# ENGINEERING CHARACTERISTICS OF WORKED PANAMANIAN LATERITIC SOIL

By

Frank Charles Townsend

Bachelor of Science

Michigan College of Mining and Technology

Houghton, Michigan

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Thesis Approved:

Phillip G. Manke
Thesis Adviser

T. Allan Haliburton

N Durham
Dean of the Graduate College

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# CHAPTER I

# INTRODUCTION

## General

Laterite is a residual soil found primarily in tropical areas such as Africa, Central and South America, and Southeast Asia. The soil is formed by the complete weathering of almost any rock: basalt, granite, gneiss, breccia and conglomerates (1). In fact, many soils throughout the world currently undergoing various weathering processes may be thought of as laterite in one stage or another.

Laterization involves the complete leaching out or removal of the silica, alkali, and alkaline earths, and concentration in the hydrated form of iron and aluminum oxides. Laterization is one of the tropical soil weathering processes, whereas podsolization is another. Laterites occur in an alkaline condition of weathering, and podsols occur under an acid condition (2).

The formation of laterite is quite complex and involves the factors of climate, elevation, rainfall, water table fluctuation, parent rock and age. (3) Because of these various factors, laterite occurs in a variety of forms from a friable soil to almost hard rock. Naturally, much controversy has arisen concerning the nomenclature of laterite lateritic soils, and latasols due to the variety of forms.

Likewise, the engineering characteristics of laterite are variable with geographic location. Generally, in situ laterite and lateritic soils possess a granular structure due to the iron and aluminum oxides coating the pore walls and filling the voids until the soil particles are knitted together in a lattice formwork. Winterkorn (4) states that "the presence of iron in laterite soils is one of the most important factors that influence their engineering properties." This granular structure is responsible for the desirable engineering properties displayed by unworked laterite and lateritic soils. In India, air drying of moist lateritic soil blocks resulted in a hardening process, and the material was used as building stone in ancient temples. In this case, the iron network was fixed thus stabilizing the granular structure. However, the high bearing strength, low plasticity, and high permeability associated with unworked laterites are lost upon "working" the soil; i.e., mixing, compaction, or any type of extensive manipulation by mechanical agents in the presence of water. When worked, the granular structure is apparently broken down, and the material becomes highly plastic with low bearing capacity and poor drainage.

## Problem

The problem is aptly stated by Winterkorn (4), "the task is to search for and adopt suitable mixing and compaction methods that would not destroy the granular structure and yet yield sufficient density." To accomplish this task it is necessary to understand better the remolding process and search for methods to prevent destruction of the granular structure or to stabilize the remolded or "worked" material. Therefore, the use of admixtures; e.g., lime, portland cement, asphalt, or chemicals, would appear to be a possible solution. However, since working by mechanical agents apparently breaks down the iron network or granular structure, it is possible that admixtures would have different stabilization characteristics depending upon the amount of working undergone by the soil during stabilization. Knowledge in this area should assist in explaining the peculiar behavior characteristics of lateritic soils and quite possibly will be of assistance to construction and soils engineers working in tropical areas where this soil type is prevalent.

## Scope of Investigation

Since working appears to change a desirable construction material into a possible engineering problem, this study was an investigation of the strength parameters and engineering properties of worked and unworked lateritic soil. The investigative tests included: Atterberg limits, grain size analysis, Proctor compaction, California Bearing Ratio, and unconfined compressive strengths of soil stabilizer mixtures.

The utilization of stabilizing admixtures for low cost road construction in lateritic soils should greatly assist the development of transportation networks in tropical countries. The common additives, portland cement, lime, and asphaltic cutback (MC-3) at economical percentages of 5 percent by weight of lime or portland cement, and 6 percent by weight of cutback were used. No attempt was made to determine the optimum percentage of stabilizer required as only indications as to the susceptibility to stabilization of worked and unworked soil was desired.

# CHAPTER II

## DEVELOPMENT OF LATERITES

The term laterite was first used by Buchanan (5) to describe the iron clay deposits found at Angadipuram, South India. The natives of this area used the red clayey material as building brick, hence the name laterite (Latin-later=brick). The soil was cut and shaped when moist and soft and then exposed to the air and sun for drying. The drying action hardened the material into bricks which were unaffected by subsequent wetting. This hardening process has been described as siderization (6) and has plagued agriculturists, for as soon as a forest or jungle is cleared a hard indurated crust forms. (3)

Ever since the original naming by Buchanan and the misleading practice of refering to any brick-red tropical clayey material as laterite, controversy as to the formation and name has existed. These differences of opinion are thoroughly discussed by Mohr (3) and echoed by Bawa (1) who states, "To date there has been no general agreement in literature regarding nomenclature and definitions for the terms relating to laterite and lateritic soils." Martin and Doyne (7) used the silica-alumina ratio as a suggested classification criterion. More recently the silica-sesquioxides ($Fe_2O_3 + Al_2O_s$) ratio has been used.

TABLE I

SOIL CLASSIFICATION BASED ON THE SILICA-SESQUIOXIDE RATIO

| Soil Type | $SiO_2/R_2O_3$ |
|---|---|
| Laterite Soil | 1.33 or less |
| Lateritic Soil | 1.33 - 2.00 |
| Non-lateritic Soil | 2.00 and over |

Generally, laterite may be defined as a tropical soil in which most of the silica, alkali, and alkaline earths have been leached out and eliminated, and in which iron and aluminum oxides in the hydrated form have been concentrated.

The primary reason for the controversy concerning laterite soils is due to the numerous factors and conditions which are necessary for laterite formations to occur. The factors of climate, elevation, rainfall, ground water fluctuation, parent rock, and age, all play an influential role in the final resulting residual soil.

## Laterite Profile

An examination of a "typical" laterite profile appears in upward sequence to be: (8)

1) parent rock
2) a decomposed zone of parent rock boulders and clay
3) a layer of reddish or yellowish clayey material (kaolinite, montmorillonites and micaceous clays (lithomarge))

4) a mottled gray zone rich in sesquioxides with small iron nodules or concretions in the upper layers

5) a very hard indurated crust or cuirasse.

The development of this profile is dependent upon the following factors: (3)

Climate - It is essential that the climate be tropical or sub-tropical and quite humid. For the required leaching of the silica and alkaline earths, the soil must be moist, and for the hardening of the sesquioxides, there must be alternating wet and dry periods. These conditions are generally found in tropical areas.

Rainfall - In climates where the organic material is rapidly oxidized the meteoric water will be slightly alkaline or neutral. Silica is soluble in such a water and goes into solution. Frequent rainfall and good drainage are necessary conditions for the required leaching and silica removal to take place. The rainfall and drainage naturally influence the ground water characteristics, which in turn influence the profile.

Ground Water Fluctuations - During the wet season the ground water level rises covering the soil layers and saturating them. At this time intense chemical weathering takes place. The parent rock is attacked, destroyed, and new minerals formed. When the saturation point is reached, seepage occurs, removing or leaching the minerals that have dissolved.

In the dry season the weathering cycle is reversed. Due to capillary action, the soil solutions advance into the upper layers where they are freely oxidized. During this period the deep-lying water, rich in alumina and iron, comes in contact with the oxidizing air. This oxidation is responsible for the hardening process which forms the concretions and eventual crust.

The mottled gray layer generally found in laterite profiles is a product of ground water fluctuations. The thickness of the mottled layer represents the extent of ground water fluctuations during wet and dry seasons. The top of this layer indicates the height to which the water table reaches. Thus, it can be seen that the upper horizons possess a deep red color due to the upward movement of the iron, while a lower gray zone develops due to the loss of this iron. (3)

Parent Rock - The thickness of a laterite profile is related to the iron richness of the parent rock. Basic rocks, such as basalt, usually produce thick laterites, whereas iron poor acid rocks are associated with thin layers. If the rock is highly ferrous, a larger iron than aluminum content would naturally be observed, while a rock rich in aluminum would produce a profile of greater aluminum content. Basic rocks usually produce the brick red color associated with laterites. Laterites have been known to occur over basalt, granite, gneiss, volcanic breccia and tuff, and conglomerates. (1)

Age - The indurated iron crust only occurs in the final stages of laterization. (9) Age is the primary difference between laterite formations. The process of laterization is an end product, an extreme example of soil forming processes taking place over a large portion of the earth's surface. Indirectly most clays may be in one stage of laterization or another, the final product being the crust formation. (10)

Because these forementioned factors vary so much throughout the world, there naturally exists much variation among lateritic materials. Just as limestone has extremes from marl to marble, laterite varies from a very friable soil to hard rock as in the crust. (11)

## Genesis of Laterite

During the chemical weathering of rocks there are two processes taking place simultaneously; the feldspars are decomposing to clays and there is a lateritic alteration which forms iron and aluminum hydrates which then pass to sesquioxides.

As the parent rock decomposes the various minerals undergo alteration; for example, in acid rocks the plagioclase feldspars form gibbsite (hydrated form of aluminate), the olivine forms goethite (an iron oxide FeO.OH) and the micas, kaolinite. The gibbsite, upon contacting silica, which is in solution, changes into kaolinite. In more basic rocks sufficient silica is present to effect the direct formation of kaolinite without passing through the gibbsite stage. In both cases, kaolinite is an intermediate product and is usually present in lateritic soils. However, the harder the laterite, i.e., the more fully developed, the lesser the percentage of kaolinite. (9)

The subsequent stage of laterization involves the concentration of iron and aluminum hydrates in the upper horizons of the soil. The further decomposition of the secondary products and ferromagnesian mineral constituents of the parent rock provides the source of these hydrates. Their concentration in the upper layers is caused by upward leaching as provided by the ground water table fluctuations. As a result the zone underlying the iron rich upper layers is gray, mottled and poor in iron. The iron and aluminum-oxides are in microcrystalline form, possessing enormous specific surfaces and can pass into solution, much more readily than more crystalline minerals. They can move as sols or gels into places where they can develop the crystalline framework (10). The iron framework construction is now forming.

Several authors (10, 12, 13) using petrographic techniques report that laterite in this stage appears as tiny spherical aggregates or clusters of clayey materials (kaolin). These clusters, a few microns in diameter, are highly impregnated by iron microcrystals, goethite. In fact, the iron is adsorbed or impregnates and may even encapsulate the kaolin. The density of packing of these tiny aggregates is dependent upon the variations in the apparent degree of impregnation of the kaolin by the iron compounds.

This author believes the adsorption of the iron by the kaolin is enhanced by cationic attraction on the part of the clay particles. In tropical areas where pH values of solutions are neutral to slightly alkaline, pH 7 or 8, clays appear to develop an extra amount of negative charge. This is caused by the dissociation of the SiOH groups to SiO-H+, which increases the exchange capacity of the clay. This effect is more pronounced in kaolinite than illite or montmorillonite clays. (15)

Further laterization involves an increase in content of the iron oxides, which coat the walls of the soil pores and/or voids until a lattice-like network of iron stained kaolin aggregates is formed. The laterite observed by Buchanan was in this stage of development. However, the mere presence of concentrated sesquioxides does not insure the hardening of the soil. There must also exist favorable conditions for movement, dehydration, and crystallization of the iron. This process which changes dilute amorphous iron colloids into dense iron crystalline minerals is called "siderization" (sideros=iron) by

Mackenzie. (6) The actual process is believed to be caused by an increase in crystallinity and continuity of the goethite due to desiccation.

The final stage of the laterite development is the formation of the hard indurated crust. Continued wetting and drying of the alternating seasons provide the iron with a period of mobility and orientation followed by drying and crystalization. This build-up of iron continues until a compact concretion is formed. It is only one step further until the concretions are cemented by the iron which is filling the voids between them and forms a crust. Petrographic studies show the crust may contain free quartz and pockets of kaolinite encased by a shell of crystalline goethite or hematite.

## CHAPTER III

## ENGINEERING CHARACTERISTICS

### Physical Properties

Due to their widely varying nature and existence in numerous forms, it is quite understandable that the engineering properties of laterite will vary with locality and factors of formation. The deciding factor generally is that of age. Naturally the engineering characteristics of old laterites, i.e., the hard indurated crust, will differ appreciably from the characteristics of young laterite, i.e., a friable lateritic soil. Also, great difficulty exists in inferring engineering properties from existing literature because of the undefined nomenclature applied to laterite, lateritic soils, latasols, and laterite soils. Due to the forementioned subjects of controversy, it is impossible to predict the exact behavior of any laterite from the literature. It appears that laboratory testing to determine basic soil properties is essential for construction projects in lateritic areas. The following discussion is a general presentation of the usual properties associated with laterite and lateritic soils.

#### Specific Gravity

Results generally show the specific gravity of laterite and lateritic soils to range from 2.70 to 3.50 (1). The iron crusts of India generally exhibit specific gravities slightly higher than 3.0, whereas the lateritic soils are below 3.0 (16). The high iron content is responsible for the specific gravity being generally higher than that of non-lateritic soils.

Atterberg Limits

These limits tests have been used extensively and quite successfully to classify cohesive soils in temperate areas. However, attempts to correlate the limit results of lateritic soils to clays of temperate zones have often resulted in erroneous conclusions. It has been shown that the Atterberg limits of lateritic soils depend to a large extent on the amount of remolding or working of the sample during the test and treatment of the sample prior to testing. (17) A liquid limit deviation of ± 15% depending upon treatment prior to testing was reported for a lateritic clay near Nairobi, Kenya (13).

Winterkorn and Chandorshekhoran (4) observed a change in liquid limit from 46% to 53% depending upon the amount of remolding of the soil; there was no observed change in plastic limit. Terzaghi (13) explaining the Atterberg limit variations associated with a young lateritic soil attributed the variations to the microaggregate clusters of the clay. He stated the soil had the plasticity index and engineering properties of a relatively coarse grain soil with a low plasticity index. Yet the liquid limit was equal to that of a soil with a high liquid limit because the quantity of water which evaporated during the process of drying a sample with a water content at the liquid limit was equal to the sum of the quantity of water located between the clusters and that contained in the voids of the porous microaggregates.

Likewise, laboratory results are influenced by the amount of mixing, which causes a breakdown of the granular structure and thus increases the number of fines and the amount of plasticity.

Generally, lateritic soils exhibit the following range of values.

TABLE II

ATTERBERG LIMITS OF LATERITIC SOILS

| Type & Location | LL | PL | PI | Source |
|---|---|---|---|---|
| Porous Red Clay - Brazil | 53 | 10 | 43 | Vargas (18) |
| Dark-Red Laterite Soil - Cuba | 53 | 31 | 22 | Winterkorn & Chandorskehorn (4) |
| Hydrated Lithomarge - Kenya | 87 | 54 | 33 | Terzaghi (13) |
| Lateritic Soil - Mocambique | 69 | 31 | 38 | Nascimento (19) |
| Lateritic Soil - India | 51.5 | 16.5 | 35 | Ramachandran (20) |

## Grain Size

Laterite and lateritic soils contain all sizes from gravel to clay depending upon the degree of laterization. The clay pockets found in the crust demonstrate this wide variance in particle size. The size deposits are usually composed of larger aggregates, while the younger lateritic soils possess larger percentages of clay.

The friable nature of laterites allows a ready breakdown of the granular components (concretions) into finer particles. Therefore, the treatment prior to or during testing can greatly alter the results.

Because the particle size depends upon the amount of disaggregation prior to testing, grain size data of residual soils is extremely doubtful. (21)

## Permeability

In general, laterites in situ exhibit a granular structure which accounts for excellent drainage and porosity. The lateritic soils possess the highest degree of permeability, while the iron crust has the lowest of the lateritic materials. The permeability is generally on the order of $10^{-2}$ to $10^{-1}$ cm/second depending upon the amount of physical aggregation. (1)

The spongy skeleton consisting of microaggregates is responsible for this high permeability, which is not usually associated with soils possessing Atterberg limits of the same magnitude. Therefore, rejection of laterites as a construction material on the basis of Atterberg limits is unwarranted, as laterites usually have drainage properties suitable for highway subgrades. Yet, working by mechanical agents will alter the granular structure and cause the soil to become clayey and plastic and thus lower the permeability.

## Swelling

Lateritic soils exhibit only minor swelling tendencies when compared with clay soils of comparable Atterberg limits. In fact, they may be considered as water inert. Fruhauf (22) immersed laterite in water for 7 days and observed no change in granular structure. Swelling tests reported by the Indian Road Congress (16) on lateritic cubes revealed swelling was limited to only a few hundredths of one percent. Nascimento (19) commented on the unfortunate fact that lateritic soils usually possess Atterberg limits which would indicate prominent swelling characteristics. Yet he observed that some laterites when immersed in water did not slake.

This author credits the iron (goethite) films which coat and impregnate the clay fractions of the soil as being responsible for the water-inertness of unworked laterites. In this case, the iron serves as a barrier and prevents the water from contacting the kaolin. Also mineralogical studies of laterite show kaolinite and halloysite are generally the predominent clay minerals present. Neither of these two clay minerals possesses a highly expansive lattice.

## Erosion

The resistance of laterite to erosion ties in with its porosity. In its natural state, the material is highly porous and resistant to erosion; however, any working of the granular structure will cause a reduction in permeability and increase the tendency to erode.

In the case of the laterite crust, which is on the surface and acts as a caprock for the underlying soils, erosion may be caused by the iron passing into solution and flowing away. The result of this erosion as observed in Nigeria and Australia will be the formation of a debris slope composed of the cap fragments as talus. Continued dissolving of the upland iron caprock will often result in foot-slope and low-level laterites. (10, 23)

The original laterite described by Buchanan (5) has stood in temple walls in India for centuries, thus indicating the only real problem with erosion will result after working or remolding.

## Engineering Behavior

As construction increases in tropical areas where laterite and lateritic soils abound, construction engineers should not only be familiar with the physical properties of the soil but with its engineering behavior as well. In general, laterites possess certain physical properties, i.e., limits indicative of a clayey material, yet because of their granular structure this often is not the case. Quite frequently judgment based upon the limits only, without further testing, results in the mistaken conclusion that all laterites are troublesome and should be avoided as construction materials.

One example is the problems associated with the "abnormal" clay used in construction of the Sasumua Dam in Kenya. (13) In this case the contractor mistakenly judged the lateritic soil as unsuitable due to its physical properties. Further testing showed the soil to be quite satisfactory; the dam was constructed, and the contractor lost a costly lawsuit.

In general, the suitability of lateritic material can only be judged after an extensive study of the engineering characteristics relative to the given project has been made. Likewise, since working or remolding greatly alters the granular structure and engineering properties, normal laboratory procedures may have to be modified. (22)

Due to their widely varying nature, laterites may be acceptable or objectionable for use as a construction material. Also the degree of working undergone by the granular structure apparently determines to a large extent the acceptability of the material.

The original use of laterites for bricks is centuries old, with ancient temples and roads paved with laterite bricks still being used. (8) Crushing strength tests of laterite bricks generally give values slightly lower than results obtained with first class kiln-fired bricks. (16) Also, after proper testing and utilization of the results, lateritic materials have performed successfully in earth dams and as base materials for highways. In some cases the lateritic gravels and concretions have been used as aggregates for concrete or asphaltic cement. These forementioned examples show to some degree the adaptability of laterite and lateritic soils as materials for construction.

The use of the harder laterites as road building gravels depends largely upon their abrasion resistance, which is influenced by the iron content. Portuguese Angola engineering laboratories have adopted several abrasion limits of lateritic gravels for road construction. One limit was 65% wear in the Los Angeles Abrasion test modified not to include the ball load. This limit contrasts to the AASHO-M-147-57, Los Angeles Abrasion test, which places a maximum of 50% wear as acceptable. However, in general lateritic gravels possess wear values higher than 50 per cent and in some cases higher than 70% and still serve adequately as roadmaking gravels. Another abrasion limit used, specifies a hardness index greater than or equal to 0.80 as allowable. The hardness index is defined as a ratio of the granulometric modulus in the natural state to the granulometric modulus after a standard treatment. The granulometric modulus is the sum of the percentages of material passing the 1", 3/4", 1/2", 3/8", No. 4, No. 10, No. 40, and No. 200 U. S. Standard Sieve. (19).

## Foundations

For lateritic soils in their natural state, the bearing capacity and resistance to shear failure under foundation loads are not critical.

However, the harder crust can function either as an asset or a source of trouble. Normally it is possible to construct light structures on the crust's surface utilizing its low compressibility and high bearing value. In this case, the crust acts as a raft over the softer intermediate layers. However, in the case of deep foundations, the hard crust will seriously hinder pile driving, and quite often it is necessary to use pre-excavated piles or cassions. (1)

Settlement in lateritic soils is usually irregular due to the varying horizons; i.e., crust, red clay with concretions, gray mottled zone, and parent rock, and their individual void ratios. Likewise, it has been noticed that the strength of lateritic formations often decreases with increasing depth. (8) Because of the varying strengths and void ratios of the intermediate layers, it is often necessary to place deep foundations on the partially weathered parent rock. This practice may often present problems as the rock weathers in an irregular boundary, and prediction of the exact depth of sound rock is difficult. Many contractors take too few borings and attempt to estimate the elevations of the rock. Unfortunately, decomposing boulders are often mistaken as parent rock, and the foundation must be revised due to encountering or not encountering sound rock at the predicted depth. (24)

Due to heavy tropical rains in laterite areas, considerable ground water may be encountered. This fact coupled with the characteristics of laterite to become plastic when remolded in the presence of water necessitates adequate drainage arrangements and cautious construction techniques when laterite is involved in foundation excavations. (8)

Earth Dams

Earth dams using lateritic materials have been constructed successfully in Brazil, Kenya, Java, Australia, and the Phillippine Islands. (13, 18) When lateritic soils are used as a construction material for earth dams or when dams are constructed over these soils, consideration should be given to the following factors: compactability, strength, permeability, shrinkage, settlement of the soil, the general design of the dam structure, and depth to decayed or weathered parent rock.

It has been found that if moisture content is properly controlled during compaction, lateritic soils will exhibit low permeabilities and high strengths adequate for earth dam construction. Generally 95% compaction and moisture contents ranging from optimum to 2% below optimum are specified. (18) The proximity of the optimum moisture content to the plastic limit should be remembered at all times, and efforts made to avoid compaction at moisture contents higher than the plastic limit. If this is not done, remolding of the soil with a subsequent increase in plasticity will result in lower densities and possibly create problems in the efficient and economical utilization of equipment.

Two principal problems in earth dam construction in lateritic areas are seepage and dam foundations over weathered rock.

The problem of seepage lies in the fact that earth dams constructed of lateritic soils are generally homogeneous sections; however, the soil usually is non-isotropic, and as a result seepage in the horizontal direction may be greater than in the vertical direction causing piping and erosion on the downstream slope. Proper utilization of vertical and horizontal sand drains will help to alleviate this problem. Care should be taken to control the void ratio of the sand used in order to prevent the possibility of the laterite washing through the drains. (18)

Several schemes have been used successfully to overcome the problem caused by seams of high permeability through the weathered rock. One method is the construction of a narrow cut-off trench through the weathered boulders to sound rock. This trench is then back-filled with concrete, forming an impermeable curtain and cap over the rock.

Another method utilizes vertical sand drains drilled into the weathered layer and connected to a filter or horizontal sand drain downstream of the longitudinal axis of the dam. This system of drains collects seepage through the permeable strata and prevents uplift on the dam. (18)

## Road Construction

Laterite soils are quite suitable for road construction material as long as their granular structure remains unchanged. However, working by heavy equipment will transform laterite into an undesirable plastic material. The degree of suitability naturally varies from locale to locale. In India the hard crust is crushed, sieved, and used as "chippings" for the top layer in water-bound macadam pavement. (16) Under heavy traffic conditions, however, this lateritic gravel must be protected by a wearing coat due to the laterite's low resistance to abrasion. On the other hand, some of the more friable and less well developed lateritic soils can only be used as a base material and must be protected by some type of waterproof surfacing to prevent infiltration of water. (8) Therefore, it is evident that laboratory testing is indispensible in determining the degree of success with which the lateritic material can be used.

In cut sections of roads, slope stability must be considered, and generally good drainage must be provided. In shallow cuts, up to 20 feet, laterite often stands permanently on a vertical slope. (8) As observed by this author, benches of 10-15 feet are used in deep cuts on the transisthmian highway in Panama. Landslides may occur due to the constant moistening from heavy rains passing through the permeable laterite and collecting on the parent rock; seepage along this zone may cause undermining of the slope, creating a landslide susceptible condition. Predrainage by open ditches or gravel filled trenches should be considered. (27) It may be observed that cut slopes will harden and stabilize due to desiccation and siderization. (10)

If lateritic soils are used as a surfacing material on unpaved low traffic roads, they will probably become very soft and slippery during the wet season and quite hard and stable during the dry season. (7) The development of corrugations on these unpaved roads is also a common problem. The corrugations initially develop during the dry season when the plasticity is lower, permitting the loss of "fines." The fines are needed to maintain cohesion and prevent displacement of laterite fragments which develop corrugations in the same way as loose gravel forms corrugations on unsurfaced gravel roads. (28)

## Compaction Characteristics

The principal problem associated with compaction of laterites is the avoidance of working by heavy construction equipment, which will transform the soil into a highly plastic clayey material. Experience has shown that primitive manual compaction, which minimizes working or remolding of the material, has yielded better airfields than compaction of the same soil by heavy equipment. (4)

Bawa (1) stated that in general relatively high compacted densities would be expected due to the high specific gravity of the solids. Quite possibly, these densities are true in the case of laterite or older lateritic soils. However, in several instances, densities of young lateritic soils are quite low in comparison with their high specific gravities. This author believes this phenomenon is caused by the popcornball-like clusters of microaggregates which provide a granular structure in the soil and thus a lower density when the soil is compacted.

Experience has shown that the optimum moisture content is commonly close to or slightly below the plastic limit; however, during the wet season, the natural moisture content of lateritic soils may be slightly above the plastic limit. For this reason, quite often it is necessary to dry the soil prior to placement for compaction. (13)

The following table presents density values (Standard Proctor) and corresponding optimum moisture contents for various lateritic soils:

TABLE III

DENSITY AND OPTIMUM MOISTURE CONTENTS OF LATERITIC SOILS

| Type & Location | Gs | Dry Density | Opt. M.C. | PL | Source |
|---|---|---|---|---|---|
| Lateritic Soil Guinea, Africa | -- | 113 | 10.3 | -- | Winterkorn (4) |
| Lateritic Soil Matanzas, Cuba | 2.90 | 88 | 30 | 31.2 | Winterkorn (4) |
| Lateritic Soil Morroco, Africa | 2.87 | 126 | 13.9 | 22.7 | Remillon (25) |
| Hydrated Lithomarge Kenya | 2.83 | 79 | 50 | 54 | Terzaghi (13) |
| Laterite India | 2.70 | 121 | 12 | 16 | Ramachandran (20) |
| Lateritic Soils Brazil | 2.68 | 81-90 | 30 | 31 | Grizienski (26) |

Due to the friability of the lateritic materials, it is necessary to use new material for each point of the Proctor curve. It has been shown that the variation of the dry density with compactive effort will plot as a straight line on semilog paper. (25)

## Stabilization

Laterites and lateritic soil have been stabilized by using various admixtures: lime, portland cement, chemicals, asphalt and sand. Unfortunately, due to the varying occurrences of laterite, none of these additives is universally successful. Therefore, it is essential that laboratory and field testing be performed prior to any extensive stabilization project. (8)

The hard laterite crust or lateritic gravels are generally best stabilized by asphalt cut-backs. These hard laterites have good affinity to asphalt much as do lavas or basalt. In fact, good bituminous concrete has been made using these materials. However, the low abrasive resistance produces "fines," and the presence of silt or clay materials is detrimental to good bond between the laterite and asphalt. (11) Another problem associated with the use of asphalt for stabilization involves the impregnation of the laterites. Impregnation is only possible after a thorough moistening of the surface. Even in the case of lateritic soils with high plastic indexes, penetration is usually only satisfactory if the surface has been first moistened. (25) It should be mentioned that complete failures in attempts to stabilize clayey lateritic soils using asphalt cut-backs have been reported in Cuba (4) and India (20).

Quite possibly "oiling" with asphalt cut-backs may be the best means of constructing low-cost, low-traffic roads in lateritic areas. This method utilizes the natural strength of the soil, while the asphalt surface provides a roof to prevent moisture from altering the granular structure. If this method is utilized, it will probably be necessary to fortify the soil with antioxidants and bactericidal additives to guard against tropical weathering and microbiological deterioration. (4)

Stabilization of lateritic soils by use of portland cement has been reported in Cuba (4) and Southeast Asia (20, 29). In these cases, 8-10 percent, by weight, of portland cement was required to obtain compressive strengths in excess of 250 psi. Successful stabilization in French West Africa using lime has been reported by Schofield. (30) Three percent lime in this case was found to reduce the plasticity index from 30 to 8 percent due to base exchange. Ten percent lime was found to be sufficient to stabilize red clays from the Sasamua Dam project in Kenya. (17) It should be remembered that lime stabilization requires a longer curing time than cement, and sufficient water must be added to provide a medium for ionic exchange. Failures using lime as a stabilizing agent have been reported in Cuba (4), and calcium chloride failed to work in India. (20)

Due to ease and economy, granular soil stabilization probably has been the most widely practiced method of stabilization for lateritic soils. However, these roads suffer greatly during dry seasons when dessication permits removal of the "fines" and allows corrugations to develop. In Brazil as much as 20% coarse sand was added to lateritic soils to provide a suitable base material. (31) In India, it was found that up to 15% sand increased the strength of a laterite soil; however, it was found that further increases in percentages of sand reduced the strength. (20)

A summary of various lateritic soils and stabilizing admixtures is shown in the following table.

TABLE IV

EFFECT OF STABILIZING ADDITIVES ON UNCONFINED COMPRESSIVE STRENGTHS OF VARIOUS LATERITIC SOILS

| Stabilizer & % | Location | Compressive Strength | | Source |
|---|---|---|---|---|
| | | No Additive | Additive | |
| RC-3 5% | India | 147 psi | 18.3 psi | Ramachandram (20) |
| RC-3 10% | India | 147 psi | 7.95 psi | Ramachandram (20) |
| MC-3 6% (Moist cured) | Cuba | | 10 psi | Winterkorn (4) |
| MC-3 10% (Moist cured) | Cuba | | 13 psi | Winterkorn (4) |
| MC-3 10% (Dry cured) | Cuba | | Swelled & failed | Winterkorn (4) |
| CK-3* 2.5% | Thailand | 106.2 | 703 psi | Michelin (32) |
| Lime 8-18% (Wet-dry) | Cuba | | Failed | Winterkorn (4) |
| Lime 14% (Immersed) | Cuba | | 52 psi | Winterkorn (4) |
| Lime 18% (Immersed) | Cuba | | 41.0 psi | Winterkorn (4) |
| Lime 5% | Kenya | | 130 psi | Newill (17) |
| Lime 10% | Kenya | | 340 psi | Newill (17) |

TABLE IV (Continued)

| Stabilizer & % | Location | Compressive Strength No Additive | Compressive Strength Additive | Source |
|---|---|---|---|---|
| Portland Cement 5% | Kenya | | 120 psi | Newill (17) |
| Portland Cement 10% | Kenya | | 300 psi | Newill (17) |
| Portland Cement 8% (Immersed) | Cuba | | 150 psi | Winterkorn (4) |
| Portland Cement 18% (Immersed) | Cuba | | 354 psi | Winterkorn (4) |
| Portland Cement 5% | India | | 140 psi | Ramachandran (20) |
| Portland Cement 15% | India | | 291 psi | Ramachandran (20) |
| Portland Cement 12% | Thailand | | 900 psi | James & Yimrerehal (33) |

*Proprietary asphaltic stabilizer manufactured by Shell Oil Company.

It can be seen from Table IV that it is possible to stabilize laterites. However, the susceptibility to stabilization varies from excellent to poor depending upon the soil type and admixture used. Generally, the more well developed laterites are easier to stabilize than the younger ones.

# CHAPTER IV

## INVESTIGATIVE PROCEDURES

### Materials

#### Soil

The lateritic soil used in this investigation was obtained from a borrow pit located in Curundu, Panama Canal Zone. The soil was hand excavated from an exposed face at one end of the borrow pit. The samples were taken at random depths varying from the surface to 17 feet. The excavated soil was sealed in plastic bags for shipment. Permission to import the soil to the Oklahoma State University Civil Engineering Soil Mechanics Laboratory was obtained by permit S-688 from the U. S. Department of Agriculture, Plant Quarantine Division.

The borrow pit from which the soil was obtained was used as a source of fill material for construction of the Balboa Bridge. Drill hole studies by Mr. R. H. Stewart (34), geologist for the Panama Canal Company, yielded the following geological profile of the borrow pit:

> 0-4 ft Clay OH 3-4. Medium hard to hard, moderate strength, moderate to high plasticity, high dry strength, moderate water content, silt content increases with depth, consists of a residual saprolitic clay derived from agglomerate by normal weathering processes. Color: mottled bright reds and buff.
>
> 4-17.5 ft Silt OH 3-4. Medium hard to hard, moderate strength, low plasticity, moderate to low dry strength, moderate water content, very clayey at top becoming sandy at base; consists of residual saprolitic clayey, sandy silt derived from agglomerate by normal weathering processes. Color: mottled bright red and buff at top grading to

mottled greys and browns at base.

17.8-18.5 ft Top of sound rock Agglomerate RH 3. Hard, strong, massive jointing and bedding; consists of andesitic and basaltic pebbles ranging from 1/4" to 3.0 ft in diameter in a fine grained sandy matrix of similar composition. Color: mottled reds, browns, blue-grey in a blue-grey matrix; oxidizes rapidly to dark grey and brown on exposure to air.

A narrow gray mottled zone mentioned by several authors (3) (8), as characteristic of lateritic soils, was observed approximately 2-4 feet below the natural ground surface. During laboratory testing numerous small angular pebbles were encountered in this soil. Subsequent investigation by Dr. J. W. Shelton of O.S.U.'s geology department identified these pebbles as quartz, chalcedony and probably parent rock; no definite indication as to origin of the quartz crystals or chalcedony could be given. However, the quartz and chalcedony were probably derived by chemical precipitation of the leached silica from the upper soil horizons.

## Stabilizing Additives

Due to the inherent construction problems involved in working with this type of soil, it is, in most cases, essential to know whether a particular laterite or lateritic soil can be stabilized satisfactorily. It was desired to determine to some extent the effect of stabilizing agents on this soil. Many different materials have been used as stabilizers with varying degrees of success (Table IV, pp. 26-27). Three of the more commonly used stabilizing agents were employed in this study.

Several series of test specimens were made incorporating five percent by weight of quick lime, (CaO) five percent by weight of Type I portland cement, and six percent by total weight of MC-3 asphaltic cutback with the soil. These percentages were selected as representative of the amounts of the various stabilizers used in previous studies. No attempt was made to determine the optimum quantity of the respective stabilizers to be used with this particular soil.

## Sample and Specimen Preparation

### General

Upon receipt of the soil, it was removed from the plastic bags and recombined and mixed to assure a representative sample. Due to permit requirements all soil was stored in 30 gallon waste cans. Tested samples and used portions of the soil were incinerated in a gas fired oven at 350° F for 24 hours prior to disposal.

Working of the soil, i.e. excess mechanical manipulation to simulate the action of heavy construction equipment, was accomplished by mixing the soil with a sufficient quantity of water to surpass the liquid limit in a Hobart mixer with a whip beater (Fig. 1) for a one-half hour period. The worked slurry was then oven dried at 105° C and ground to pass the U. S. No. 10 sieve.

Unworked material was obtained by gently hand sieving the soil through a U. S. No. 10 sieve.

### Sample Mixing

A major problem in all phases of study was to prevent or minimize "working" the "unworked" material used in the various tests and the compacted test specimens. In some cases this could not effectively be done, e.g., the determination of Atterberg limits, hydrometer analysis, and compaction of the test specimens. This points out the need for development of new or modified laboratory testing procedures, as

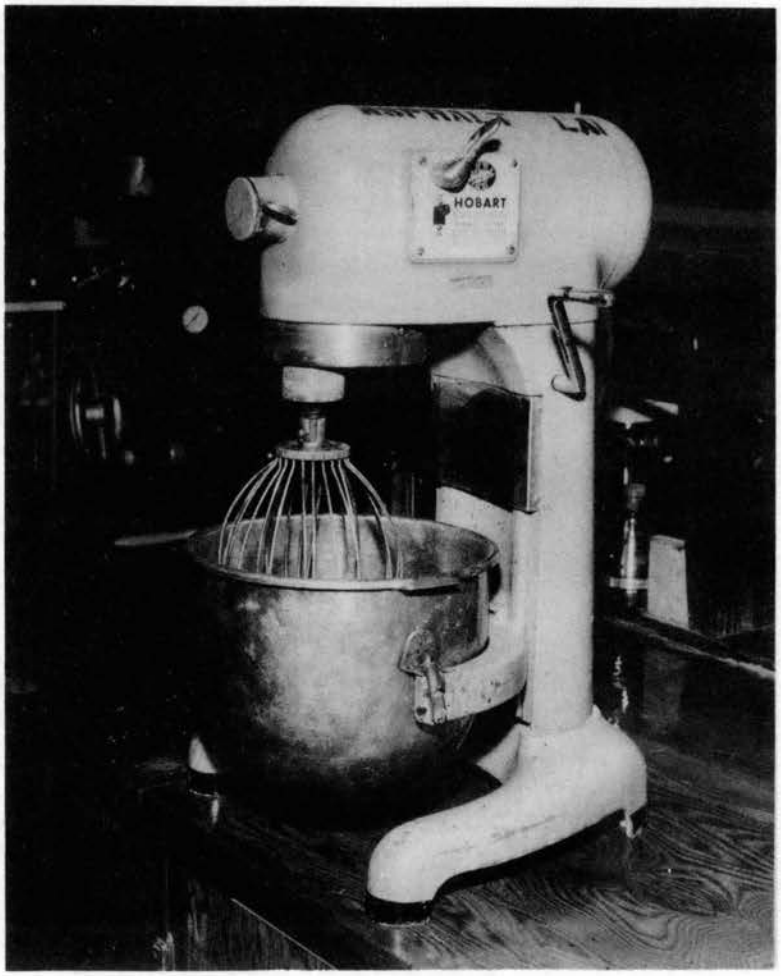


Figure 1. Hobart Mixer with Whip Beater

mentioned by Winterkorn (4), when testing a friable soil such as laterite.

Moisture was added to the unworked samples of the soil prior to compaction or addition of the stabilizer by placing the sample in tare pans, sprinkling it with the desired amount of water, then sealing the pan and contents in a plastic bag and allowing them to sit undisturbed for at least twenty-four hours to assure a uniform distribution of the added moisture. For the worked samples, the soil, additive, and required water to moisten the mixture were mixed in a Hobart mechanical mixer.

Lime

Lime stabilization required large quantities of water to serve as a medium for cationic exchange. Unfortunately, mixing with large quantities of water remolds or works the soil; therefore, the following mixing procedure was used for soil-lime mixtures:

1. The soil and lime were mixed by hand for unworked samples or by mechanical mixer for worked samples.

2. The soil-lime mixture was spread at a 2 to 3 inch depth in a pan.

3. A predetermined amount of water, which was sufficient to raise the moisture content to 50% (approximately midway between the plastic and liquid limits) was sprinkled over the mixture.

4. The mixture was allowed to air dry until the moisture content was below the optimum value of 35% (approximately 4 days).

5. The dry mixture was stirred gently to break up any large soil-lime agglomerates and steps 3 and 4 repeated.

6. The moisture content of the dry mixture was determined.

7. The mixture was then brought to optimum moisture content for compaction.

This procedure resembles a possible construction technique of mixing and spreading the lime and soil with a road grader and then using a water truck to supply the needed moisture.

## Portland Cement

The required amount of cement was added to the soil sample which had been previously moistened to the optimum moisture content. The moist unworked soil-cement was hand mixed prior to compaction, while the moist worked soil-cement was mixed in a mechanical mixer.

## MC-3 Asphaltic Cutback

The mixing and curing procedures used for the soil-asphalt cutback mixture are described in Appendix A.

## Compaction of Test Specimens

To compare objectively the strength values of various soil-stabilizer mixtures, it was necessary that all specimens be compacted to the same density and possess relatively the same particle orientation. Prior to compacting the test specimens, Proctor compaction tests were made on the respective mixtures using varying compactive efforts.

The compactive efforts used were: fifteen blows, twenty-five blows, and thirty-five blows per layer on three layers per test specimen. These tests were conducted using the same compaction mold as was used to mold the unconfined compression test specimens. This method allowed the approximate determination of the number of blows per layer required to achieve a density of 82.5 pcf and the approximate optimum moisture content for this compactive effort. (Table V)

All specimens were compacted in a Harvard Miniature compaction apparatus which had a diameter of 1 5/16 inches and a height of 2.8 inches (Fig. 2). Approximately 110 grams of soil-stabilizer mixture were compacted in three layers by a drop hammer of 0.825 lbs weight, with a face diameter of 0.70 inches and a drop height of 6 inches to a density of 82.5 pcf.

TABLE V

COMPACTION DATA OF SOIL MIXTURES

| Mixture | d | $w_{opt}$ | No. blows/layer |
|---|---|---|---|
| unworked-no additives | 82.5 | 35% | 18 |
| worked-no additives | 82.5 | 35% | 30 |
| unworked + 5% lime | 82.5 | 34% | 42 |
| worked + 5% lime | 82.5 | 33.5% | 32-33 |
| unworked + 5% PC | 82.5 | 33% | 20-25 |
| worked + 5% PC | 82.5 | 32% | 25-28 |
| unworked + 6% MC-3 | 82.5 | 35%* | 25-27 |
| worked + 6% MC-3 | 82.5 | 33%* | 15 |

*See Appendix A

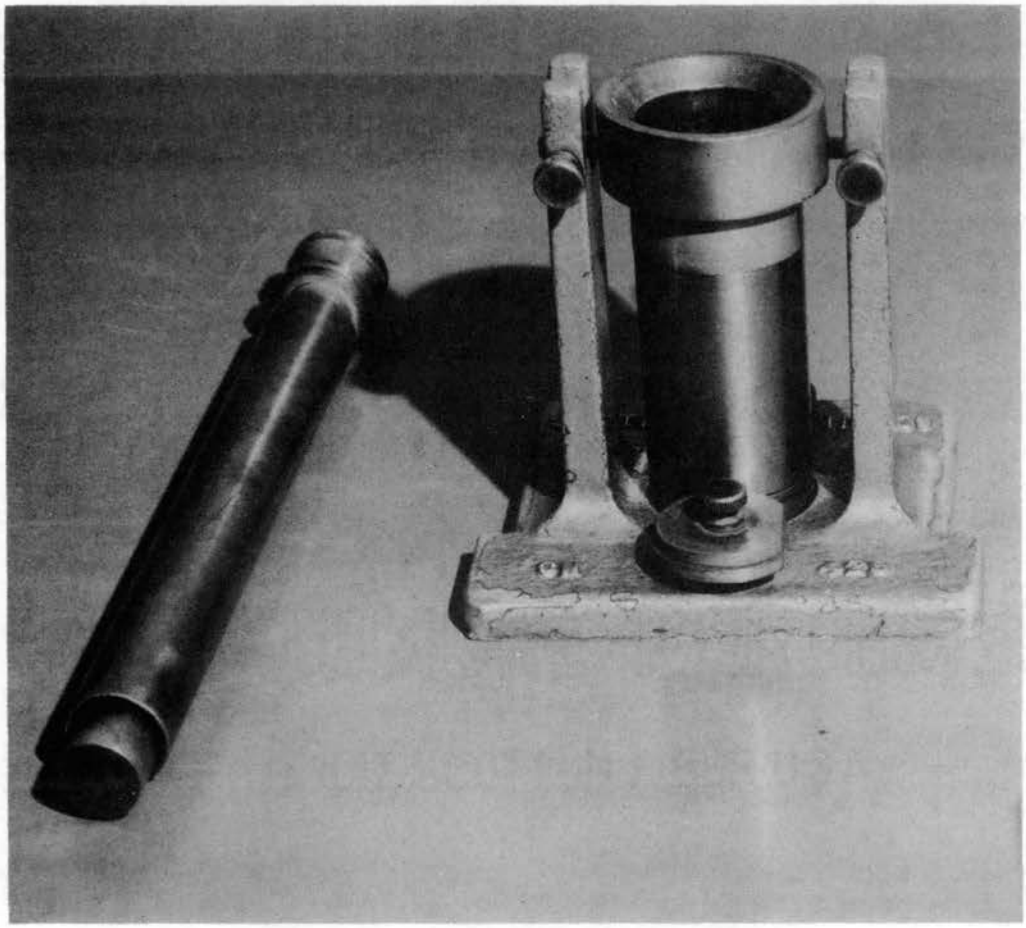

Figure 2. Harvard Miniature Compaction Apparatus

### Curing Test Specimens

After compaction, the soil-lime and soil-cement specimens were wrapped in Saran Wrap, waxed and stored in a moist room for a specified curing period. (Fig. 3) Three curing periods were used: twelve days, twenty-eight days, and sixty days.

The soil-asphalt mixtures were cured in an oven at 150° F prior to compaction according to the procedure in Appendix A. Compacted specimens were not subjected to additional curing before testing.

## Testing Equipment and Procedures

### Atterberg Limits

Liquid limits tests were conducted using a standard liquid limit device which was electrically operated. It was necessary to modify normal mixing procedures for the unworked soil samples. Spatula manipulations were minimized to reduce the amount of working for the unworked samples. Worked samples were mixed by the usual method with no limitations.

### Grain Size Analysis

Grain size determinations utilized slightly modified mechanical and hydrometer methods of analysis. The sample was soaked in distilled water plus a deflocculant, Calgon, for 24 hours prior to conducting the mechanical analysis. The soaked sample was washed through a set of U. S. Standard sieves. After drying, the percentages retained on the various sieves were calculated. The material passing the No. 100 and retained on the No. 200 sieve was recombined with the material passing the No. 200 and used for the hydrometer analysis. By this procedure, an overlap point (a point determined by both sieve and hydrometer

Figure 3. Waxed Specimens

analysis) was obtained, and a smooth continuous grain size curve could be plotted. Unworked samples were not beaten in an electric mixer as suggested by normal procedures.

## Unconfined Compression Tests

After the specified curing time, the samples were stripped of their wax coatings, and their unconfined compressive strength determined. (Fig. 4 and 5) The tests were conducted at a constant deformation rate of 0.05 inches/minute on a Karol Warner compression machine (model 550). (Fig. 6) The reported results are the average of at least three and in some cases five tests. The peak stress was chosen to represent failure. Moisture contents of the broken samples were determined to insure that testing was done approximately at optimum moisture content, where applicable.

## California Bearing Ratio Tests

This test was conducted only on worked and unworked samples, with and without an asphaltic MC-3 cutback. The primary purpose of the test was to compare the swelling tendencies of the samples in order to evaluate more carefully the waterproofing ability of the cutback. The miniature CBR apparatus which is adapted to a Harvard Miniature compaction mold of 1 5/16 inches in diameter was used for this test. (Fig. 7) The samples were compacted to a dry density of 82.5 pcf and soaked four days prior to testing.

The percentage swell was recorded on a daily basis. The constant rate of penetration was 0.05 inches per minute. The same Karol Warner compression machine used for the unconfined compression tests was used for these tests.

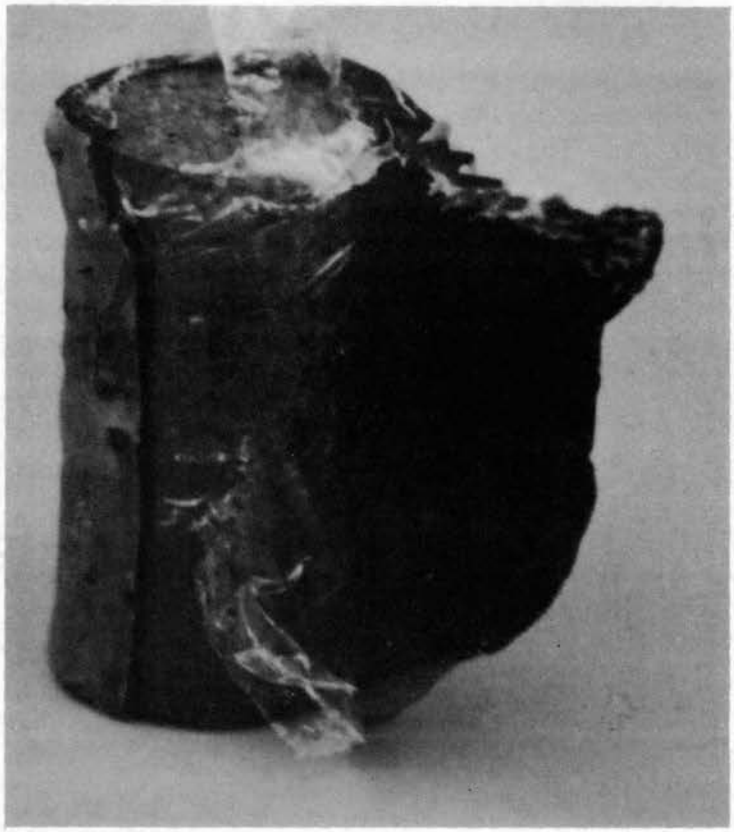

Figure 4. Waxed Coating Partially Stripped from Specimen

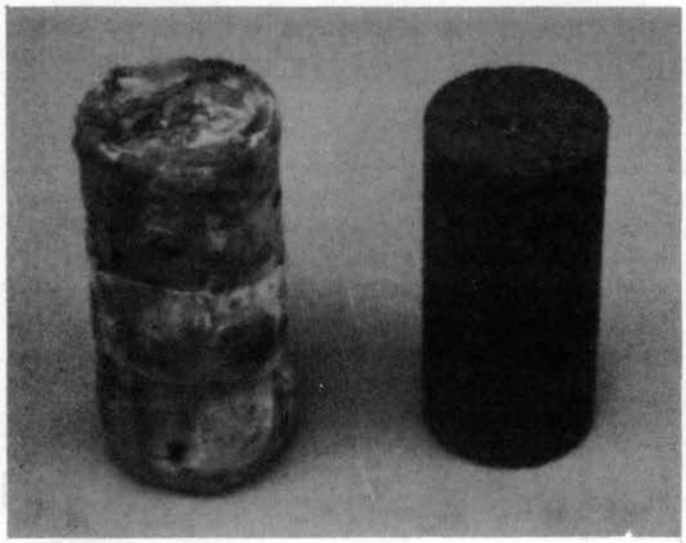

Figure 5. Specimens With and Without Protective Wax Coating

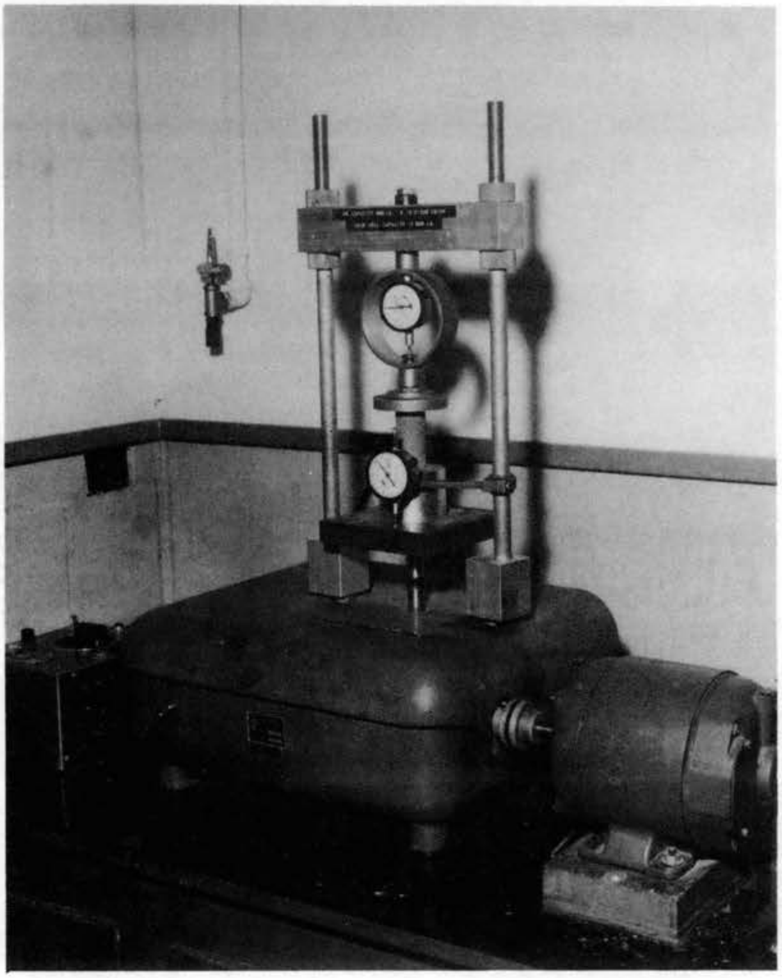

Figure 6. Karol Warner Compression Machine

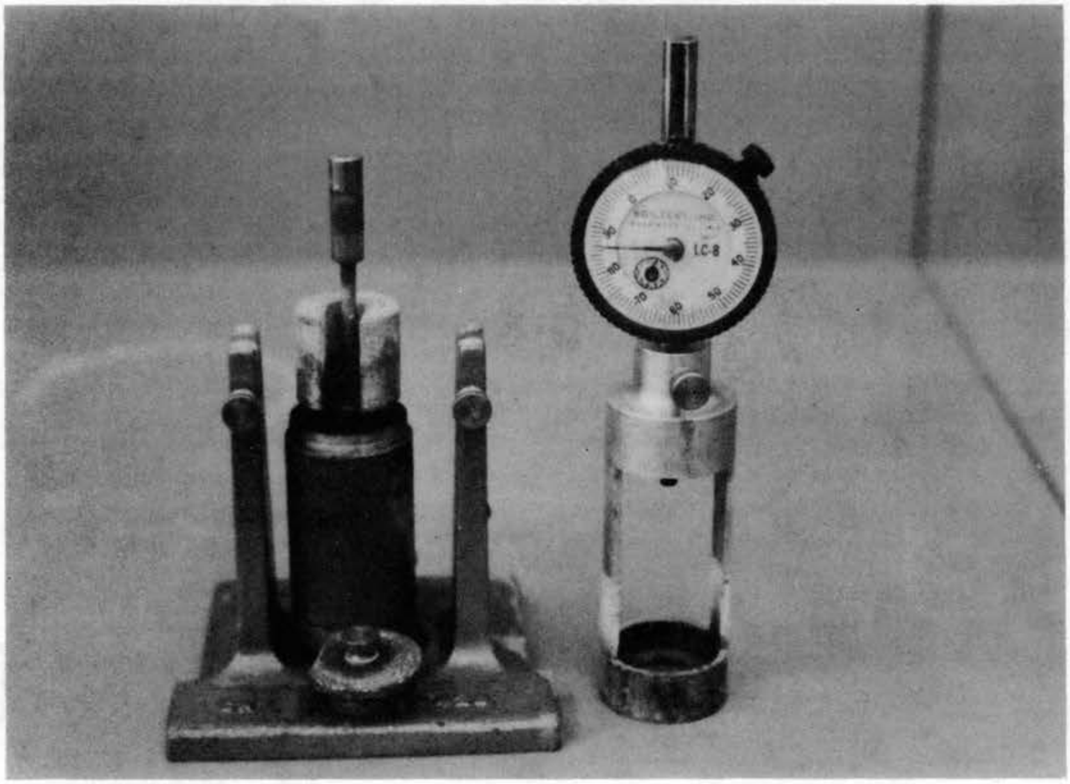

**Figure 7.** Miniature CBR Apparatus Adopted for Use with Harvard Miniature Compaction Mold

# CHAPTER V

## RESULTS AND DISCUSSION

As cited in literature (1) (4) and demonstrated by test results, working causes a breakdown of the lateritic soil particles and an increase in plasticity. The following explanation is hypothesized by this author:

> During the laterization process kaolinite, a clay, is one of the early primary minerals formed by weathering. Further weathering leaches the silica and alkaline earths leaving an abundance of iron and aluminum oxides in hydrated form. These sesquioxides coat and impregnate the clay particles (10) and thus satisfy the electro-magnetic charge possessed by the clay and suppress its characteristics, i.e., plasticity. Working, by mechanical agents in the presence of water, of these coated clay particles causes the iron oxide coating to be abraded away from the clay and thus allows the clay characteristics to become more prevalent. Therefore, the integrity of the iron coatings which provide the granular structure largely determines the engineering properties and behavior of lateritic soils.

However, one might also argue that the change in plasticity of the soil by mechanical working is due solely to the increased amounts of finer particles created by the breakdown of the granular structure. (35) The results of the tests performed in this study do not offer sufficient evidence to prove or disprove either concept. Additional investigation using more sophisticated techniques, e.g., differential thermal analyses, petrographic examination, will be necessary to determine conclusively what actually occurs during mechanical working of this type of soil.

## Atterberg Limits

The Atterberg Limits test results in Table VI, which are the average results of five tests, show that working increased the liquid limit of the soil from 60.5% to 69.6% and that the plastic limit remained basically the same. Similar results were obtained by Winterkorn (4) and Newill (17) who both reported increased liquid limits due to remolding of lateritic soils.

TABLE VI

PHYSICAL PROPERTIES OF WORKED AND UNWORKED LATERITIC SOIL

| Property | Worked | | | Unworked | | |
|---|---|---|---|---|---|---|
| | N.A.* | L.S. | P.C.S. | N.A. | L.S. | P.C.S. |
| Atterberg Limits | | | | | | |
| Liquid Limit | 69.6% | 53.2% | -- | 60.5% | 46.5% | -- |
| Plastic Limit | 40.1% | 31.7% | -- | 39.5% | 40.0% | -- |
| Plasticity Index | 29.5% | 21.5% | -- | 21.0% | 6.5% | -- |
| Specific Gravity | 2.80 | -- | -- | 2.80 | -- | -- |
| Proctor Density | 83.0pcf | 82.0pcf | 84.6pcf | 84.5pcf | 80.5pcf | 85.5pcf |
| Opt. Moisture Content | 34.5% | 34.0% | 32.0% | 35.0% | 35.0% | 32.0% |

*N.A. - No Additive
L.S. - Lime Stabilized
P.C.S. - Portlant Cement Stabilized

## Grain Size Analysis

The grain size distribution curves in Fig. 8 show that the worked soil has a larger percentage of finer particles than the unworked soil. This indicates that mechanical working does cause a disaggregation of the soil structure into finer particles. The curves show that the unworked soil was fairly well graded while approximately 50% of the worked soil was composed of uniform sized particles, 0.004 mm in diameter. This indicates that the unworked soil is large aggregates or clusters composed of uniform small sized particles.

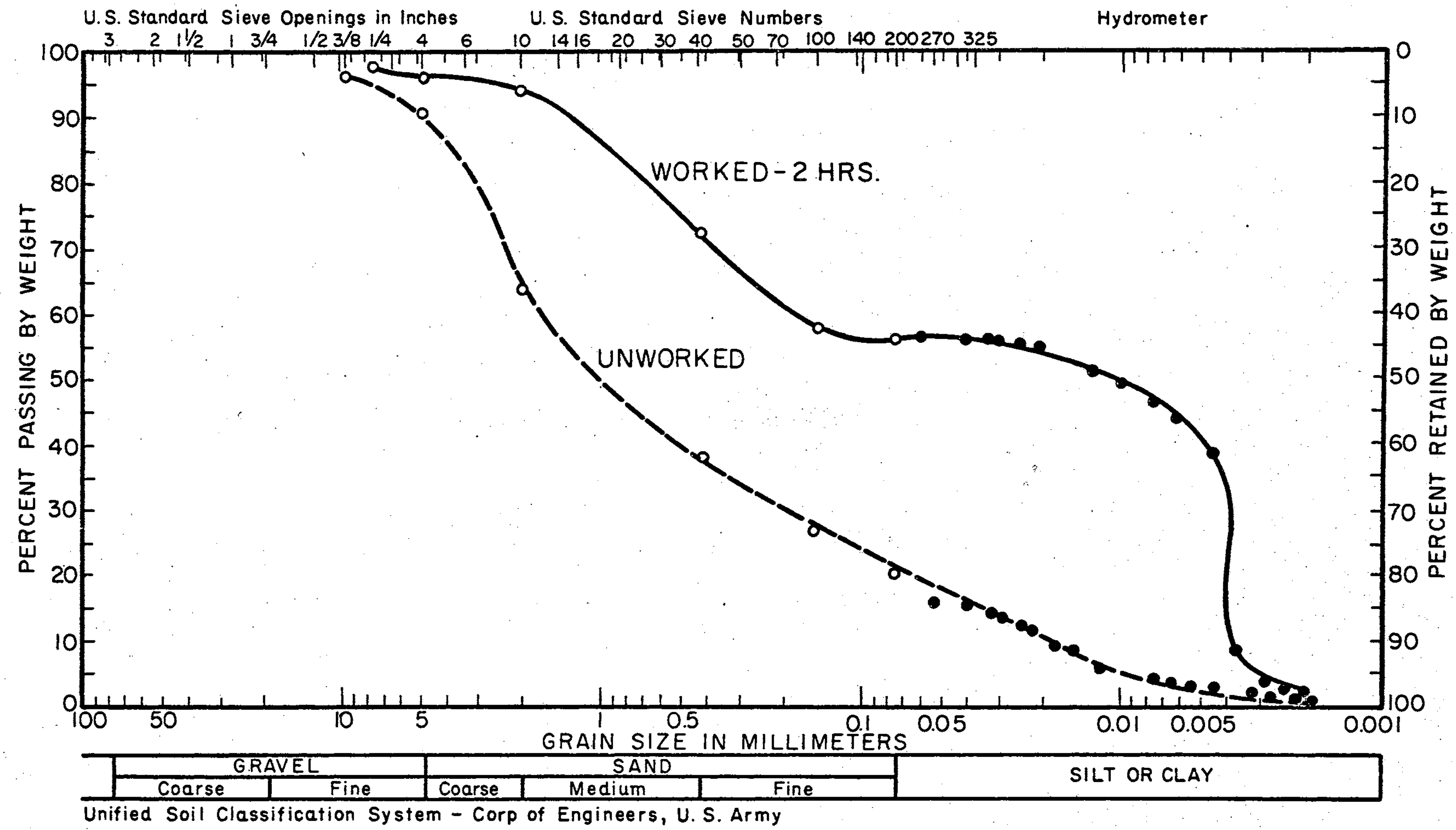


Figure 8. Grain Size Analyses of Worked and Unworked Lateritic Soil

## Proctor Compaction

Density values determined by miniature Standard Proctor compaction tests are listed in Table V and show that the density of the soil was very low in comparison to normal clays of temperate regions which possess similar Atterberg limits. For example, permian red clay from Stillwater, Oklahoma, with a specific gravity of 2.7 has a plasticity index of 22% and standard Proctor density of 106 pcf. Working caused only a slight decrease (1.5 pcf) in maximum density of the soil; however, the moisture contents remained essentially the same.

If two soils differing only in the percentage of finer particles are compacted using the same compactive effort, the soil with the higher percentage of fines will ordinarily reach a greater density at a lower moisture content. This is directly related to the void ratios of the two soils at their maximum density. In the case of the worked and unworked soil this was not true. The unworked soil with a smaller percentage of finer particle sizes achieved the higher density. This can be interpreted as an indication of an increase in effective or active clay content of the worked soil due to the removal of the sesquioxide coatings from the clay particles. For a given dry density below optimum moisture content, the worked soil required more moisture than the unworked soil due to the increased water adsorption by the clay particles.

A comparison of the optimum moisture content and plastic limit indicates compacting on the wet side of optimum will probably cause the soil to become highly plastic and complicate field compaction with heavy equipment.

## Effects of Stabilizers

The results of unconfined compression tests of the various stabilized soil mixtures are tabulated in Table VII. Each value is an average of not less than three tests. The comparison of stress-strain characteristics of worked and unworked stabilized soils under similar conditions of density and curing time is graphically shown in Appendix B.

TABLE VII

UNCONFINED COMPRESSIVE STRENGTHS OF STABILIZED LATERITIC SOIL

| Stabilizer | Curing Time (days) | Unconfined Compressive Strength (Psi) | |
|---|---|---|---|
| | | Worked | Unworked |
| Natural Soil | 0 | 20 | 22 |
| MC-3 | 0 | 13.8 | 17.5 |
| Lime | 12 | 38 | 13.8 |
| | 28 | 75 | 51 |
| | 45 | 80 | -- |
| | 75 | -- | 67 |
| Portland Cement | 12 | 70 | 113 |
| | 28 | 87 | 117 |
| | 45 | 95 | -- |
| | 60 | -- | 140 |

Lime

The discovery that the workability of a clay soil would be greatly improved by the addition of lime, $Ca(OH)_2$, dates back to remote times when the Romans constructed the Appian Way. Although lime has been used quite extensively, the reactions of lime with soil are still for the most part a mystery. Reports describing the benefits of lime stabilization mention that the plasticity of the soil is reduced, and if properly cured, a substantial increase in strength occurs. The most accepted explanation of these results is that a base exchange reaction occurs with a replacement of certain ions, i.e. the replacement of sodium or hydrogen with calcium. At the same time but much more slowly, pozzalanic reactions, which are the formation of calcium silicates by the reaction of the lime with free silica, result in cementing compounds. (36)

A comparison of the Atterberg Limits values in Table V shows a reduction in plasticity of the worked and unworked soils. However, this reduction in plasticity is much greater in the unworked soil; in fact, it is almost double that of the worked soil. This is an indication that unworked soil contains a less effective clay content that the worked and will require less lime for base exchange to reduce plasticity.

The data presented in Table VI with one exception shows an increase in unconfined compressive strength of both worked and unworked soil with the addition of lime. The graphical presentation of data in Fig. 9 shows some interesting aspects. Both the worked and unworked soils exhibit increases in strength, with the worked soil showing more rapid and greater strength gains with curing time. The data for the

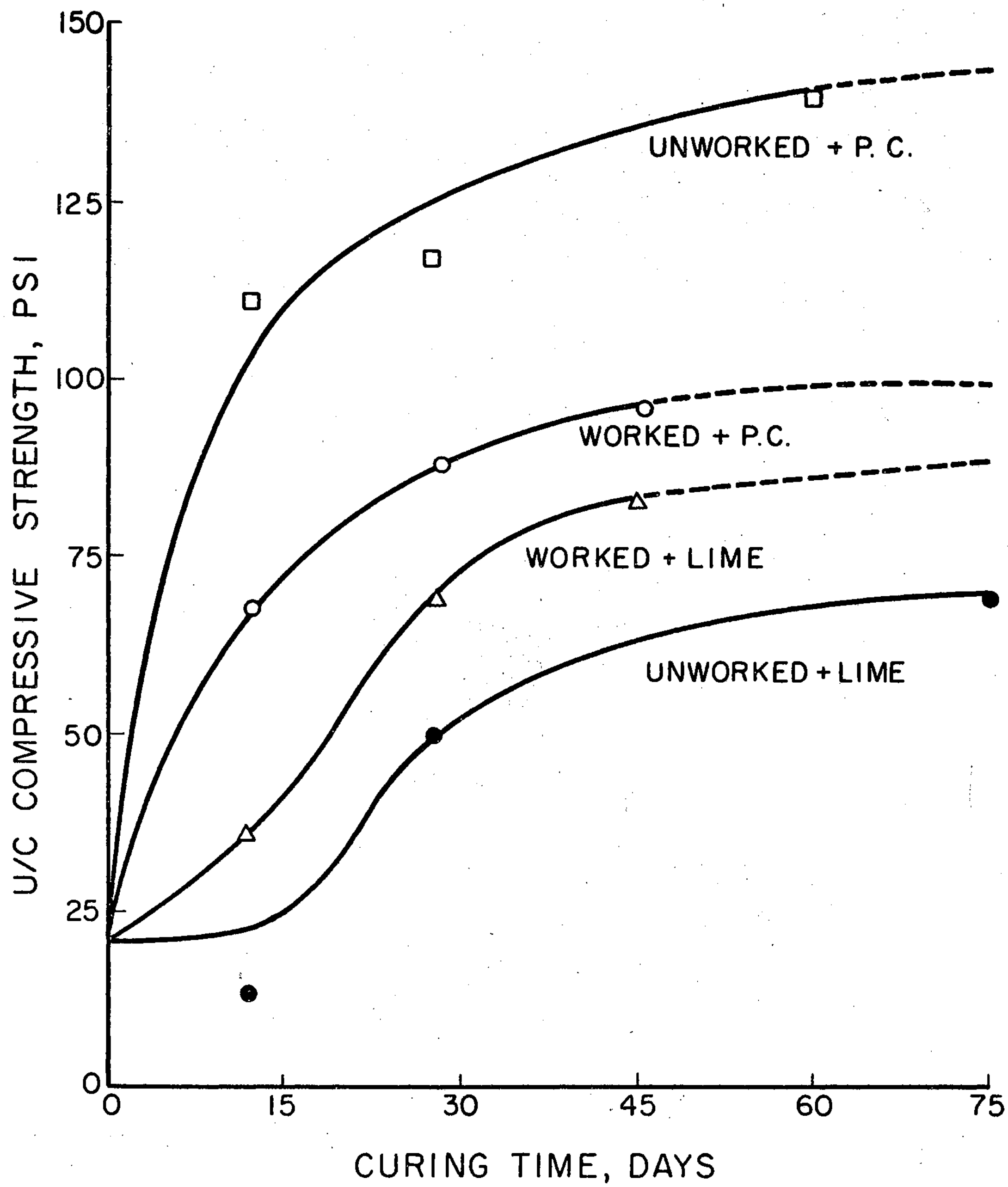


Figure 9. Effect of Curing Age on Unconfined Compressive Strength of Laterite + 5% Lime or 5% Portland Cement

unworked soil indicates a reduction in strength after 12 days curing. Although this value represents the highest strength of several tests specimens, there is some doubt as to the accuracy of this test series. The curve was drawn to reflect no change in strength of the soil until after this period of curing. With regard to the trends of the two curves, it appears reasonable that the strength of the unworked soil plus lime specimens after 12 days curing would be the same as the initial strength or slightly greater.

Since most or all free silica has been leached from lateritic soils, the silica available for pozzalanic action with lime should be primarily found in the clay particles. Thus, the more rapid and greater strength gains of the worked soil reflect a higher effective clay content. It appears that pozzolanic action between the lime and unworked soil is slower than similar reactions between lime and the worked soil as little or no initial strength gain is observed for unworked soil plus lime. It may be possible that the sesquioxide coatings on the clay particles in unworked soil inhibit the calcium and clay (silica) reactions. These sesquioxide coatings are replaceable by calcium, but insufficient moisture to ionize completely these coatings would limit the rate of ionic exchange by the calcium. Strength gains in unworked soil would be limited to pozzolanic action between small amounts of unleached silica and the "exposed" clay particles present in the soil. Conversely, worked soil, in which these coatings may have been partially removed by working action, presents greater amounts of exposed clay particles with which the calcium readily reacts producing greater strength gains at a more rapid rate.

Reduced plasticity reflected by a more granular texture caused a

4.0 pcf decrease in Proctor density of the unworked soil, while only a 1.0 pcf decrease occurred in the worked soil (Table V). Apparently the mixing process described in the Investigative Procedures provided sufficient moisture for the base exchange phenomenon to occur readily. Since the unworked soil possessed a lower effective clay content, aggregation caused by flocculation of the clay occurred more readily than in worked soil. The worked soil with a higher effective clay content apparently possessed a more dispersed structure which would require more time and a greater amount of ionic exchange to achieve the same degree of aggregation as for unworked soil.

## Portland Cement

As shown in Table VI the portland cement stabilized test specimens had higher strengths for both the worked and unworked soil than the lime or MC-3 stabilized specimens. A comparison of the stress strain curves in Appendix B for soil-cement and natural soil shows that the soil-cement exhibited a more brittle type of failure. The curves for these specimens peak sharply at the maximum stress which is more or less characteristic of concrete mixtures. This indicates a rupture or breakdown of the skeletal structure formed by the hydrating cement between the soil particles.

The hydration of the cement would be relatively unaffected by the chemical composition of the soil particles. This is evidenced by the rapid gains in strength by both the worked and unworked specimens during the first few days of curing (see Fig. 9). However, the curves for these specimens in contrast with the lime stabilized specimens show higher strengths for the unworked soil. This is interpreted as an indication of the increased amount of finer particle sizes in the worked soil. Since, in general, the finer the texture of a soil, the greater the amount of cement required to harden it to a satisfactory degree, the worked soil would require a larger percentage of cement to reach strength values comparable to the unworked soil.

The 28 day compressive strength values were lower than expected. No definite explanation can be given for this except that it is a result of experimental or operator error.

MC-3 Asphaltic Cutback

Successful stabilization of fine grained plastic soils with asphalt has been somewhat limited due to the problem of achieving good mixing of the asphalt and the soil. Yet in the case of lateritic soils, if it were possible to waterproof the soil particles, the remolding effect of water on this soil type would be greatly reduced. During preparation of the soil asphalt specimens, it was found that a soil-asphalt mixture completely cured in an oven could not be mixed with water. This indicates that if it were possible to compact properly such a mixture, even with relatively low percentages of asphalt, the resulting material would be quite waterproof and considerably more stable. The test results and experience of this study indicate that aeration or curing of lateritic soil-asphalt mixtures is quite critical if good results are to be achieved with this type of stabilizer.

Test results in Table VI indicate the addition of MC-3 cutback caused a reduction in strength for both worked and unworked soil. This reduction in strength is considered to be due to incomplete curing of the soil-asphalt mixture. Since the curing and compaction procedure adopted for this series of test specimens left a considerable amount of the volatile constituents of the cutback in the compacted mixture, the less viscous asphalt cement may have acted as a "lubricant" instead of a "binder" between the soil particles and thus reduced the unconfined compressive strengths of the specimens. With proper aeration it is quite probable that these mixtures would show an increased strength due to cementing action of the base asphalt cement.

The unworked soil-asphalt specimens had a slightly higher strength than the worked specimens. This was also true in the unstabilized test specimens. As the plasticity of a soil increases, its cohesion generally increases also, and the stability of highly plastic soil-asphalt mixtures depends more on the cohesive characteristics of the soil than on the cementing value of the asphaltic material.

The CBR tests were performed on the soil-asphalt mixtures to determine primarily any decrease in swelling tendencies of asphalt stabilized soil. These results also indicated a decrease or reduction in strength of both worked and unworked soil with the addition of the cutback. The swell tests indicated rather low swelling characteristics for this particular lateritic soil, but due to incomplete curing of the mixtures and the limited number of tests performed, no satisfactory indications could be made, and no data is presented.

## Testing Procedures

The test results presented indicate that there exists a significant difference between physical properties and behavior of worked and unworked lateritic soil. Laboratory procedures often remold or work the soil during testing, and such working will definitely influence the results. For example, the amount of spatula working during a liquid limit test and the use of an impact type of compaction will work or breakdown the soil structure to some extent. Newill (17) also reported that the method of treatment prior to testing will affect the test results.

It appears that standard laboratory testing procedures should be modified to prevent or minimize "working" of lateritic type soils, perhaps along lines similar to the procedures used in this study. The use of some type of static compaction equipment for density studies and compacting test specimens would improve the test results.

## CHAPTER VI

## CONCLUSIONS

This investigation was a study of the engineering characteristics of a Panamanian lateritic soil and an attempt to determine the suitability of various additives for stabilizing purposes. Within the limits of the type of soil and testing procedures employed, the following conclusions can be made:

1. Mechanical working causes a breakdown of the soil particles and increases the percentage of finer particles particularly the 0.004 mm equivalent size.
2. Mechanical working of the soil apparently increases its "effective" clay content. The breakdown or stripping away of the sesquioxide coatings allows the indigenous clay particles to behave in a more characteristic fashion.
3. Stabilization of the soil by various additives is influenced to a considerable extent by mechanical working. The type and quantity of stabilizer necessary in field application will depend to a large extent on the construction equipment and techniques employed.
    a. From the standpoint of strength, portland cement is the most effective of the three stabilizing additives used, and better results can be achieved with the soil in the unworked condition.

b. Lime is an effective stabilizer for this type of soil and reacts more favorably, i.e., develops higher strengths, with the soil in the worked condition.

c. Proper aeration or curing of the mixtures is a critical factor if effective stabilization of the soil is to be obtained using cutback asphalt as an additive. The primary benefit from asphaltic stabilization will be in waterproofing the soil.

4. New or revised laboratory techniques are necessary for extensive evaluation of the properties of laterites and lateritic soils.

## Recommendation for Research

The following are suggestions for further research on lateritic soils:

1. A more comprehensive investigation using more sophisticated equipment and techniques such as differential thermal analysis and petrographic studies to determine what effect working has on the physical and chemical properties of lateritic soil. Such an investigation could give more insight as to the existence of sesquioxide coatings and their behavior as hypothesized in this study.
2. The development of standard testing procedures which would reduce or minimize remolding or working of the soil; for example, static pressure compaction of specimens rather than impact compaction as used in this study.
3. An investigation to determine the optimum percentages of the

stabilizing additives used in this study, as well as the effects of other additives. The results of this type of investigation could be correlated with actual field studies.

4. An investigation similar to this study, but using lateritic soils from different areas of the world. Such an investigation would determine if the conclusions of this study are applicable to a wide variety of lateritic soils.

## BIBLIOGRAPHY


1. Bawa, K. S. "Laterite Soils and Their Engineering Characteristics," Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 83, Nov. 1957.

2. Wooltorton, F. L. D. "Engineering Pedology and Soil Stabilization," Bulletin 108, H.R.B., 1949, p. 29.

3. Mohr, E. C. J. Tropical Soils. New York, New York: Interscience Publishers, 1954.

4. Winterkorn, H. F. and Chandorshokhoran, E. C. "Laterite Soils and Their Stabilization," Bulletin 44, H.R.B., 1951.

5. Buchanan, F. "A Journey from Madras through the Countries of Mysore, Canara, and Malabar, etc. (in 1800-1801)," The East India Co., London, 1807, Vol. 3, p. 440.

6. Mackenzie, R. C. The Aging of Sesquioxide Gels I: Iron Oxide Gels," Mining Mag. (London), Vol. 32, pp. 153-165.

7. Martin, F. J. and Doyne, H. C. "Laterite and Lateritic Soils in Sierra Leone," J. Agric. Sci., Vol. 17, 1927, p. 530.

8. Nixon, I. K. and Skipp, B. O. "Airfield Construction on Overseas Soils Part 5: Laterite Soils," Proc. Inst. of Civil Eng., Vol. 8, Nov. 1957, p. 253.

9. Humbert, R. P. "The Gensis of Laterite," Soil Science, Vol. 65, 1948, p. 281.

10. Alexander, L. T. and Cady, J. G. "Gensis and Hardening of Laterite," USDA Tech. Bull. 1282.

11. Christophe, L. "Les Laterites et Leur Uilization Comme Maleriau de Construction des Routes," Rev. Gen. des Routes et des Aerodromes, Paris, No. 212, Sept. 1949, p. 38.

12. Sirarajasingham, S, Alexander, L. T., Cady, G., and Cline, M. G. "Laterite," Adv. in Agronomy, Vol. 14, 1962, pp. 1-56.

13. Terzaghi, K. "Design and Performance of Sasamua Dam," Proc. Inst. of Civil Eng., Vol. 9, April, 1958, p. 369.

14. Ramashkevich, A. I. "Micromorphological Indications of the Processes Associated with the Formation of the Krasnozems

(Red Earths) and the Red-Colored Crust of Weathering in the Transcaucasus," Soil Micromorphology, Ed. Jongerius, Elsevier Publishing Co., Amsterdam, 1964, p. 261.

15. Taylor, A. E. "Physico-Chemical Properties of Soils: Ion Exchange Phenomena," Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 85, 1959, p. 19.

16. Zipkes, E. "An Investigation on Some Laterites," Indian Roads Congress, Vol 18, Dec., 1953.

17. Newill, D. "A Laboratory Investigation of Two Red Clays from Kenya," Geotechnique, Vol. VII, 1962, p. 302.

18. Vargas, M. and Silver, F. P. "Residual Earth Dams in State of Sao Paulo, Brazil," Sixth Int. Conf. on Soil Mechanics and Foundations Engineering, Vol. II, 1965, p. 578.

19. Nascimento, U. "Swelling and Petrifaction of Laterite Soils," Formento, Vol. 1, 1963, p. 23.

20. Ramachandran, E. K. "Stabilization of Lateritic Soils," Indian Roads Congress Road Research Bulletin No. 4, Nov., 1957.

21. Lambe, T. W. Soil Testing for Engineers. New York: John Wiley and Sons, Inc., 1967, p. 29.

22. Fruhauf, B. A. "A Study of Lateritic Soils," Proceedings, H.R.B., Vol. 26, 1946, p. 579.

23. Dowling, J. W. F. "The Mode of Occurrence of Laterites in Northern Nigeria and Their Appearance in Aerial Photography," Eng. Geology, Vol. 1, 1966, pp. 221-233.

24. Sowers, G. F. "Soil Problems in Southern Pidmont Regions," Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 80, Nov., 1954.

25. Remillion, A. "Stabilization of Lateritic Soils," Bulletin 108, H.R.B., 1955, p. 96.

26. Gizienski, S. F. "Disc Lateritic Soils and Their Engineering Characteristics," Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 83, Nov., 1957.

27. Deere, D. Seepage and Stability Problems in Deep Cuts in Residual Soils," Amer. Railway Eng. Bull., Vol. 535, Feb., 1957.

28. Giraud, M. "The Behavior of Earth Roads Under Traffic," Abstracts H.R.B., Vol. 28, 1958, p. 9.

29. Tongmark, K. "Strength and Modulus of Elasticity of Lateritic Soil-Cement," M.S. Thesis No. 157, SEATO Graduate School of Engineering, Bangkok, Thailand.

30. Schofield, A. N. "Lime Stabilization of a Nodular Clayey Pea Laterite in Nyasaland," Road Res. Laboratory (HMSO) Bull. No. 3, London, 1957.

31. Medina, J. "Disc-Laterite Soils and Their Engineering Characteristics," Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 83, Nov. 1957.

32. Michelin, F. S. "Asphaltic Stabilization of Lateritic Soils with the Proprietory Additive CK3 and Aluminum Sulfate," Tech. Note No. 28, SEATO Graduate School of Engineering, Bangkok, Thailand, Feb., 1967.

33. Jones, J. H. and Yimsirikul, C. "Cement Treatment of Selected Lateritic Soils," Tech. Note No. 22, SEATO Graduate School of Engineering, Bangkok, Thailand, May, 1965.

34. Stewart, R. H. Geological Field Log, Panama Canal Col., Eng. and Const. Bureau, Hole AB-17, Oct., 1958.

35. Eades, J. L. and Grimm, R. E. "Reaction of Hydrated Lime with Pure Clay Minerals in Soil Stabilization," Bulletin 262, H.R.B., 1960, p. 51.

37. Martin J. R. and Wallace, H. A. Design and Construction of Asphalt Pavements. New York: McGraw-Hill Book Co., 1958.

# APPENDIX A

# APPENDIX A

## CURING AND COMPACTION PROCEDURES FOR LATERITIC SOIL-ASPHALT SPECIMENS

In soil-asphalt mixtures incorporating a cutback asphalt, the mixture must be aerated prior to compaction. This is necessary to reduce the amount of both moisture and volatile constituents in the mixture. If the mixture were compacted with a relatively large percentage of moisture and volatiles, low stability values, e.g., low unconfined compressive strengths, would result. Ideally the moisture content should be reduced to approximately 75% of optimum for the soil and 65 to 75% of the volatile constituents of the cutback evaporated prior to compaction: (37)

Aeration of the lateritic soil-cutback asphalt mixture was accomplished by drying in an oven and stirring at frequent intervals. Both the moisture and volatile contents of the mixture were reduced by evaporation during this drying process. However, compliance with the above criteria for better stability was not checked. While the percentages of moisture and volatiles remaining in the mix after various drying periods can be determined by distillation procedures, the purpose of this phase of the study did not warrant the determination of the optimum drying or aeration time of the mixture. A more comprehensive study of stabilizing this type of soil with asphaltic materials would of necessity include such a determination as well as the determination of the most desirable type and grade of cutback to use and the optimum asphalt content to achieve maximum strength and waterproofing.

Since the study was made primarily to determine the relative effects of various stabilizers and was not a complete evaluation of any particular one, the following procedure was adopted for drying and compacting the soil-asphalt mixtures.

Unworked Soil

1) The soil was brought to optimum moisture, as described under the section on Sample Mixing, prior to the addition of the cutback. Since moisture acts as a carrier for the asphalt, this provided sufficient moisture necessary for good mixing and coating of the soil particles.

2) The percentages of volatile constituents and asphalt cement in the cutback were determined. From these values the amount of liquids (volatiles) and solids (asphalt cement) being added to the soil were calculated.

3) Six percent of the MC-3 cutback on a total weight basis was added to the wet soil and hand mixed to minimize working of the soil.

4) The soil-asphalt mixture was placed in an oven at 150° F and stirred at frequent intervals until a predetermined weight loss was obtained. This loss in weight was due to the evaporation of a portion of the liquids, i.e., both moisture and volatiles in the mixture.

5) The remaining weight of liquid (volatiles and moisture) in the mixture was used to calculate the "moisture" content of the solids (soil and asphalt cement) prior to compaction. These values are indicated in Table V.

6) The compactive effort was then varied to achieve a dry density of 82.5 pcf for the finished test specimens.

Worked Soil

1) The necessary amount of water to bring the dry worked soil to optimum moisture content was added to the soil and mixed by mechanical means.

2) Six percent of MC-3 cutback on a total weight basis was added to the wet soil and mixed by hand.

3) The drying and compaction procedures were the same as presented above for the unworked soil-asphalt material.

APPENDIX B

STRESS-STRAIN CHARACTERISTICS

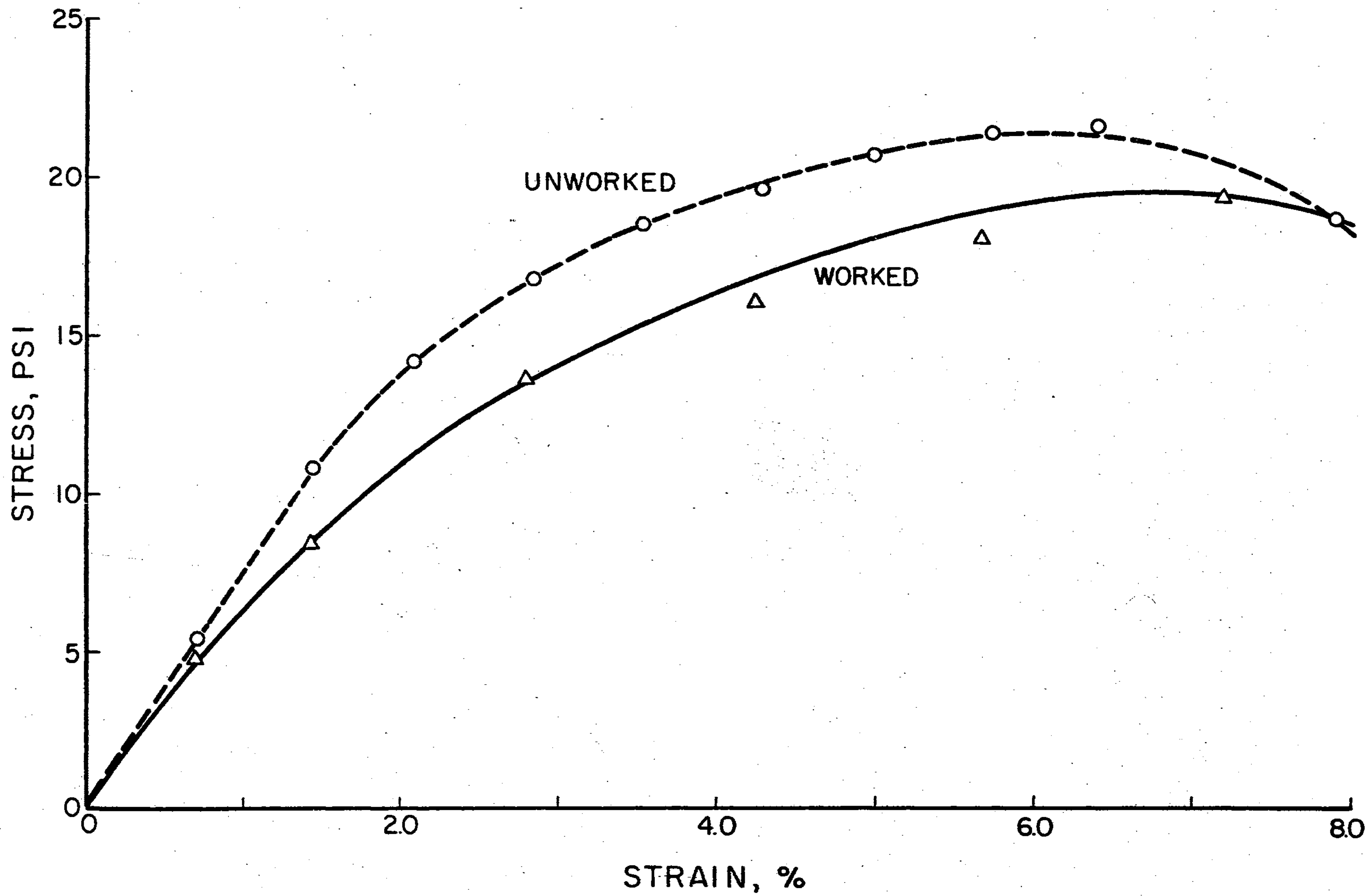


Figure 10. Stress-Strain Characteristics, Laterite + No Additives

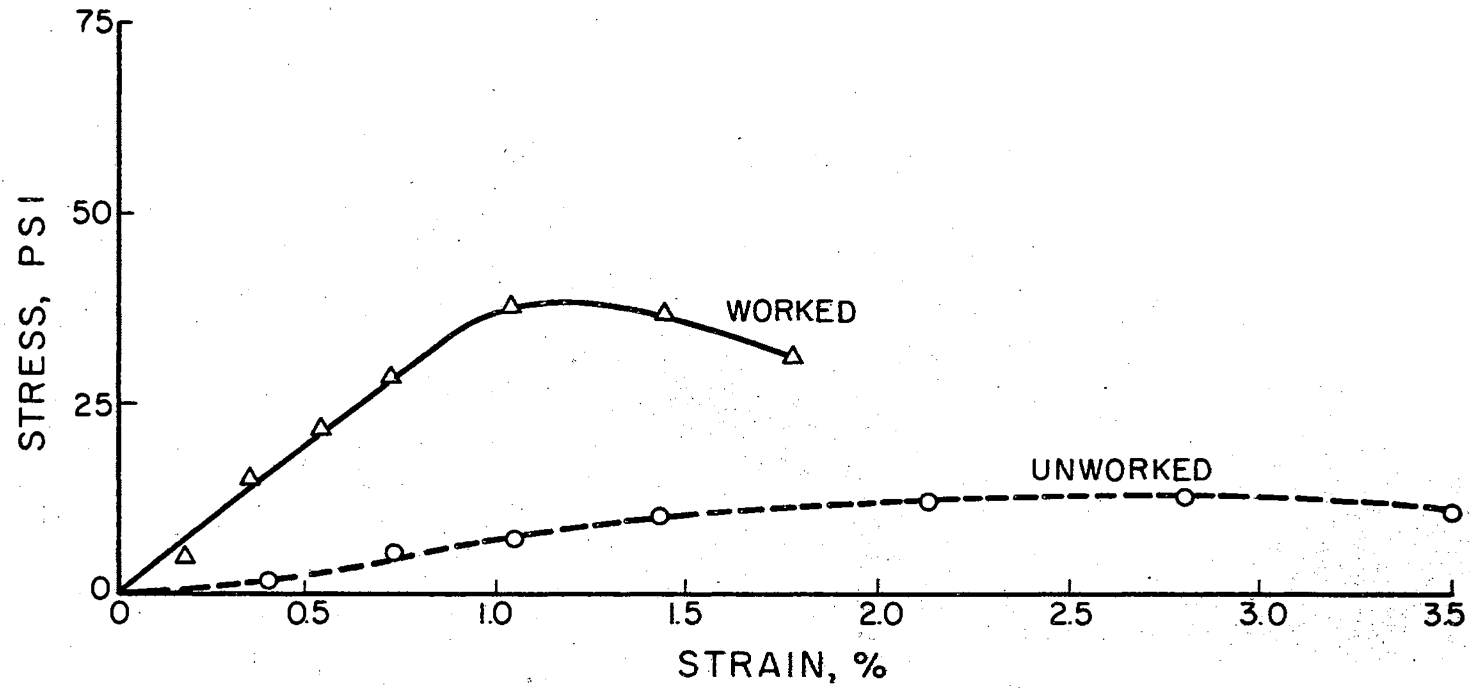


Figure 11. Stress-Strain Characteristics, Laterite + 5% Lime, 12 Days Cure

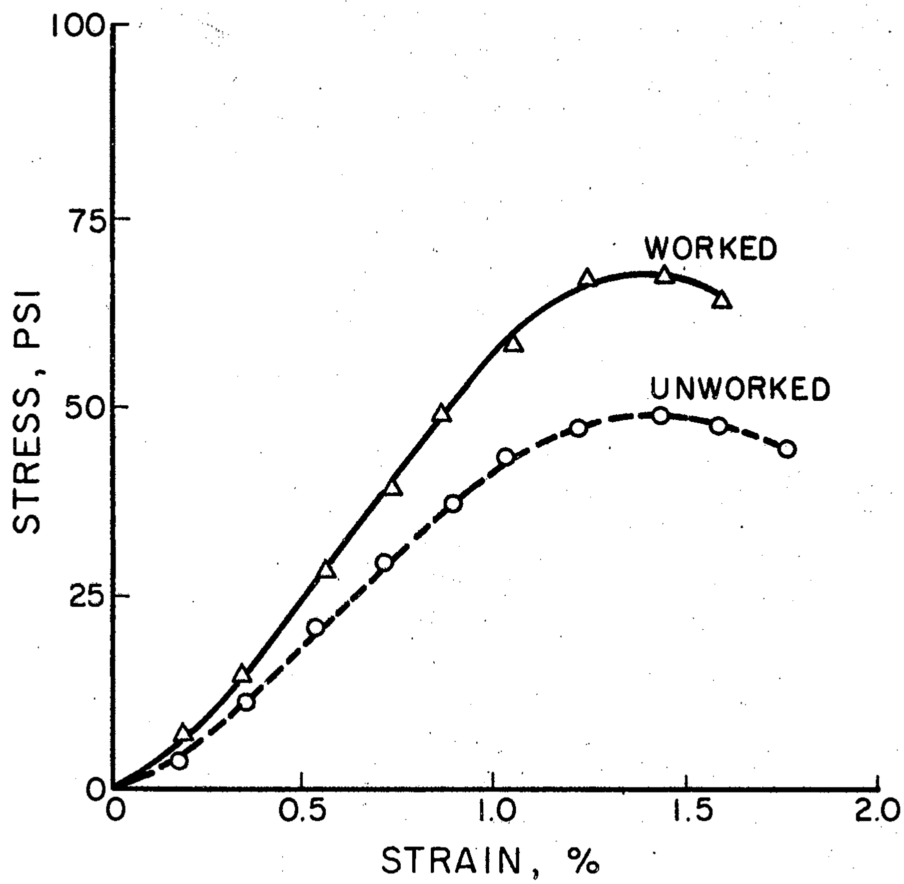


Figure 12. Stress-Strain Characteristics, Laterite + 5% Lime, 28 Days Cure

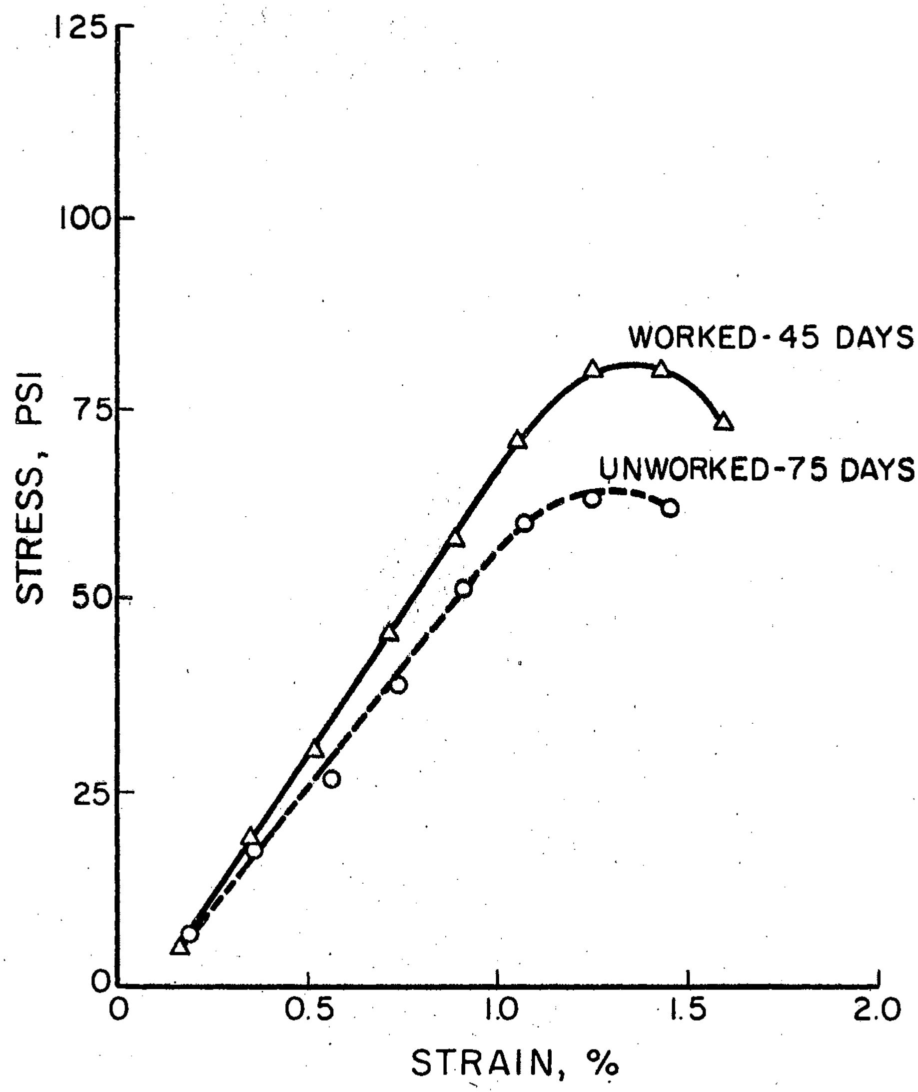


Figure 13. Stress-Strain Characteristics, Laterite + 5% Lime, 45 and 75 Days Cure

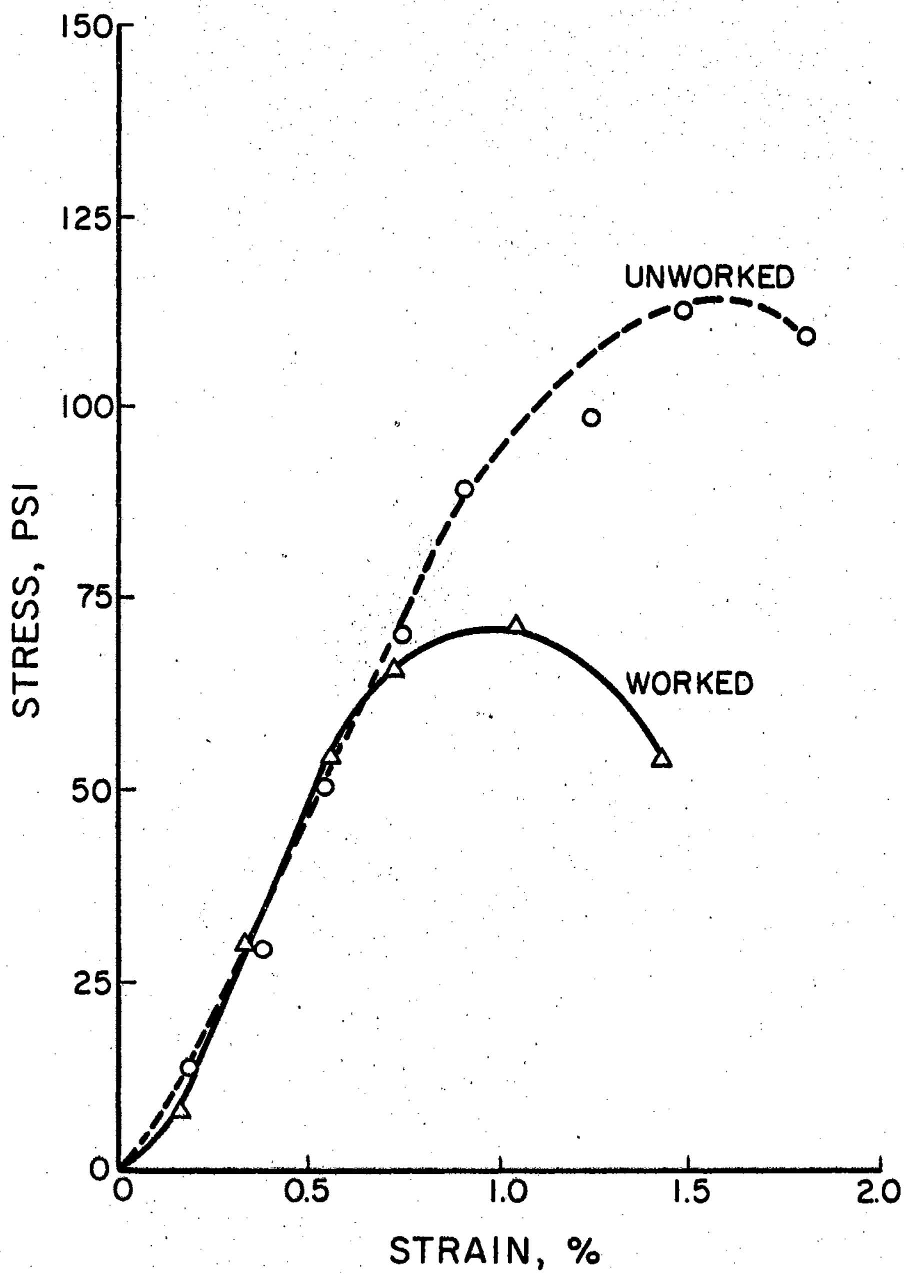


Figure 14. Stress-Strain Characteristics, Laterite + 5% Portland Cement, 12 Days Cure

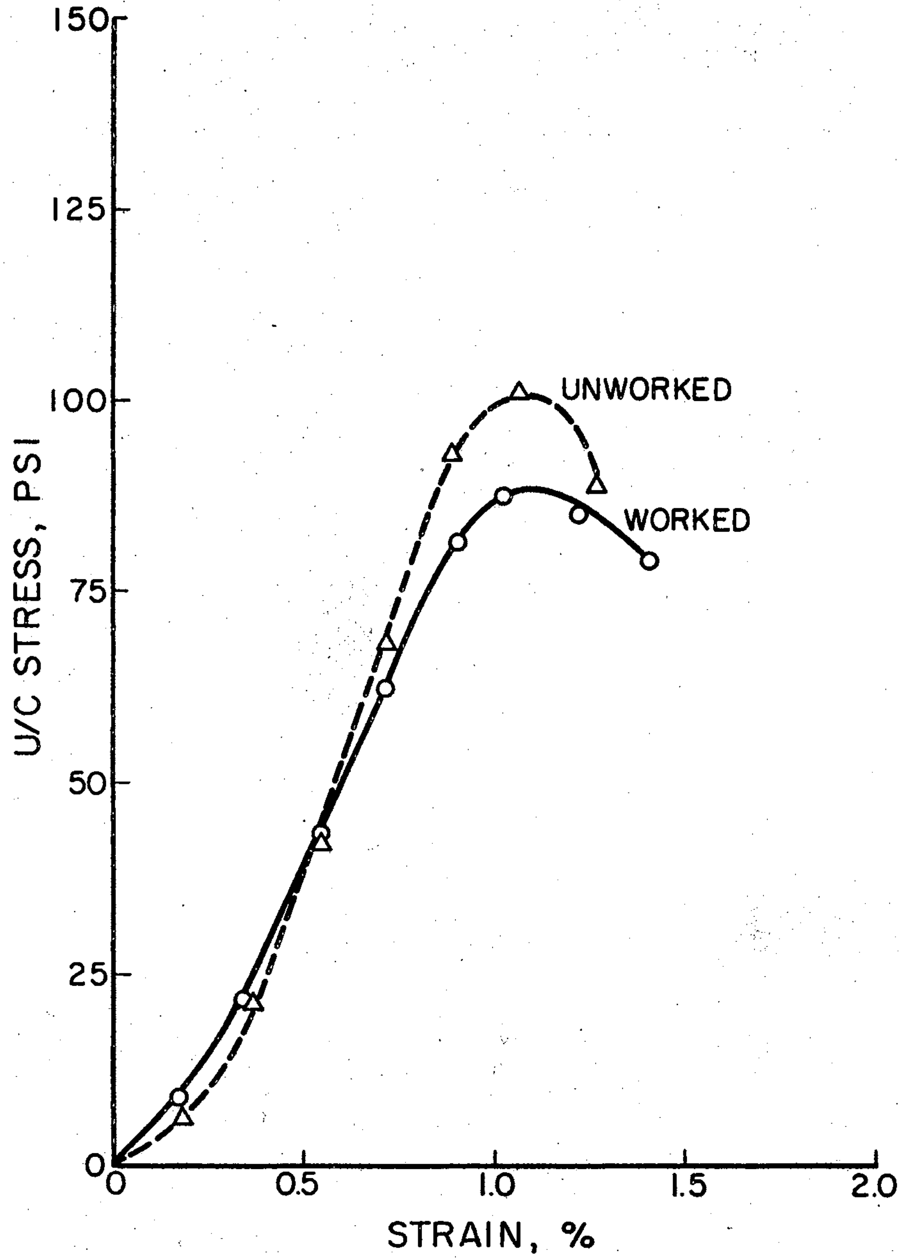


Figure 15. Stress-Strain Characteristics, Laterite + 5% Portland Cement, 28 Days Cure

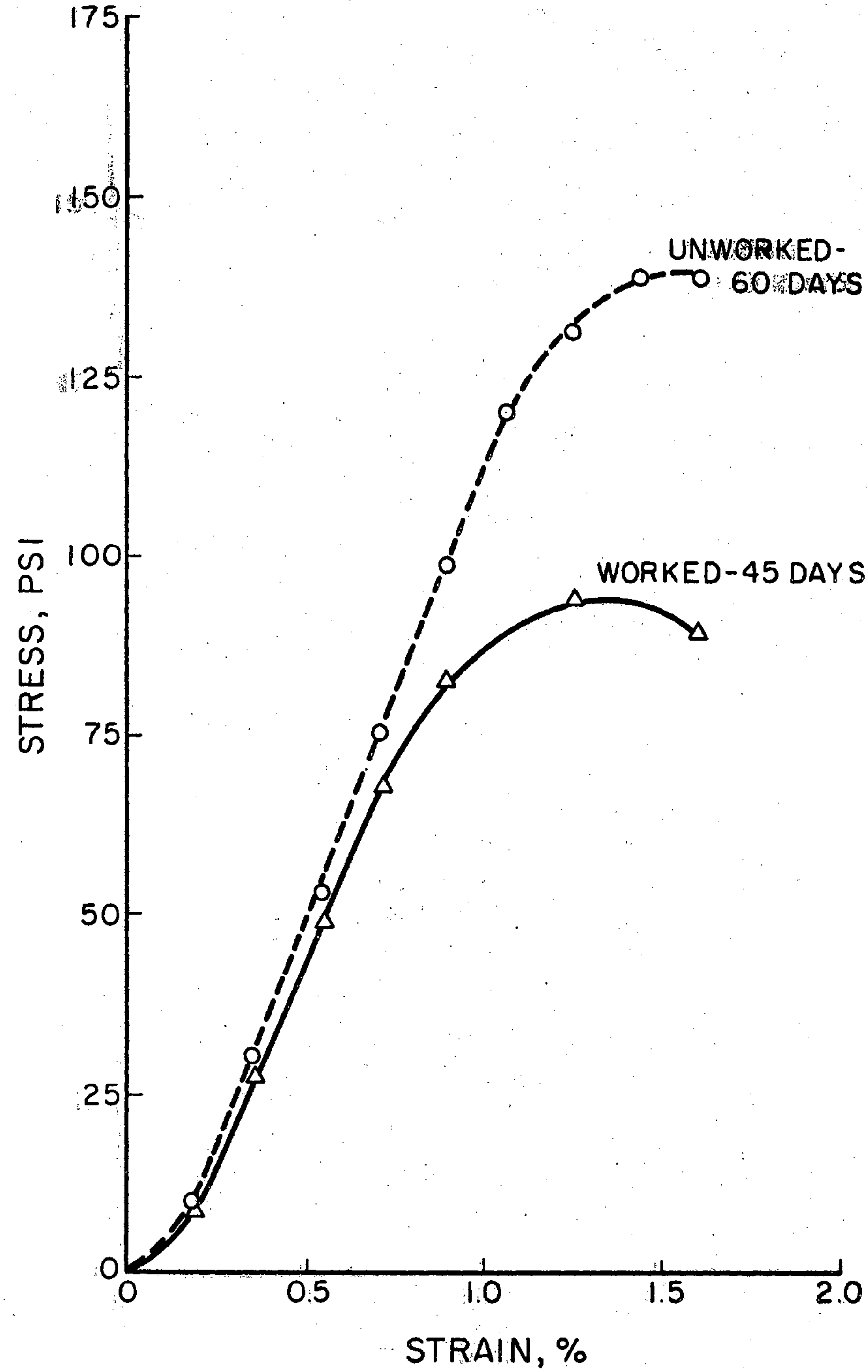


Figure 16. Stress-Strain Characteristics, Laterite + 5% Portland Cement, 45 and 60 Days Cure

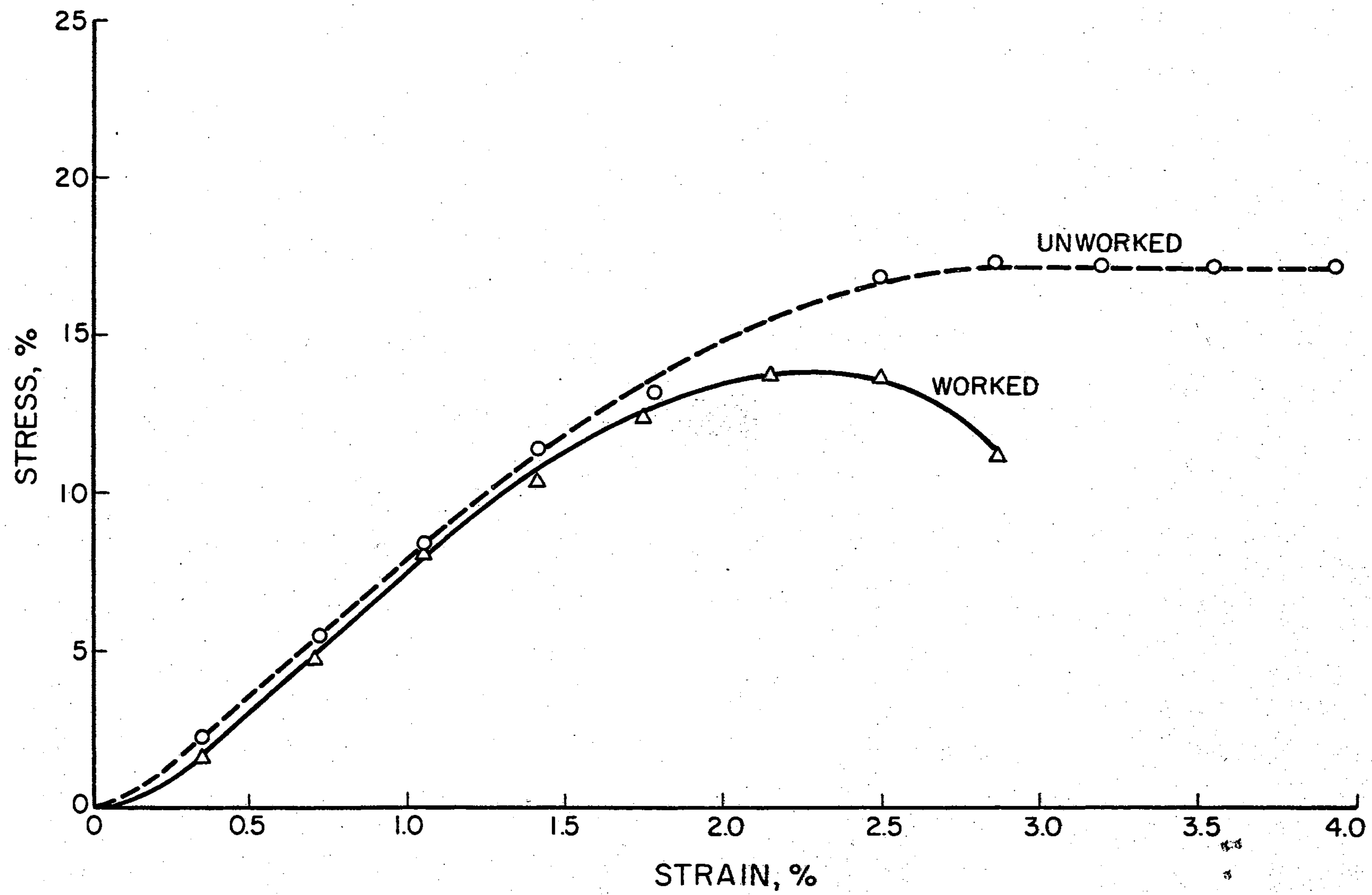


Figure 17. Stress-Strain Characteristics, Laterite + 6% MC-3 Cutback

# VITA

Frank Charles Townsend

Candidate for the degree of

Master of Science

Thesis: ENGINEERING CHARACTERISTICS OF WORKED PANAMANIAN LATERITIC SOIL

Major Field: Civil Engineering

Biographical:

Personal Data: Born November 24, 1940, at Panama City, Republic of Panama, the son of Wesley and Virginia Townsend.

Education: Attended elementary school in Balboa, Canal Zone; graduated from Balboa High School, Balboa, Canal Zone in 1958; received a Bachelor of Science in Civil Engineering degree from Michigan College of Mining and Technology, Houghton, Michigan in June, 1962; completed requirements for the Master of Science degree in May, 1968.

Professional Experience: Surveying assistant for Section of Surveys, Panama Canal Company, Balboa, Canal Zone, the summers of 1960 and 1961; Highway Engineer (Trainee), Bureau of Public Roads, Arlington, Virginia, from August, 1962 to November, 1962; First Lieutenant, Corps of Engineers, U. S. Army, from November, 1962, to February, 1966; Highway Engineer (Trainee), Bureau of Public Roads, Kingman, Arizona, from June, 1966 to August, 1966; Graduate Assistant in the Civil Engineering Department, at Oklahoma State University, from September, 1966 to date.

Professional Societies: Associate Member American Society of Civil Engineers.