensar SS2) (after NETLON, 1986).

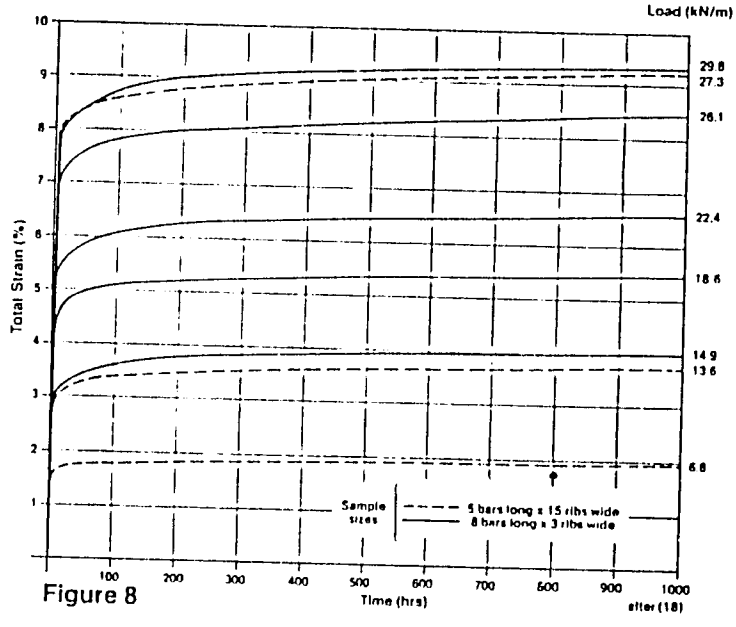


Fig.50 Typical Creep Curves for Tensar SR2 at 20°C (after NETLON, 1986).

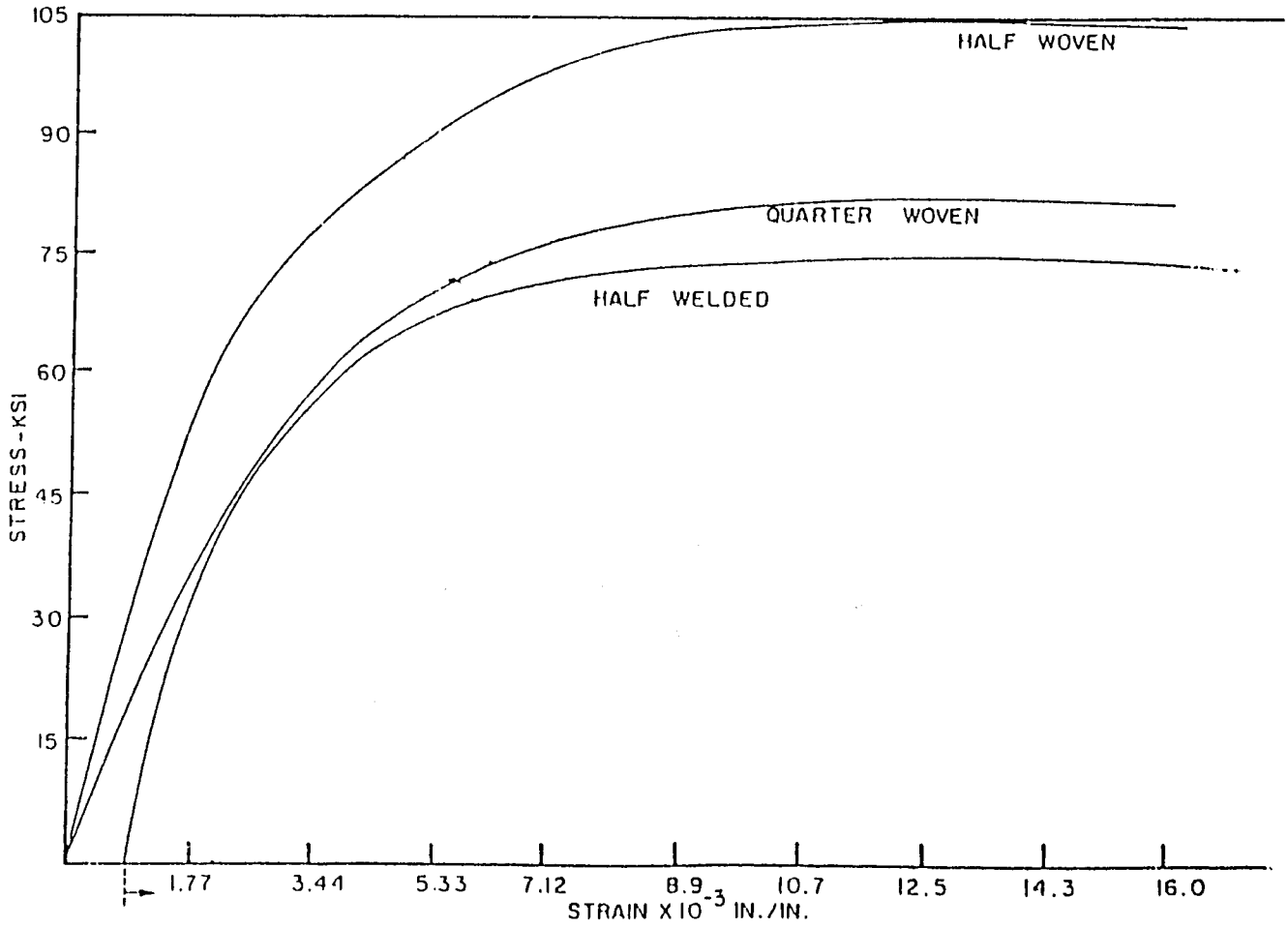


Fig.51 Stress-Strain Curve for Steel Meshes

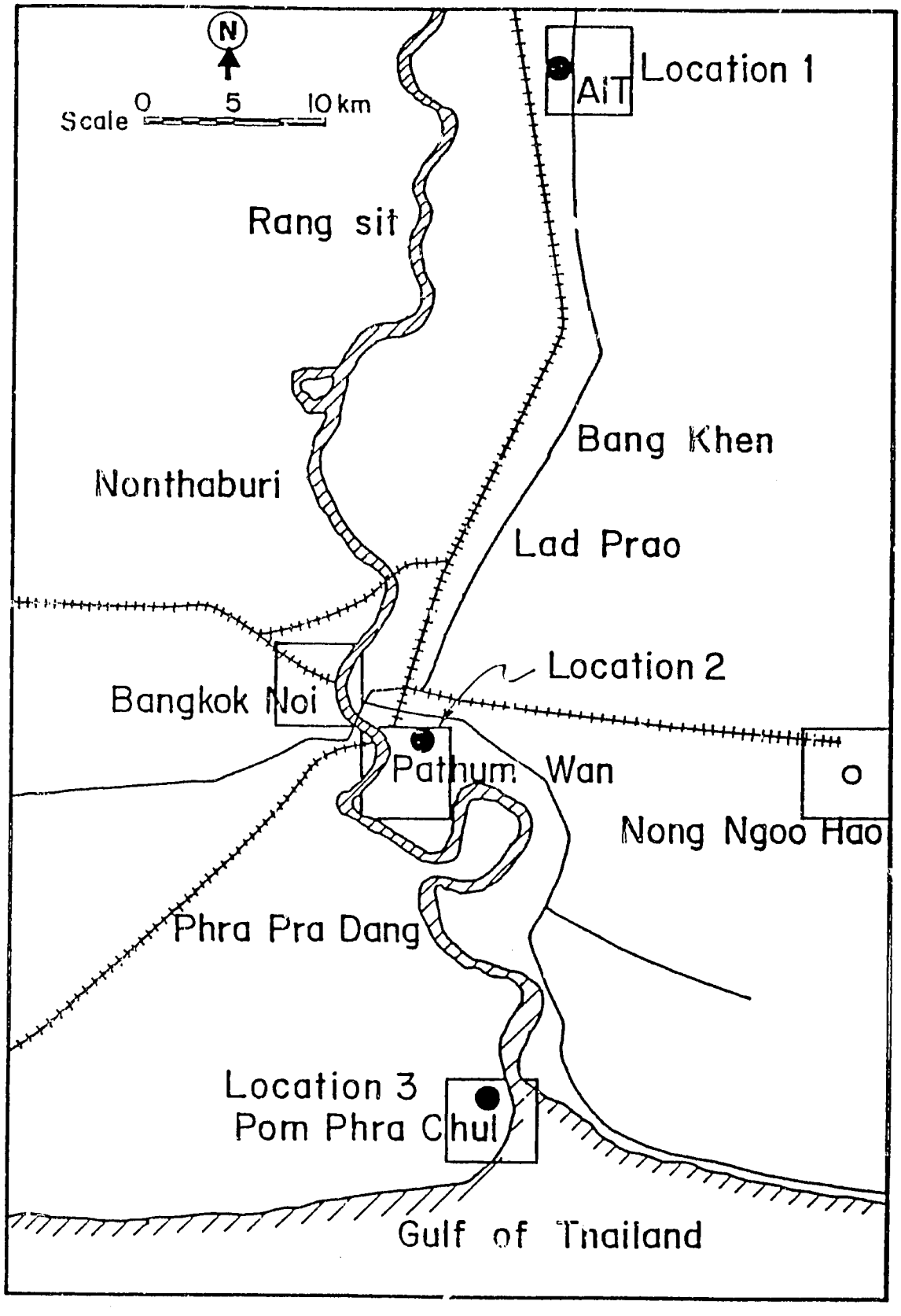


Fig.52 The Chao Phraya (Bangkok) Plain.

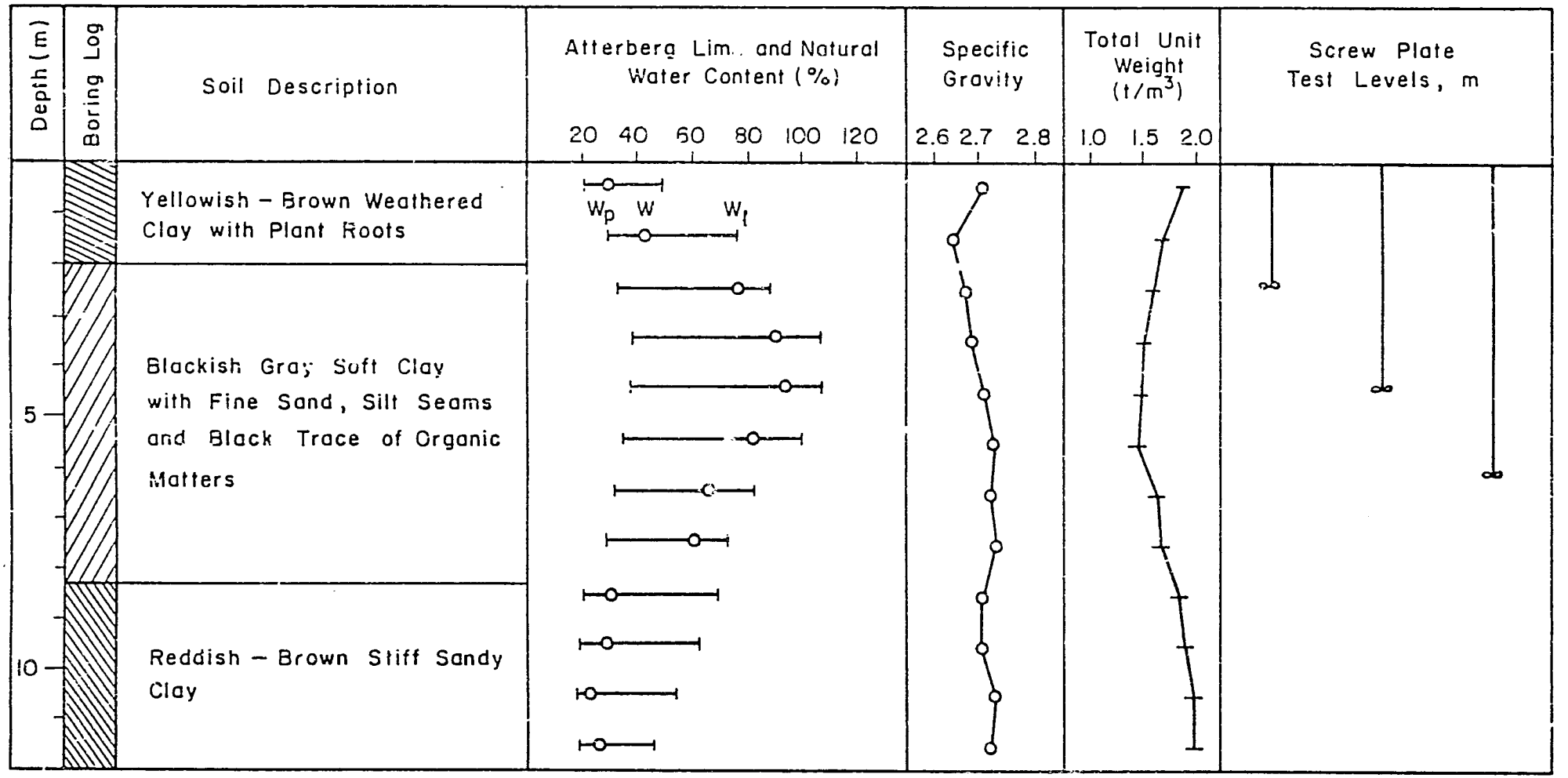


Fig.53 General Subsoil Profile and Soil Properties at the AIT Campus (after BERGADO & HUAN, 1987).

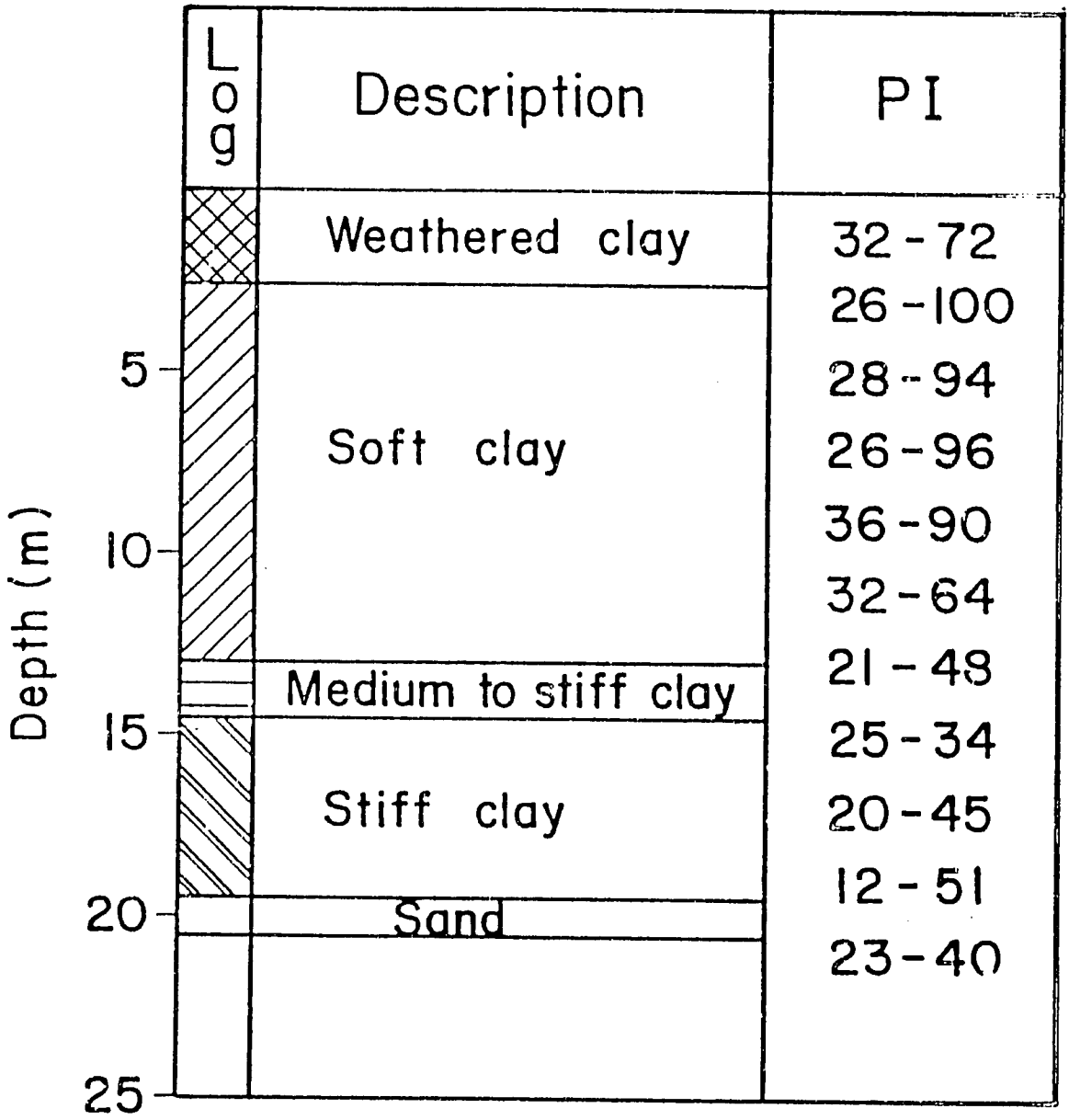


Fig.54 Typical Subsoil Profile at Location 2 (Pathum Wan).

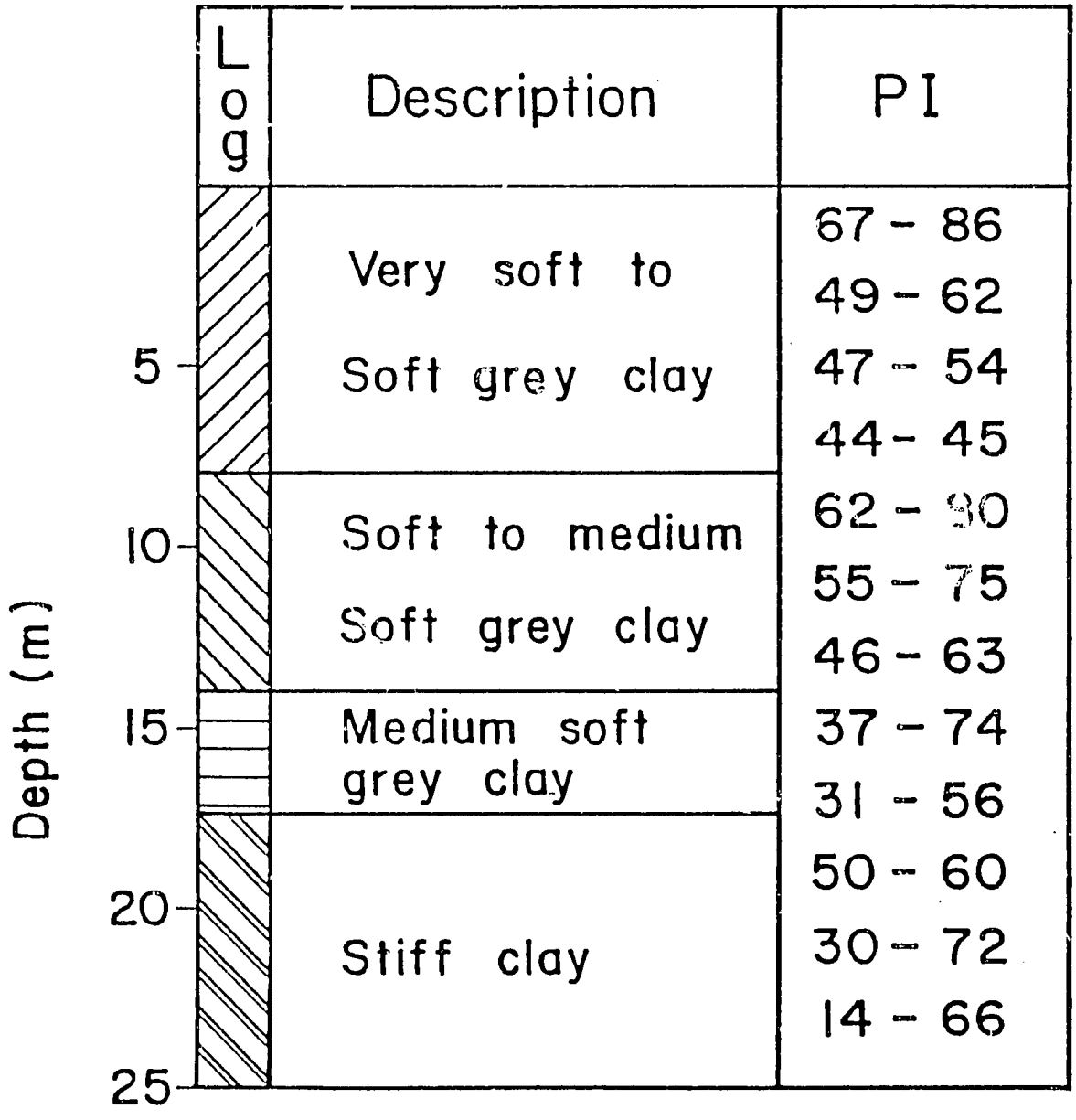


Fig.55 Typical Subsoil Profile at Location 3 (Pom Phra Chul).

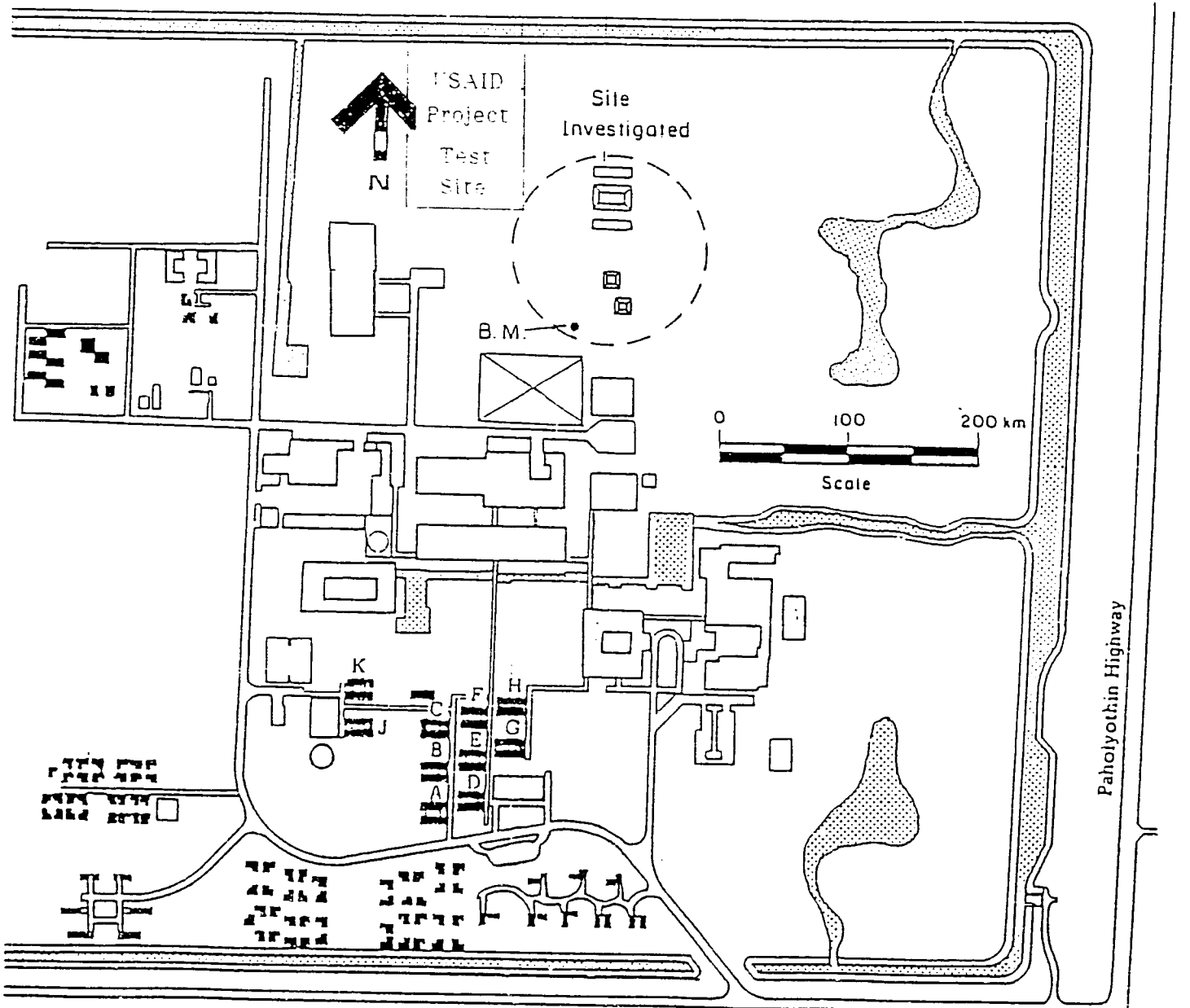


Fig.56 Site Plan of AIT Campus Showing the Proposed Test Embankment Site of the USAID Reinforced Earth Project.

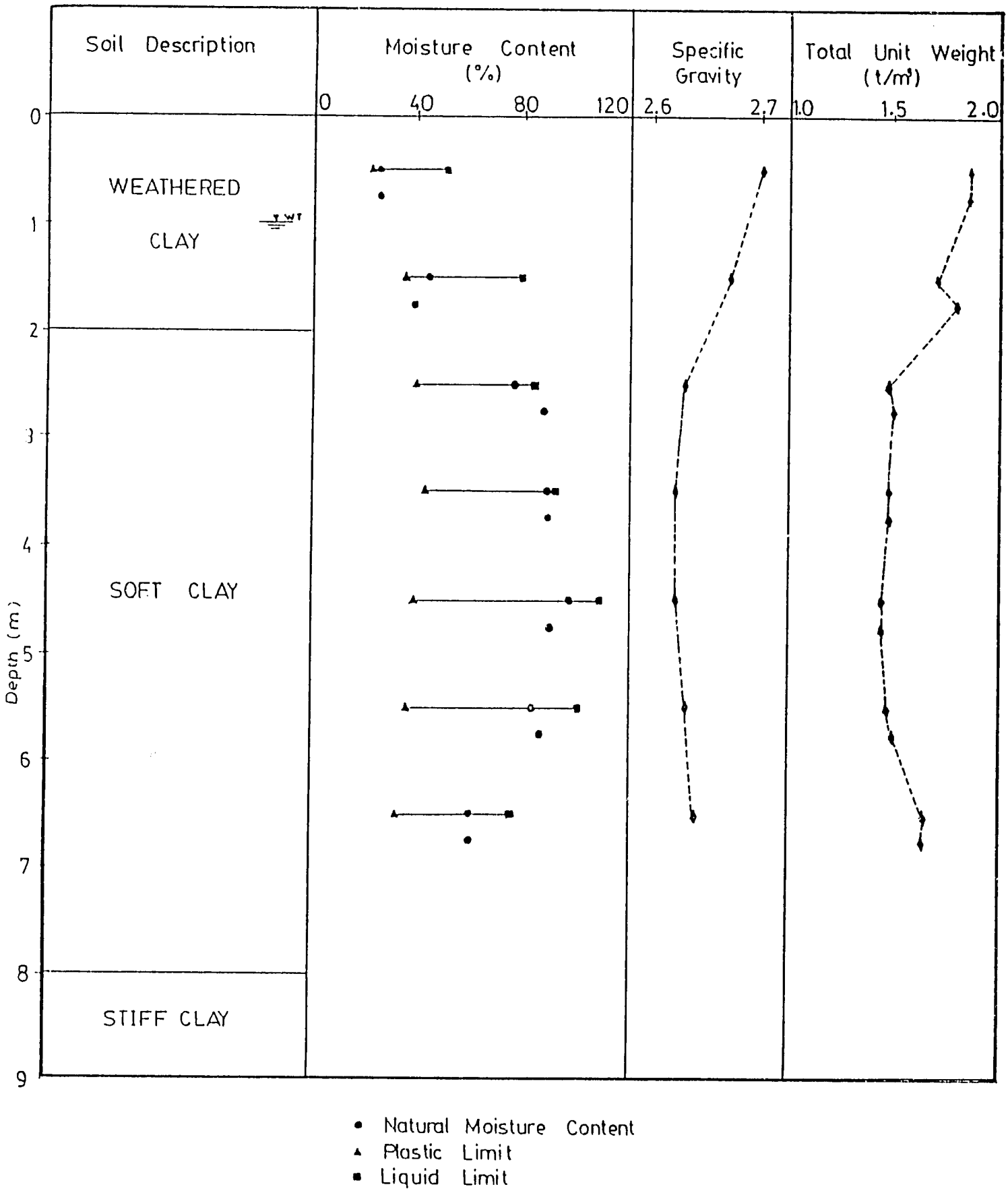


Fig.57 Soil Profile and General Soil Properties at the Site (after MOY, 1984).

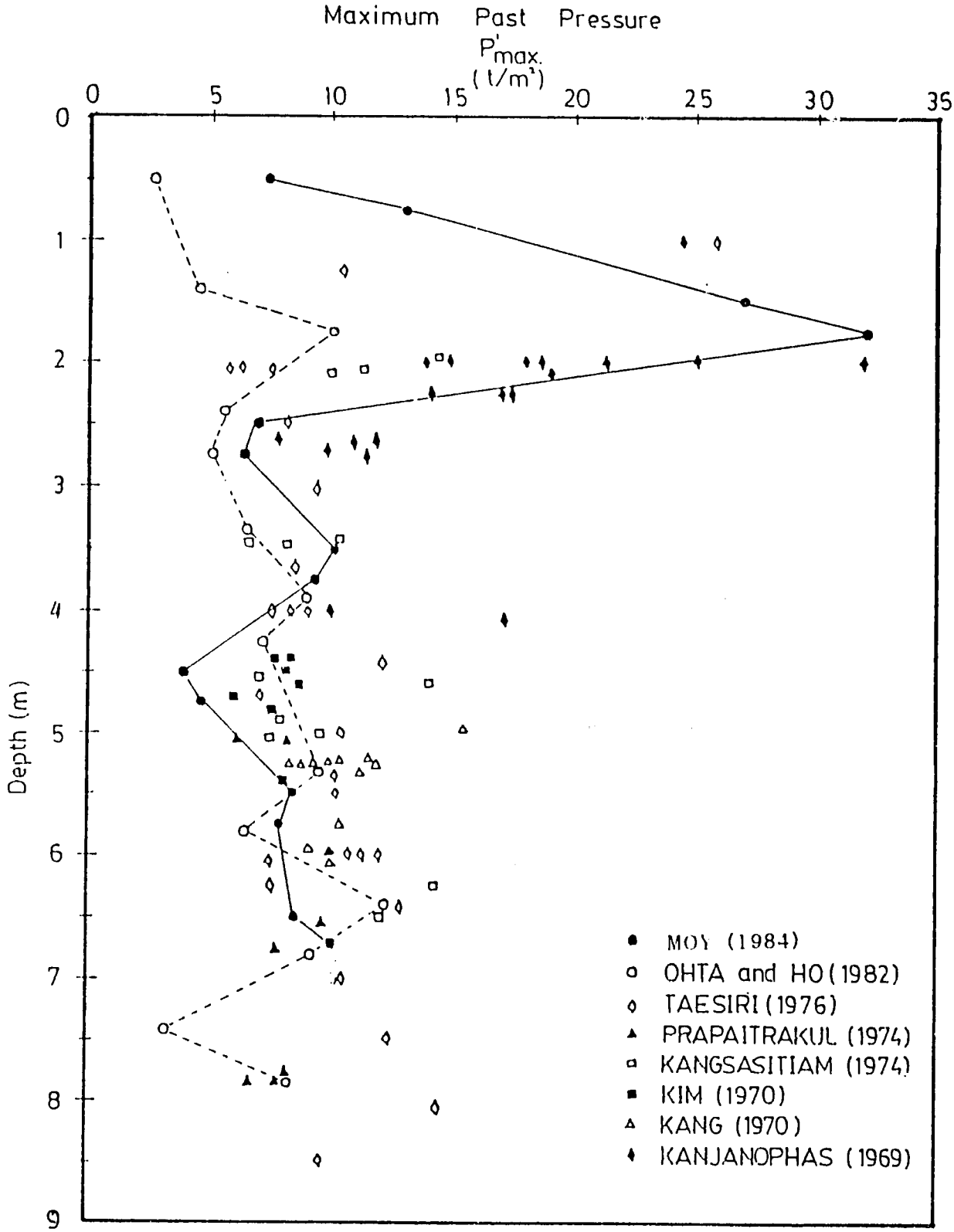


Fig.58 Variation of Maximum Past Pressure with Depth at the AIT Campus as Determined by Various Investigators.

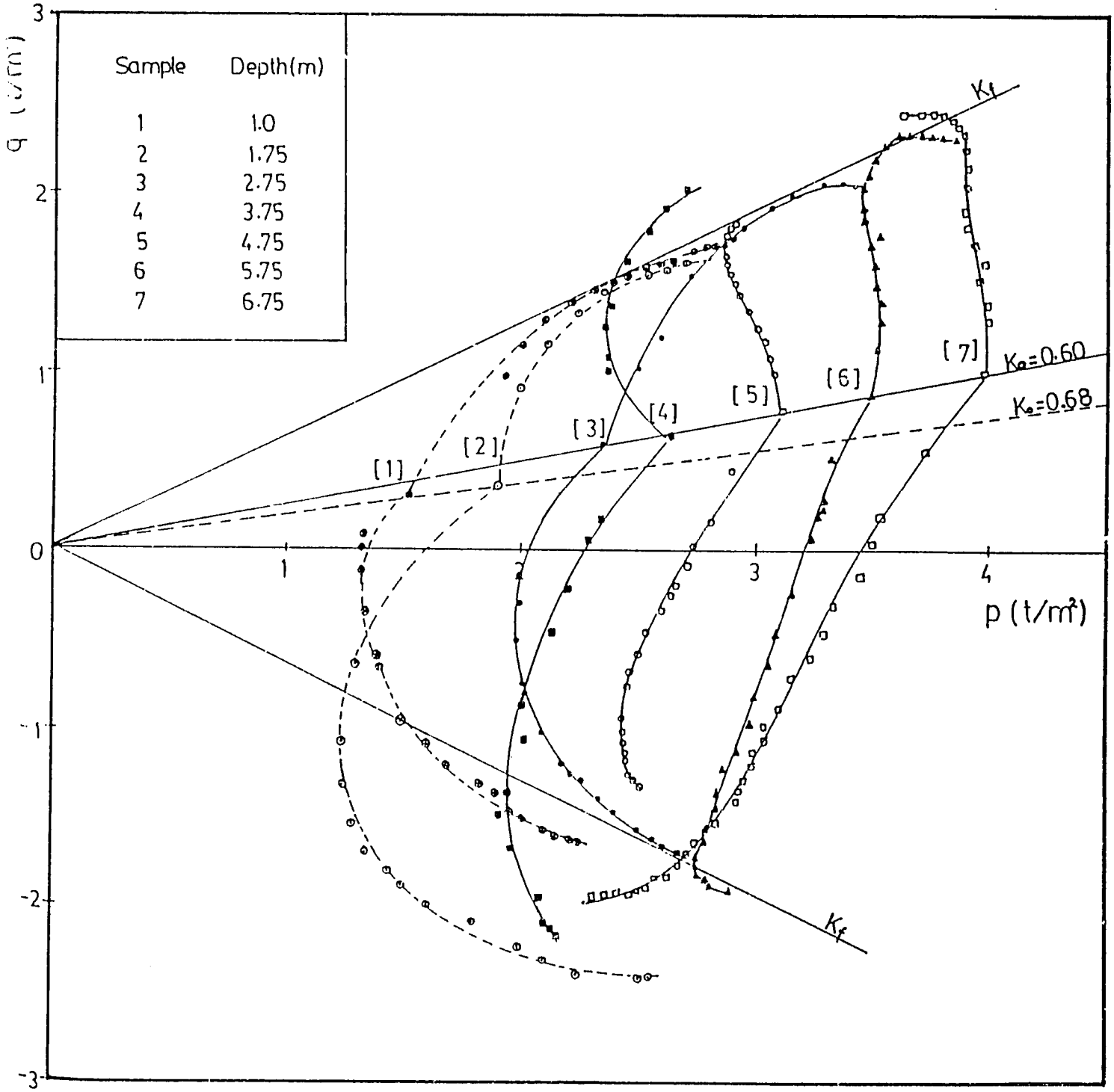


Fig.59 Effective Stress Paths from Triaxial Compression and Extension Tests (after MOY, 1984).

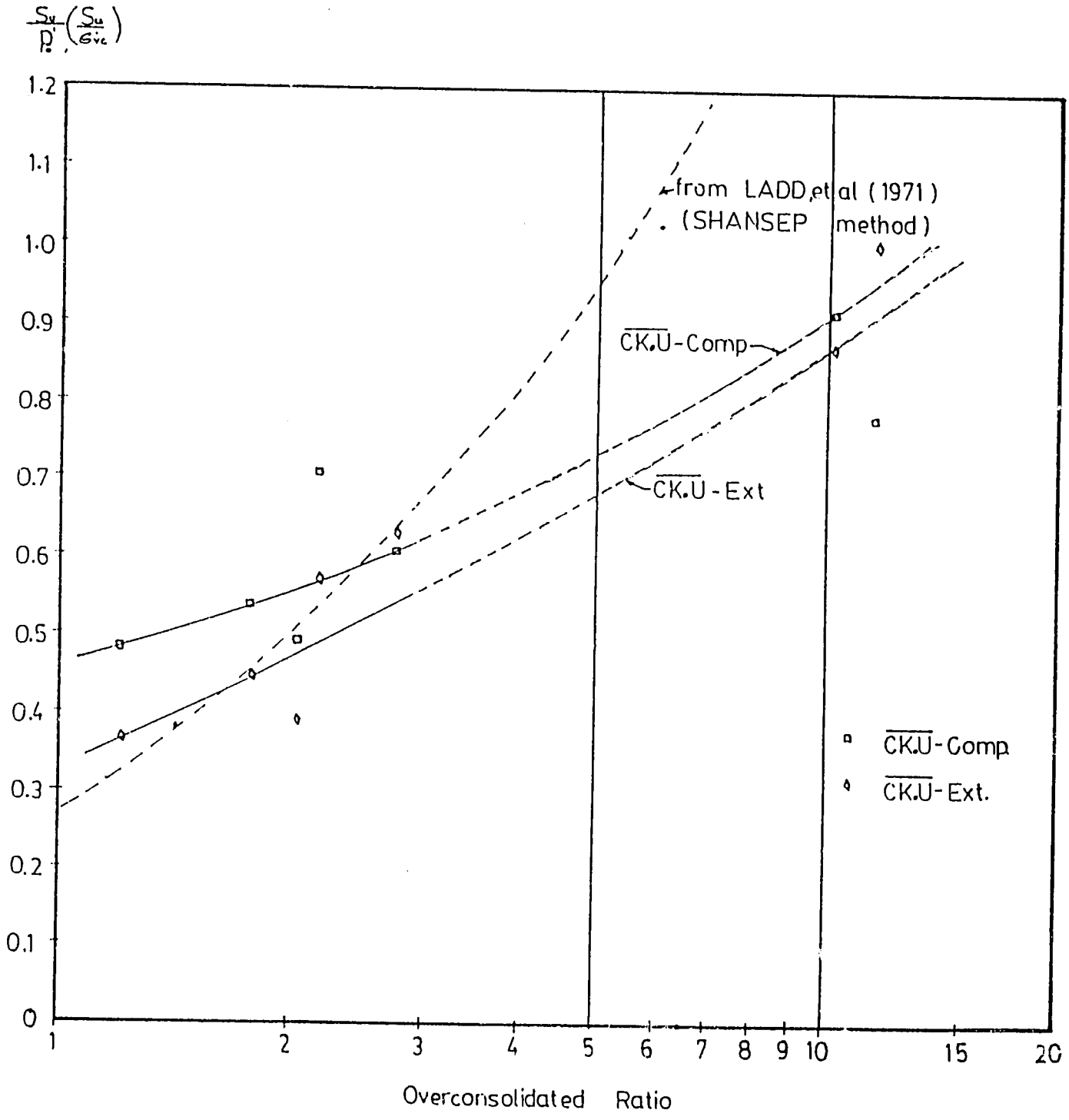


Fig.60 Variation of the Normalized Undrained Shear Strength from Triaxial Tests with OCR (after MOY, 1984).

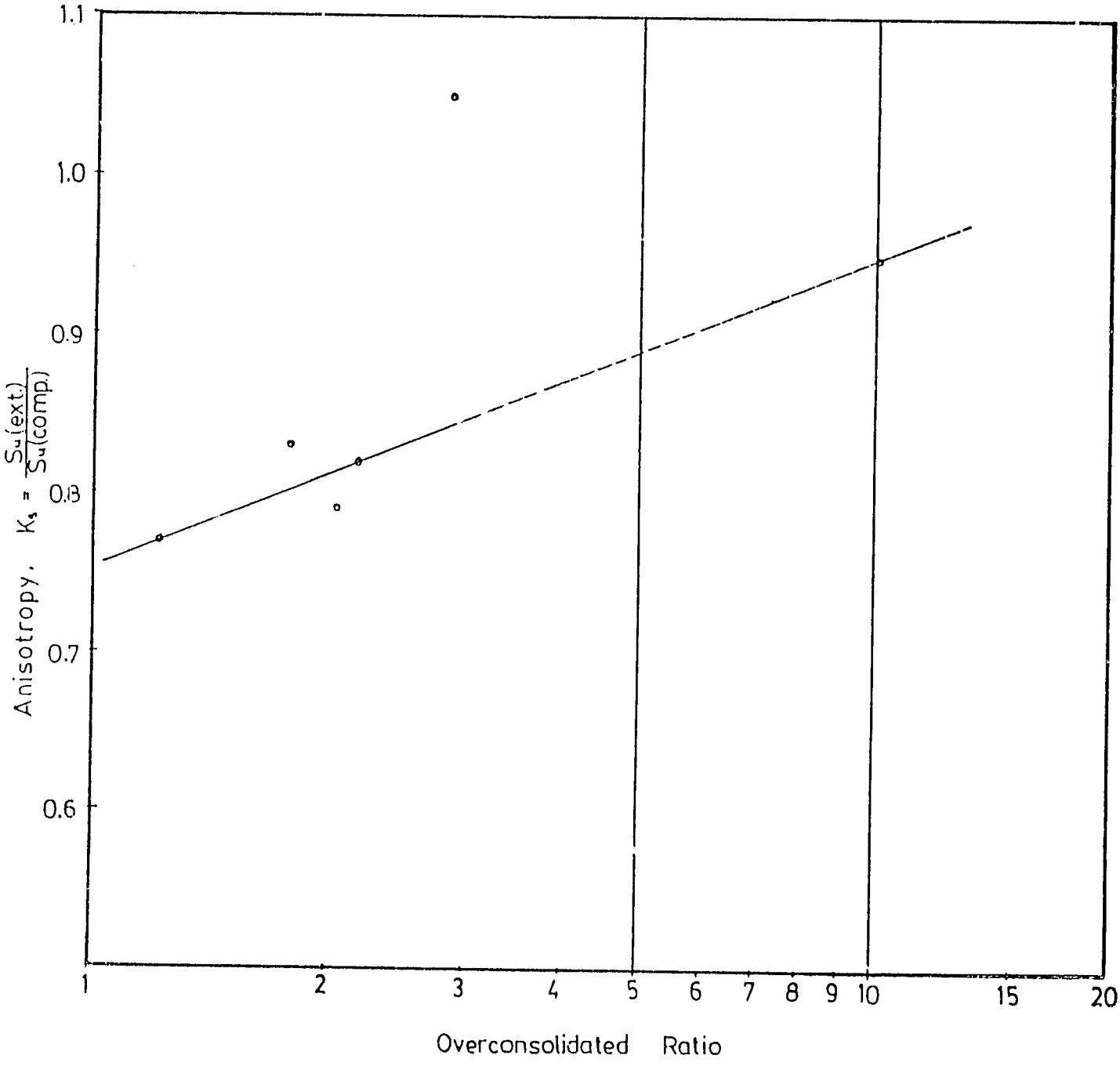


Fig.61 Variation of Anisotropy K_s from Triaxial Tests with OCR (after MOY, 1984).

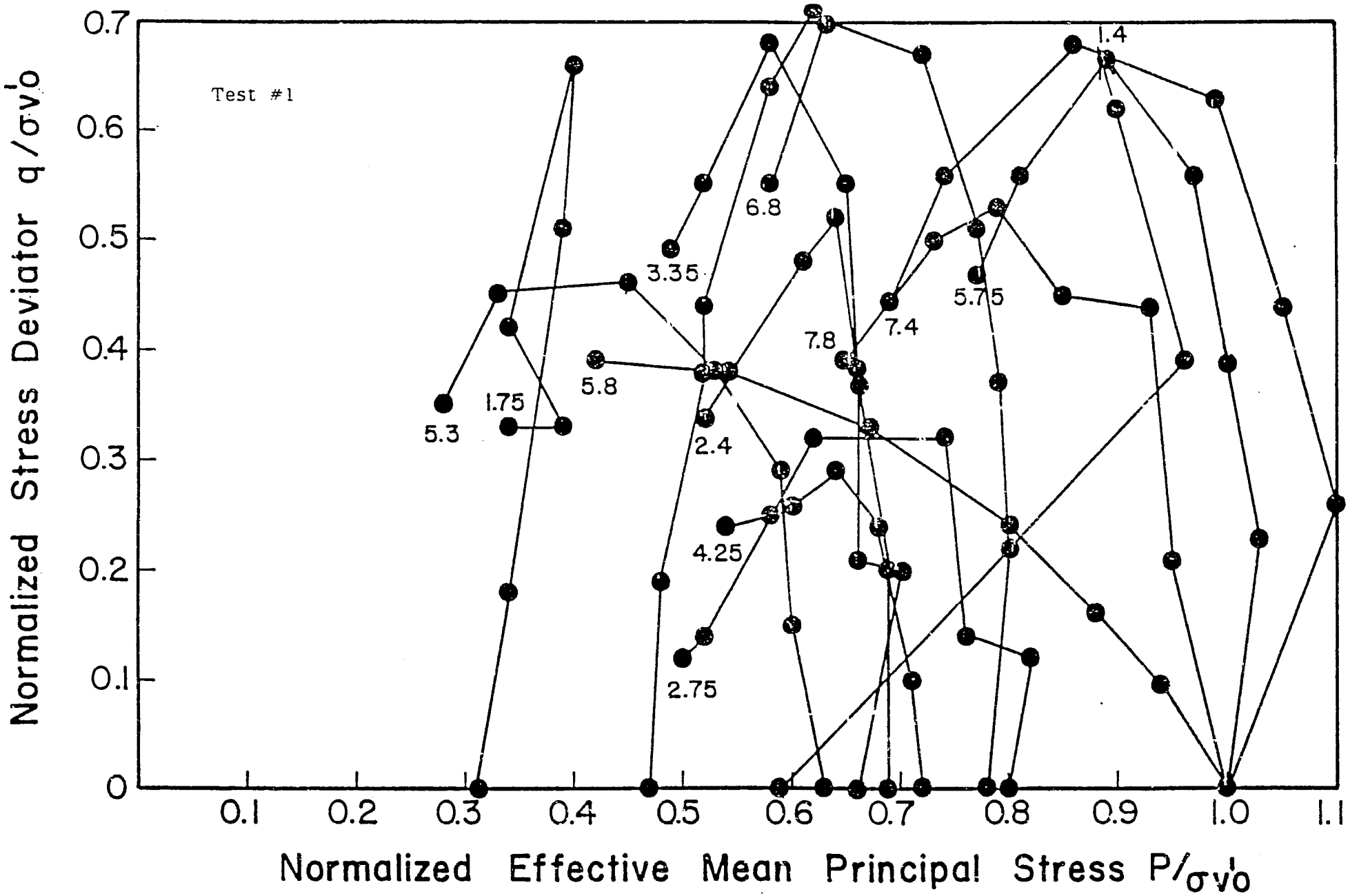


Fig.62 Normalized Stress Paths from CIU Tests (after OHTA & HO, 1982).

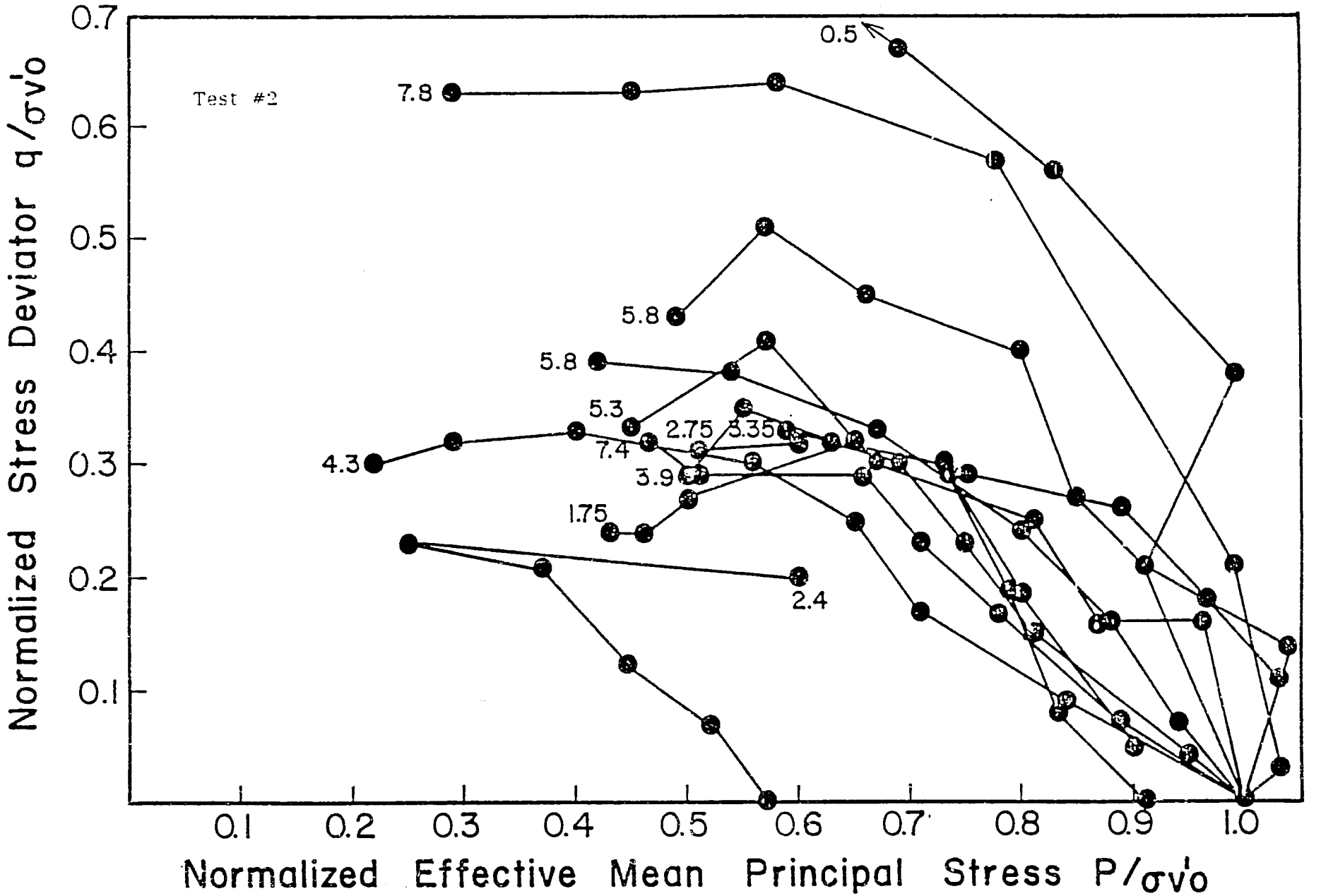
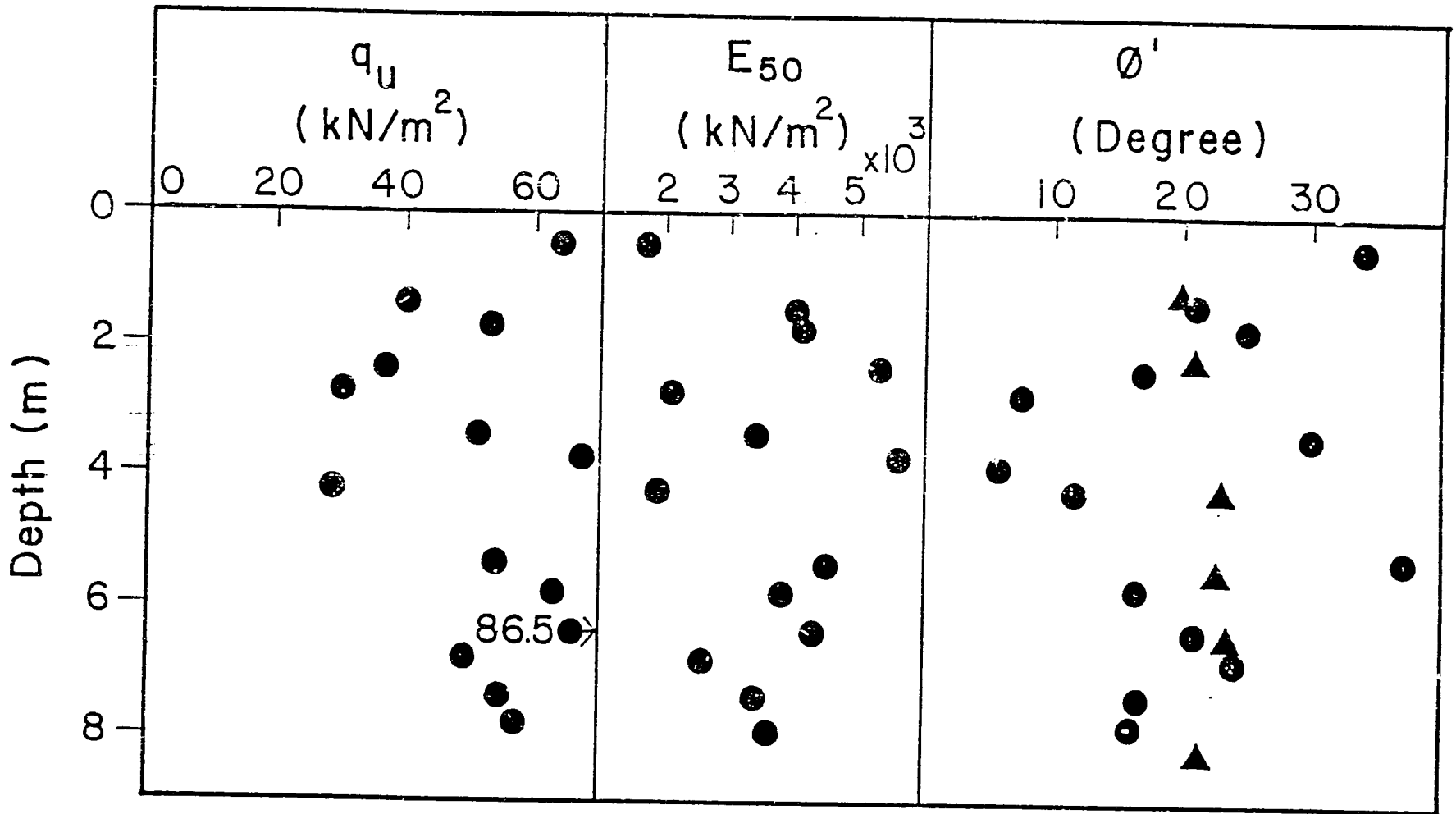


Fig.63 Normalized Stress Paths from CIU Tests (after OHTA & HO, 1982).



● OHTA & HO (1982)

▲ Mean Value from Previous Theses

Fig.64 Variations of q_u, E_{50} and ø' with Depth (after OHTA & HO, 1982).

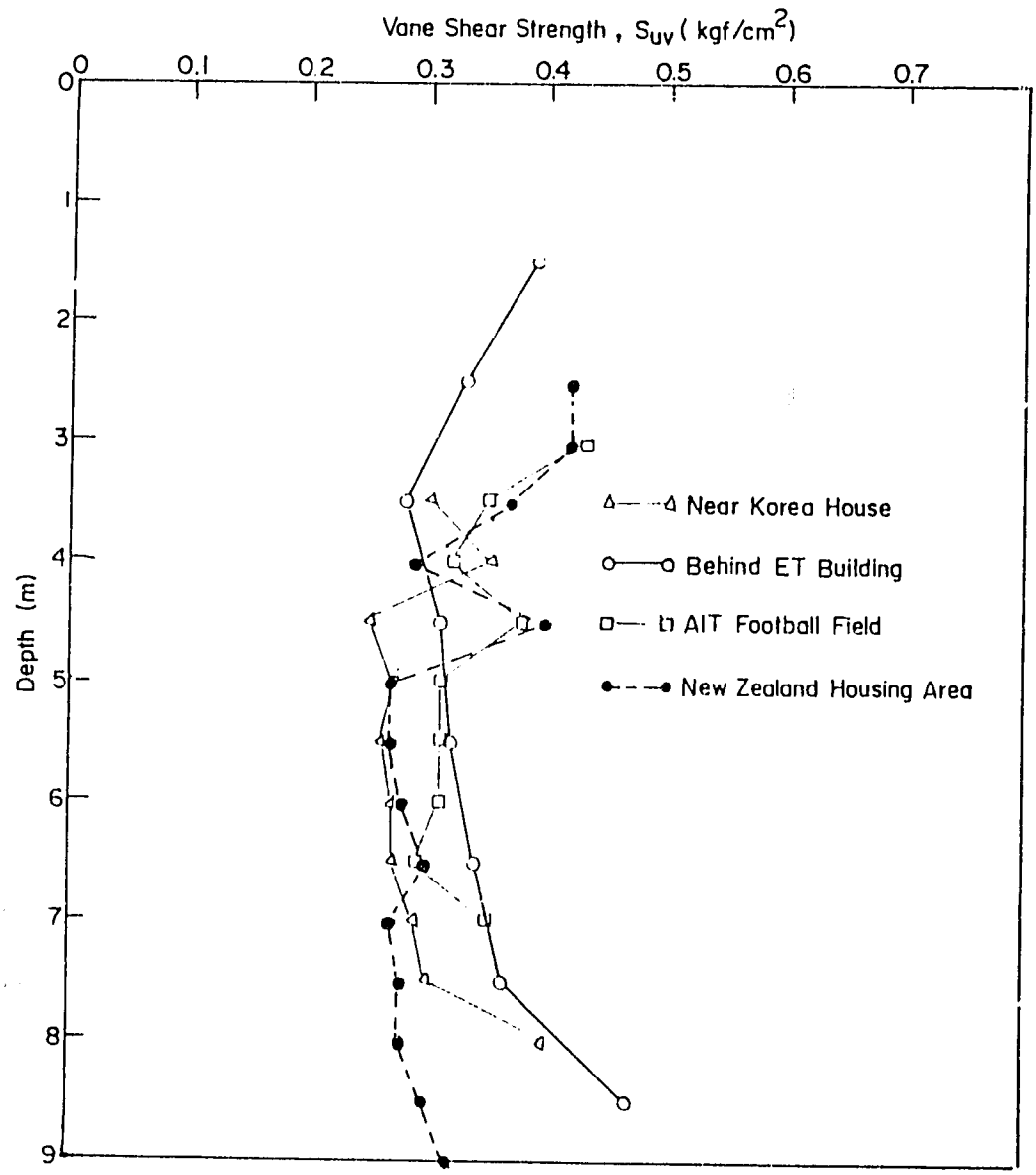


Fig.65 Variation in Vane Shear Strength with Depth (after BERGADO et al, 1986).

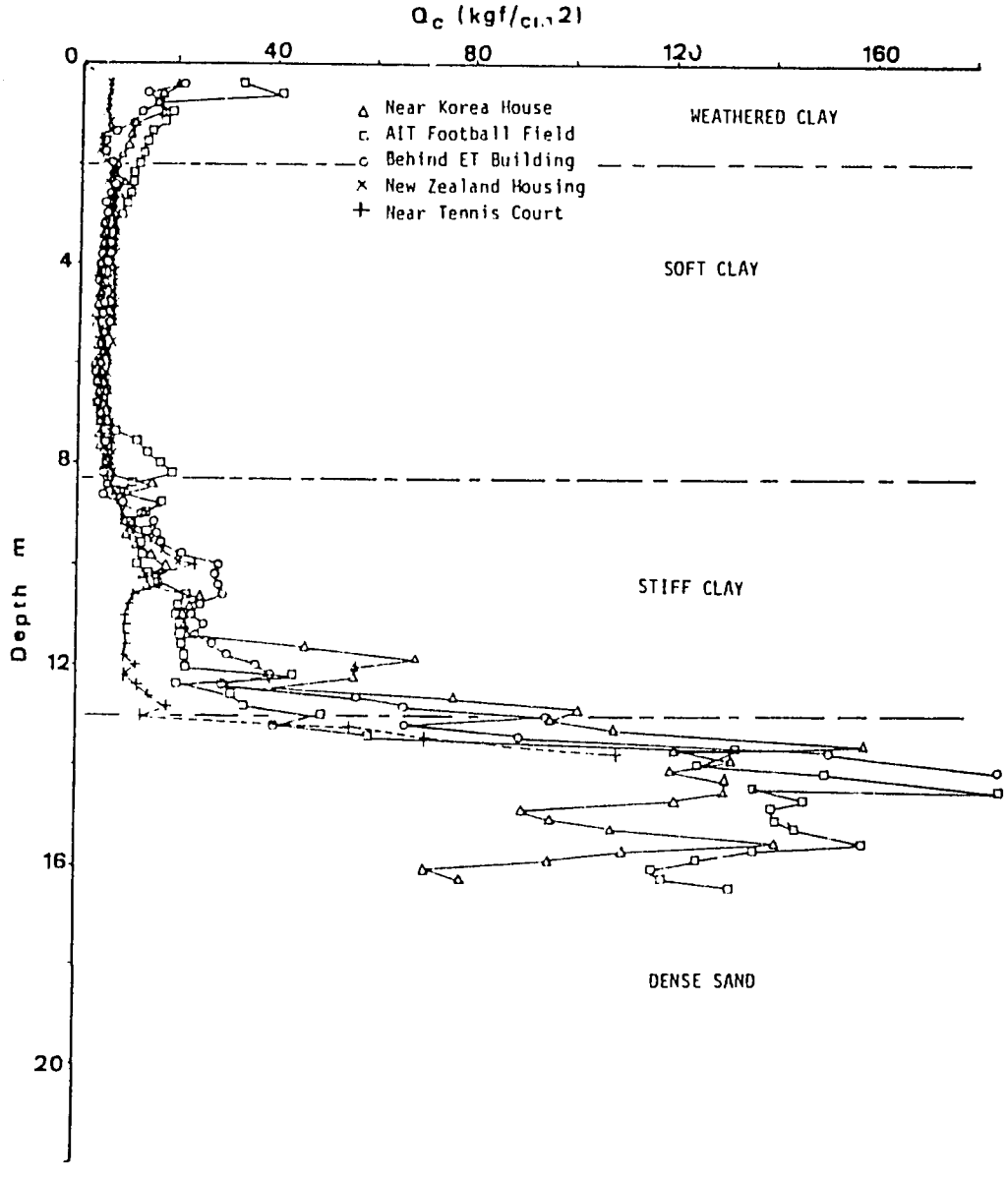


Fig.66 Variation of Dutch Cone Resistance with Depth (after BERGADO et al, 1986).

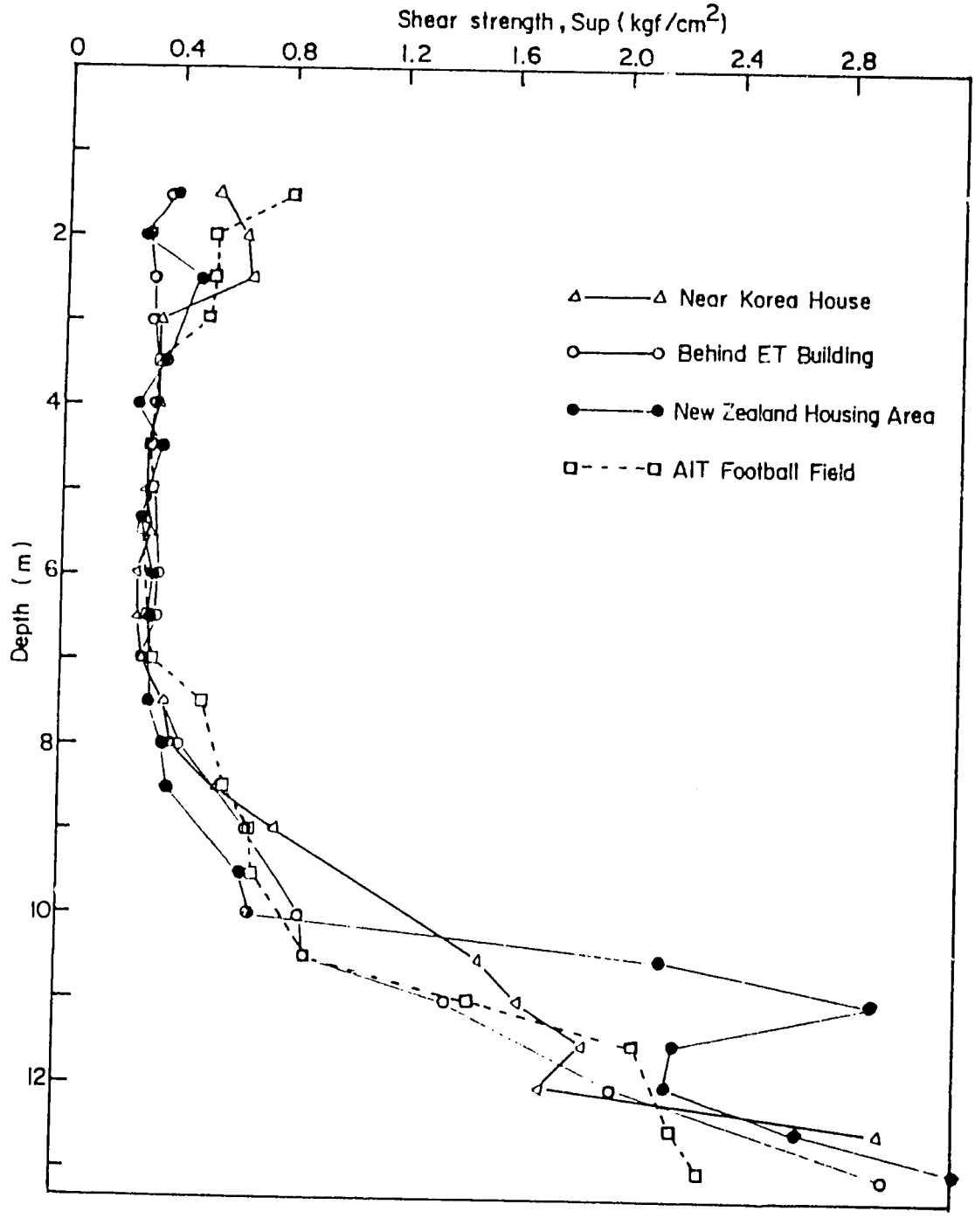


Fig.67 Variation in Shear Strength with Depth Obtained from Pressuremeter Test (after BERGADO et al, 1986).

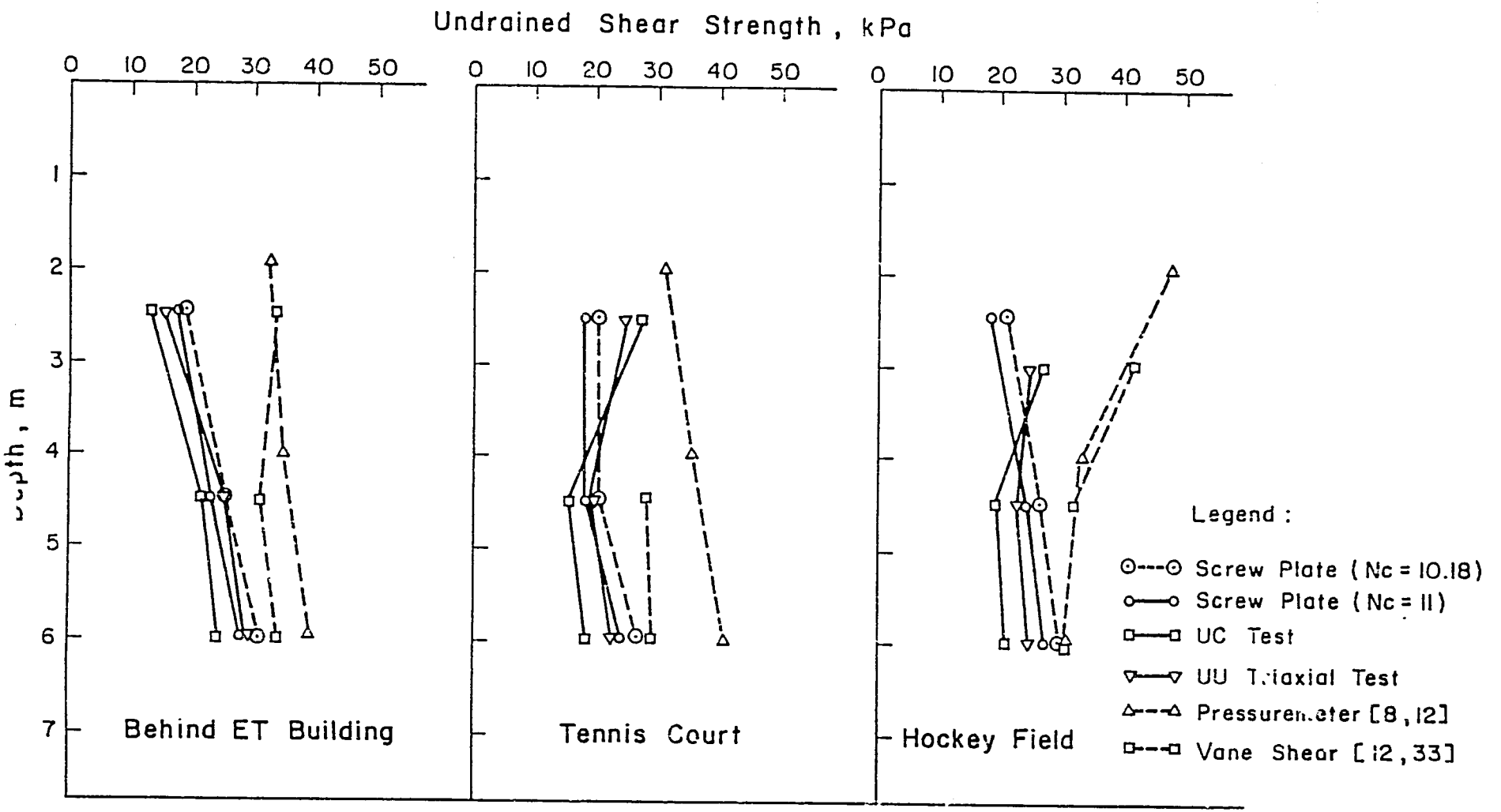


Fig.68 Comparison of Undrained Shear Strength for Screw Plate, Unconfined Compression (UC), Unconsolidated Undrained (UU) Triaxial, Vane Shear, and Pressuremeter Tests (after BERGADO & HUAN, 1987).

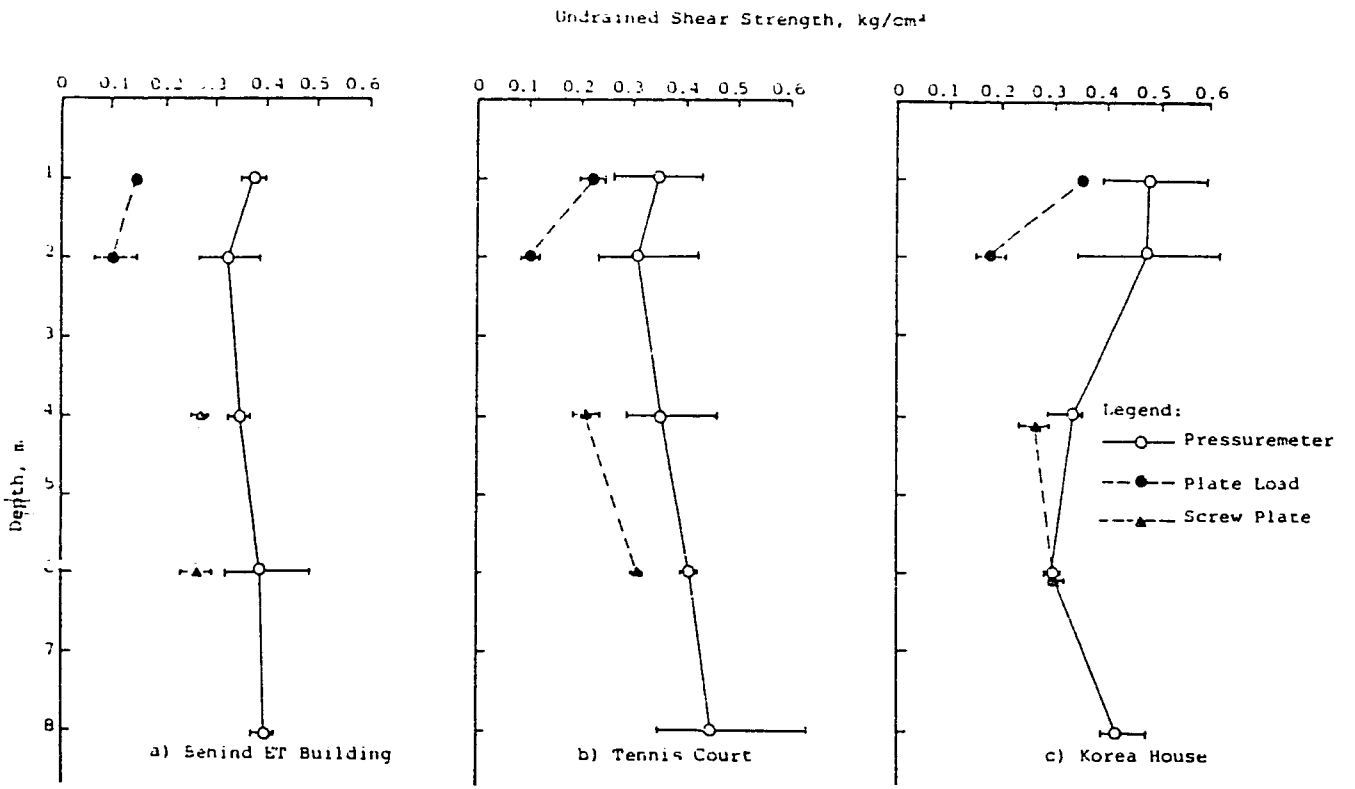


Fig.69 Comparison of Undrained Shear Strength from Pressuremeter, Plate Load, and Screw Plate Tests (after BERGADO et al, 1986).

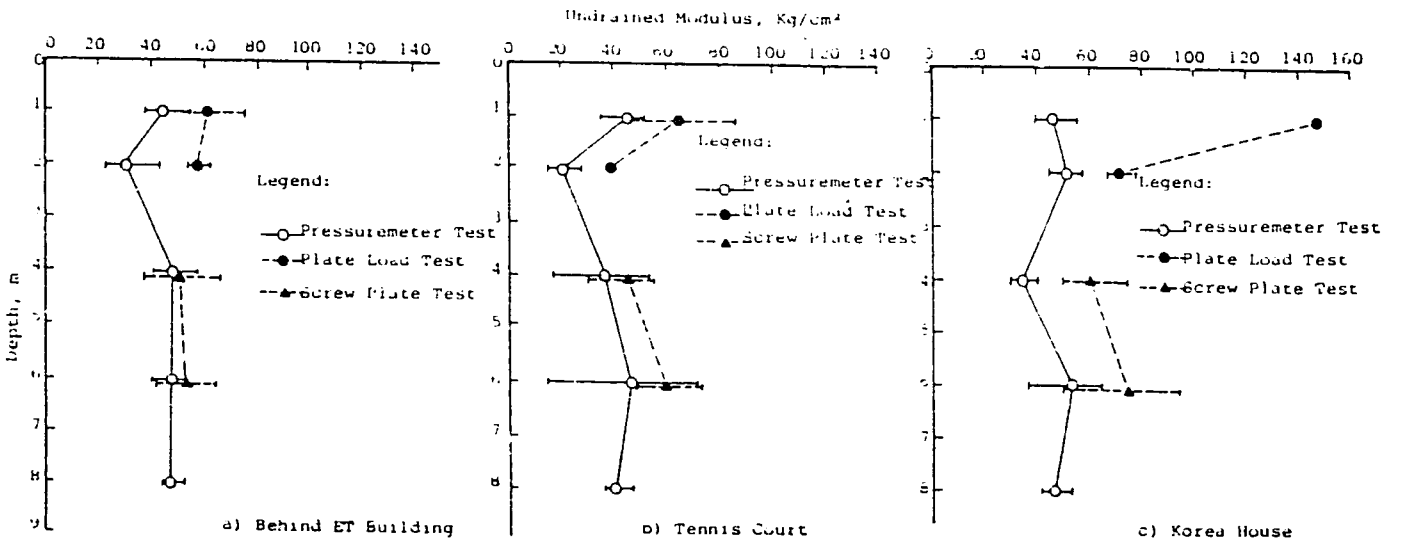


Fig.70 Comparison of Undrained Modulus from Pressuremeter, Plate Load, and Screw Plate Tests (after BERGADO et al. 1986).

APPENDIX

Properties of Tensar Geogrids

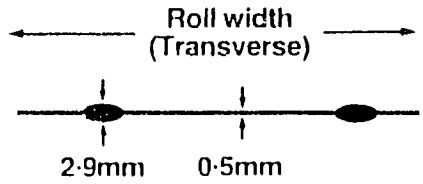
Tensar SS1 Geogrids

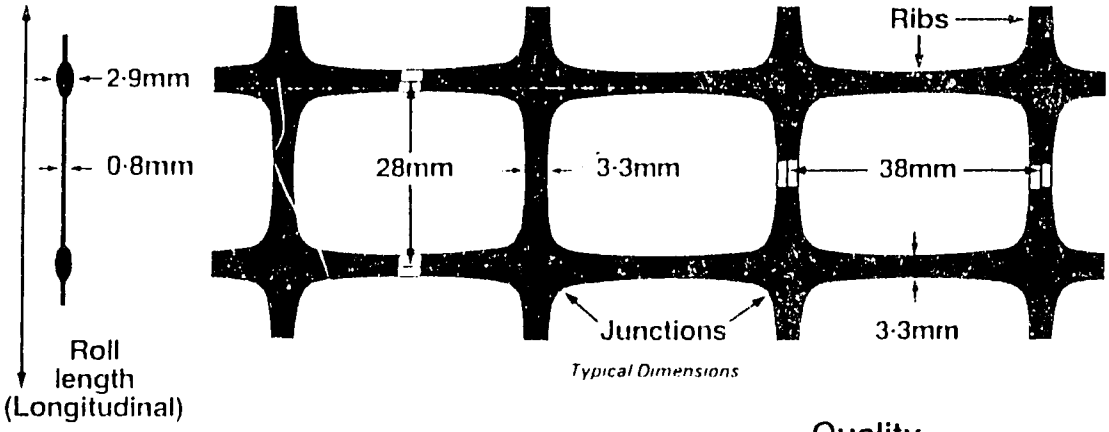
Tensar®

Specification

'Tensar' SS1 Geogrids were developed for stabilising weak, low load bearing soils.

e.g. for the construction of temporary and permanent roads.





Typical Dimensions

Roll dimensions

Length:	50m
Width:	4m
Approx diameter:	0·4m
Approx weight:	44 kg

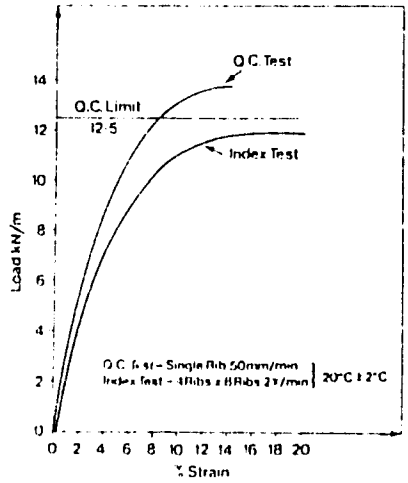
Physical properties of the grids

Weight:	0·2 kg/m²
Colour:	Black

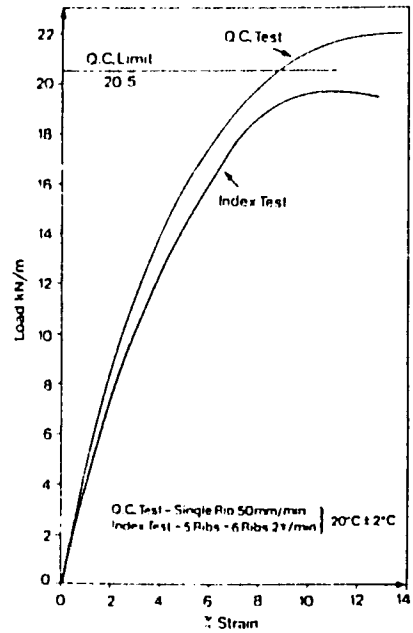
Quality Control Properties

	*Quality Control Strength (kN/m)	Approx Peak Strain (%)
Transverse	20·5	13·8
Longitudinal	12·5	14·0

* Determined as a 95% lower confidence limit



Typical tensile test results for 'Tensar' SS1 Geogrids (Longitudinal direction)



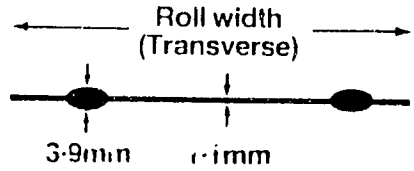
Typical tensile test results for Tensar' SS1 Geogrids (Transverse direction)

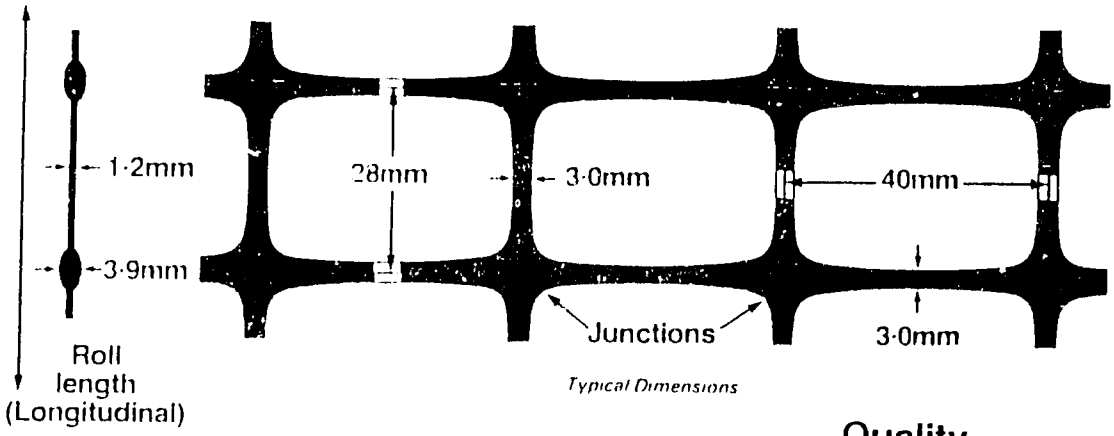
Tensar SS2 Geogrids

Tensar®

Specification

'Tensar' SS2 Geogrids were developed for stabilising weak, low load bearing soils, e.g. for the construction of temporary and permanent roads.





Typical Dimensions

Roll dimensions

Length:	50m
Width:	4m
Approx diameter:	0·5m
Approx weight:	68kg

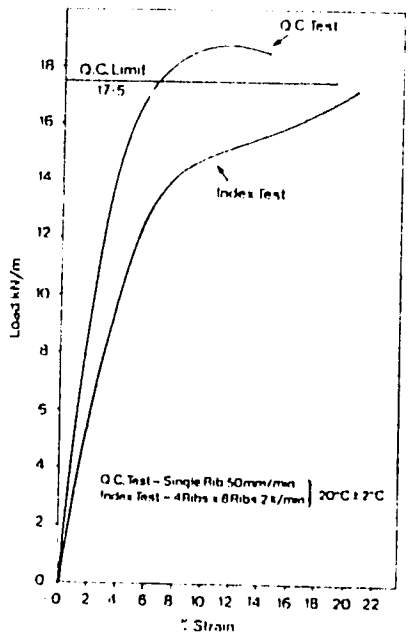
Physical properties of the grids

Weight:	0·3 kg/m²
Colour:	Black

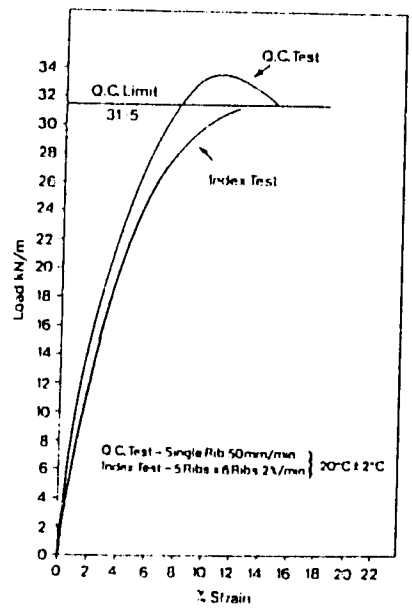
Quality Control Properties

	*Quality Control Strength (kN/m)	Approx Peak Strain (%)
Transverse	31·5	11·0
Longitudinal	17·5	12·0

* Determined as a 95% lower confidence limit



Typical tensile test results for 'Tensar' SS2 Geogrids (Longitudinal direction)



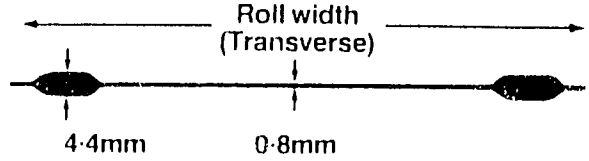
Typical tensile test results for 'Tensar' SS2 Geogrids (Transverse direction)

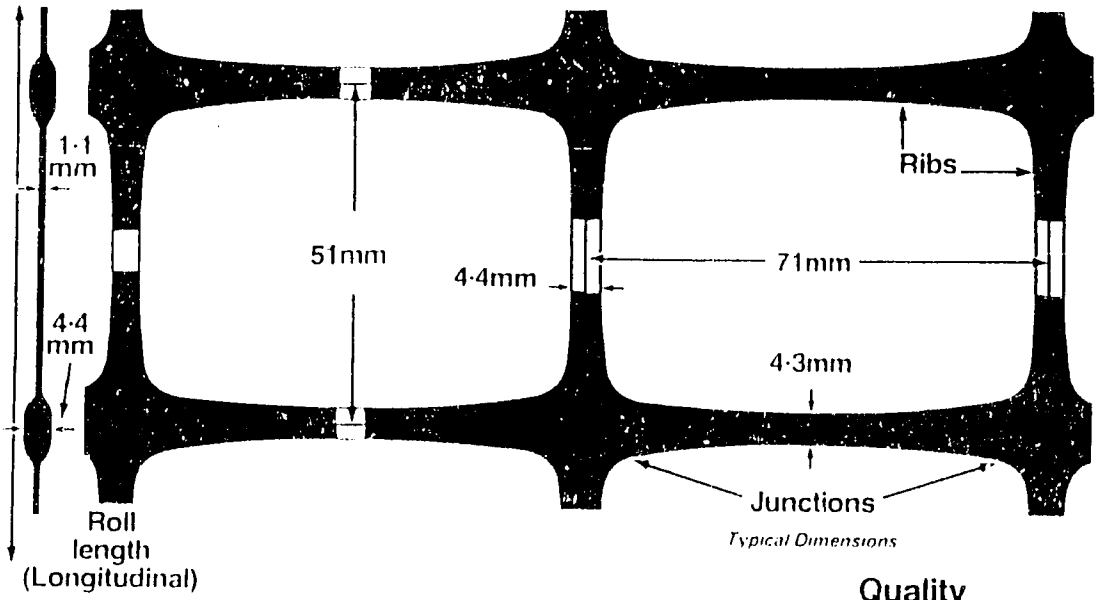
Tensar SS3 Geogrids

Tensar®

Specification

'Tensar' SS3 Geogrids were developed for stabilising weak, low load bearing soils. e.g. for the construction of temporary and permanent roads.





Typical Dimensions

Roll dimensions

Length:	50m
Width:	4m
Approx diameter:	0·5m
Approx weight:	56kg

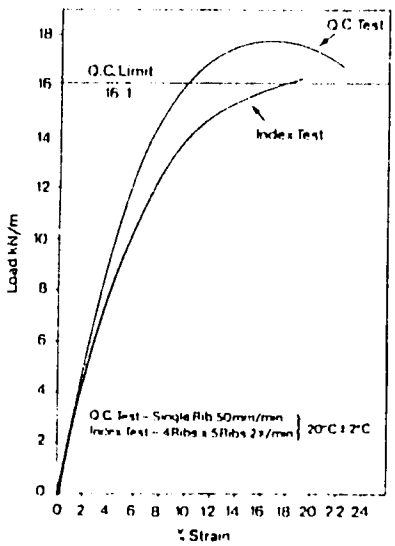
Physical properties of the grids

Weight: 0·24kg/m²
Colour: Black

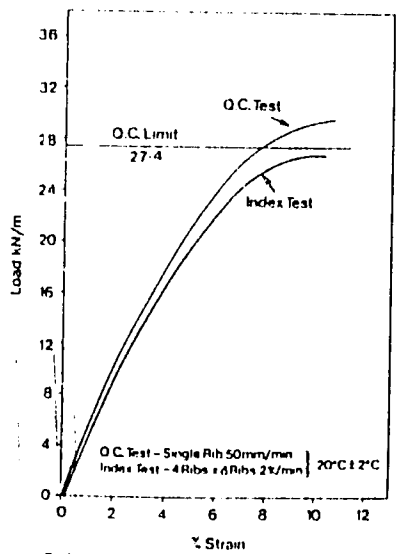
Quality Control Properties

	*Quality Control Strength (kN/m)	Approx Peak Strain (%)
Transverse	27·4	10·6
Longitudinal	16·1	16·5

* Determined as a 95% lower confidence limit



Typical tensile test results for 'Tensar' SS3 Geogrids (Longitudinal direction)



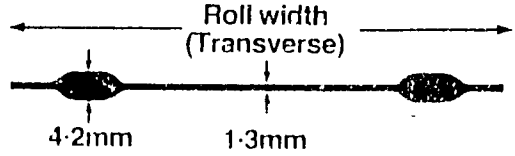
Typical tensile test results for 'Tensar' SS3 Geogrids (Transverse direction)

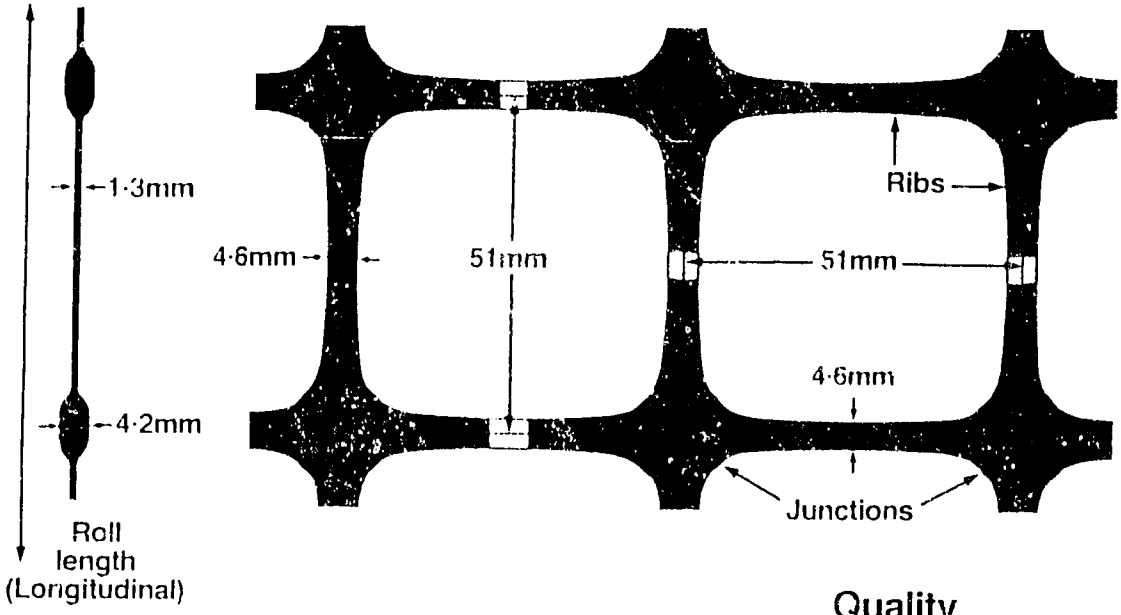
Tensar GM1 Geogrids

Tensar®

Specification (Provisional)

'Tensar' GM1 Geogrids were developed for rock face protection and for the fabrication of gabions and mattresses.





Roll dimensions

Length:	50m
Width:	3m
Approx diameter:	0·5m
Approx weight:	56 kg

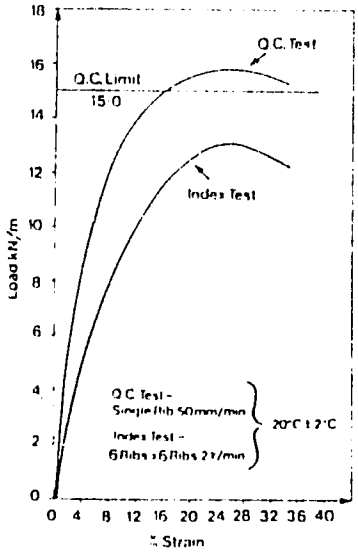
Physical properties of the grids

Weight: 0·32 kg/m²
Colour: Black

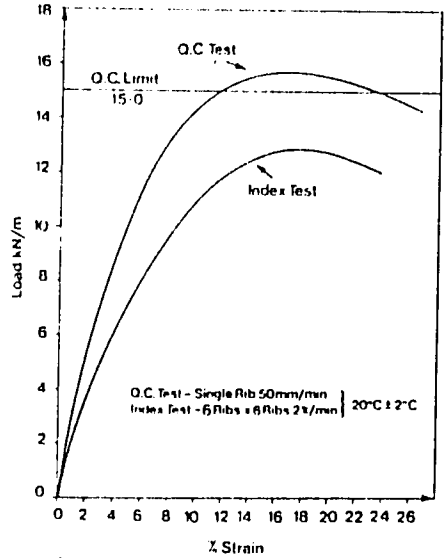
Quality Control Properties

	*Quality Control Strength (kN/m)	Approx Peak Strain (%)
Transverse	15·0	16·0
Longitudinal	15·0	26·0

* Determined as a 95% lower confidence limit



Typical tensile test results for Tensar GM1 Geogrids



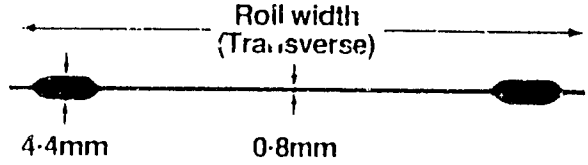
Typical tensile test results for Tensar GM1 Geogrids

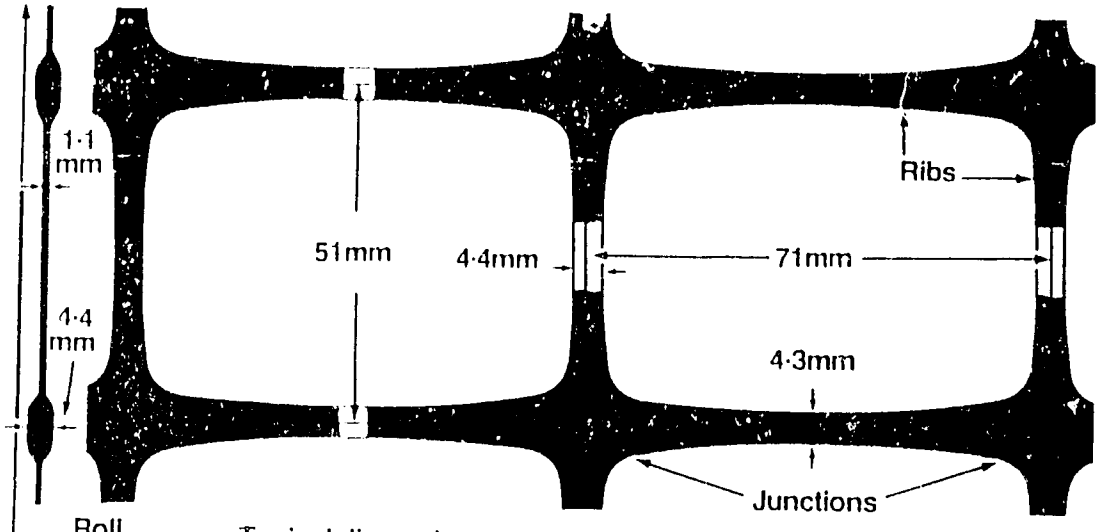
Tensar AR1 Polymer Grids

Tensar®

Specification

'Tensar' AR1 polymer grids were developed for reinforcing asphalt roads and pavements to minimise rutting and reflective cracking.





Roll length (Longitudinal)

Typical dimensions

Roll dimensions

Length:	50m
Width :	upto 4 0m
Approx diameter	0·5m
Approx weight	54kg

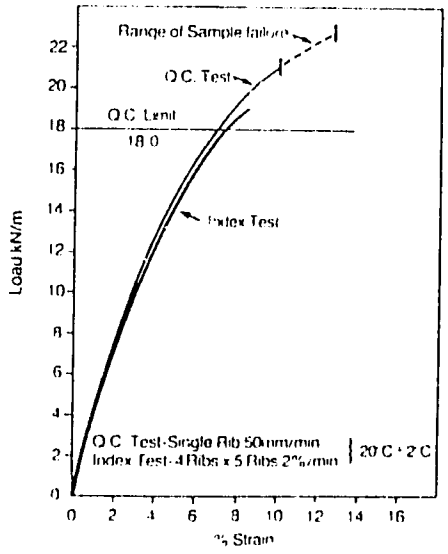
Physical properties of the grids

Weight: 0·24 kg/m²
Colour: Black

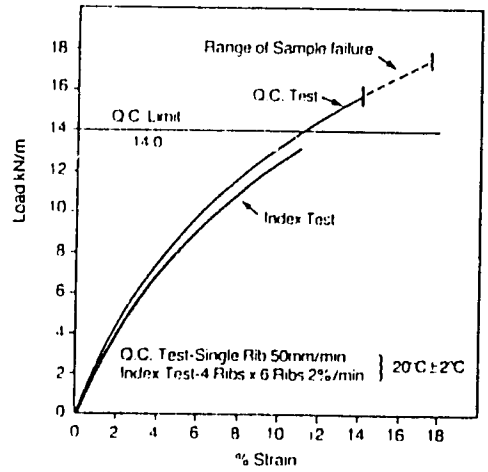
Quality Control Properties

	*Quality Control Strength (kN/m)	Approx Peak Strain (%)	**Maximum Shrinkage (%)
Transverse	18·0	10·0	4·0
Longitudinal	14·0	14·0	4·0

*Determined as a 95% lower confidence limit
**Determined as the free relaxation in a forced circulation hot air oven at 140°C for 30 mins.



Typical tensile results for 'Tensar' AR1 polymer grids (Transverse direction)



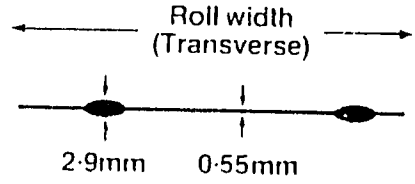
Typical tensile results for 'Tensar' AR1 polymer grids (Longitudinal direction)

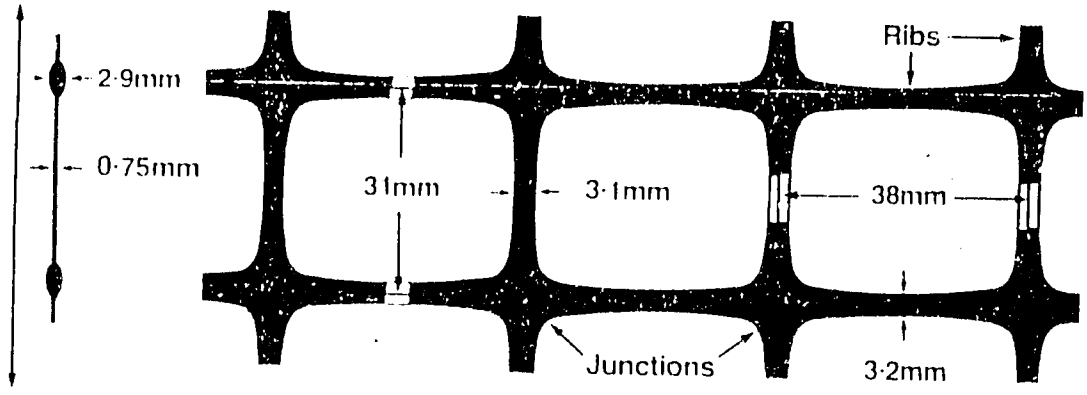
Tensar GS1 Polymer Grids

Tensar®

Specification

'Tensar' GS1 polymer grids were developed for continuous exposure applications such as fencing and slope protection





Roll length (Longitudinal)

Typical dimensions

Roll dimensions

A range of roll sizes are available to suit the end use.

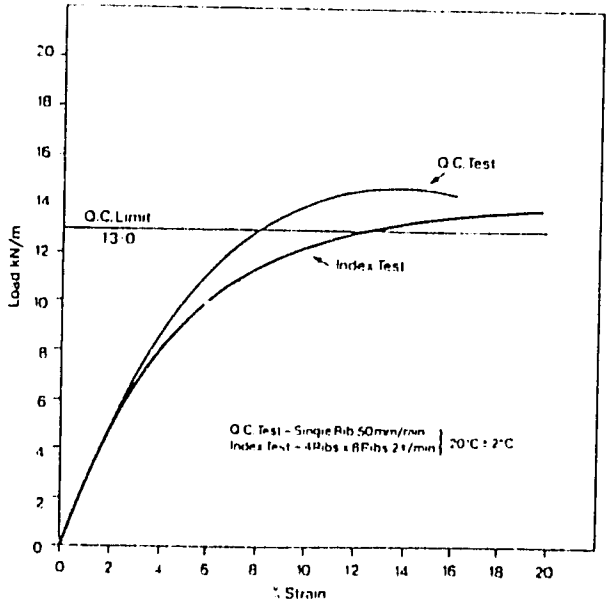
Physical properties of the grids

Weight: 0·18 kg/m²
Colour: Black

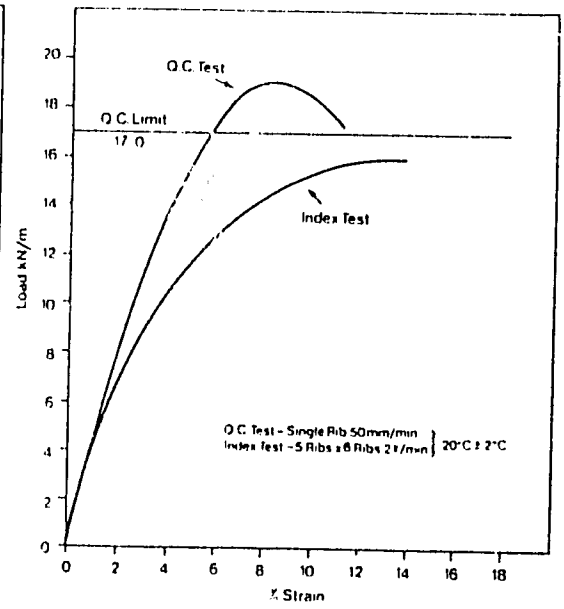
Quality Control Properties

	*Quality Control Strength (kN/m)	Approx Peak Strain (%)
Transverse	17·0	8·3
Longitudinal	13·0	14·0

* Determined as a 95% lower confidence limit



Typical tensile test results for Tensar GS1 polymer grids (Longitudinal direction)



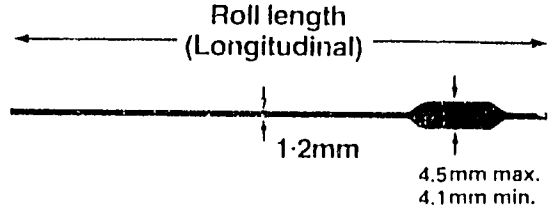
Typical tensile test results for Tensar GS1 polymer grids (Transverse direction)

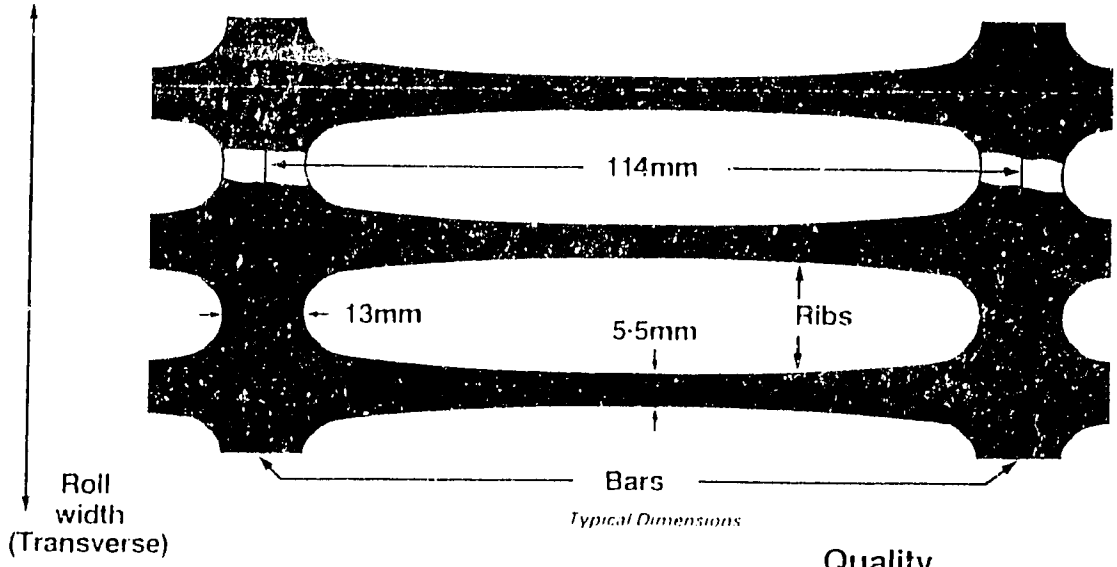
Tensar SR2 Geogrids

Tensar®

Specification

'Tensar' SR2 Geogrids were developed for reinforcing soils.
e.g. in the construction of embankments and vertical walls.





Typical Dimensions

Roll dimensions

Length:	30m
Width:	1m
Approx diameter:	0·4m
Approx weight:	28 kg

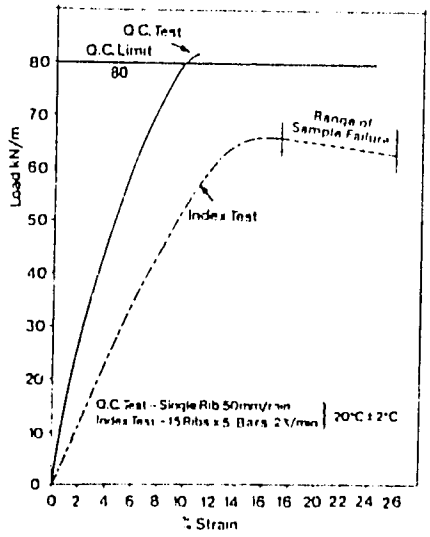
Physical properties of the grids

Weight: 0·85 kg/m²
Colour: Black

Quality Control Properties

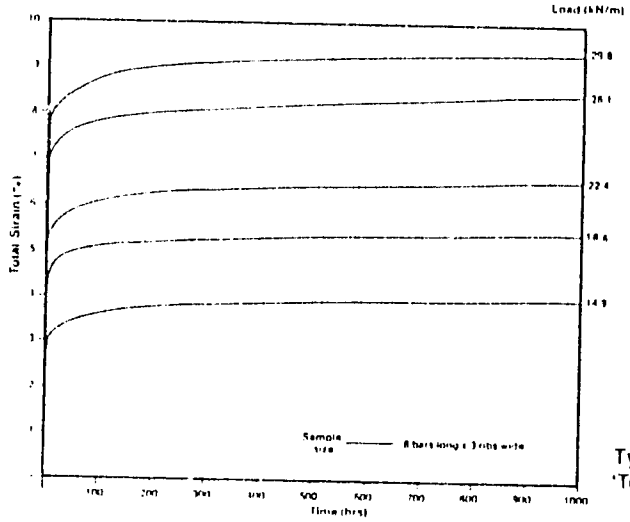
	*Quality Control Strength (kN/m)	Approx Peak Strain (%)
Longitudinal	80·0	10·5

* Determined as a 95% lower confidence limit

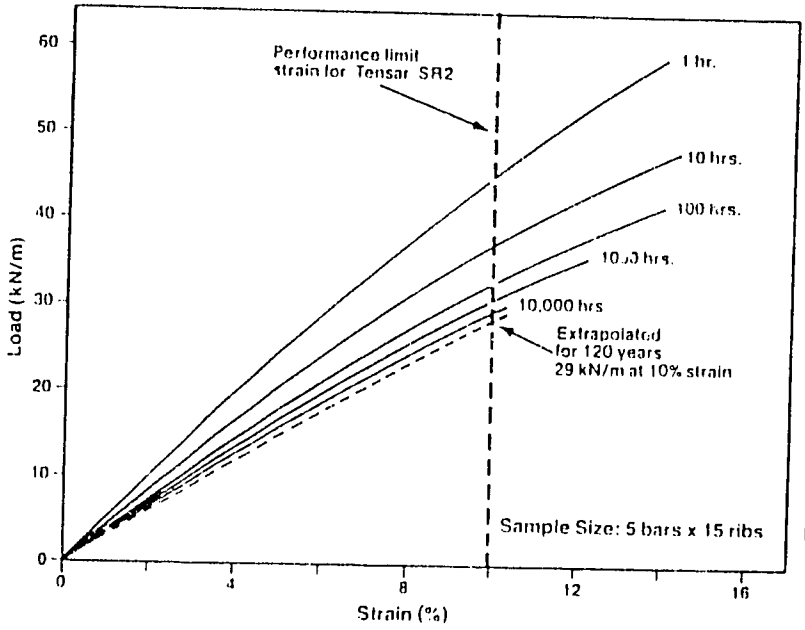


Typical tensile test results for 'Tensar' SR2 Geogrids (Longitudinal direction)

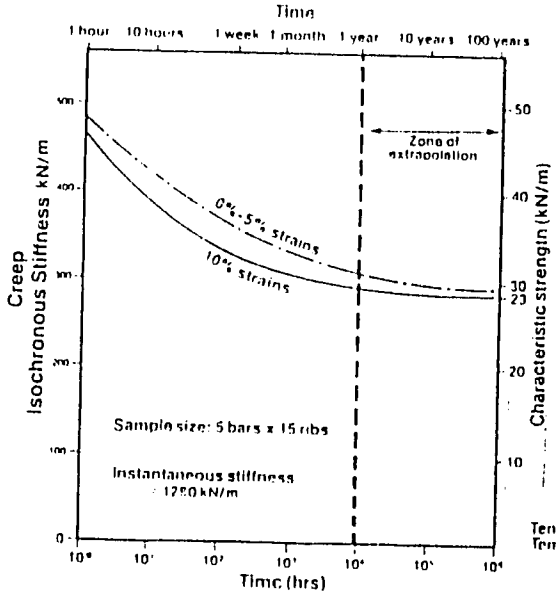
Characteristic strength of 'Tensar' SR2 Geogrids.



Typical creep curves for 'Tensar' SR2 at 20°C



Isochronous Curve (SR2)
Temp. 20°C



Tensar SR2
Temp: 20 C