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COLLAPSIBLE SOILS IN LOUISIANA

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State Project No. 736-01-25

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(Suspended Moisture in Silts)

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LOUISIANA STATE UNIVERSITY
DIVISION OF ENGINEERING RESEARCH
for the
LOUISIANA DEPARTMENT OF HIGHWAYS
in Cooperation with the
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I. ABSTRACT

In southwest Louisiana, some surface silts (up to 3 feet deep) collapse under load after moisture is added. These soils were indistinguishable from nearby normal silts through routine tests. The deposits occur as low, gently-sloping ridges. Very similar collapsible soils were found in the well-known pimple mounds of the area. In the stable silts, vermiculite is the dominant clay mineral; in the collapsible silts, montmorillonite. Kaolinite and illite are also present. Moisture is held in the pores of collapsible silts by electrochemical forces. When collapsible silts were remolded with water or ethylene glycol (both polar liquids), no water passed through under 2 meters of head.

In settled mixtures of collapsible silts and a solution of Calgon, (commercial brand of Sodium Hexameta phosphate) the supernatant liquid was black. The color change is attributed to lignins. Four criteria were established for identifying collapsible silts: an in-place unit weight less than 80 lbs/cu ft; a maximum dry unit weight less than 104 lbs/cu ft; a black liquid after the soil solids have settled out in a 3% solution of Calgon; and a total strain of at least 15% in the collapse test (a modified consolidation test of an oven-dried, undisturbed sample that is then saturated under pressure to 16 tons/sq ft).

KEY WORDS: silts, organic silts, loess, soil stability, collapsible soils, test procedures, soil tests, soil stabilization, pimple mounds

II. SUMMARY

In southwest Louisiana, surface deposits of a gray collapsible silt have been encountered within a 30-mile wide band that extends from Lafayette both westward to Texas and southeastward to the Gulf of Mexico. A large irregular area surrounding this band, as well as parts of eastern Louisiana, may also contain deposits of this material.

In dry weather, the silt will support farm tractors, construction equipment, houses, and concrete slabs. However, when additional moisture is added from rain, it loses strength and, under pressure, will decrease in volume and flow like soft mud.

Construction of the Interstate Highway through these deposits entailed over 2.5 million dollars in cost overruns and many delays because the collapsible silt was indistinguishable from nearby stable silts of the area, through both visual observation and routine soil laboratory tests. When construction equipment bogged down after a rain, the unstable soil in the right-of-way had to be removed and backfilled with suitable material to ensure trouble-free pavement sub-bases and equipment traction.

Large-scale excavation and backfill procedures may be costly remedies for poor in-place soil problems. The laboratory studies reported in the following pages resulted in criteria for identifying collapsible silts in Louisiana and showed that convential lime or cement treatment will stabilize them.

III. IMPLEMENTATION OF RESEARCH FINDINGS

During recent construction of the interstate highway in southwest Louisiana, collapsible silts were responsible for delays and large cost overruns. These unstable soils--indistinguishable from the normal silts of the region, through either field observation or routine laboratory tests--would not support construction equipment after a rainfall. Such deposits had to be excavated and backfilled where necessary to ensure suitable pavement subbases and vehicle traction.

The results of the experiments reported in this bulletin may be used to prevent unexpected encounters with unstable silts through the application of four identification criteria that either are part of or may be incorporated in routine soil testing procedures. Two criteria require undisturbed samples (in-place unit weight determination and collapse test); the other two are suited to disturbed samples (maximum dry density and Calgon tests). Any two of the four tests will identify unstable silts in the study area.

New testing procedures, along with revisions to existing tests, are necessary. The Calgon test requires that the laboratory technician note the color of the supernatant liquid 24 hours after the soil/water/Calgon mixture has settled. The collapse test is a modified consolidation test of an oven-dried specimen. After a maximum load of 8 tons per square foot, the dry specimen is rebounded to 4 tons per square foot, then submerged and reloaded up to 16 tons per square foot for determining possible additional subsidence (collapse). The tests for in-place unit weight and maximum dry density are standard. Any collapse characteristics of a silt are indicated by a certain range of subnormal weights.

Application of the collapse criteria does not require new or special testing equipment.

If the necessary construction equipment can traverse the area, collapsible silts may be stabilized with Portland Cement (6 to 14%) or lime (1 to 4%). Standard compression, pH, and brush tests will indicate the optimum mixture.

The potential benefits resulting from the research include savings in costs and avoidance of delays. Deposits of unstable silt, when detected, may be either removed or stabilized. If excavation and backfill are chosen as a remedy, the area may be quickly delimited to prevent unnecessary removal of normal soils.

IV. BACKGROUND INFORMATION

Problem Statement

Over $2\frac{1}{2}$ million dollars have been spent on the removal of unstable surface silts in southwest Louisiana and the replacement of highway structures founded on such soils (Table 1).

Early reports from field engineers blamed moisture. Unstable silts which were too wet for construction equipment to traverse (i.e., they underwent large volume changes when subjected to loads) and could not be drained seemed to hold moisture suspended in the pores.

Deposits of these unstable silts exist in a band 30 miles wide from Lafayette westward to Texas and southeastward to the Gulf of Mexico (Figure 1). Areas of potentially unstable silt surround the known deposits and generally include, but are not limited to, the soil areas of the Coastal Prairies, Loessal Hills, and Mississippi Terraces (Figure 2). Potential and known areas of unstable silt were determined from the sampling program described herein, highway construction records, and engineering experience.

Unstable silts are massive (homogeneous) in structure and usually less than three feet thick. When dry they have sufficient shear strength to support tractors, construction equipment, houses and other similar structures. B. A. Touchet (1970), a Soil Conservation Service Geologist, gave the following description of these soils in the Lafayette, Louisiana, area:

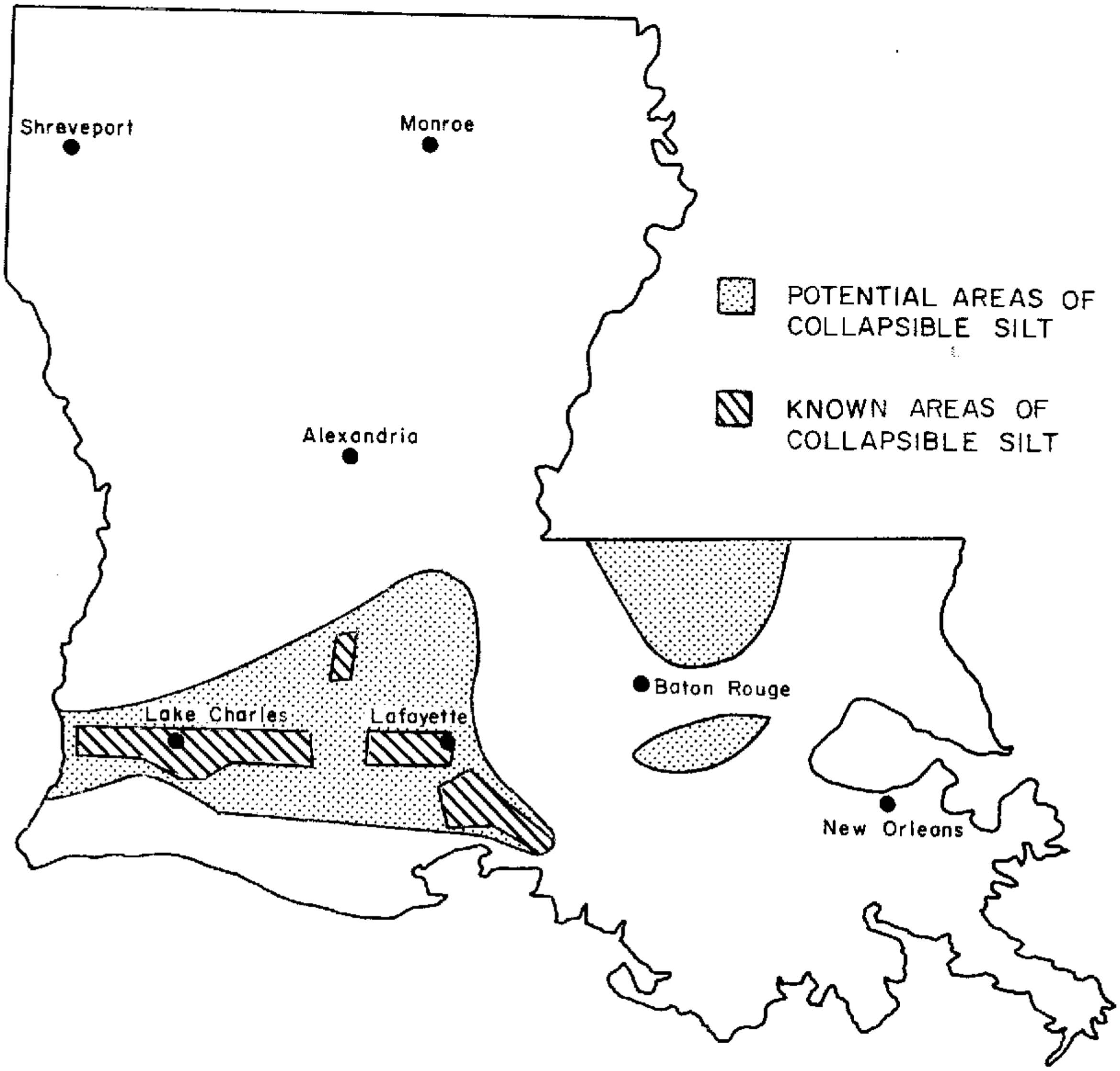
> The Crowley, Caddo, Calhoun, Mowata and other soils with weak engineering characteristics in the top 20 inches are problems when building low cost roads or when building houses or other structures. It has been noted that the thick A2 horizon-rated ML is very firm and even hard when dry and has a high bearing strength. Structures built on this ML material will stay stable until wetting occurs. When wet, this ML (silt), which is very porous, turns to liquid, flows, or collapses readily and the structure will

TABLE 1

CONSTRUCTION CONTRACTS FOR UNSTABLE SILT REPLACEMENT

Highway Route	Project No.	Highway Segment	Cu. Yd. Removed	Year	Cost*
I-10	450-02-31 450-02-29	Sulphur-Toomey	184,000	1965	$ 143,000
I-10		Iowa-Laccasine	30,000	1963	27,000
I-10		Laccasine-Jennings	71,000	1963	60,000
I-10	450-02-26 450- -05	Lake Charles-Iowa	105,000	1963	50,000
US 90	424-04-07	New Iberia-Jeanerette	8,000	1966	7,000
I-10	450-05-04	Duson-Lafayette	212,000	1966	569,000
I-10	450-05-06	Lafayette-Intersection La 3052	479,000	1967	1,783,000

*Cost includes price of material to backfill undercut areas



COLLAPSIBLE SILTS IN LOUISIANA

FIGURE 1

> shift. Excavating to the bearing soil (CL or CH) and back filling and compacting or reserving the dryness affords some solution for these conditions. These conditions are very noticeable on driveways by the cracks and shifts near the edge, and in homes where water gets under the house or encroaches against the foundation. Doors jam and walls crack when this occurs.

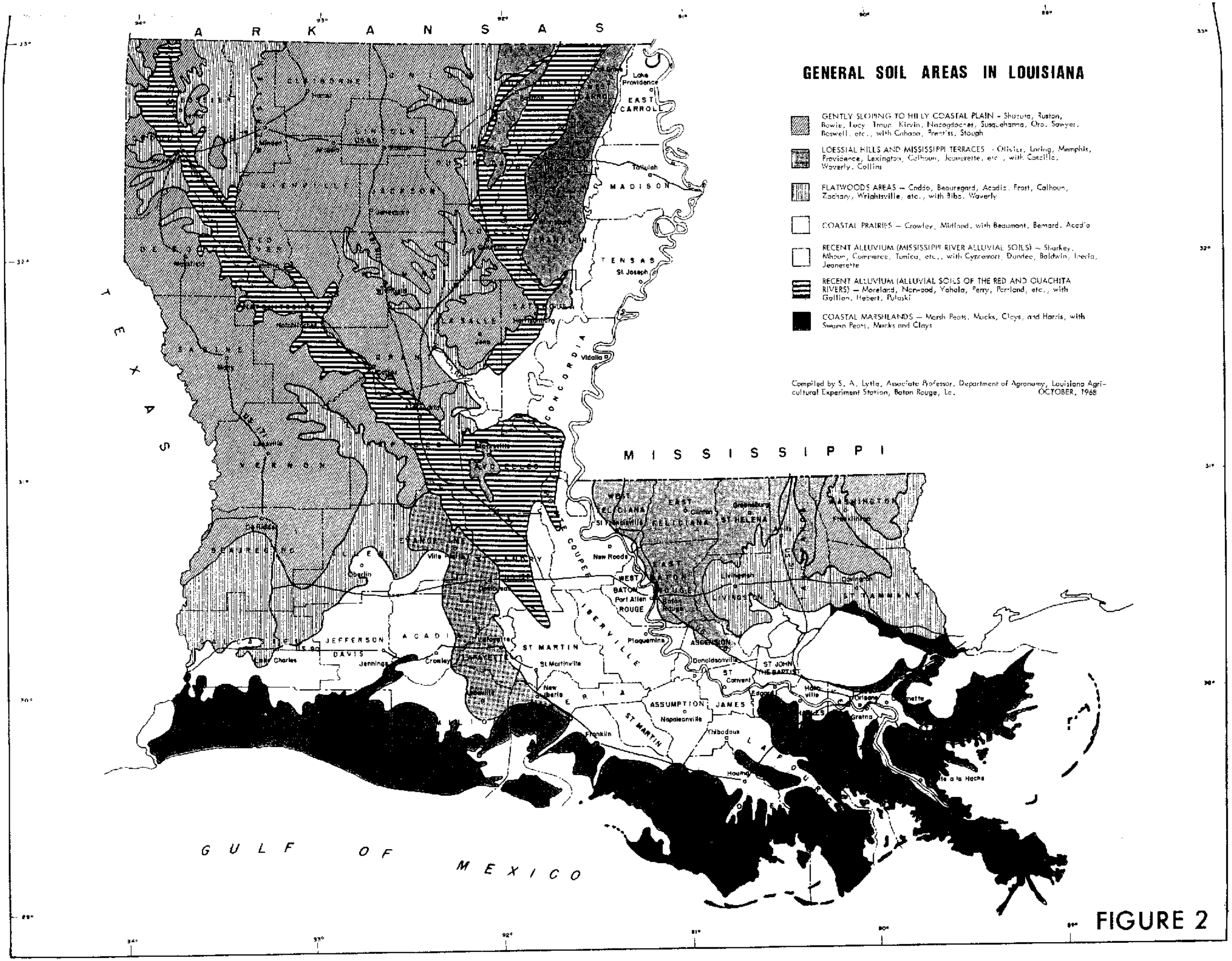
Many deposits of stable silts in southwest Louisiana are similar in appearance to the unstable ones and may be located within a few hundred feet, with their contacts frequently gradational. Neither the grain size or shape, structure, fossil content, nor Atterberg limits indicate the difference in behavior of the stable and unstable silts.

Inability to distinguish the unstable silts is a serious engineering problem. Structures resting on dry unstable soil (especially those with mat foundations) risk foundation failures due to differential settlement or loss of bearing capacity when the soil becomes saturated. Unidentified unstable silt causes cost overruns and delays in the completion of highway construction projects because it must be stabilized or replaced if construction equipment is unable to traverse the material. The practice of replacing all silts could be instituted, but the replacement of stable silt would only add unnecessary cost.

Loess in Mississippi

The unstable silt deposits are similar in structure and grain size to the loess deposits of Mississippi, and like them, also contain over 70% of silt-size particles, are massive in structure, and collapse under load in the presence of water. The fact that the silts occur downstream from the Mississippi loess deposits suggests that they may be either loess of similar origin or alluvially redeposited loess.

The Mississippi loess is composed primarily of silt-size grains of quartz (Snowden and Priddy, 1968, p. 139). Quartz comprises 65% of the soil volume. Montmorillonite is the dominant clay mineral, followed by illite and kaolinite in decreasing order of abundance (Snowden and Priddy, 1968, p. 139). The



GENERAL SOIL AREAS IN LOUISIANA

Compiled by S. A. Lytle, Associate Professor, Department of Agronomy, Louisiana Agricultural Experiment Station, Baton Rouge, La.
OCTOBER, 1968

FIGURE 2

specific gravity of the solids ranges between 2.66 and 2.73 (Krinitzsky and Turnbull, 1967, p. 46); porosity, between .43 and .54; and the plasticity index, between 2 and 12. The liquid limit is 24. Silt size-particles constitute 89% of the soil; the remaining 11% is clay (Krinitzsky and Turnbull, 1967, p. 24).

Definition of Soil Collapse

Soil collapse differs from consolidation, which is the subsidence resulting from loss of adsorbed water and reduction of voids caused by excess hydrostatic pressure. Collapse is a subsidence resulting from the addition of water or load. Most reported collapsible soils exist in a partially-saturated or dry condition because they usually are located in arid climates.

A collapsing soil is defined as one that undergoes an appreciable loss of volume upon wetting, load application, or both (Sultan, 1969, p. 1). Collapses are reported to be between 4 and 20 percent of the original soil height (Knight and Dehlen, 1963, p. 31). The magnitude of collapse depends on the moisture content, load, and nature of the collapsible soil. Soil collapse is not instantaneous, but may take several minutes or hours.

Collapse Criteria

The identification and prediction of collapse have proved elusive because no single criterion can be applied to all collapsible soils. Both routine and more sophisticated testing have failed to indicate the presence of collapsing soils.

Because low unit weights indicate a loose structure, the in-place dry unit weight is a good criterion for collapse prediction. For loess or loessial material, an in-place dry unit weight of 80 pounds per cubic foot (pcf) or less indicates a collapsible soil (Clevenger, 1956, p. 60). Unit weights between 80 and 90 pcf are transitional, while soils heavier than 90 pcf will settle very little. Holtz and Hilf (1961, p. 677) added the moisture content as a criterion. Soils with less than 10 percent water content have a high resistance to settlement. Water contents over 20 percent, however, permit full settlement to occur under load.

Denisov (Sultan, 1969, p. 6) introduced a coefficient of subsidence and a set of ranges for this coefficient which correlate with the degree of collapse:

$$\text{Coefficient of Subsidence } K = \frac{e_L}{e_o}$$

where e_L = void ratio at the liquid limit

e_o = natural void ratio

K value	Degree of Collapse
0.5 to 0.75	Highly collapsing soil
1.0	Non collapsing loams
1.5 to 2.0	Non collapsing soil

If the natural void ratio is higher than the void ratio at the liquid limit, the soil would collapse spontaneously upon saturation. Denisov's theory, however, did not account for the applied stress level or structure of the soil.

An interesting criterion incorporating a variation of the dry unit weight and water content, proposed by Gibbs and Bara (1962, p. 234), was successfully applied to the San Luis Canal in the San Joaquin Valley. The soil voids must be sufficient to contain the moisture of the soil at its liquid limit. This criterion applies only if the soil is uncemented, the liquid limit is applicable, and if the liquid limit is greater than twenty.

The 1962 Soviet Building Code predicts collapse for soils that are less than 60% saturated. According to this code (Sultan, 1969, p. 9), a value of λ greater than minus 0.1 indicates collapse, where

$$\lambda = \frac{e_o - e_L}{1 + e_o}$$

Several workers have attempted to identify collapsing soils from the consolidation test or a variation of it (Bull., 1964; Knight and Dehlen, 1963, p. 33). Milovic (1969, p. 4), using a variation of the normal test, suggests the concept of a specific coefficient of settlement, which he defines by the equation

$$i_m = \frac{e_n - e'_n}{1 + e_n} = \frac{\Delta e_n}{1 + e_n}$$

where e_n = void ratio corresponding to the natural water content at vertical stress σ_n

e'_n = void ratio corresponding to the increased water content at the same vertical stress σ_n

Δe = change in void ratio after wetting at the same vertical stress σ_n

Subsidence due to the addition of water can be calculated by multiplying the coefficient of settlement by the thickness of the soil layer.

Use of the double consolidometer was suggested by Knight and Dehlen (1963, p. 33). One sample is tested at its natural moisture content while the other is run submerged in the confining ring. The test has the advantage of indicating the magnitude of collapse over a wide range of soil stresses.

Feda (1966, p. 216) proposed a subsidence index for predicting collapse in partially saturated soils. This index (K_L) is a modification of the liquidity index

$$K_L = \frac{(w_n/S_n) - w_p}{I_p}$$

where w_n = natural moisture content (%)

w_p = Plastic Limit (%)

S_n = natural degree of saturation (%)

I_p = Plasticity Index (%)

Soils with a subsidence index greater than 0.85 are subsident. Those which are subsident and also have a natural saturation less than 60 percent would be expected to collapse. Feda's criterion applies only to plastic soils because the Plasticity Index appears in the denominator. Since many loessial and granular soils are nonplastic, Feda suggested that they are subject to collapse if their natural porosity exceeds 40 percent (1966, p. 213).

In 1961, Gibbs developed a new criterion, the collapse ratio, that relates the water content at saturation to the liquid limit (Sultan, 1969, p. 10).

$$R = \frac{w_{sat}}{w_L}$$

or

$$R = \frac{(\gamma_w / \gamma_d) - (1/G_s)}{w_L}$$

where R = collapse ratio

w_{sat} = water content at 100% saturation

w_L = liquid limit

γ_w = unit weight of water

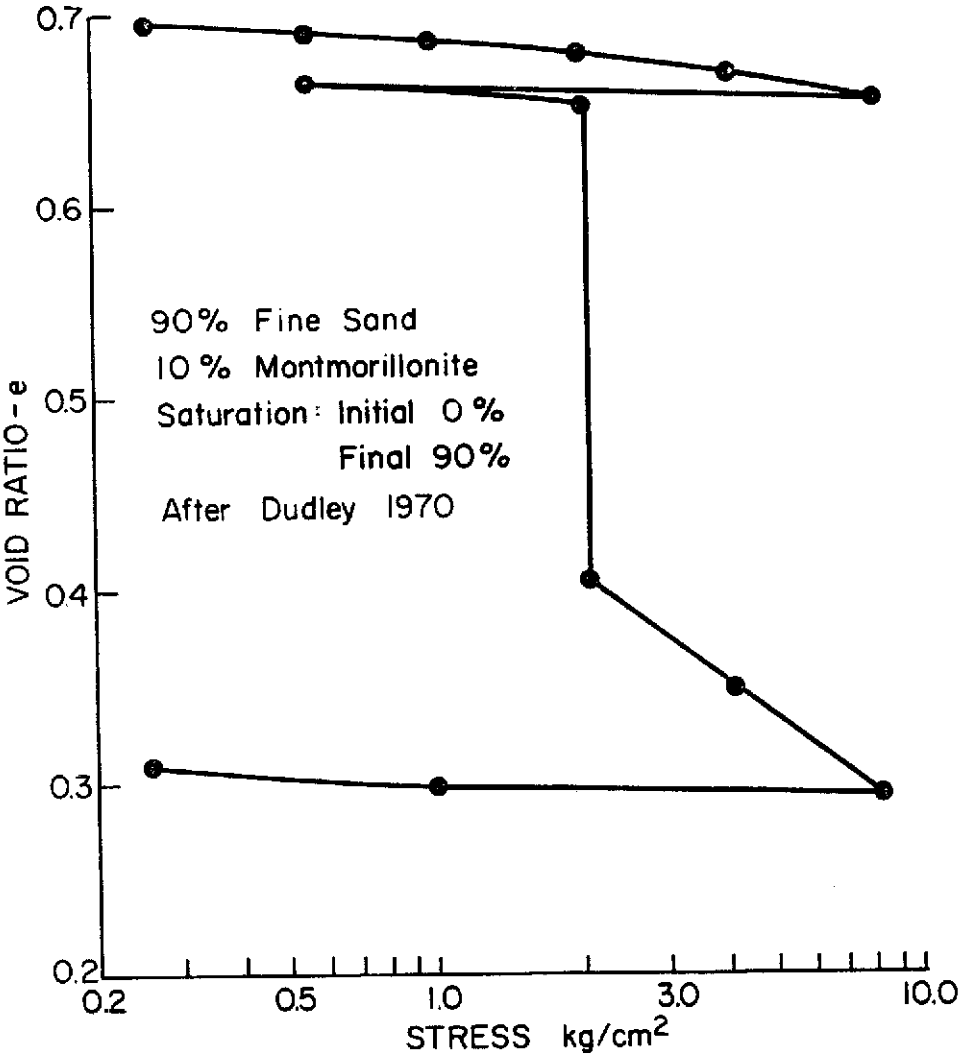
γ_d = dry unit weight of soil

G_s = specific gravity of solids

A soil with a collapse ratio greater than unity will be near a liquid state when saturated and therefore subject to collapse. The R values do not account for the type or nature of bond between the soil grains.

Kassiff and Henkin (1967, p. 16) proposed the product of dry density and moisture content ($\gamma_d \times w$) as a predictor of collapse for loess. When this product exceeds 15, large settlements may occur.

According to Dudley (1970, p. 927), the only requirements for collapsible soils are a loose structure and a moisture content less than full saturation. This criterion is of little use, however, because any soil above the water table might be classified as collapsing, depending on the interpretation of the term "loose." Dudley also presented a description of collapse with a pseudo consolidation test (Figure 3). In this test a sample was fabricated from 90% fine sand and 10% montmorillonite. After the sample has been loaded and rebounded in a dry state, it is flooded and the amount of collapse recorded. Dudley disagrees with the use of dry unit weight as a failure criterion stating (p. 928): "The range of dry unit weights where collapse has been experienced varies from 64



COLLAPSE IN A CONSOLIDATION TEST

FIGURE 3

pounds to 105 pounds per cubic foot. Where cobbles and larger particles are present, the unit weights run even higher."

The Collapse Mechanism

Addition of water as the triggering action is a widely-used explanation of collapse. Collapse may occur, however, as the result of load application, wetting, or both. It is important to remember that the relationship between soil stress and shear strength determines stability. Thus collapse can occur either by increasing the stress above the strength or by lowering the strength below the stress.

Partially-saturated soils have shear strength as the result of capillary forces, which are destroyed at full saturation (Holtz and Hilf, 1961, p. 678). In the well known Coulomb equation, shear strength is the sum of the apparent cohesion and a function of the effective grain-to-grain pressure:

$$\tau = c + \bar{\sigma} \tan \varphi$$

where τ = shear strength

c = apparent cohesion

$\bar{\sigma}$ = effective stress

φ = angle of internal friction

The grain-to-grain stress is determined in part by the capillary forces and is directly related to the effective stress. Effective stress is the total stress minus the pore water pressure.

$$\bar{\sigma} = \sigma_T - u$$

where σ_T = total average stress due to vertical loads

u = pore water pressure

Because the meniscus is developed by surface tension, the pore water pressure is negative in partially-saturated soil. Negative pore pressures increase the intergranular pressure, thereby increasing the shear strength. Saturation destroys the capillary forces, thus reducing the intergranular, or effective stress, which causes a loss of shear strength (Figure 4). If the shear strength reduction is

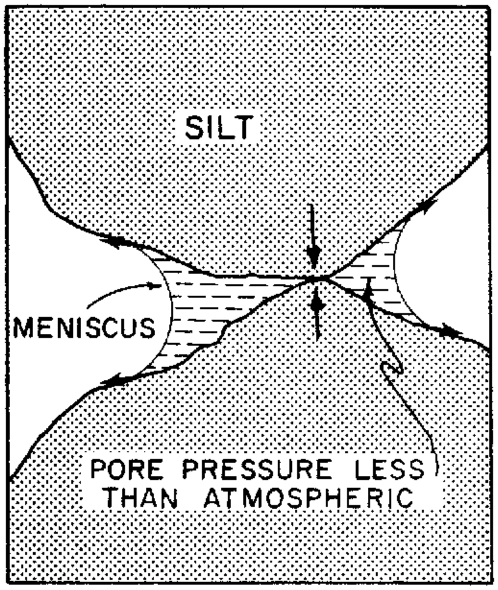


FIGURE 4 how capillarity increases the grain to grain stresses in a partially saturated soil.

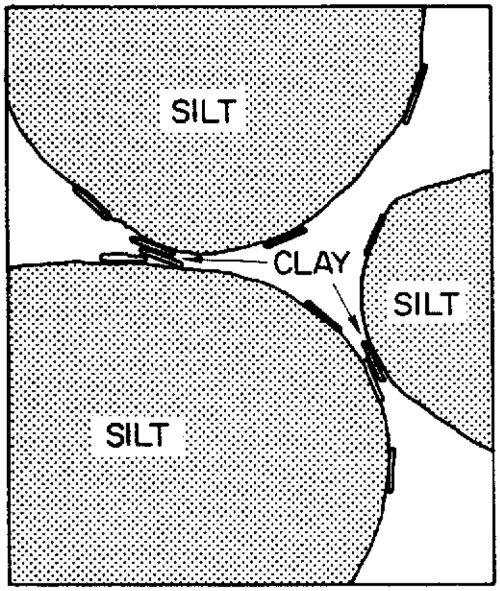


FIGURE 5 silt particles whose strength depends on the moisture content of the clay binder.

After Kane, 1969, p. 34.

lower than the applied stress, collapse will occur.

A hypothetical failure mechanism involving clay-covered silt particles is shown in Figure 5. The strength of this soil depends on the amount of the clay binder (Kane, 1969, p. 34). At a given binder content, the strength of the clay depends on its water content. When water is added to the dry soil, the clay becomes more plastic and loses strength.

Soil particles supported by clay minerals and associated ions (Figure 6) may undergo collapse when saturated. Buttresses of clay-size material flocculated by ions may form around silt particles as the soil dries (Dudley, 1970, pp. 937-8). Capillary forces may add to the strength. The addition of water, besides destroying the capillary forces, will reduce the ion concentrations, which will, in turn, increase the repulsive forces between particles and cause a dispersion of the supporting buttresses.

Flocculated clay particles may form buttresses which separate the silt particles (Figure 7). Lang (1969, p. 87), while working with a mixture of 94% foundry sand and 6% montmorillonite, found "a pronounced tendency for individual platelets to stand on edge between sand grains and combine in ribbon-like strands which fill the space between adjacent sand grains." He illustrated this with a scanning-electron micrograph (Figure 8) to show parallel-oriented montmorillonite platelets (shown schematically in Figure 7). Saturation of the clay buttresses in this case would cause a "mud flow" type of failure.

Remedies Against Soil Collapse

If the unstable deposit is not deep, either removal of the collapsing soil or its compaction is the best solution (Holtz and Hilf, 1961, p. 679). In the description of the failure of a road constructed on a collapsible soil, Knight and Dehlen (1963, p. 33) recommend compacting the wetted soil with vibrating rollers.

If the deposits are deep, the use of piles and caissons is the best solution. Displacement piles, such as timber or concrete piles, driven in pre-wetted soil,

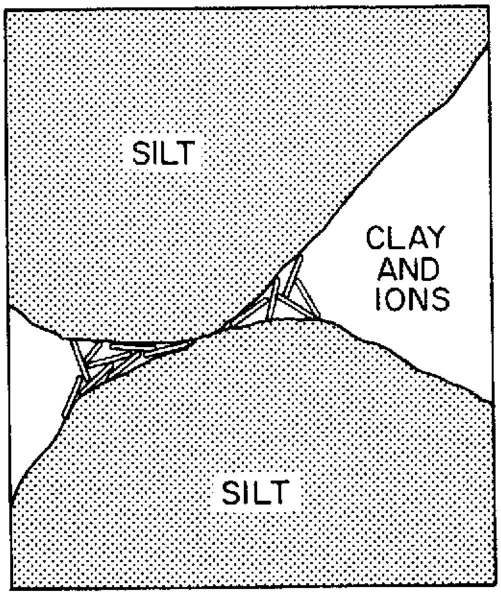


FIGURE 6

soil particles supported by ring buttresses.

After Dudley, 1970, p. 937.

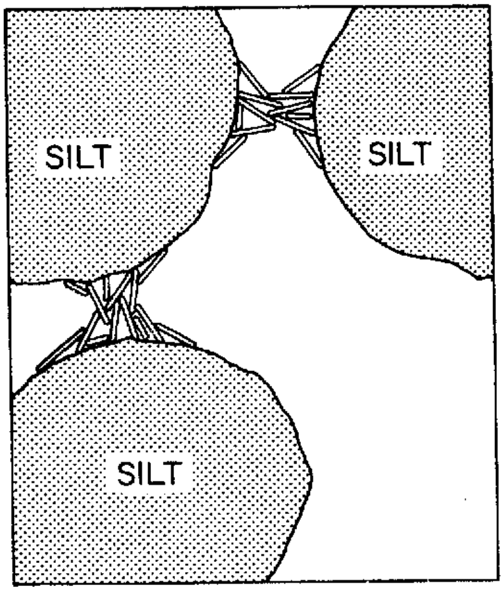


FIGURE 7

silt particles separated by flocculated clay buttresses.

After Lang, 1969, p. 87.

Figure 8

Sand Grains Separated By Parallel-Oriented Montmorillonite Platelets (After Lang, 1969) offer the advantage of compacting the loose soil as they are driven, making the subsoil more stable (Holtz and Hilf, 1961, p. 679). Clevenger (1956, p. 24) recommended cast-in-place concrete piers in bedrock for Denver area residences located over a 5 to 6 foot layer of collapsible loess.

Peck and Ireland (1956, p. 173) recommend footings or a raft foundation for structures where the subsoil can be kept dry:

> Because the load settlement curves for (dry) loess often exhibit a fairly well-defined break marking the pressure at which the open structure of the soil begins to crush or collapse, the designer may choose to restrict the footing pressure to a value well below the break in the curve so that the structure will experience minor settlements. However, he may choose a high value and accept larger settlements. The latter alternative is feasible because there is little likelihood of a complete bearing-capacity failure by rupture in the usual sense. The second alternative has often been used by constructors of grain elevators, whereas the first alternative is more suitable in connection with conventional structures such as school buildings.

Compaction of the soil by surcharge or prewetting, or a combination of both, has been most successful for large hydraulic structures. Medicine Creek Dam, a 2,750,000 cubic yard rolled-earth structure which is 5,665 feet long and about 115 feet above the stream bed, was built after prewetting and surcharging the base (Clevenger, 1956, p. 14). Dry loess of 40-foot average depth existed where the structure was to be built. Prewetting caused much of the settlement to occur while the structure was being built instead of after construction and reservoir filling. Field ponding was used to compact collapsing soils along the San Luis Canal in the San Joaquin Valley, California (Gibbs and Bara, 1962, p. 231).

In addition to the more usual treatments of collapsing soils, Johnson (1953) injected silt slurry into the collapsing loess of Nebraska. He reported successful stabilization of canals and the reduction of settlement.

Formation and Distribution of Collapsible Soils

Collapsible soils are found on almost every continent. Sultan (1969, p. 10) cited the USSR, Bulgaria, Israel, Czechoslovakia, Argentina, South Africa, Hungary, Rumania, and the United States. Dudley (1970, p. 918) reported collapsing soils in several other parts of Africa as well as Australia, India, and other countries where loess is found. Figure 9 is a map showing collapsible soils in the conterminous United States.

Deposits of collapsible soil have been formed by aeolian (wind blown), alluvial, or weathering processes. Regardless of the process, most collapsible soils are geologically young.

Aeolian soils may be loose sand, like dunes, but more often they are loessial silt. The term loess represents a megascopically massive silt deposit of wind-blown origin. The soil particles in loess are loosely bonded by clay or calcareous material that gives it a low density. Saturation or flooding of loess weakens the bonds and reduces its strength. Increases in applied stress may also break the cementing bonds.

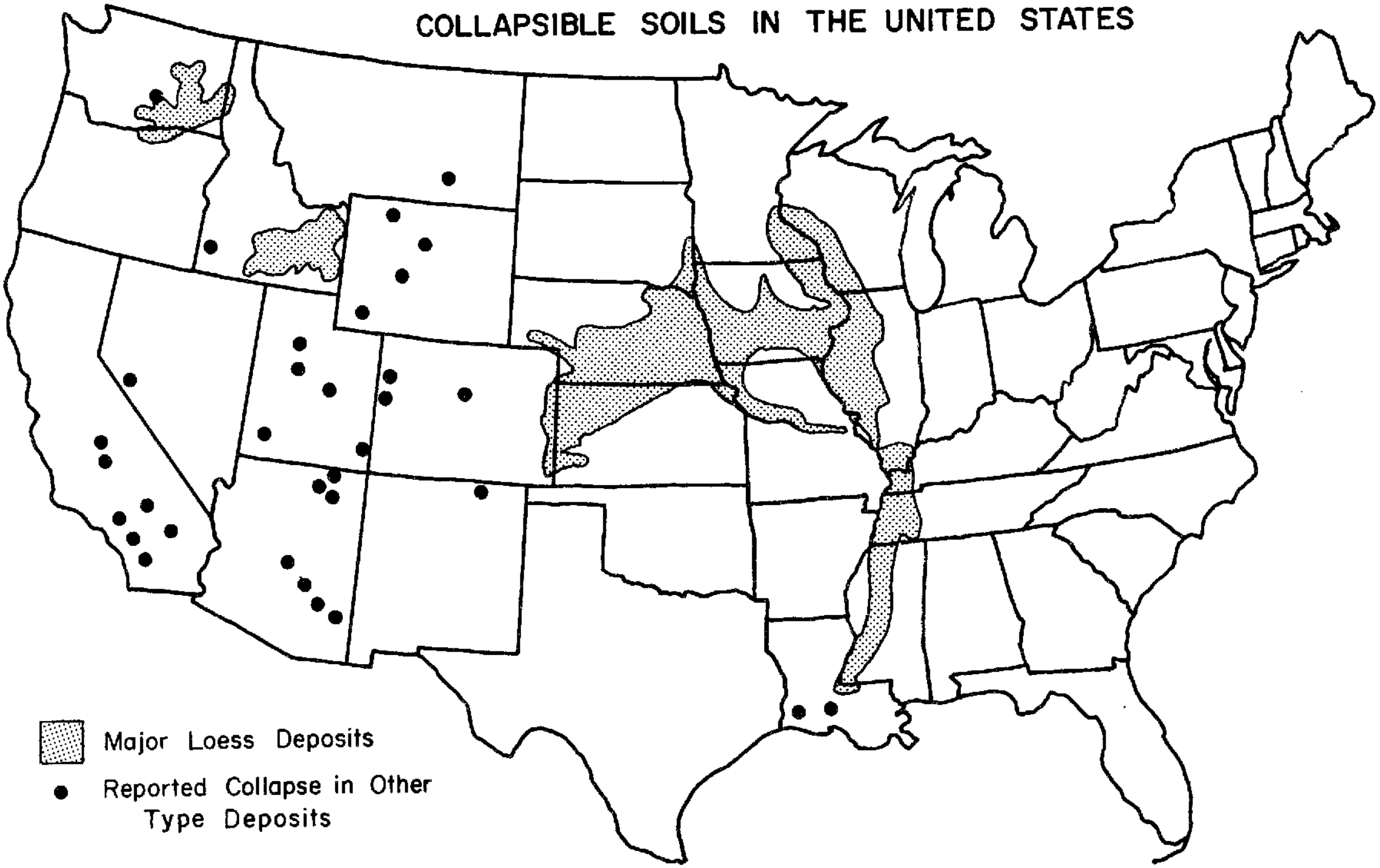
Collapsible soils of alluvial origin include flood plain deposits, alluvial fans, and mudflows. A major part of the flood plain deposits are the "loess-like" deposits, which are sometimes mistaken for loess. The major difference between loess and loess-like deposits is the sand content, which rarely exceeds 3% in loess, but which is usually over 5% in alluvial silt. A high sand content is not found in loess because the wind energy needed to transport sand is much greater than that for silt.

Residual soils, formed from the in-place weathering of rock, have also been reported to collapse. Sultan (1969, p. 2) cites weathered granitic soils in South Africa and decomposed gneiss in India.

Formation and Distribution of Collapsible Soils In Louisiana

Known areas of collapsible silt in Louisiana (shown in Figure 1) are found within a 30-mile band extending from Sulphur to Lafayette including the parishes (counties) of Calcasieu, Jefferson Davis, Acadia, Lafayette, and St. Martin. Slopes are gentle in the area. Rainfall, which averages 60 inches per year, runs off to the Gulf.

Of the general soil types in Louisiana (Figure 2), flatwoods, coastal



COLLAPSIBLE SOILS IN THE UNITED STATES

After Dudley, 1970

FIGURE 9

prairies, and loessial hills are the types found in the study area. The loess of the Lafayette - New Iberia area is of special interest because the term loess is almost synonymous with collapsible soil. Daniel and Young (1968) rejected an alluvial origin for the silts in that area for the following reasons: (1) the silty deposits occur on the highest parts of the landscape and on the terraces; (2) the silty deposit thins in relation to the distance from an excarpment which marks its eastern boundary; (3) it mantles the underlying topography; and (4) there is an absence of bedded sand and clay lenses.

Geologists generally have attributed the surface deposits in Southwest Louisiana to alluvial origin. Howe and Moresi (1933, p. 17) describe Acadia and Jefferson Davis Parishes as broad, flat alluvial terraces.

The area between Lafayette and Lake Charles is marked with meander scars of the ancestral Mississippi and Red Rivers. Red-colored sediments, similar to those found in the headwaters of the Red River, can be seen along ditch cuts. The curving, intertwining low ridges in the area, along with remnant oxbow woods and swamps, are slowly becoming obscured by agricultural activity.

Calcasieu Parish is dotted with "pimple mounds," which are low, round hummocks of silt from 20 to 85 feet across and 1 to 3 feet high height. Their soil structure is also thought to be unstable. Many theories have been proposed for their formation, including whirlwinds, Indian mounds, ant hills, fish nests, sand dunes, and accumulated deposits in holes in stagnant ice (Holland, et. al., 1952, p. 51). The same authors (p. 60) suggest: "the following combination of factors is responsible for the formation of pimple mounds: (1) a sandy or silty soil with a low percentage of collodial clay, (2) an initial surface of very low relief, (3) sufficient rainfall to cause erosion, and (4) some type of vegetation peculiar to the pimple mound areas." It is interesting to note that the B soil horizon in areas where pimple mounds are found contains calcareous nodules similar to those reported by Snowden and Priddy (1968, p. 139) in the Mississippi loess.

V. TESTING PROGRAM AND TEST RESULTS

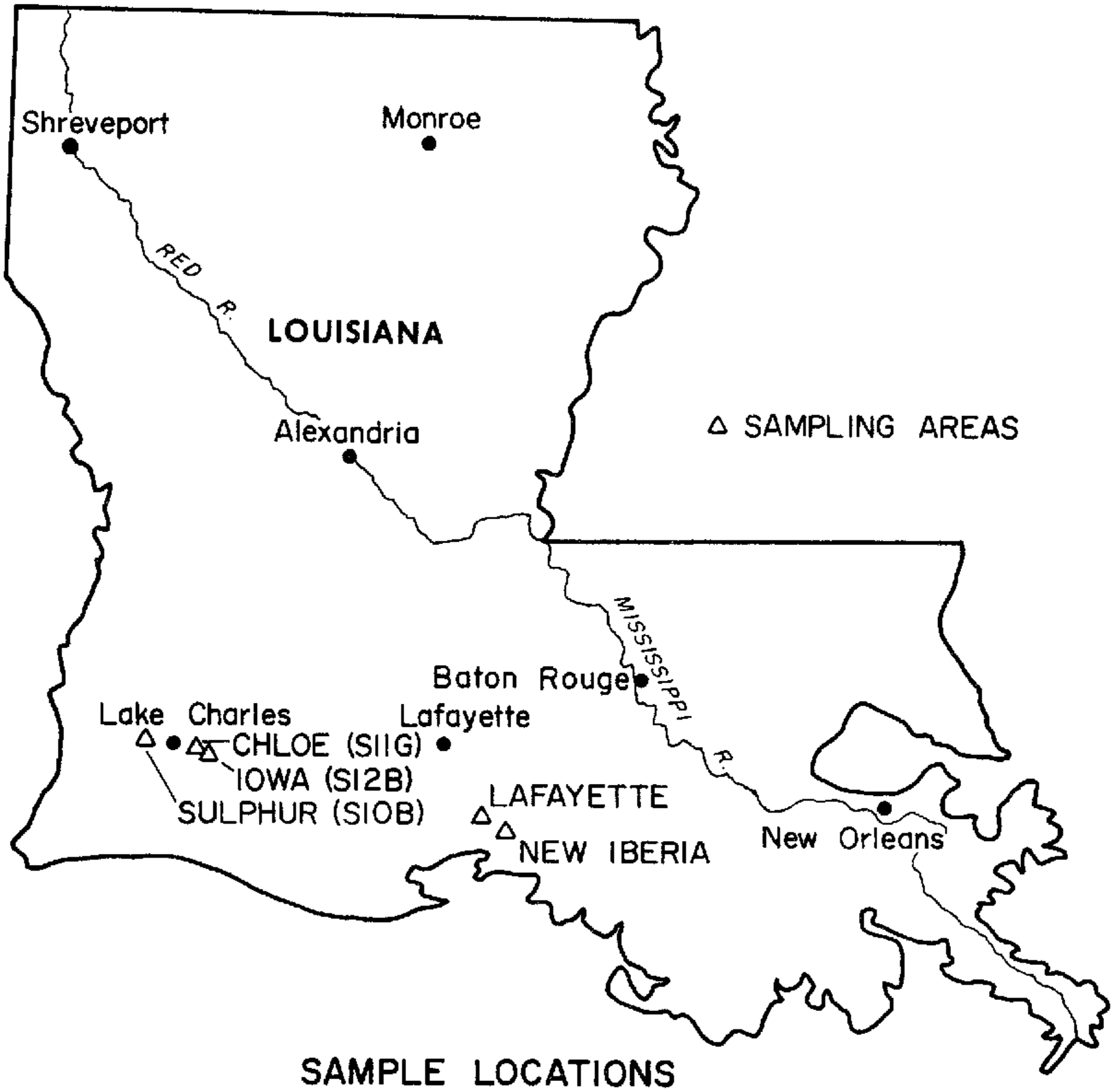
General Description of Tests

Initially, three sites in Calcasieu Parish, Louisiana, were chosen for sampling and testing. Two sites contained unstable silts; the other, stable silt. The unstable silts from Sulphur and Iowa (Figure 10) were given the designation S10B and S12B, respectively. (The suffix B stands for "bad.") The stable silt from Chloe (Figure 10) was designated S11G (G denoting "good.")

Because the nature of the problem was not known, disturbed samples of S10B, S11G and S12B were tested for classification, permeability, and maximum density. Sample preparation followed the ASTM D2217 procedure. Drying temperatures were held below 60°C to prevent major changes in the chemical properties of any organic matter. The liquid and plastic limits were determined according to ASTM D 423 and D 424. Grain size was determined by the ASTM D 442 procedure. Permeability tests were made in a falling-head permeameter with a maximum head of 2 meters. ASTM D 698 was followed in performing the standard compaction test.

Two special tests, the surcharge compression test and Calgon test, were developed and applied to the samples. (Calgon is a commercial brand of sodium hexametaphosphate.) The first is a test to find the maximum density under an assumed overburden pressure. The loose, dry material was placed in a 2½-inch round, Teflon-coated aluminum cylinder between two porous stones (Figure 11). Pressures varying between 300 and 1,600 lbs per sq ft were applied through a piston which fits inside the cylinder. The samples were submerged under pressure in water for three days, after which the dry weight density was determined.

The Calgon test was developed because of the black color of the liquid produced by samples S10B or S12B in the solution of Calgon used for hydrometer analyses. (The color of sample S11G was brown after the soil particles settled from suspension.) About 12 hours were required to achieve color stability.



SAMPLE LOCATIONS

FIGURE 10

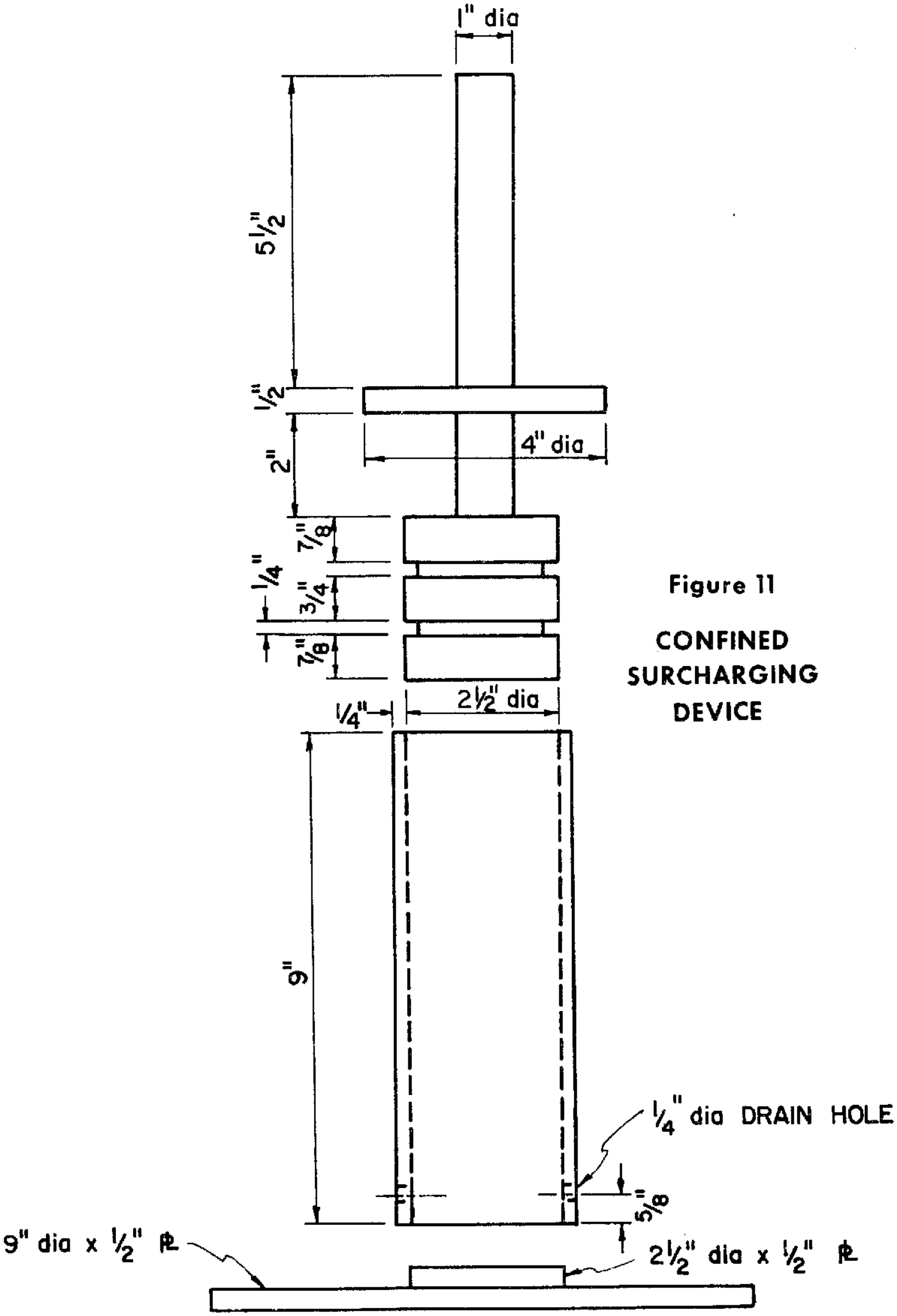


Figure 11

CONFINED SURCHARGING DEVICE

Both electron microscopy and X-ray diffraction were used to determine particle shapes and types of clay minerals. (A scanning electron microscope, JEOL Model JSM2 and a transmission electron microscope, Phillips Model EM100C, along with a Norelco high-angle goniometer, equipped with a proportional counter and a copper target X-ray tube operated at 40 kilovolts and 20 milliamperes, were used.)

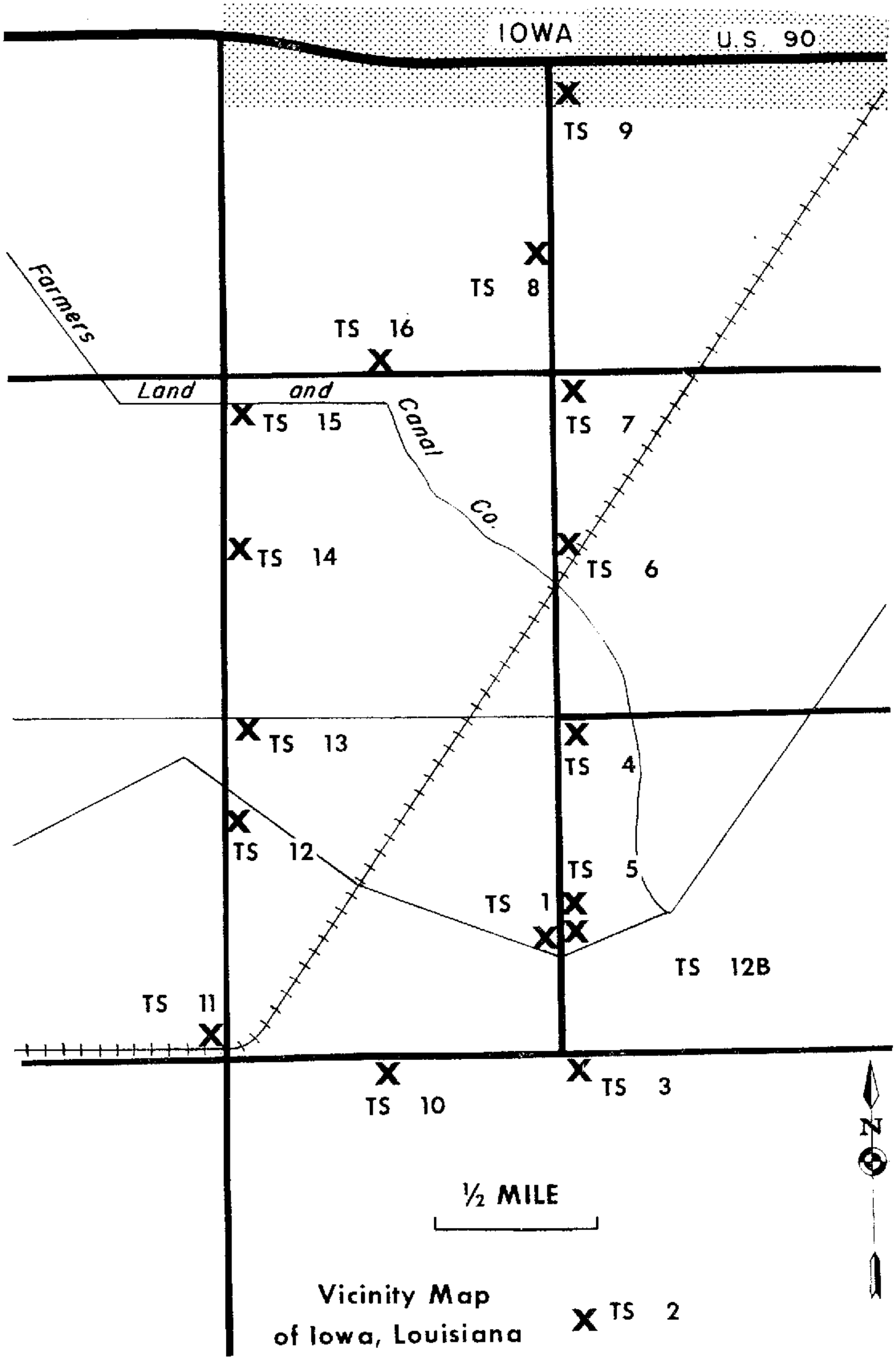
Differential thermal analyses (DTA) and specific gravity determinations were also performed on samples S10B, S11G, and S12B. The DTA's were run with equipment manufactured by Robert L. Stone Co. In-place unit weights of the undisturbed soils were determined with a nuclear density device in conformance with Louisiana Highway Department Standard Procedure TR 401 (Appendix A).

Undisturbed samples were obtained with a 3-inch diameter Shelby tube driven by a tractor-mounted hydraulic ram. The Shelby tubes were temporarily sealed with plastic bags in the field and then capped with paraffin after they were returned to the laboratory. All undisturbed samples were shipped and stored in Shelby tubes until ready for testing.

A direct shear test was conducted on undisturbed samples of S12B to determine its cohesion and angle of internal friction.

A special collapse test was performed on undisturbed samples to determine the subsidence characteristics of the soil, both when dry and submerged. This test was run in accordance with ASTM D2435 (Consolidation Test) with the following exceptions: A sample was first oven-dried, loaded dry in a load increment ratio of two from 1 to 8 tons per sq ft, rebounded to 4 tons per sq ft, and then flooded with water. Dudley (1970, p. 927) ran a similar test on a fabricated sample (Figure 3).

A soil survey was made to determine the extent of the unstable S12B (Iowa) deposit. Disturbed and undisturbed samples were taken at about ½-mile intervals along a north-south centerline throughout the area (Figure 12). Standard compaction, in-place density, grain size, Calgon, and liquid limit tests were run on



SAMPLE LOCATIONS

Figure 12

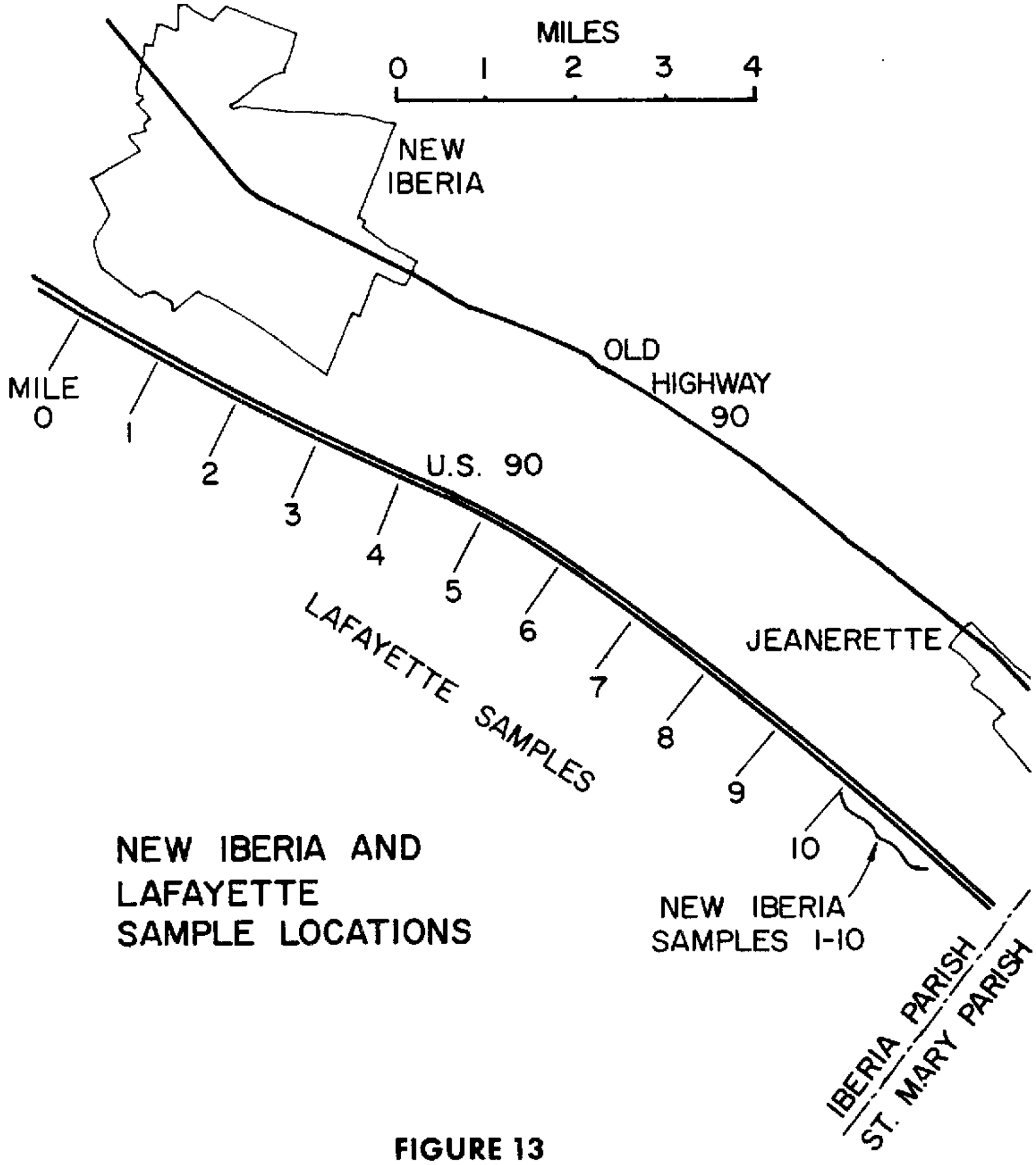
these survey samples.

Soil profile samples were collected at varying depths down to 5 feet and subjected to chemical and mechanical tests to determine if a soil binder had been leached out and redeposited in the underlying soil. Unstable samples S12B, S10B, TS7, and TS17, and stable silt S11G were included in these tests, which included liquid and plastic limits, grain size, pH, organic content, and weak-acid extractable phosporous, potassium, calcium, and magnesium. The acid extraction was as described in Bulletin, No. 632, "Analytical Methods and Procedures Used in the Soil Testing Laboratory" (Brupbacher et. al., 1968). (It should be noted here that the chemical test for organic content gives results which are one third less than those given by the combustion test.)

Highway department engineers from the Lafayette district furnished ten disturbed samples from a section of U. S. Highway 90 at one mile intervals where construction difficulties were encountered in 1966 (Figure 13). Standard compaction and Calgon tests were performed on the disturbed samples. The in-place unit weight tests were run in the field at a later date.

Samples were also taken at 1,000-foot intervals along the centerline of a segment of U. S. Highway 90 that was under construction south of New Iberia, Louisiana, and adjacent to the Lafayette sampling sites (Figure 13). Standard compaction, in-place unit weight, grain size, collapse, and Calgon tests were performed on the New Iberia samples.

A "Calgon-Washed" test was developed to determine if the material that turns black in the Calgon solution is responsible for the instability. In this test, the loose soil is mixed with dry Calgon (3% by weight of soil), washed in 3 ml of distilled water per gram of soil, and then centrifuged. The soil is again rinsed with distilled water and centrifuged at 1850 rpm for 15 minutes two more times to ensure removal of all Calgon and dissolved solids. No visible suspended solids or clays were lost in the washing operation. Calgon-washed tests were performed on S10B, S11G, S12B, Lafayette and New Iberia samples.



NEW IBERIA AND LAFAYETTE SAMPLE LOCATIONS

FIGURE 13

The pimple mounds (Figure 29) were sampled and tested because they also occur in the same region as the unstable silts. Results of the pimple mound tests are included in Appendix B because the samples had 24 to 42% sand content, which is much more than that of the silt samples (sand content 1 to 20%).

The stabilizing effects of cement and lime on the collapsible sample S12B and stable sample S11G were evaluated by laboratory tests. Silt and cement mixtures, prepared and tested in accordance with procedures recommended by the Portland Cement Association (PCA, 1959, p. 27), were subjected to unconfined compression and durability tests. All soil-cement samples were molded in accordance with ASTM D559. Compressive tests were run after samples had been cured for 7 to 14 days. Durability, as measured by the brush test, was tested in accordance with ASTM D559.

The liquid limit, plastic limit, grain size, pH, compressive strength, and standard compaction tests were also made on lime-treated samples. Grain size, pH, and compressive-strength testing procedures are included in Appendix A. Liquid limit, plastic limit and standard compaction tests were performed according to the ASTM standards.

Permeability Tests

Samples averaging ten centimeters in length were placed in a loose, dry state in a Lucite cylinder between porous carborundum stones. Water was permitted to flow vertically through the sample until the sample was saturated. Only after saturation were any readings taken. The initial permeability tests were inconclusive because after the test was repeated 24 hours later, the permeability was less.

Due to the erratic permeability test results, a drainage test was devised. Samples were prepared at moisture contents above their liquid limit, placed in Lucite cylinders over porous carborundum stones, and allowed to drain. The samples, Lucite containers, and stones were weighed each day to determine the weight loss. Less than 17% (.3 lbs.) of the water drained or evaporated from any sample within a week (Table 2). This same drainage test was tried with samples used for the first permeability tests. After the permeability tests were completed, the samples were allowed to drain freely for one week. Virtually no water was lost from these specimens.

The permeability tests, when repeated, showed that the permeability of silts decreased with time. A linear relationship exists between log time and log permeability (Figure 14). The equation is of the general form

$$K = At^{-0.7}$$

where K = permeability (cm/sec)

A = permeability at $t = 1$

t = lapsed time (days)

Some reduction in sample height took place as a result of the 2-meter head, which provided seepage force. The height reduction was 0.7 cm in 10.7 cm for sample S12B, 0.3 cm in 8.9 cm for sample S11G, and 0.3 cm in 7.8 cm for sample S10B. When the permeability samples were removed from the cylinders, remolded, and returned to the permeameters, the permeability was reduced by a factor of 20 (from 6.2×10^{-4} to 3.3×10^{-5} cm/min) for S12B, 10 (1.6×10^{-4} to 1.6×10^{-5} cm/min) for S11G, and 4 (9.7×10^{-5} to 2.8×10^{-5} cm/min) for S10B.

Permeability tests on sample S12B with polar and nonpolar liquids indicated that water, a polar liquid, is somehow attracted by and held to the soil. Sample S12B was saturated with carbon tetrachloride, a nonpolar liquid, mixed with a mechanical mixer, and then placed under a 2-meter head of carbon tetrachloride. The permeability from this test was 5×10^{-4} cm per second. (Only one test was run with carbon tetrachloride because of toxic vapors.) Sample S12B was also mixed with ethylene glycol, an extremely polar liquid, and placed under 2-meters head of water. After one hour, no flow had taken place and the porous stone below the sample remained dry.

TABLE 2

DRAINAGE TEST

(Wt. of Soil and Cylinder)

Days Drained	Moisture Above Liquid Limit		Samples Saturated From Permeability Test	
	S12B w=24%	S10B w=42%	S12B	S10B
0	7.00	6.53	4.89	4.89
1	6.93	6.47	4.89	4.89
2			4.89	4.89
3			4.88	4.88
4	6.81	6.39	4.88	4.88
5	6.77	6.36	4.88	4.87
6	6.74	6.33	4.88	4.87
7	6.70	6.30	4.88	4.86

$$w = \frac{\text{wt. of water}}{\text{wt. of soil}}$$

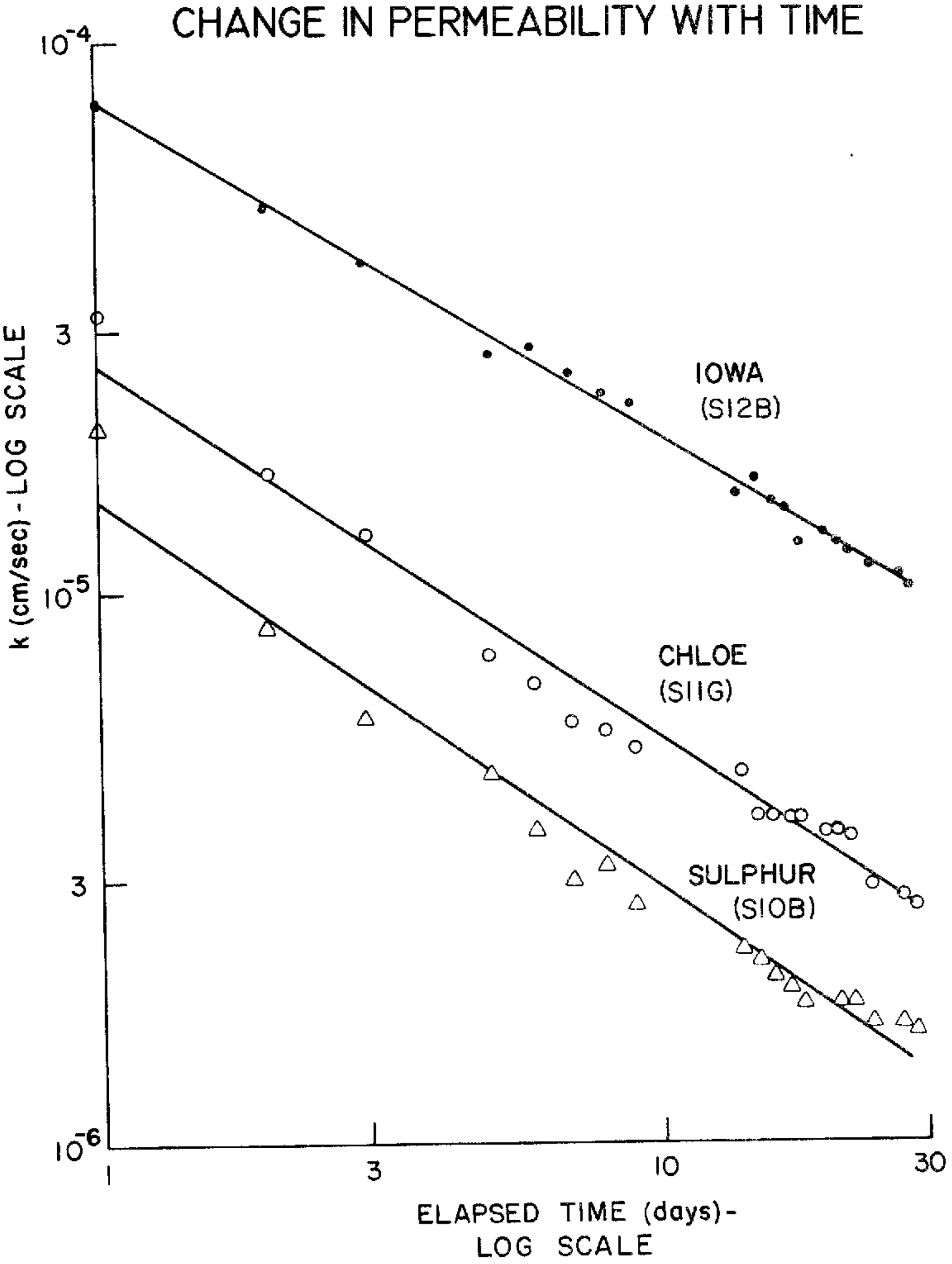

CHANGE IN PERMEABILITY WITH TIME
10^{-4}
3
10^{-5}
3
10^{-6}
k (cm/sec) - LOG SCALE
IOWA
(S12B)
CHLOE
(S11G)
SULPHUR
(S10B)
1
3
10
30
ELAPSED TIME (days)-
LOG SCALE

FIGURE 14

Classification Tests

The mechanical analyses show that no significant difference in grain size exists between stable and unstable silts. All three samples contained 10 to 20% sand, 70% silt, with the remainder being clay (Figure 15).

The liquid limits for S10B, S11G, and S12B were 32, 20, and 23, respectively. All three samples were non plastic.

The specific gravities of S10B, S11G, and S12B were 2.53, 2.62, and 2.60, respectively.

Standard Compaction Test

The maximum density obtained from the standard compaction tests was higher for the stable silt than for the unstable ones (Figure 16). Stable soil S11G had an average maximum density of 107 lbs per cu ft at 14.4% moisture. Unstable samples S12B and S10B reached a maximum density of only 102.8 and 101.5 lbs per cu ft respectively at 16.2 and 17.9% moisture. The difference in maximum densities (5 pounds per cubic foot, at a lower moisture content) was the first consistent difference found between stable and unstable silts.

Table 3, a summary of standard compaction tests for samples S10B, S11G, and S12B, also includes four tests with 1% of an additive (data from Table 4). Samples used for compaction tests were mixed with varying amounts of barium sulphate, gypsum, lime, and cement (from 1 to 10% by weight) and compacted immediately. The effect on density, even for 10% additive, was negligible (Table 4).

Surcharge Compression

Sample S11G gained a denser structure under vertically-applied loads in the surcharge compression test than did samples S12B and S10B (Figure 17). The unit weights attained by S11G averaged 88 lbs per cu ft, while those for S10B and S12B averaged 78 and 80, respectively. The amount of vertical stress (range 200 to 1600 psf) had little effect on the dry density.

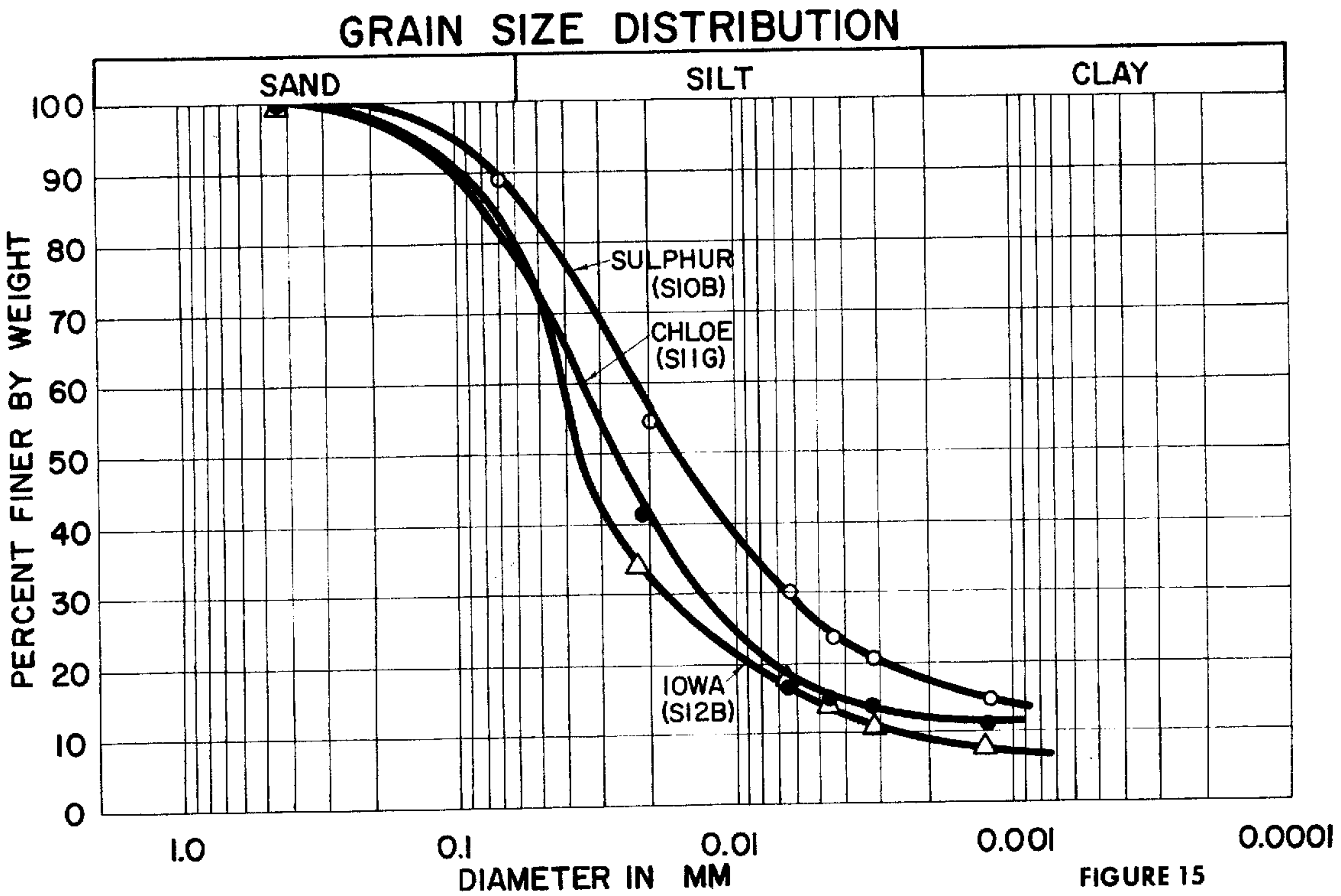
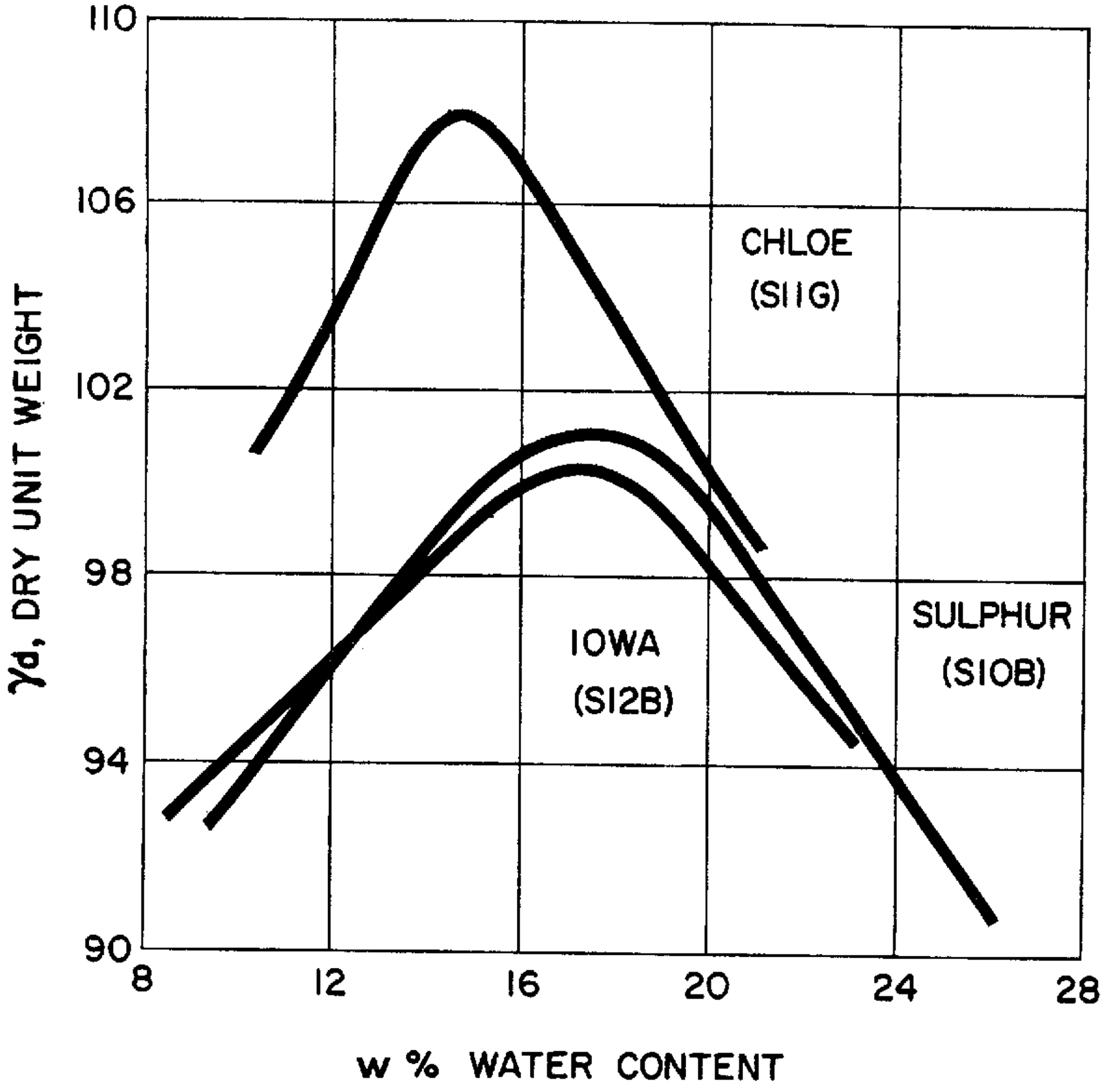

GRAIN SIZE DISTRIBUTION
SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
90
80
70
60
50
40
30
20
10
0
1.0
0.1
0.01
0.001
0.0001
DIAMETER IN MM
SULPHUR (S10B)
CHLOE (S11G)
IOWA (S12B)

FIGURE 15



TYPICAL STANDARD COMPACTION CURVES

FIGURE 16

TABLE 3

SUMMARY OF STANDARD COMPACTION RESULTS
(Maximum Density and Optimum Moisture Content)

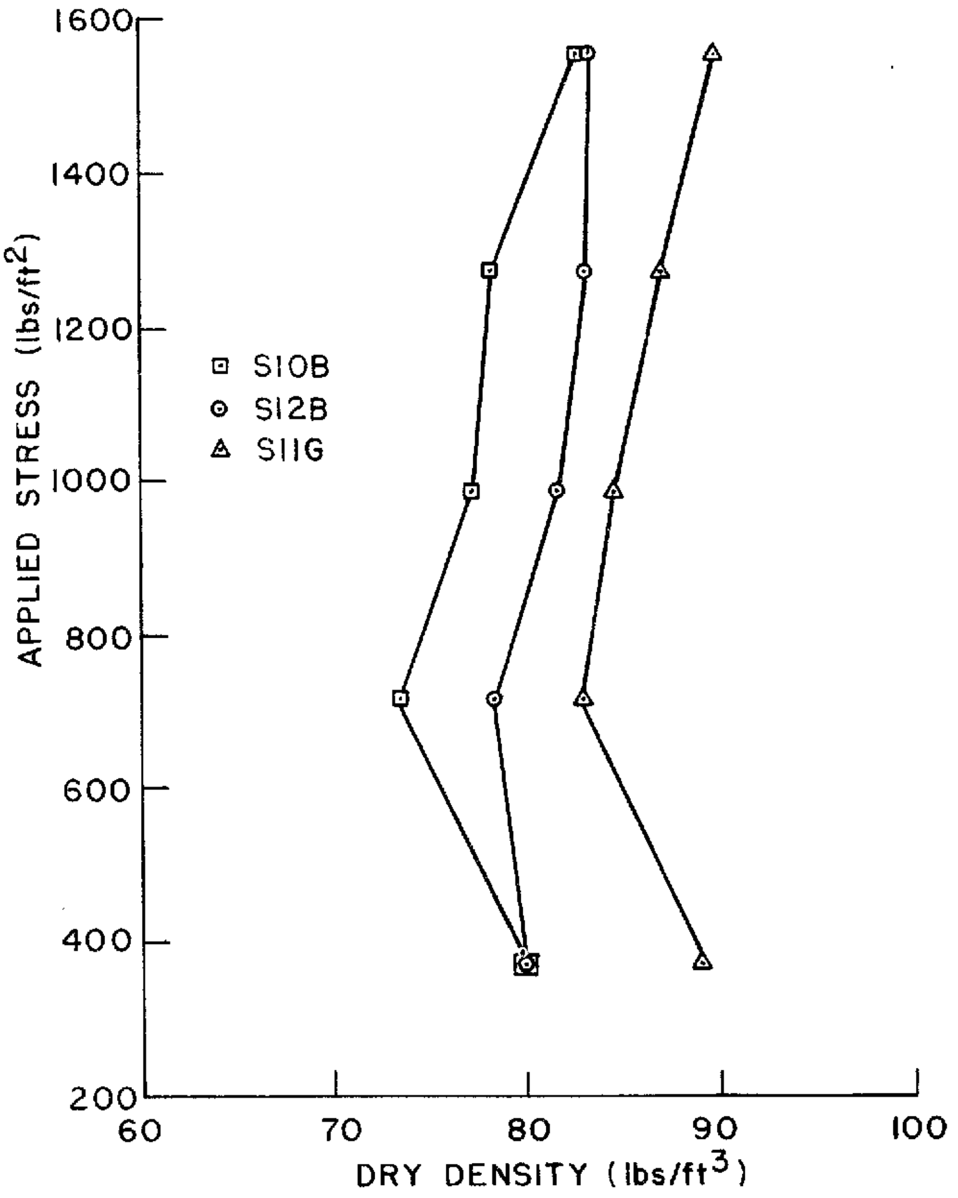
Run	Iowa (S12B)		Sulphur (S10B)		Chloe (S11G)	
	$\gamma_{d)MAX}$ lbs/cu ft	w % OPT	$\gamma_{d)MAX}$ lbs/cu ft	w % OPT	$\gamma_{d)MAX}$ lbs/cu ft	w % OPT
1	105.0	16.7	102.6	18.4	108.6	15.0
2	105.5	15.9	103.0	18.7	107.0	14.6
3	106.0	15.4	102.3	17.5	109.6	13.1
4	104.0	14.8	101.1	17.5	106.4	14.3
6	105.0	15.8	103.0	18.7	107.2	14.2
7	102.6	15.4	102.3*	16.7	108.6	15.0
8	103.6	15.8	101.2*	17.8	107.2*	14.5
9	100.5	17.1	99.8*	18.3	108.0*	15.0
10	101.6*	16.5	99.8*	18.3	108.1*	14.2
11	101.0*	17.0			106.6*	14.5
12	101.7*	17.2				
13	99.8*	16.5				
Average	102.8	16.2	101.5	17.9	107.6	14.4

*Contains 1% additives (See TABLE 4)

TABLE 4

SUMMARY OF STANDARD COMPACTION TESTS WITH ADDITIVES
(Maximum Density and Optimum Moisture Content)

Additive	S10B		S11G		S12B	
	$\gamma_{d)\mathrm{MAX}}$ lbs/cu ft	w % OPT	$\gamma_{d)\mathrm{MAX}}$ lbs/cu ft	w % OPT	$\gamma_{d)\mathrm{MAX}}$ lbs/cu ft	w % OPT
None	103.0	18.7	108.6	15.0	100.5	17.1
1% Gypsum	102.3	16.7	107.2	14.5	101.6	16.5
3% Gypsum			106.5	15.1	101.4	16.6
10% Gypsum	101.9	17.3	106.8	14.8	102.4	16.9
1% Ba Sulphate	101.2	17.8	108.0	15.0	101.0	17.0
10% Ba Sulphate	103.0	18.1	110.0	14.2	103.3	17.3
1% Cement	99.8	18.3	108.1	14.2	101.7	17.2
10% Cement	99.1	19.3	106.8	15.4	103.0	15.8
1% Lime	99.8	18.3	106.6	14.5	99.8	16.5
10% Lime	93.1	20.5	101.3	16.5	97.1	18.7



SURCHARGE COMPRESSION TEST RESULTS

FIGURE 17

Calgon Test

The supernatant liquid produced by unstable silts in the Calgon test was black. In the same test, stable silts left a brown or gray liquid--not very different from that produced in plain water without Calgon. Figure 18 shows unstable samples S10B and S12B and stable samples S11G as they appear in (1) water (Figure 18A--100g soil in 300 ml of distilled water), (2) the Calgon test (Figure 18B--110g soil and 3g Calgon in 300 ml of distilled water), and (3) the mechanical analysis (Figure 18C). A description of the Calgon Test is included in Appendix A.

Electron Micrographs

No significant difference in particle shape between samples S11G and S12B could be seen from electron microscope studies. Many soil particles from both samples were well rounded (Figure 19). Both samples contained particles that were agglomerations of smaller soil particles (Figure 20). Microfossils ranging from 1 to 10 microns in size were the only fossils present (Figure 21). These debris are similar to those reported by Beutelspacher and Van Der Marel (1968, p. 264) as silicious diatoms.

Clay Mineralogy

The clay-size fraction of a soil is especially important. It gives the soil many of its physical properties, such as plasticity, shrinkage and swelling characteristics, and cohesion. Clays are small crystalline particles (less than .002 mm.) which are members of a group of minerals called clay minerals (Grim, 1968, p. 21), whose crystalline structure determines their physical properties and provides the basis for their identification. When subjected to X-ray diffraction, each mineral exhibits characteristic peaks. Because the diffraction characteristics of many clay minerals are similar, however, other techniques--such as furnace heating (minerals are destroyed at different temperatures), acid treatment (some minerals are destroyed by acid, others are not), and glycol saturation (some minerals expand in glycol to a

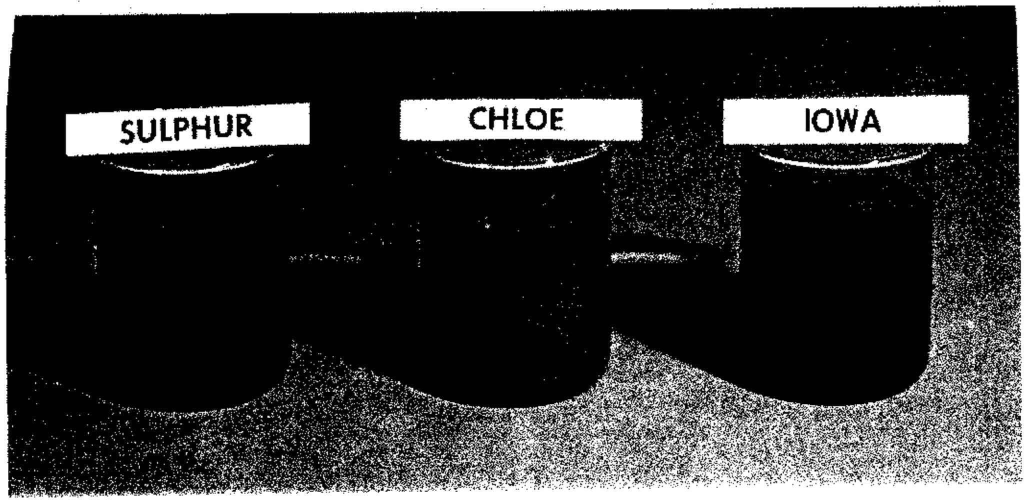


FIGURE 18A

SOIL IN WATER--SETTLED OVERNIGHT

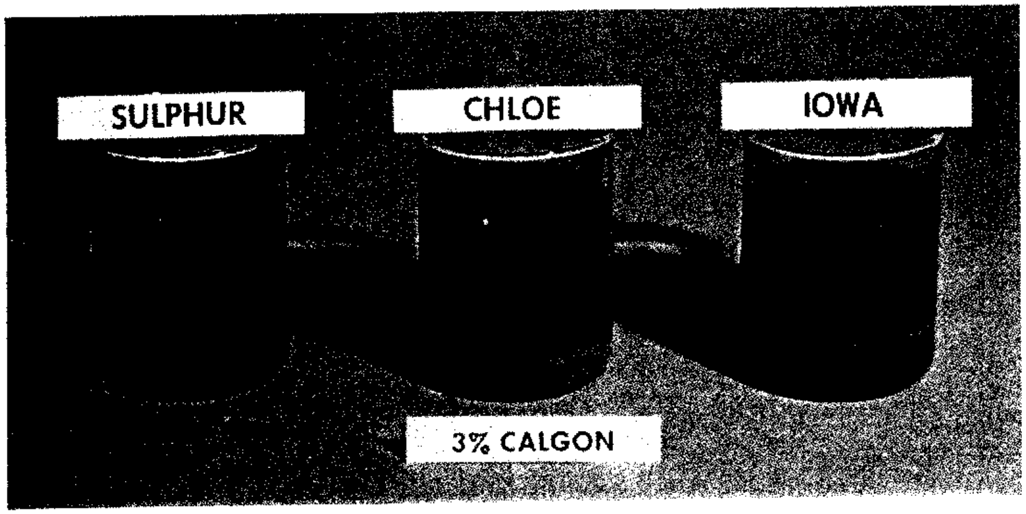


FIGURE 18B

SOIL IN CALGON AND WATER--SETTLED OVERNIGHT

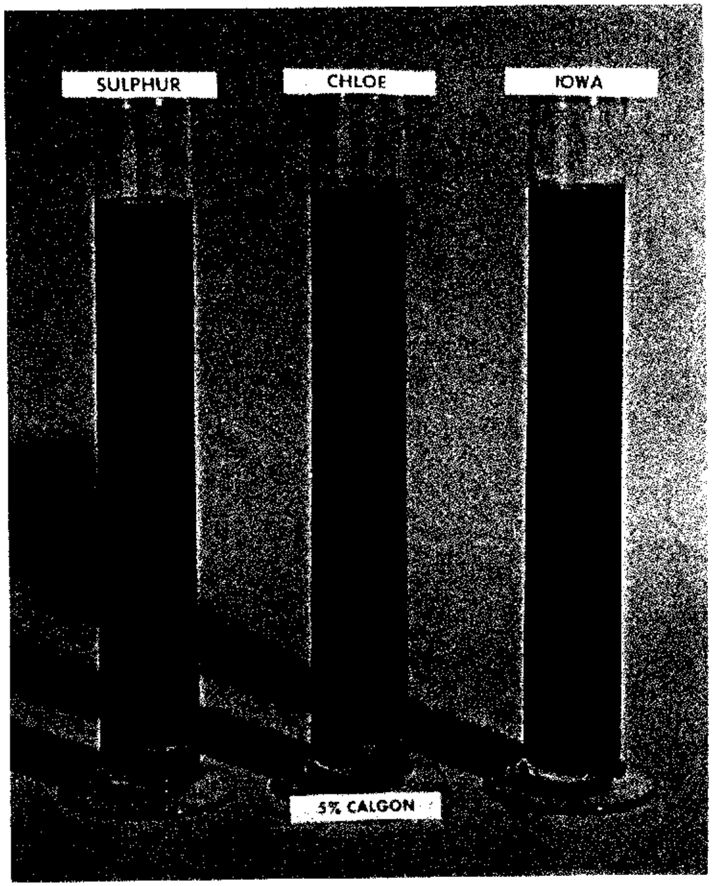
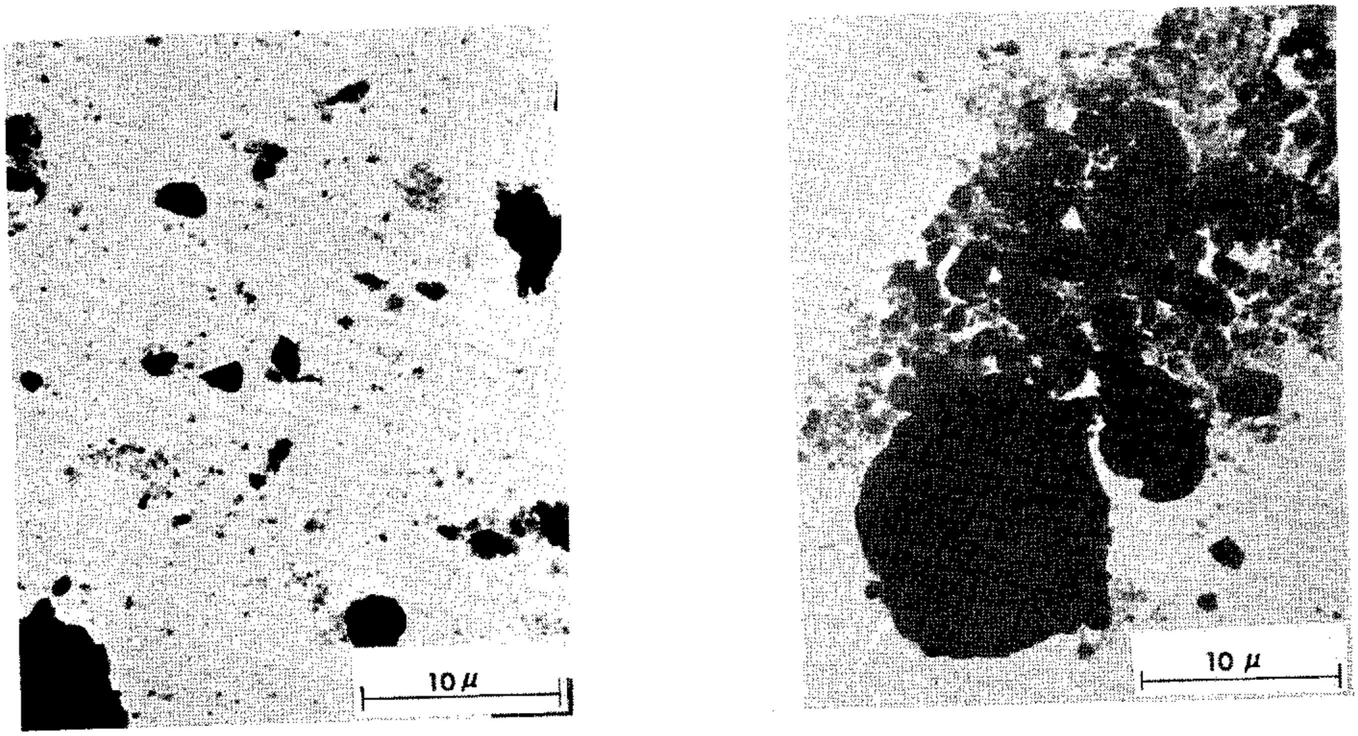
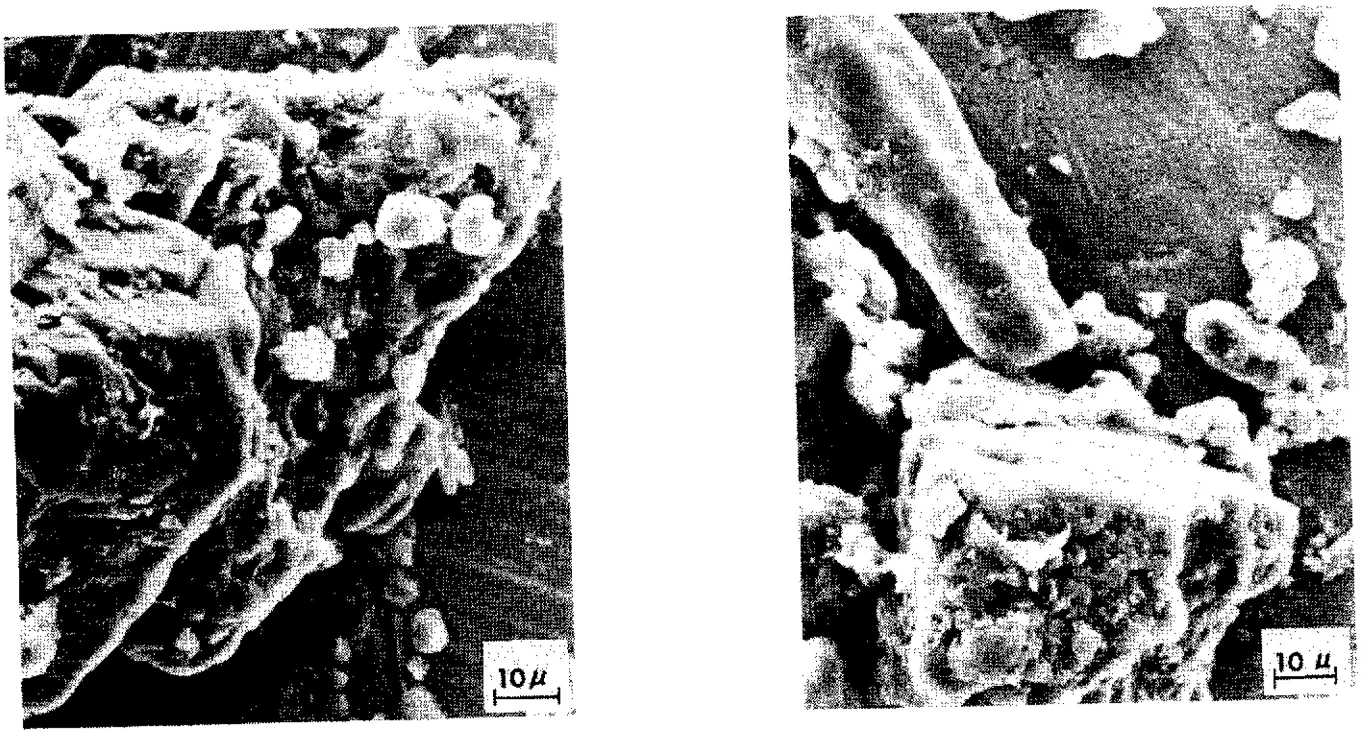


FIGURE 18C

SAMPLES AT TIME OF 1-DAY READING
IN THE MECHANICAL ANALYSIS

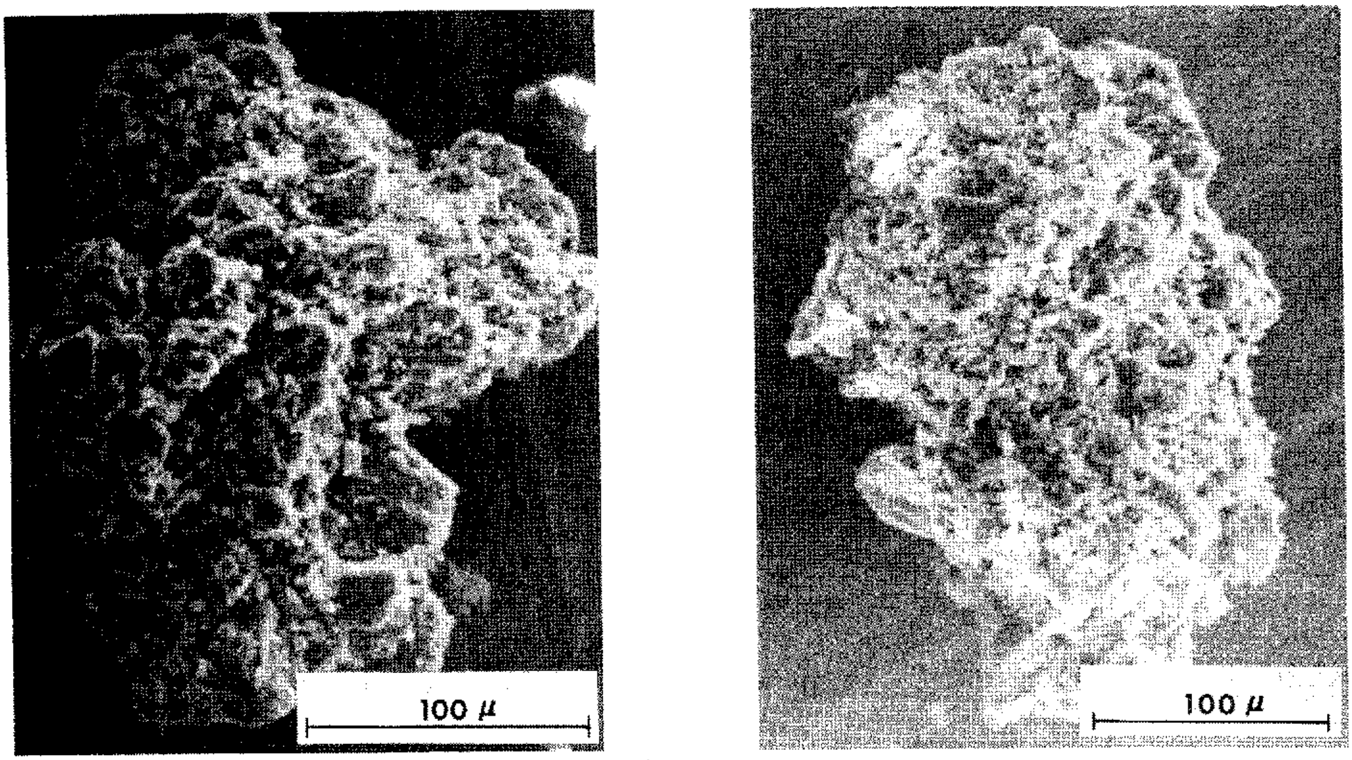


Chloe-S11G TRANSMISSION ELECTRON MICROGRAPHS SHOWING CLAY FRACTION Iowa-S12B



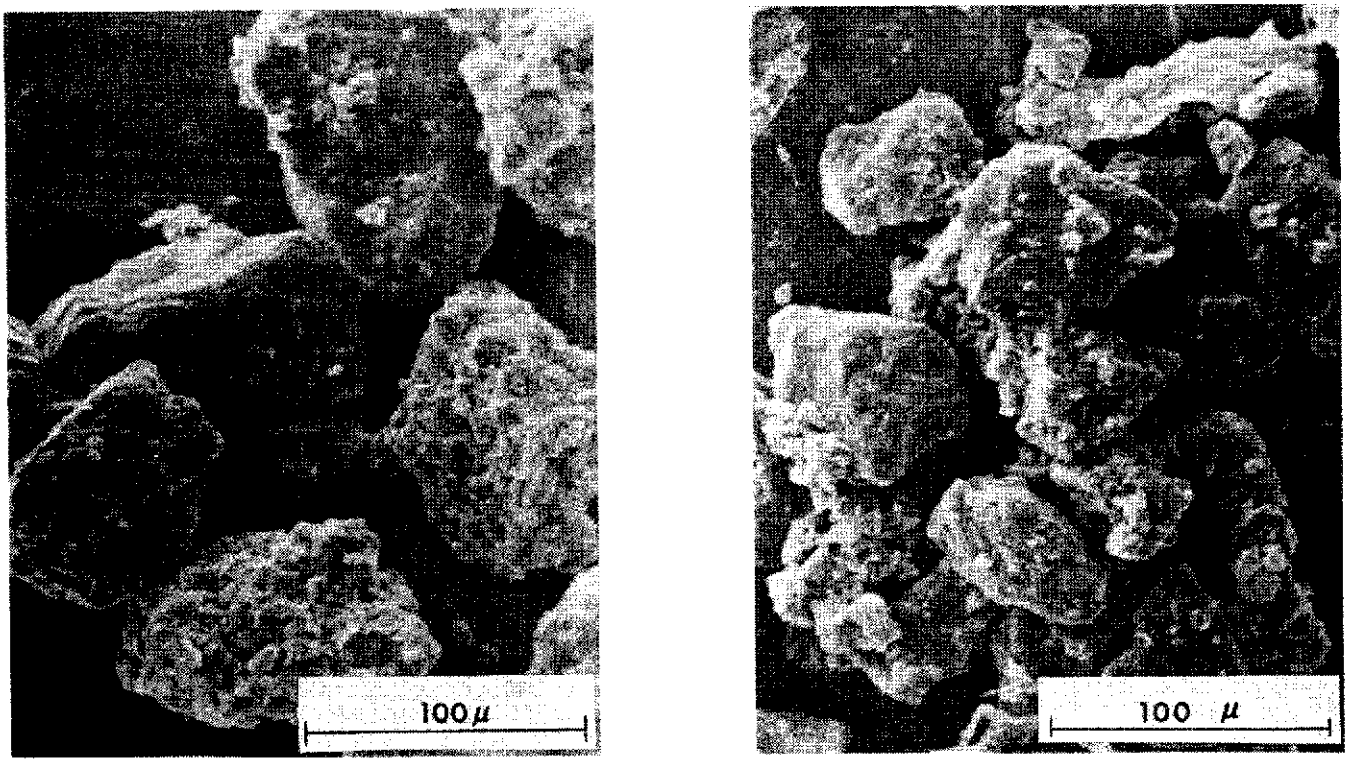
SCANNING ELECTRON MICROGRAPHS SHOWING ROUNDED SILT PARTICLES

FIGURE 19



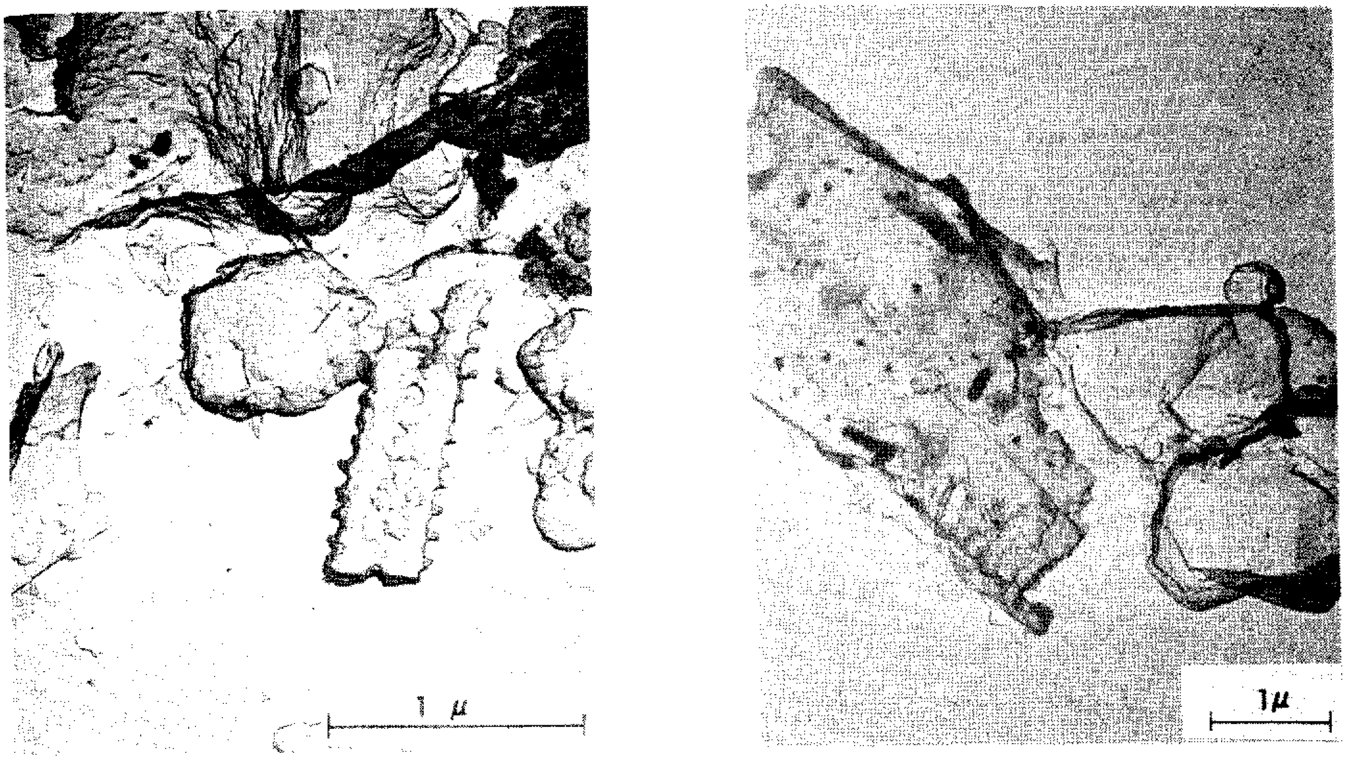
Chloe-S11G

Iowa-S12B

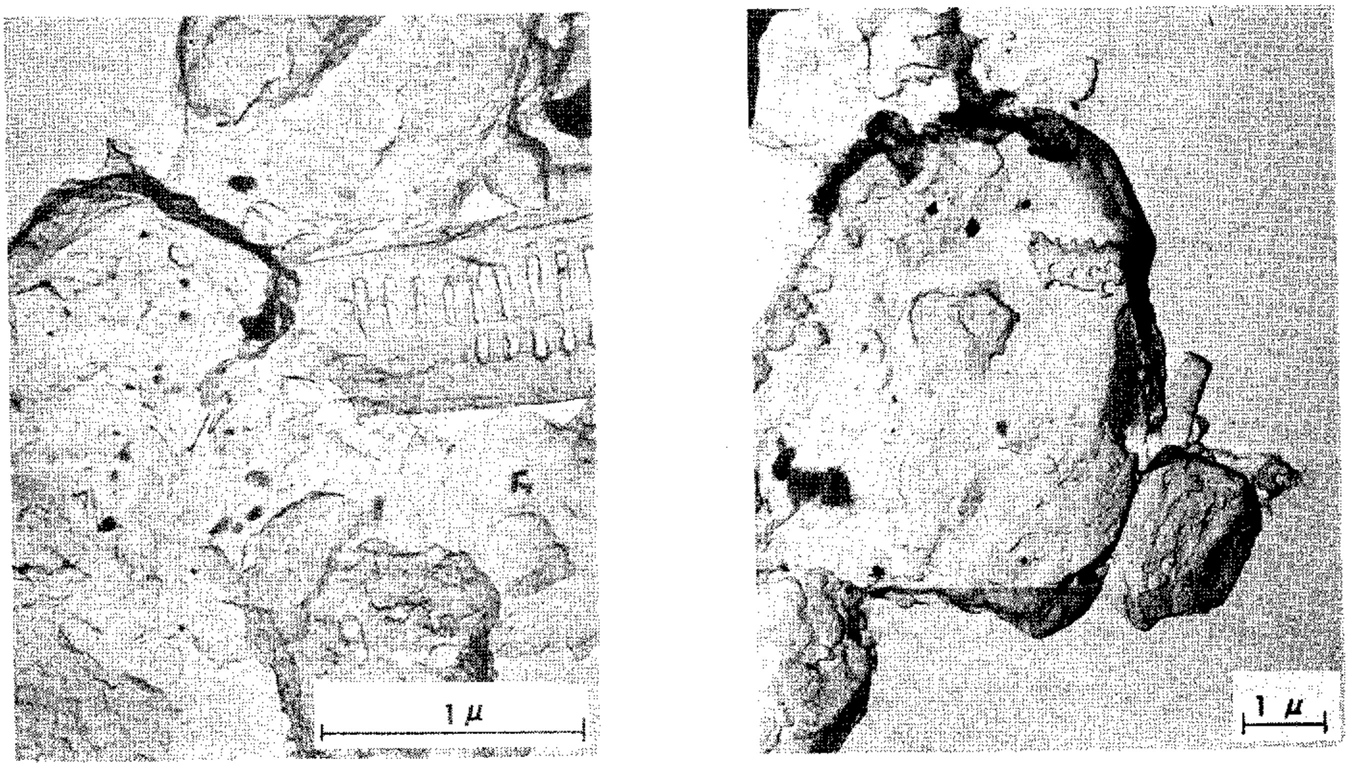


SCANNING ELECTRON MICROGRAPHS
SHOWING AGGLOMERATED SAND AND SILT PARTICLES

FIGURE 20



Chloe-S11G Iowa-S12B



TRANSMISSION ELECTRON MICROGRAPHS
SHOWING SILICIOUS DIATOMS OR PLANT DEBRIS

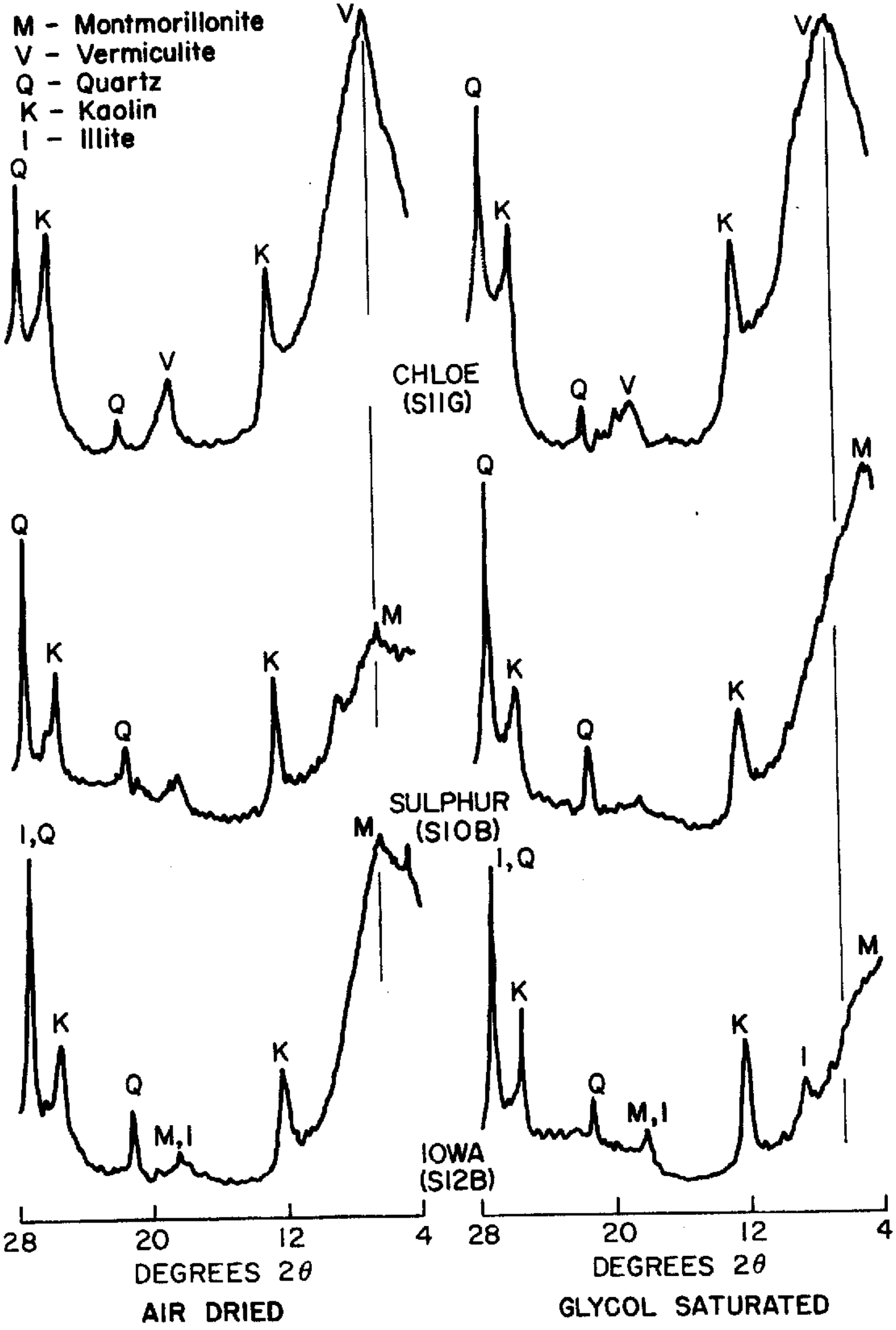
FIGURE 21

specific spacing)--must be used in conjunction with diffraction analysis for a positive identification.

The clay fractions of S12B (Iowa), S10B (Sulphur), and S11G (Chloe) silts were analyzed by X-ray diffraction to determine which clay minerals were present. The diffraction patterns, obtained from air-dried, parallel-oriented samples with particles smaller than two microns, were compared with similar samples that had been: (1) glycolated; (2) heated from 250°C to 600°C at 50°C increments; or (3) treated with 1N HCl.

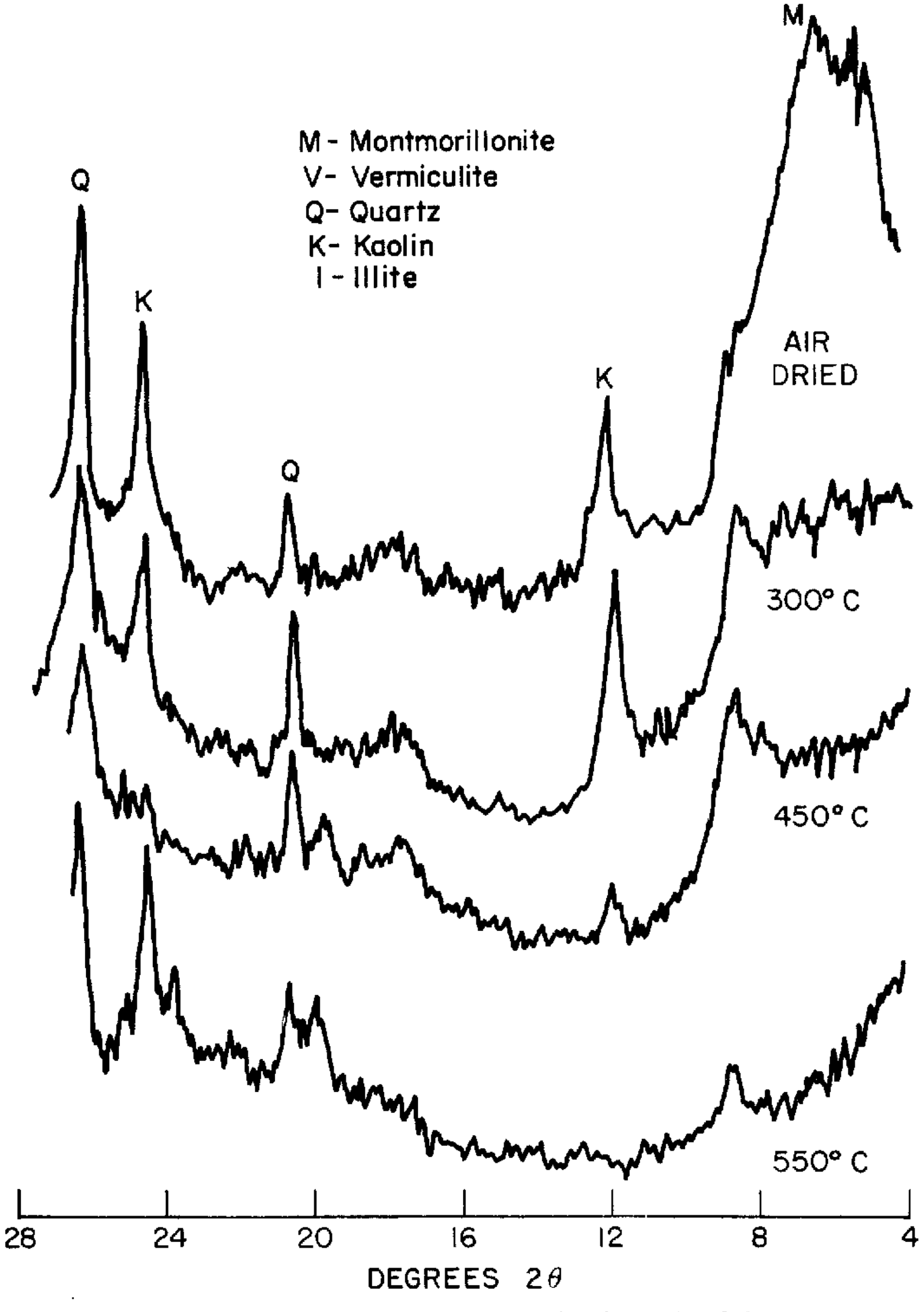
The presence of montmorillonite in samples S10B and S12B and illite in sample S12B was indicated by the diffractions patterns from air-dried and glycol-saturated samples (Figure 22). Air-dried montmorillonite has a broad diffraction peak between 5-8° 2θ (12Å - 15Å). In glycol-saturated samples, the montmorillonite peak expands to 5.2° 2θ (17Å) (Jackson, 1956, p. 212). Illite has a diffraction peak at 8.8° 2θ (10Å) which is partially obscured in the air-dried sample S12B by the broad montmorillonite peak. Glycol saturation, which has no effec on illite, enhances the illite peak by shifting the montmorillonite peak to the right.

Comparisons of diffraction patterns from air-dried and heated samples confirmed the presence of montmorillonite in samples S10B and S12B and indicated the presence of vermiculite in S11G. The air-dried montmorillonite peaks (5-8° 2θ) of S10B (Figure 23) and S12B (Figure 24) collapse to 8.6° 2θ (10Å) when heated to 300°C. The montmorillonite peaks remain collapsed and were not destroyed at 450°C and 550°C. The 6.0° 2θ (14.2Å) vermiculite peak in air-dried S11G (Figure 25) collapses at 300°C to 8.6° 2θ (10Å). Vermiculite, however, is not destroyed at 450°C or 550°C. Loss of the 12.2° 2θ (7Å) peak, which occurred in sample S11G (Figure 25) at 450°C, was attributed to intergradational material, i.e., incomplete "brucite-islands" between vermiculite layers (Johns, et. al., 1954, p. 243).



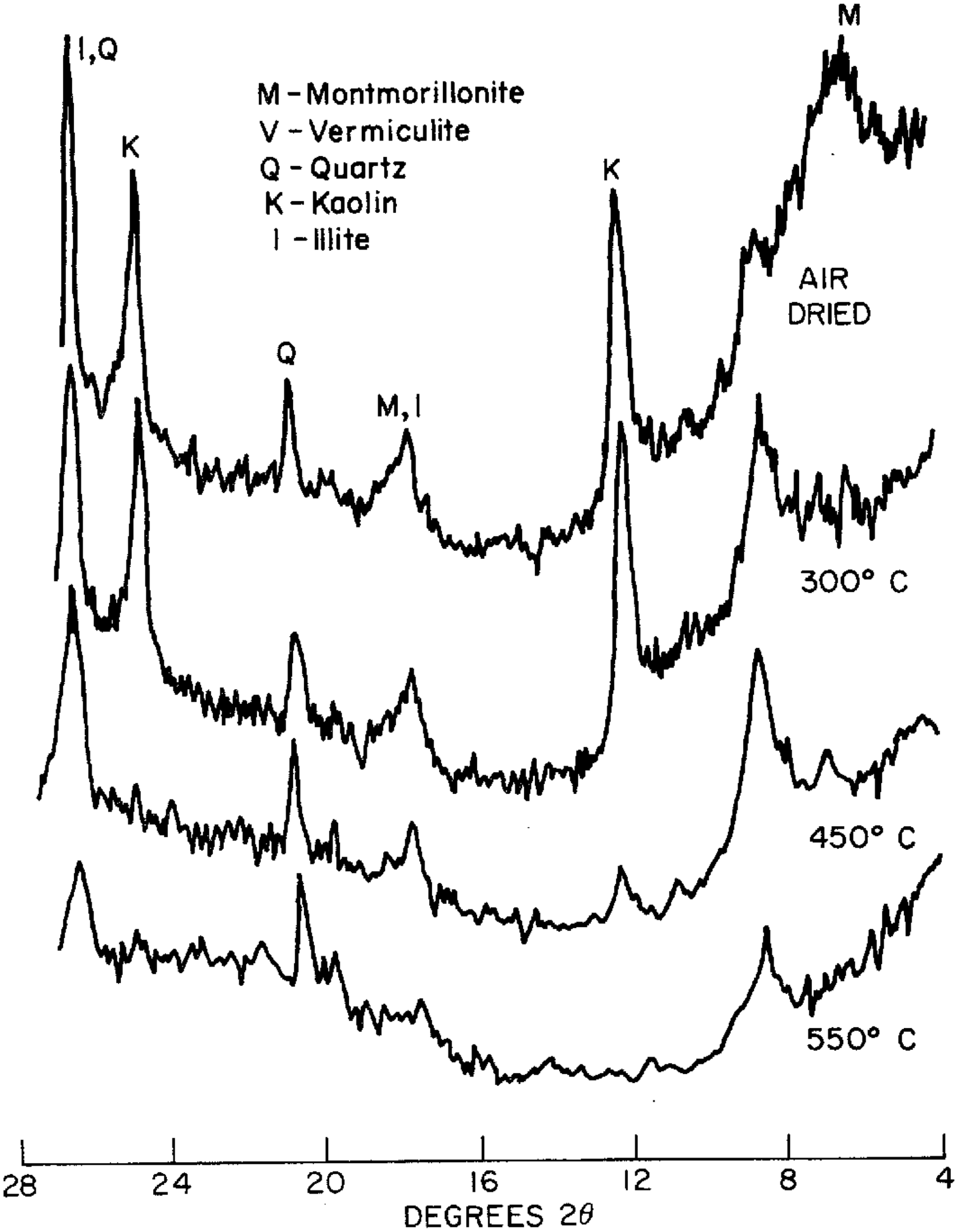
X-RAY DIFFRACTION OF CLAY FRACTION

FIGURE 22

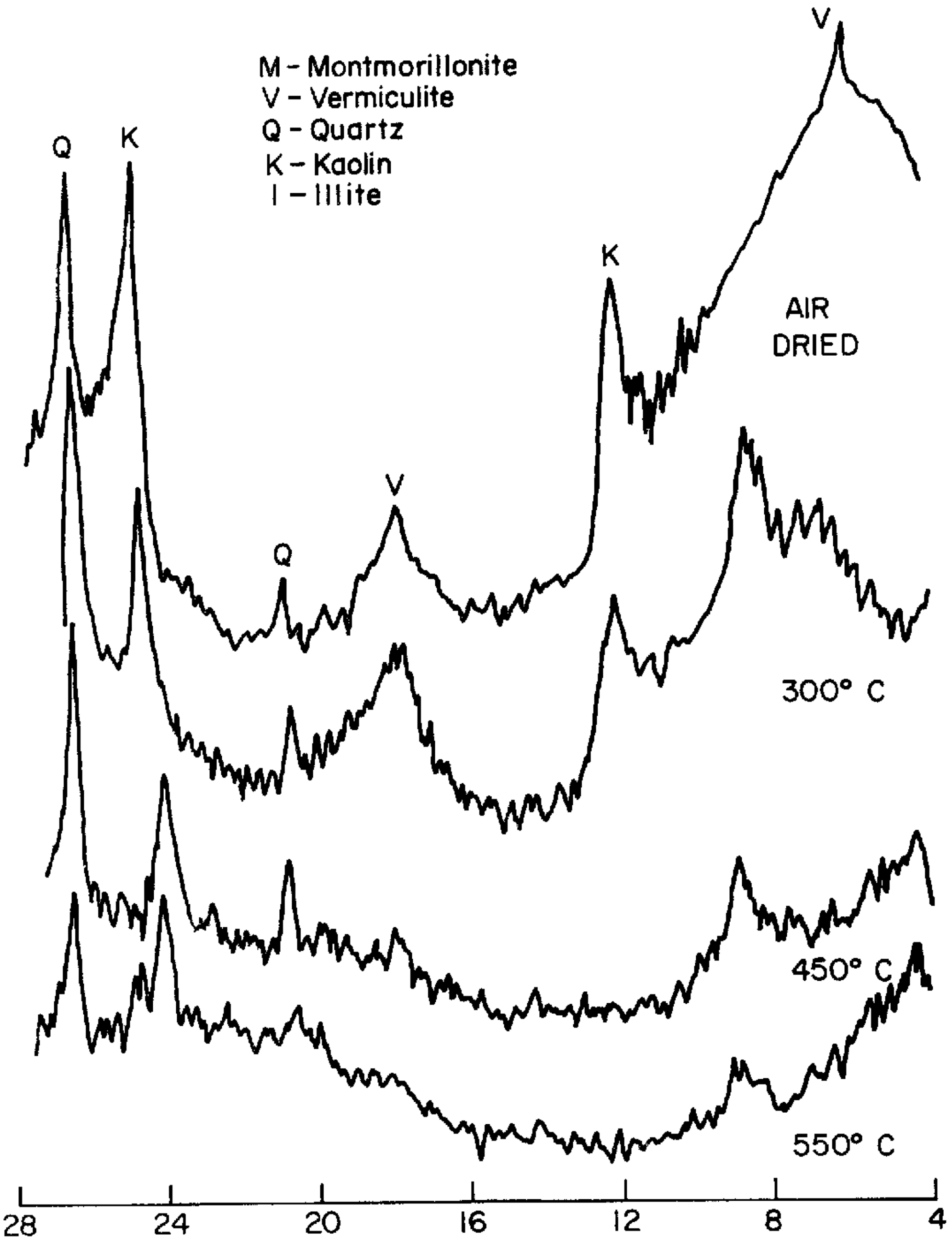


X-RAY DIFFRACTION OF CLAY FRACTION SULPHUR-SIOB

FIGURE 23



X-RAY DIFFRACTION OF CLAY FRACTION
IOWA-S12B
FIGURE 24



X-RAY DIFFRACTION OF CLAY FRACTION

CHLOE-SIIG

FIGURE 25

The presence of kaolinite in all three samples was confirmed from comparison of the diffraction patterns (Figure 26) of air-dried samples and 1N HCl treated samples. Acid treatment destroys the 12.4° 2θ (7.2Å) and 24.8° (3.6Å) peaks, if they are due to the presence of chlorite, but leaves kaolinite unaffected.

Quantitative analyses of quartz, a stable, highly weather-resistant mineral, were made in an effort to find a difference between stable and unstable silts. Samples S10B, S11G, and S12B contained 46%, 76%, and 60% quartz by weight, respectively. (Details of the tests are included in Appendix C.) Because sample S11G did not contain significantly more or less quartz than S10B and S12B, this analysis is useless.

In-Place Unit Weights

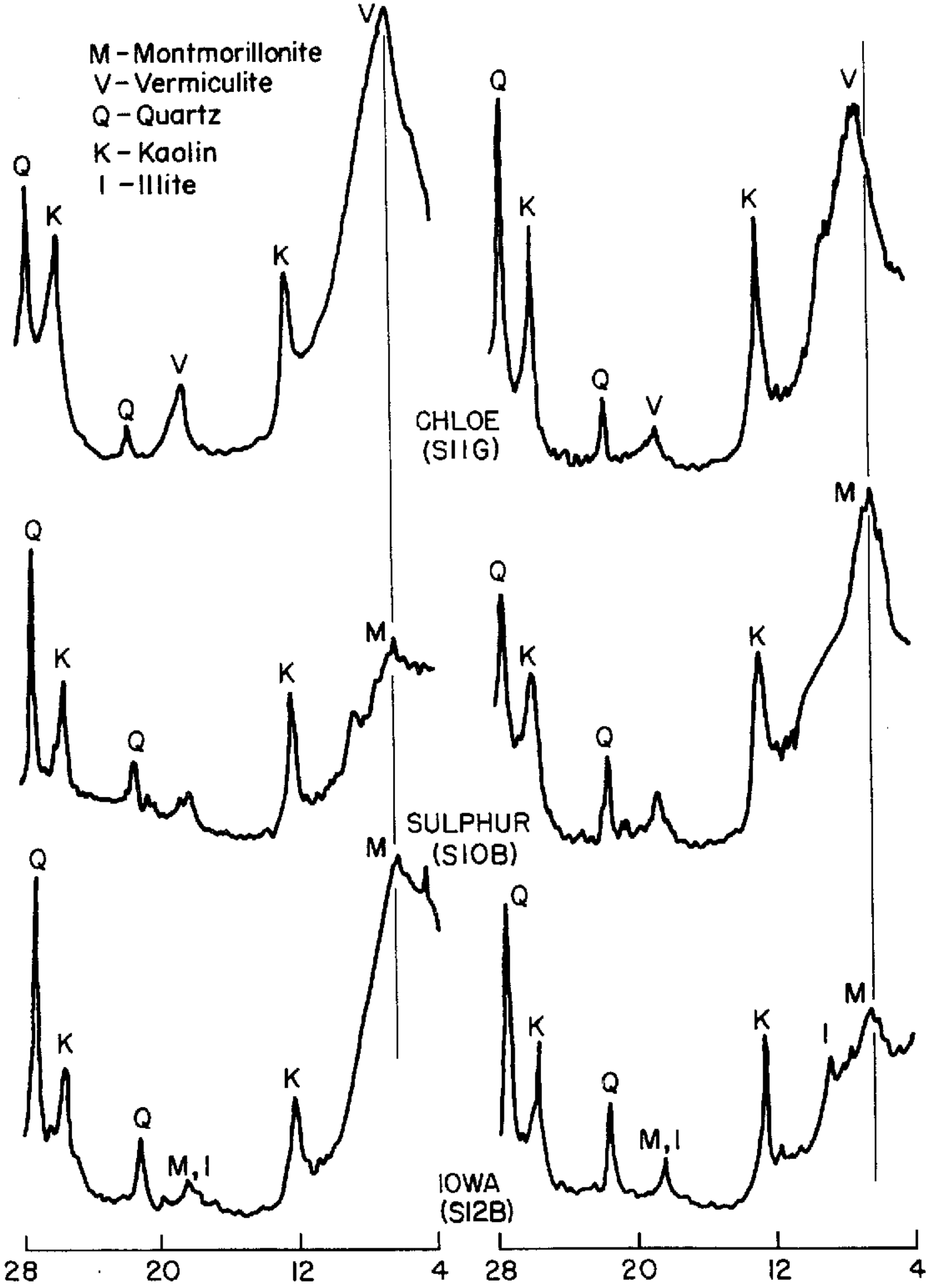
The in-place unit weights of the undisturbed silts, measured with a nuclear density device, were less for unstable soils (samples S12B and S10B) than for the stable one (S11G) (Table 5). Samples S12B and S10B averaged less than 80 lbs per cu ft; at disturbed sites, between 95 and 110 lbs per cu ft. All in-place unit weights were measured at sites least likely to have been disturbed by construction or farm equipment. The constant difference in weight of 8 lbs per cu ft furnished a third property, along with the black Calgon liquor and standard compaction density, for distinguishing stable and unstable silts.

Direct Shear Tests

Sample S12B had an apparent cohesion of 0.4 tons per sq ft and an angle of internal friction of 30 degrees (Figures 27 and 28).

Soil Profile at Iowa, Louisiana

The topographic features of the unstable silt deposits at Iowa were studied from aerial photographs and by site visits. The deposits were found to be located on low, but well-drained, ridges. Slopes from the ridges are gentle (less than 1 in 100 feet) to the northwest and southeast. For determining the soil profile, disturbed and undisturbed samples were taken at approximately half-mile intervals along the slopes (Figure 12).



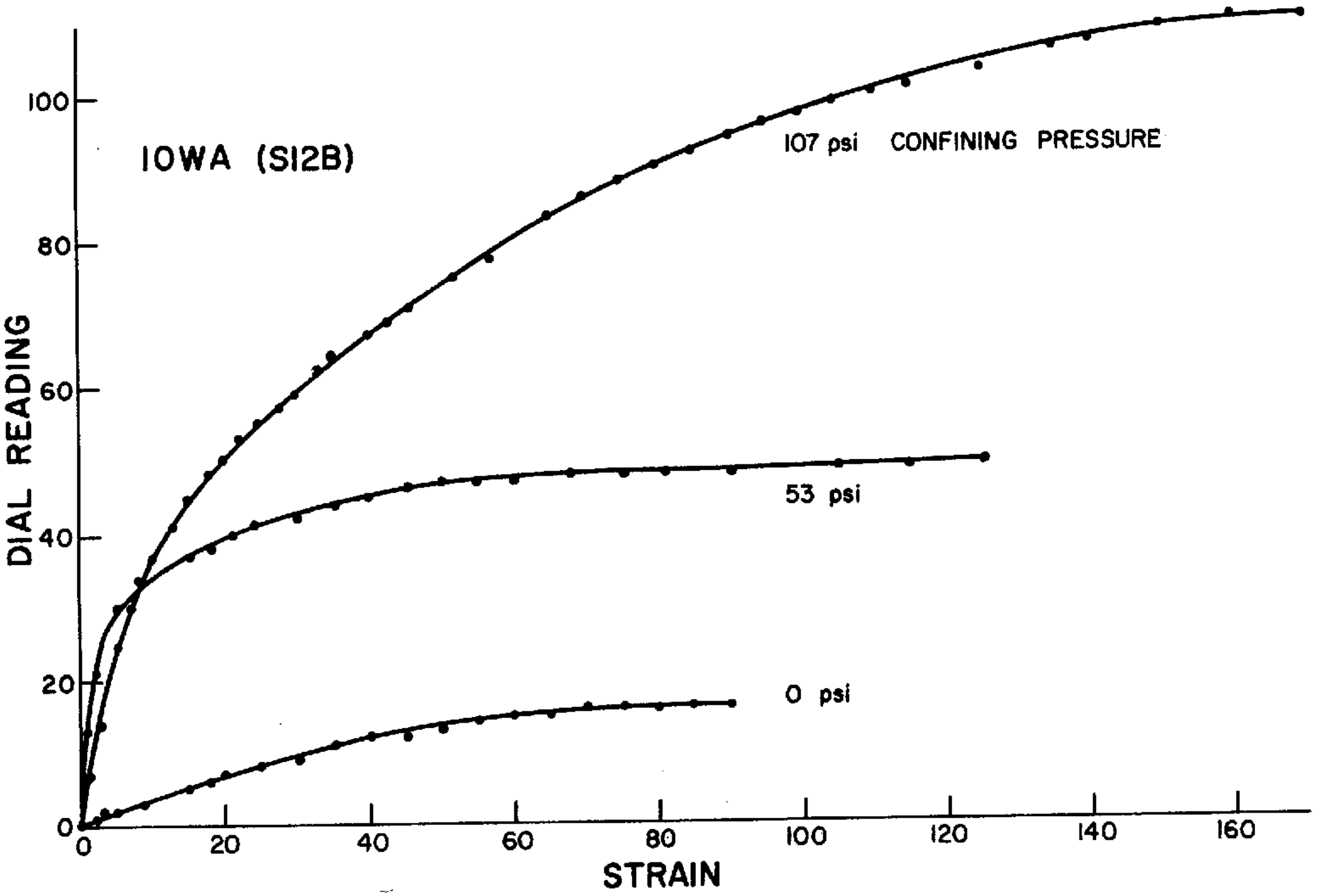
X-RAY DIFFRACTION OF CLAY FRACTION

FIGURE 26

TABLE 5

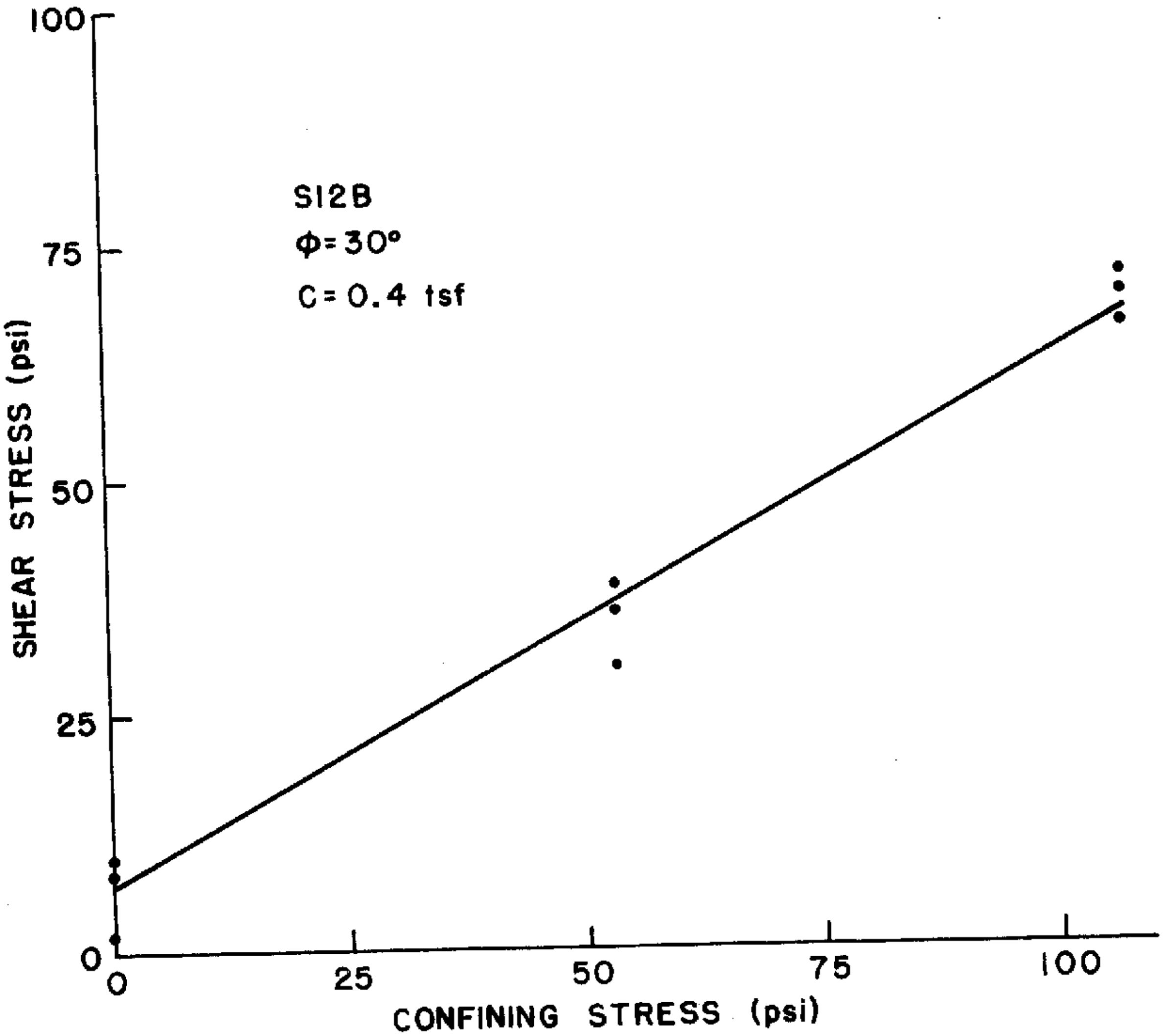
IN PLACE DRY UNIT WEIGHTS AND MOISTURE CONTENT AS MEASURED AT DIFFERENT TIMES OF THE YEAR

Date	Iowa		Sulphur		Chloe	
	lbs/cu ft	w%	lbs/cu ft	w%	lbs/cu ft	w%
Oct. 69	75.0	21.3	77.0	18.0		
Jan. 70	75.7	22.6	82.5	23.6	88.3	23.3
July 70	82.4	15.3				
July 70	78.7	12.2				



TYPICAL DIRECT SHEAR DATA

FIGURE 27



DIRECT SHEAR-IOWA

FIGURE 28

The sampling results indicate that farming has changed the soil composition (Table 6)--apparently by turning part of the underlying clay pan up into the surface silts. TS1, for example contains 19% clay, whereas the original undisturbed S12B sample (located only 100 feet away) contains only 9% clay.

When the Calgon test was applied to Iowa soil profile survey samples TS1 through TS16, three of them, TS1, TS7, and TS11, turned black (Table 6). Field investigation of test sites 7 and 11 revealed that both were on low ridges. TS11, although one mile away, was a part of the same ridge from which the original S12B and TS1 samples were taken. TS7 had a similar grain size distribution to S12B and significantly lower Standard Proctor density, 105.2 lbs per cu ft, than samples at the other test sites.

Three additional test sites, TS17, TS18, and TS19, containing unstable silt similar to S12B, were discovered on a field survey made southeast of the S12B site (Figure 29) through observation of strata sequences along ditch cuts and measurement of in-place densities. Test results are shown in the last part of Table 6. All three test sites were on ridges. The samples had in-place unit weights, maximum standard compaction densities, and grain sizes similar to sample S12B and turned black in the Calgon test.

The slope of the liquid limit plot (log no. of blows vs. water content) was tried out as an identification method. Because no trend in the slopes was apparent, the plot has no value. Appendix C contains liquid limit curves for TS1-19.

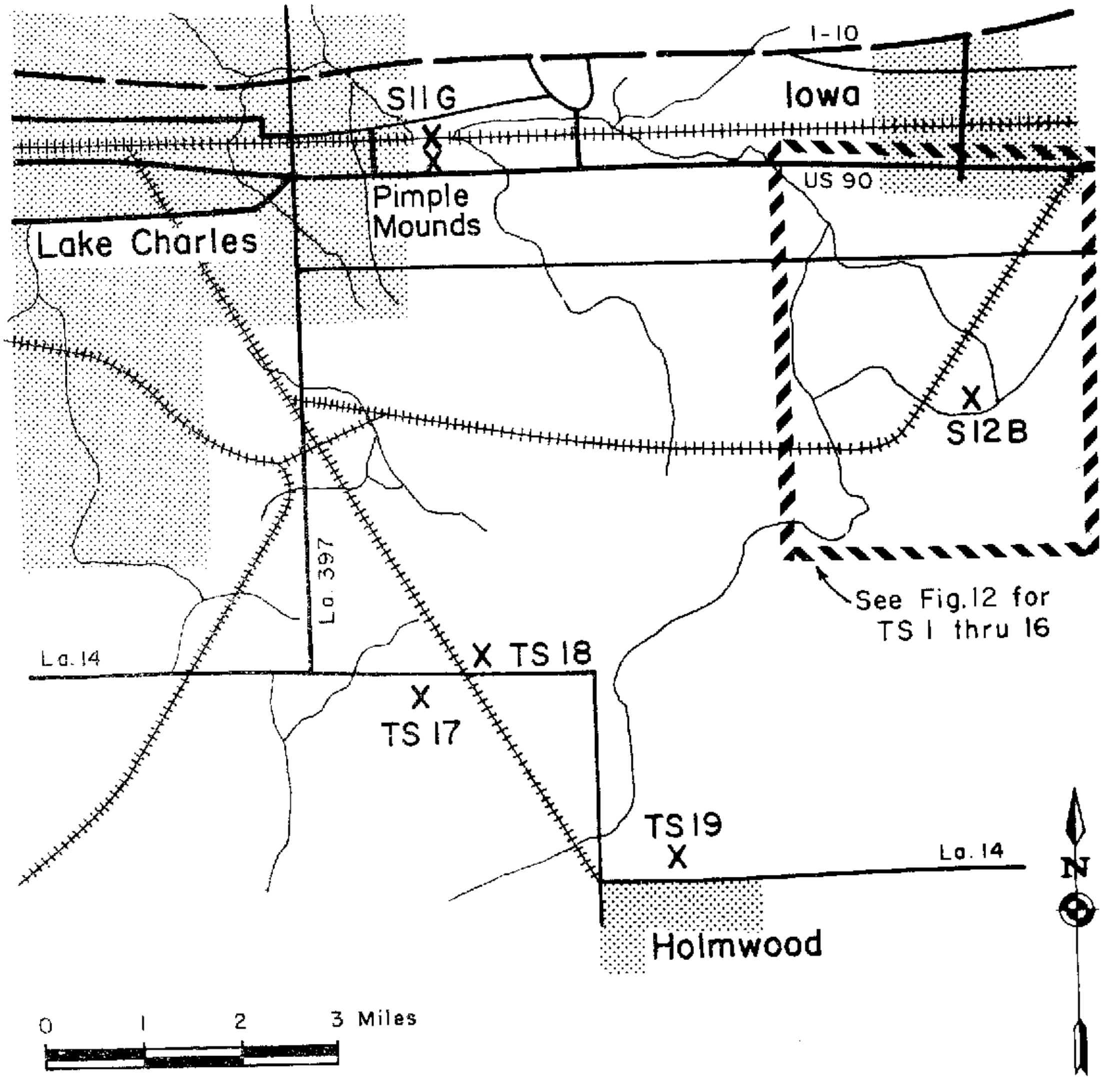
Collapse Test

The collapse test shows that unstable silts compress more under load than stable silts (Figure 30). Unstable silts underwent strains up to 23.6% under a load of 16 tons per sq ft, whereas strains in stable silt were only 14.5% (Table 7). Collapse, or subsidence, in the lab takes a few minutes to occur (Figure 31). A standard consolidation test on S12B (Figure 32) gave approximately the same strain results between 0 and 16 tons per sq ft as the collapse test, but it did not

TABLE 6

SUMMARY OF TEST RESULTS OF SOILS FROM IOWA*

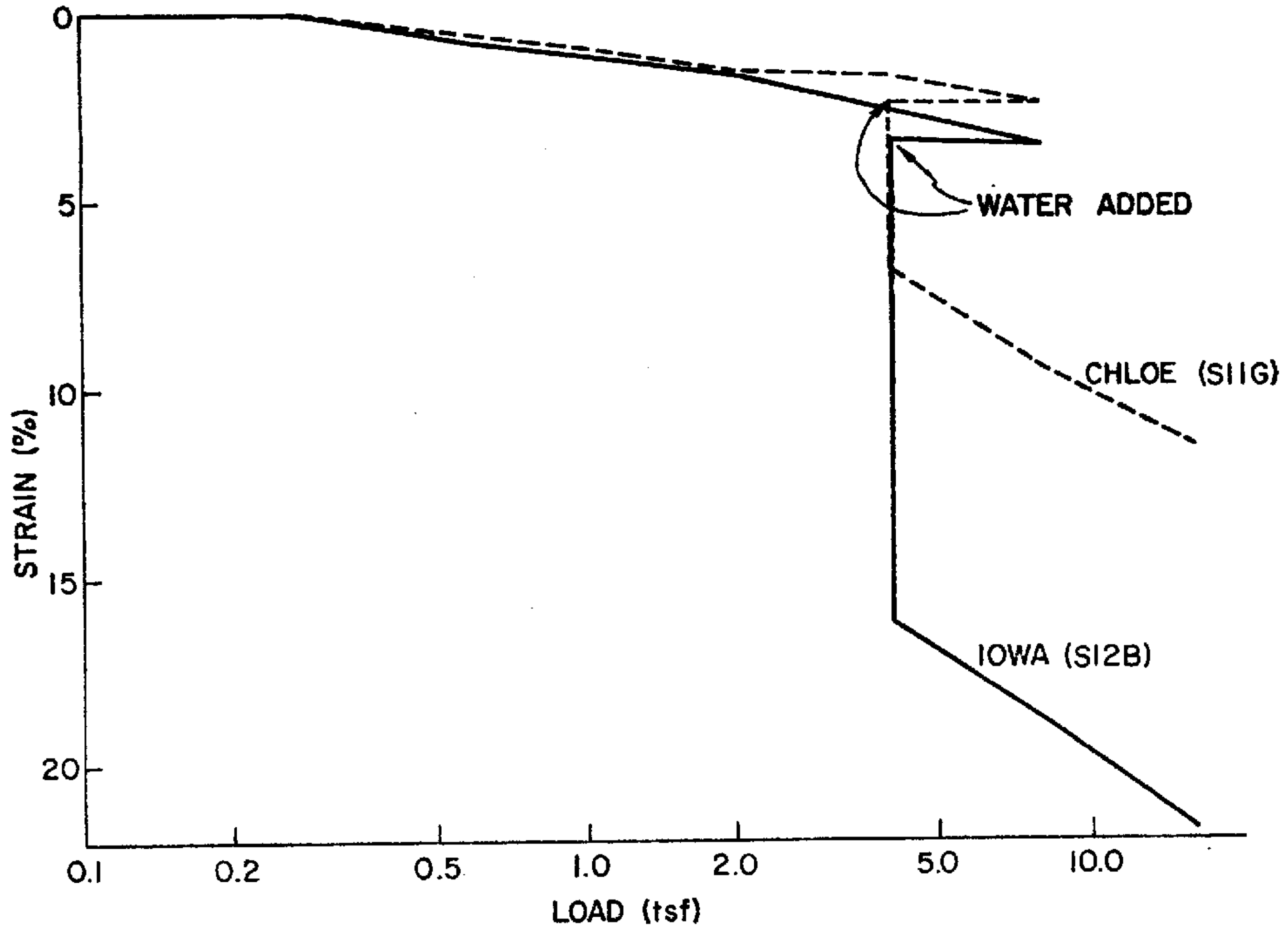
Sample	Standard Compaction $\gamma_d)_{MAX}$ (lbs/cu ft)	Standard Compaction W_{OPT} (%)	In Place Density (lbs/cu ft)	Grain Size % Silt	Grain Size % Clay	Calgon Color Test
S12B	102.8	16.2	78	71	9	Black
TS1	107.6	13.5	94.7	67	19	Black
TS2	112.6	13.3	8.9	62	19	Brown
TS3	110.4	14.1	95.1	70	17	Brown
TS4	109.5	14.1	106.8	65	25	Dark Gray
TS5	110.2	13.4	101.0	75	13	Brown
TS6	112.4	12.9	104.1	74	10	Brown
TS7	105.2	15.5	100.4	64	9	Black
TS8	112.2	12.7	102.4	58	13	Dark Gray
TS9	112.0	13.0	104.4	57	17	Dark Gray
TS10	109.1	13.9	99.1	73	12	Gray
TS11	111.4	13.6	96.3	61	25	Black
TS12	112.4	12.9	104.9	61	14	Gray
TS13	110.8	14.3	-----	63	19	Gray
TS14	112.2	12.6	110.5	61	25	Dark Gray
TS15	112.5	13.2	105.5	47	22	Brown
TS16	112.2	12.8	110.6	42	17	Brown
TS17	103.5	16.2	73.2	72	5	Black
TS18	105.1	14.8	83.1	68	13	Black
TS19	102.3	15.8	66.8	74	5	Black



Lake Charles Area

SAMPLE LOCATIONS

FIGURE 29



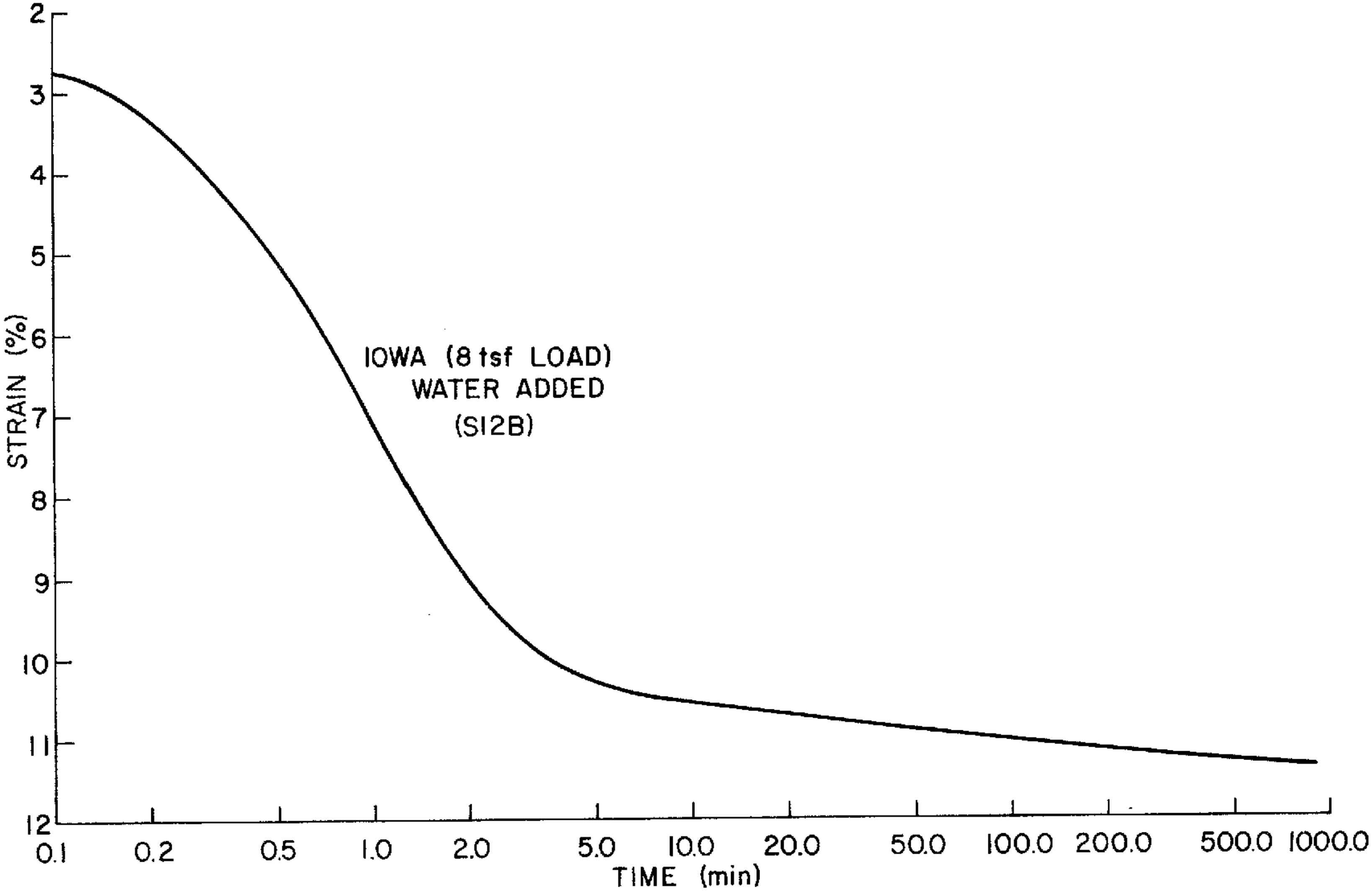
COLLAPSE IN A CONSOLIDOMETER

FIGURE 30

TABLE 7

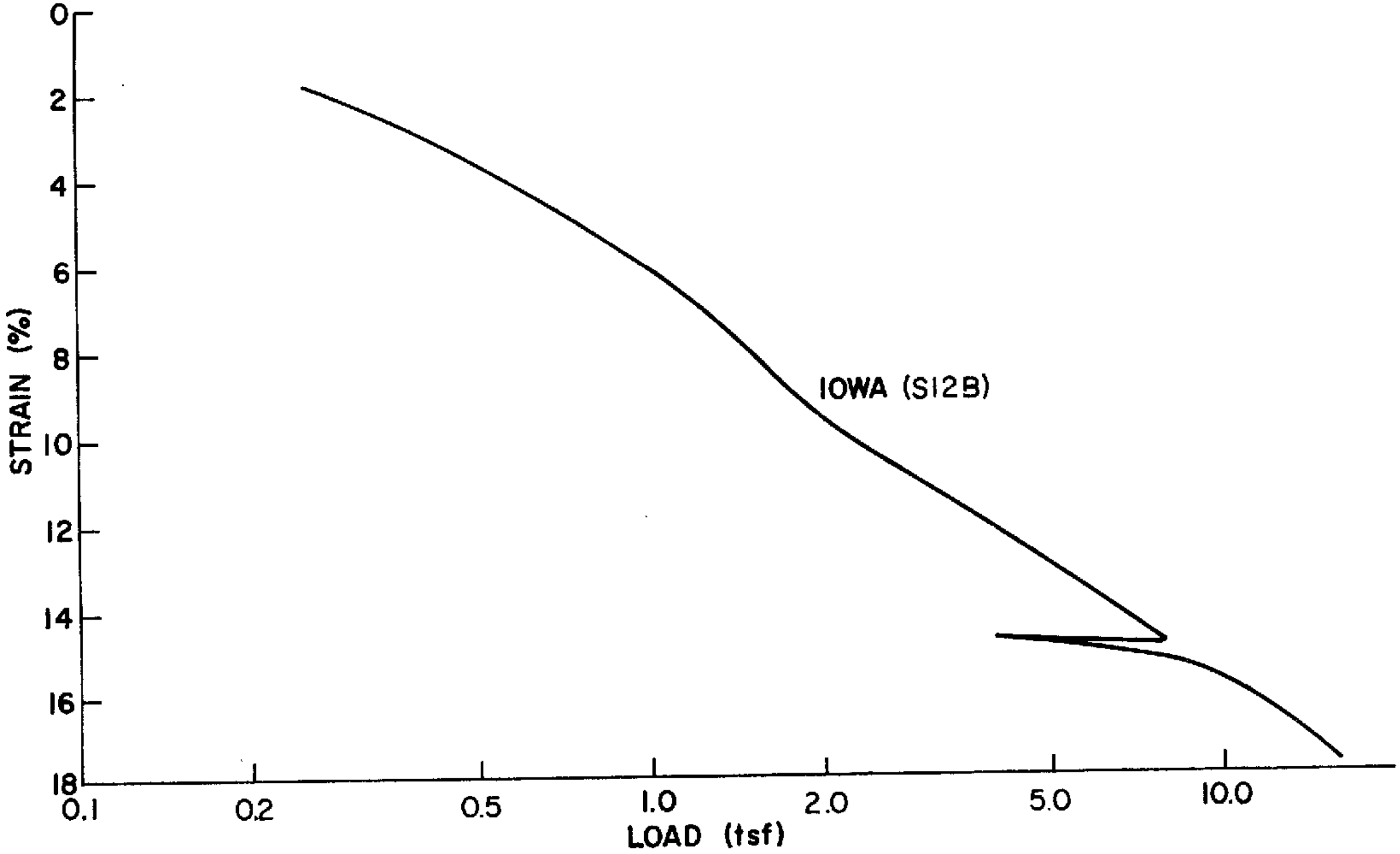
SUMMARY OF COLLAPSE TESTS FOR SOILS FROM IOWA AND CHLOE

Sample	% Strain in per in 0-16T	 8T-8R	Sample	% Strain in per in 0-16T	 8T-8R
S12B-1	15.0	5.0	TS-2	11.3	6.6
S12B-11	6.1	1.6	TS-4	5.9	2.8
S12B-12	5.6	1.3	TS-5	8.5	4.3
S12B-13	9.8	3.4	TS-6	6.1	2.4
S12B-2	15.3	5.5	TS-8	11.0	4.0
S12B-22	11.4	2.6	TS-9	10.4	4.5
S12B-23	10.2	2.3	TS-10	11.8	4.0
S12B-33	11.6	0.0	TS-11	16.5	8.7
S12B-4	21.9	12.8	TS-12	8.0	2.5
S12B-41	15.3	6.6	TS-13	7.4	3.4
S12B-43	12.1	3.7	TS-14	6.0	2.3
S12B-44	12.4	0.0	TS-15	6.6	2.2
S12B-5	8.6	0.0	TS-16	6.2	2.0
S12B-x	16.9	6.6	TS-17	19.6	6.5
S11G-11	14.3	6.2	TS-18	23.6	13.0
S11G-12	11.7	5.3			
S11G-14	14.5	3.0			
S11G-2	11.5	4.4			



RATE OF COLLAPSE IN A CONSOLIDOMETER

FIGURE 31



CONSOLIDATION TEST

FIGURE 32

show the effect of water on a dry soil.

Soil Profiles

The chemical analyses of the A and B horizon soils of the subject areas (Table 8) did not indicate extensive leaching of soil minerals from the unstable silts and their redeposition in lower horizons. The clay content increases in all samples from 15% at the top to 15% at the bottom of the deposit. As was to be expected, the liquid limit and the plasticity index increased as the clay content increased. Tests for pH indicated acidic (4.5) soil for samples S11G, S12B, and TS17 and neutral (6.8) for samples S10B and TS7. The organic content of all samples was less than 1.5%. Leaching of extractable nutrients has been more severe in stable silt S11G because larger amounts are found at greater depth. Acidity and alkalinity in soil affect soil particle charges and organisms that degrade organic matter. Certain microorganisms can exist only in a suitable environment; soil that is too acidic or basic will not support microbes. Jackson (1968, p. 42) points out that higher pH values are obtained when the ratio of water to soil is increased. A soil/water ratio of 1:1 by weight was used in the pH determination.

Samples from Lafayette Area, Louisiana

All of the Lafayette samples, except the one identified as "Mile Three," exhibited the same disturbed soil characteristics as unstable samples S10B and S12B (Table 9); i.e., they turned the Calgon-water solution black, had a maximum standard compaction density less than 104 lbs per cu ft, and had in-place unit weights below 80 lbs per cu ft.

A linear relationship exists between the maximum compaction density and optimum moisture content of the Lafayette samples (Figure 33). This relationship is thought to exist as the result of the elctrochemical forces which hold moisture in the silts. Stronger forces cause a looser structure thereby lowering the maximum compaction density.

TABLE 8

SOIL PROFILE SURVEY RESULTS*

Sample	Depth (in)	Grain Size			Liquid Limit	Plastic Index	pH 1:1 Soil Water	Organic** Material %	Extractable Nutrients (PPM)			
		% Sand	% Silt	% Clay					P	K	Ca	Mg
S12B	0-14	14	71	15	23	0	6.2	1.5	20	115	880	116
	14-20	5	67	28	24	9	4.5	0.6	5	25	240	172
	20-26	9	55	36	31	17	4.4	0.6	5	40	480	325
	26-32	10	51	39	40	25	4.7	0.3	5	75	680	425
	32-38	12	55	33	37	25	5.7	0.1	5	70	800	491
	38-44	14	53	33	40	27	6.2	0.0	5	80	880	589
S11G	0-8	19	68	13	20	0	4.5	1.2	12	90	210	56
	8-14	12	61	27	26	12	4.1	0.4	5	50	160	121
	14-20	7	44	49	47	29	4.2	0.4	5	90	680	306
	20-26	5	43	52	52	33	4.3	0.4	5	120	1200	428
	26-32	5	43	52	52	34	4.4	0.4	5	140	1520	498
	32-38	4	41	55	57	40	4.8	0.4	5	150	2000	576

*Chemical tests performed by LSU Department of Agronomy Laboratories

**By acid dissolution-approximately 2/3 of value by combustion

TABLE 8

SOIL PROFILE SURVEY RESULTS* (CONTINUED)

Sample	Depth (in)	Grain Size % Sand	% Silt	% Clay	Liquid Limit	Plastic Index	pH 1:1 Soil Water	Organic** Material%	Extractable Nutrients (PPM) P	K	Ca	Mg
S10B	22-2B	6	65	29	27	12	6.0	0.6	5	45	880	548
	28-34	6	61	33	28	15	6.7	0.2	5	45	720	608
	24-40	6	56	38	35	21	6.9	0.2	5	50	800	713
	40-46	5	53	42	46	33	7.1	0.1	5	70	880	880
	46-52	4	56	40	40	26	7.1	0.1	5	60	840	781
TS 7	9-15	14	49	37	40	25	6.5	0.8	5	110	1760	298
	15-21	11	43	46	56	40	6.4	0.4	5	140	2000	490
	21-27	12	45	43	51	36	6.5	0.2	5	120	1800	530
	27-33	12	47	41	44	30	6.3	0.1	5	100	1680	523
	33-39	13	47	40	41	29	6.6	0.1	5	100	1600	484
TS 17	20-26	17	65	18	21	5	4.2	0.6	5	20	120	35
	26-32	16	63	21	21	5	4.2	0.3	5	25	120	56
	32-38	15	60	25	24	9	4.4	0.2	5	30	120	96
TS 17	38-44	12	55	33	36	22	4.5	0.2	5	45	160	250
	44-50	13	50	37	40	23	4.6	0.3	5	60	320	334

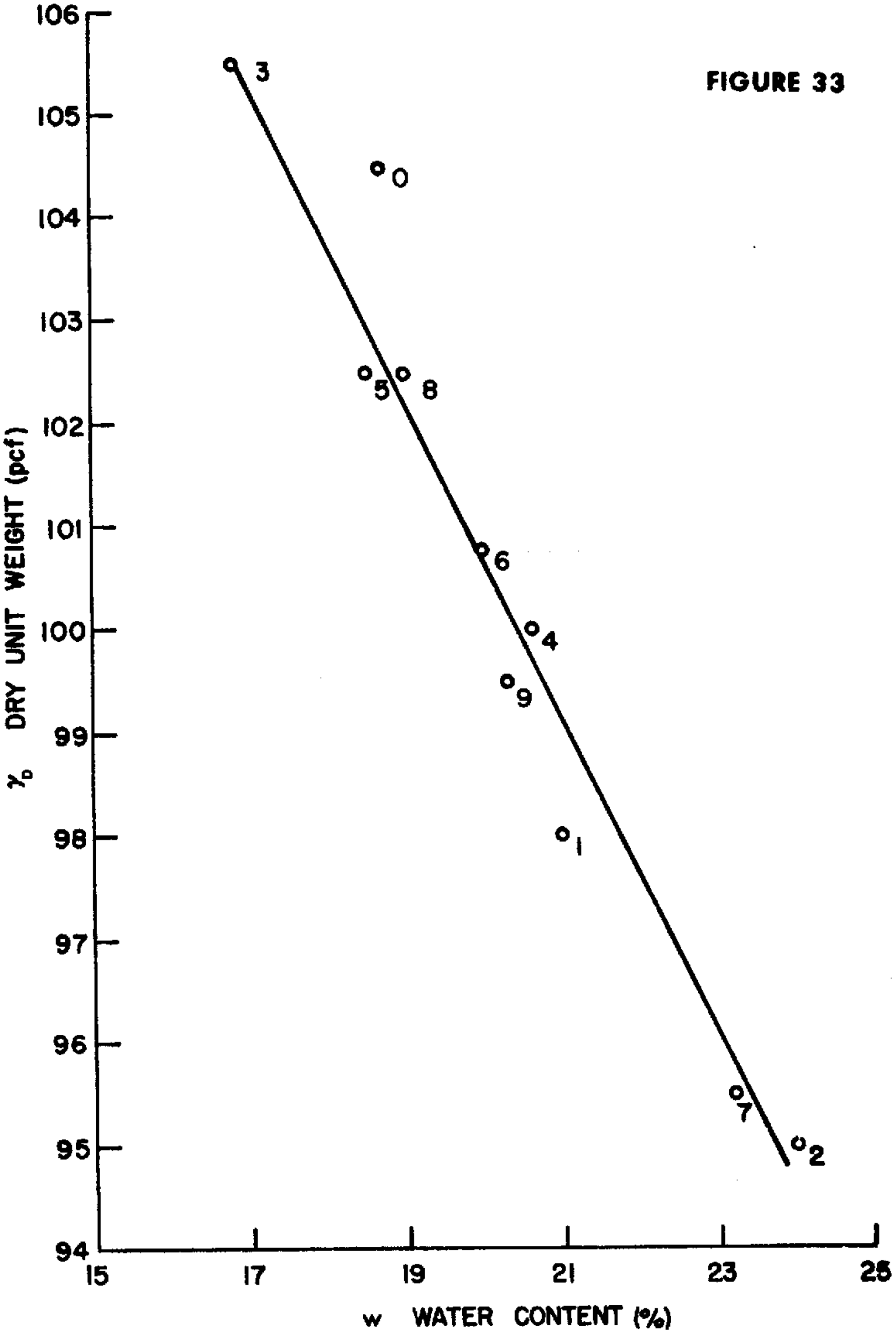
*Chemical tests performed by LSU Department of Agronomy Laboratories

**By Acid dissolution-approximately 2/3 of value by combustion

TABLE 9

SUMMARY OF LAFAYETTE RESULTS

	Std. Compaction			
Sample	$\gamma_{D max}$ lbs/cu ft	w_{opt} (%)	Calgon Test Color	In-Place Unit Weight lbs/cu ft
Mile 0	103.5	18.7	Black	
Mile 1	98.0	21.0	Black	68
Mile 2	95.0	24.0	Black (Muddy)	63
Mile 3	105.5	16.8	Brown	69
Mile 4	100.0	20.6	Black	
Mile 5	102.5	18.5	Black	
Mile 6	100.8	20.0	Black	63
Mile 7	95.5	23.2	Black	76
Mile 8	102.5	19.0	Black (Muddy Near BTM)	75
Mile 9	99.5	20.3	Black	66



STANDARD PROCTOR TEST — LAFAYETTE SOILS

Samples from New Iberia, Louisiana

Samples obtained at the New Iberia sites exhibited several properties common to unstable silts. Standard compaction densities were below 103 lbs per cu ft for all ten sites (Table 10A). A linear relationship exists between the maximum compacted unit weight and the water content at which it occurs (Figure 34). All the samples subsided in excess of 7% during the collapse test between 0 and 16 tons per sq ft (Table 10B).

All the sites were disturbed as the result of extensive sugarcane farming. Because of this, all in-place unit weights were between 86 and 96 lbs per cu ft (Table 10A). Sample NI15 was the only sample which turned black in the Calgon test (Table 11).

Calgon Liquor Extract

An infra-red spectrograph (Figure 35) of the dry Calgon liquor salts of S12B showed five areas of minimum transmittance (3400, 2900, 2820, 2350, and 1510 cm^{-1}) that are common to lignin (Brauns, 1952, p. 222). The peak at 1625 cm^{-1} (Figure 35) is characteristic of Calgon.

A second infra-red spectrograph of S12B (Figure 36), which had the liquid phase extracted with chloroform, showed three peaks at 2955, 2920, and 2860 cm^{-1} that are characteristic of cellulose and lignin products.

Results from the chemical, X-ray diffraction, and differential thermal analyses of the Calgon liquor salts from S12B (because they were not instrumental in identification) are included in Appendix E. The chemical analysis showed that the Calgon liquor contained materials normally found in Calgon and clays.

Alexander (1961) reports the following about lignin:

> Natural materials rich in lignin are less readily utilized by microorganisms than lignin poor products. It is not uncommon to find that the rate of decay of plant debris is proportional to their content of lignin (1961, p. 154)... The rate and extent of lignin decomposition are affected by temperature availability of nitrogen, anaerobiasis, and by constituents of the plant residue undergoing decay (1961, p. 201).

TABLE 10A

SUMMARY OF TEST RESULTS OF SOILS OBTAINED FROM NEW IBERIA

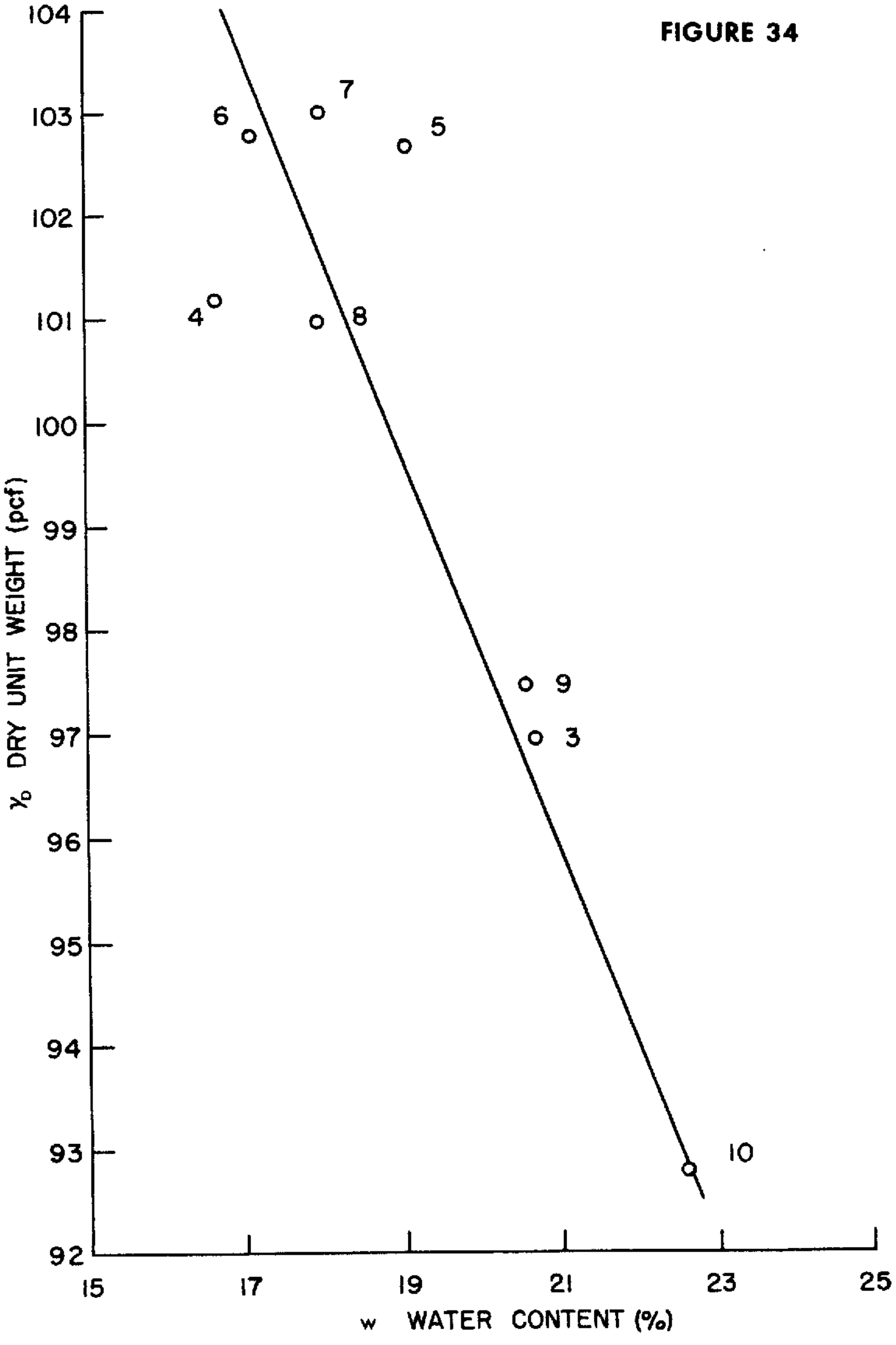
Sample	Standard Compaction $\gamma_{d)MAX}$ lbs/cu ft	Standard Compaction w% OPT	In-Place Density lbs/cu ft	In-Place Density w%	Grain Size Silt % .005 to .070mm	Grain Size Clay % .005mm
NI 1			96	22.6		
NI 2						
NI 3	97.0	20.7	89	28.6	67	32
NI 4	101.2	16.7	91	26.0	84	15
NI 5	102.7	19.1	86	19.5	74	25
NI 6	102.8	17.1	90	16.7	78	17
NI 7	103.0	18.0				
NI 8	101.0	18.0			74	24
NI 9	97.5	20.6	90	28.5	66	33
NI 10	92.8	22.6	91	25.5	62	36

TABLE 10B

SUMMARY OF TEST RESULTS OF SOILS OBTAINED FROM NEW IBERIA

Sample	Collapse % Strain in per in		Calgon Test
	0-16T	8T-8R	Color
NI 1	15.5	7.4	
NI 2	9.5	4.7	
NI 3	13.5	10.0	Dark Brown
NI 4	7.2	4.2	Black
NI 5	14.8	9.9	Brown
NI 6	17.0	10.7	Very Dark Brown
NI 7			Very Dark Brown
NI 8	11.5	5.9	Very Dark Brown
NI 9	15.4	4.5	Very Dark Brown
NI 10	12.1	4.6	Dark Brown

FIGURE 34



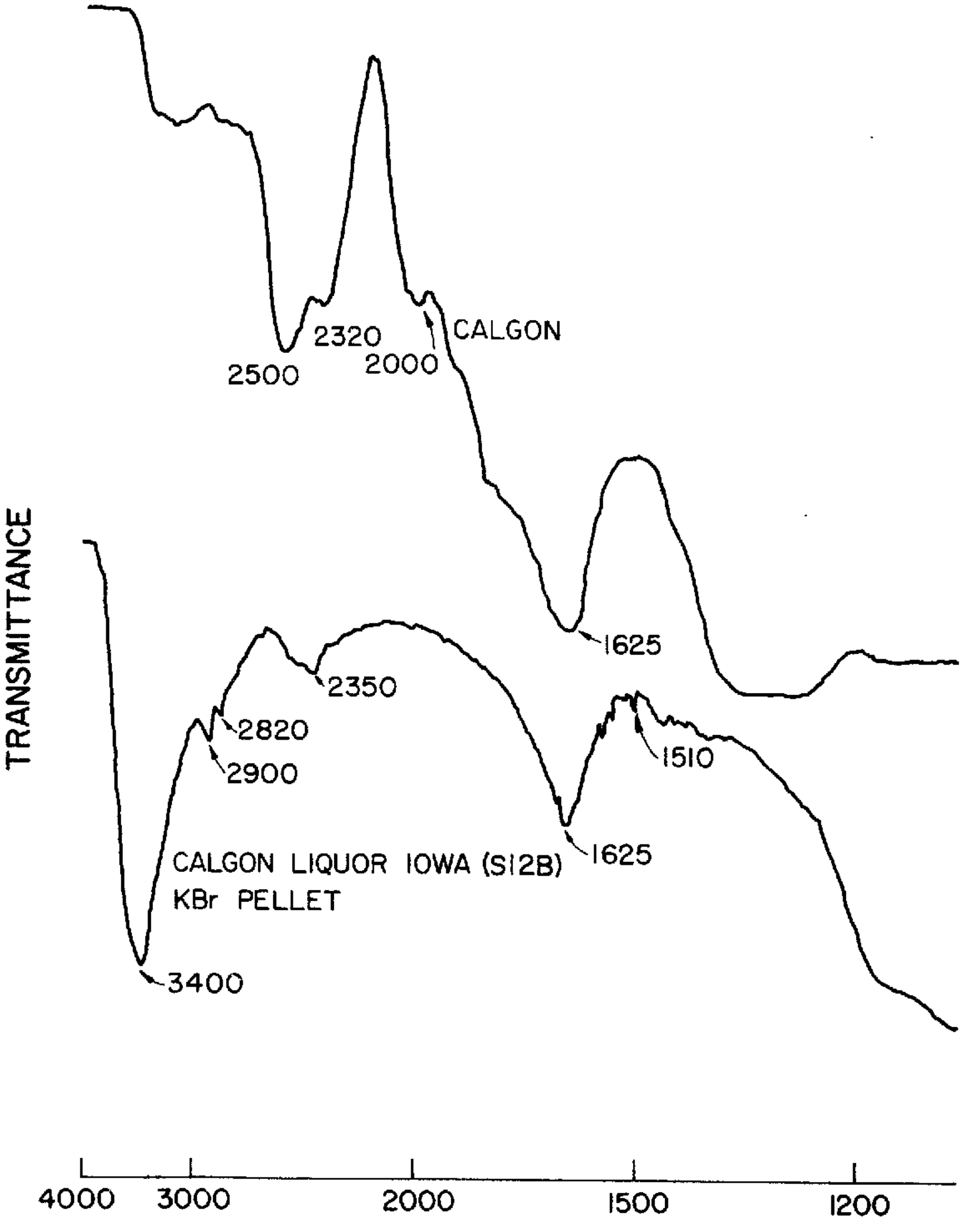
STANDARD PROCTOR TEST — NEW IBERIA SOILS

TABLE 11

SUMMARY OF "CALGON WASHED" LIQUID LIMIT RESULTS

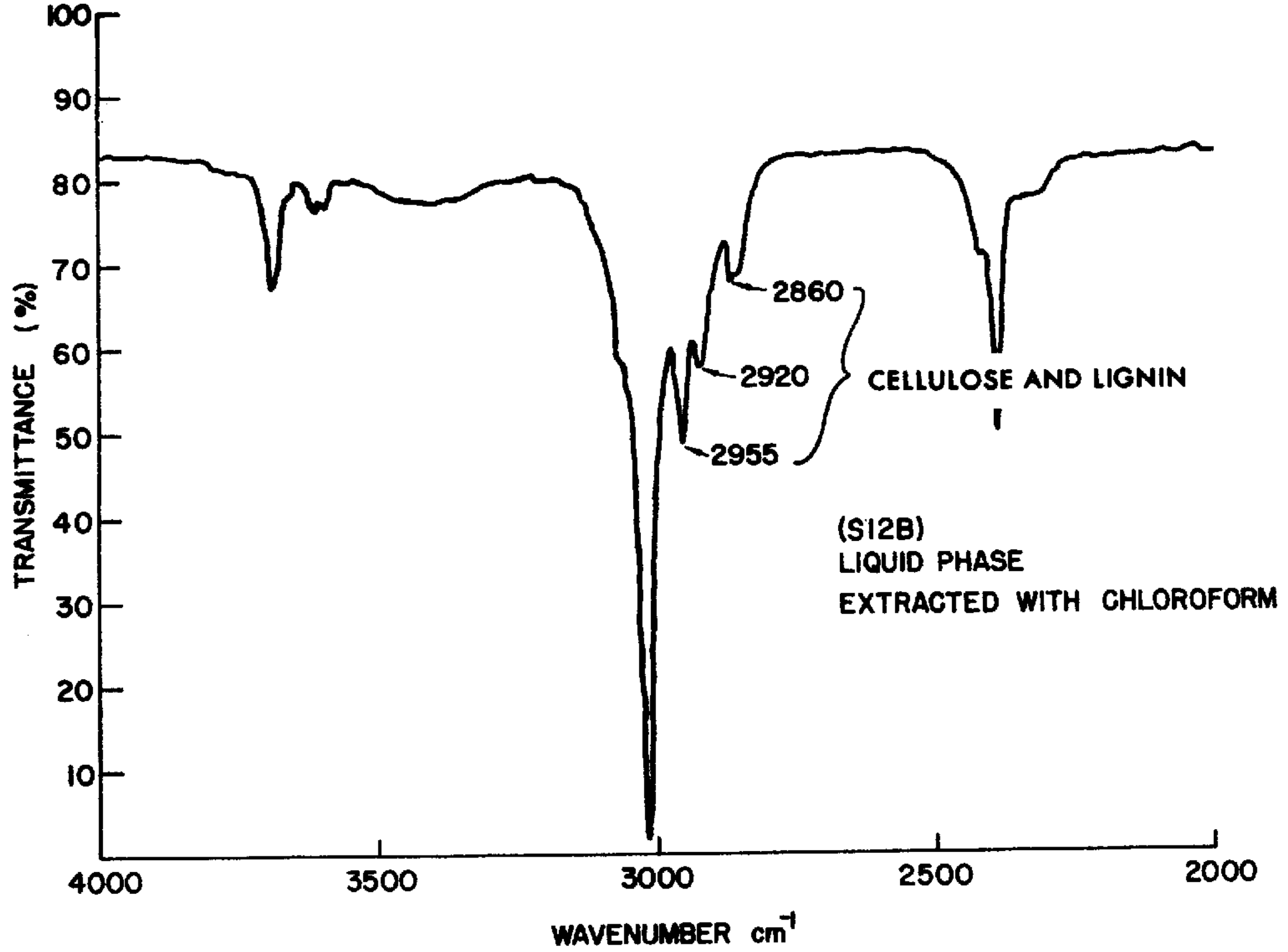
Sample	Liquid Limit	Calgon Washed Liquid Limit	Change in Liquid Limit
S11G	20	20*	0
S10B	32	23*	-9
S12B	23	19*	-4
Lafayette Mile 0	33	28	-5
Lafayette Mile 1	36	38	2
Lafayette Mile 2	45	40	-5
Lafayette Mile 3	31	27	-4
Lafayette Mile 4	37	29	-8
Lafayette Mile 5	31	31	0
Lafayette Mile 6	32	31	-1
Lafayette Mile 7	40	43	3
Lafayette Mile 8	32	24	-8
Lafayette Mile 9	30	27	-3
New Iberia TS 3	41	41	-1
New Iberia TS 4	29	24	-5
New Iberia TS 5	31	27	-4
New Iberia TS 6	27	24	-3
New Iberia TS 7	35	43	8
New Iberia TS 8	35	41	6
New Iberia TS 9	40	50	10
New Iberia TS 10	47	59	12

*Average of three tests



INFRARED SPECTROGRAPH

FIGURE 35



INFRARED SPECTROGRAPH OF IOWA SILT

FIGURE 36

Properties of Soil Washed with Calgon

"Calgon washing" lowered the liquid limit of unstable silts except when the organic content of the soil, as determined by the combustion method (heating at 440°C for five hours) (Arman, 1969, p. 30), exceeded 4%. Calgon-washed liquid limit tests were run on S11G, S10B, S12B, Lafayette, and New Iberia samples (Table 11). The organic content affected the results by causing the Calgon-washed liquid limit to increase with increasing organic content. A linear relationship exists between the organic content of New Iberia samples and the change in liquid limit due to Calgon washing (Figure 37).

Electro-Osmotic Stabilization

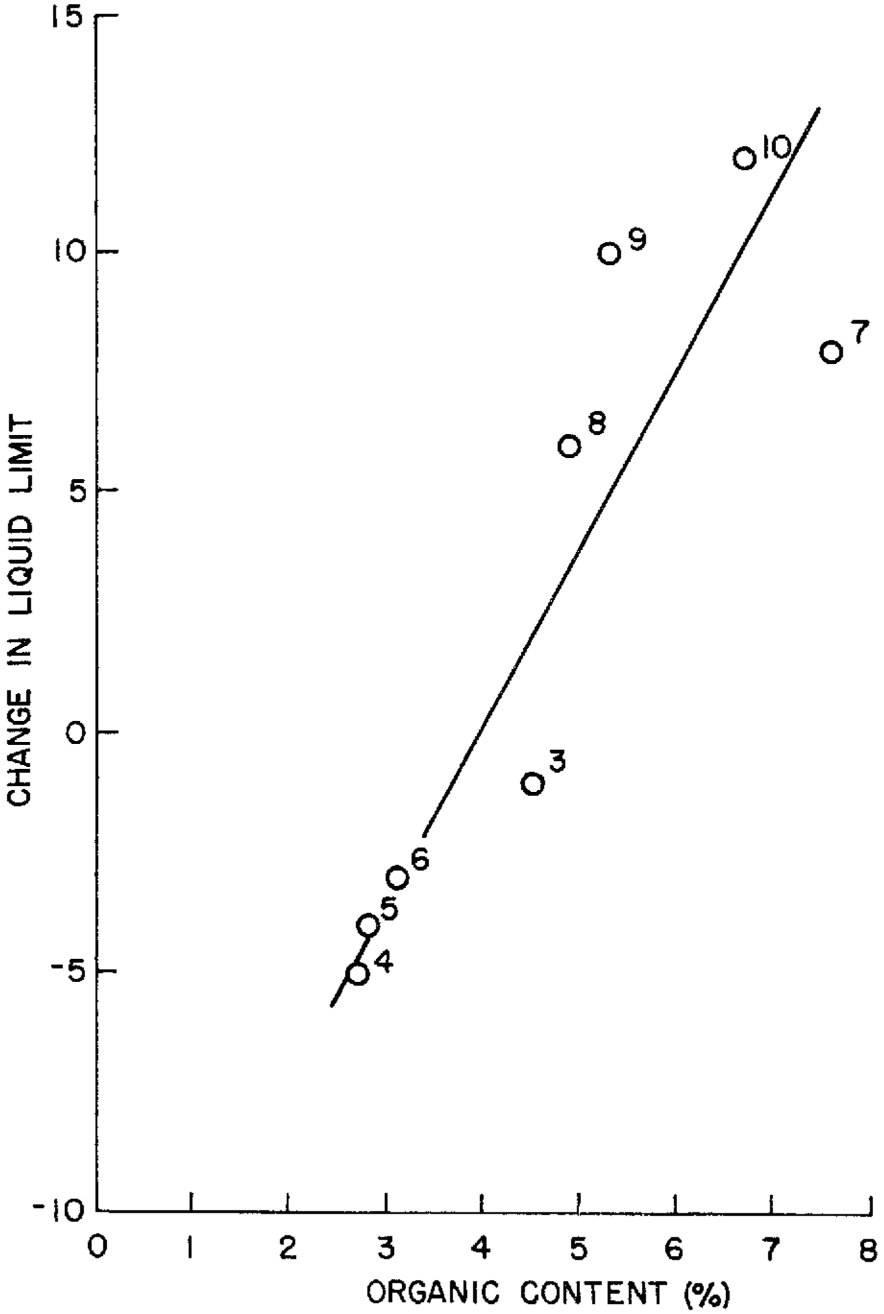
Stabilization of silts by removal of moisture through electro-osmosis was not tried because electro-osmosis is not an effective means of dewatering silt. L. Casagrande reported on his construction experience (1952, p. 77):

> "It is of particular interest to note that during the period of approximately six months during which electro-osmosis was kept in operation on this project, the average water content of the affected soil decreased only about one-half per cent.
> Although electro-osmosis could not be regarded as an economical proposition if applied with the intention of decreasing moisture content of a material to any considerable extent,... such undertaking may still be justified in exceptional cases."

Cement Stabilization

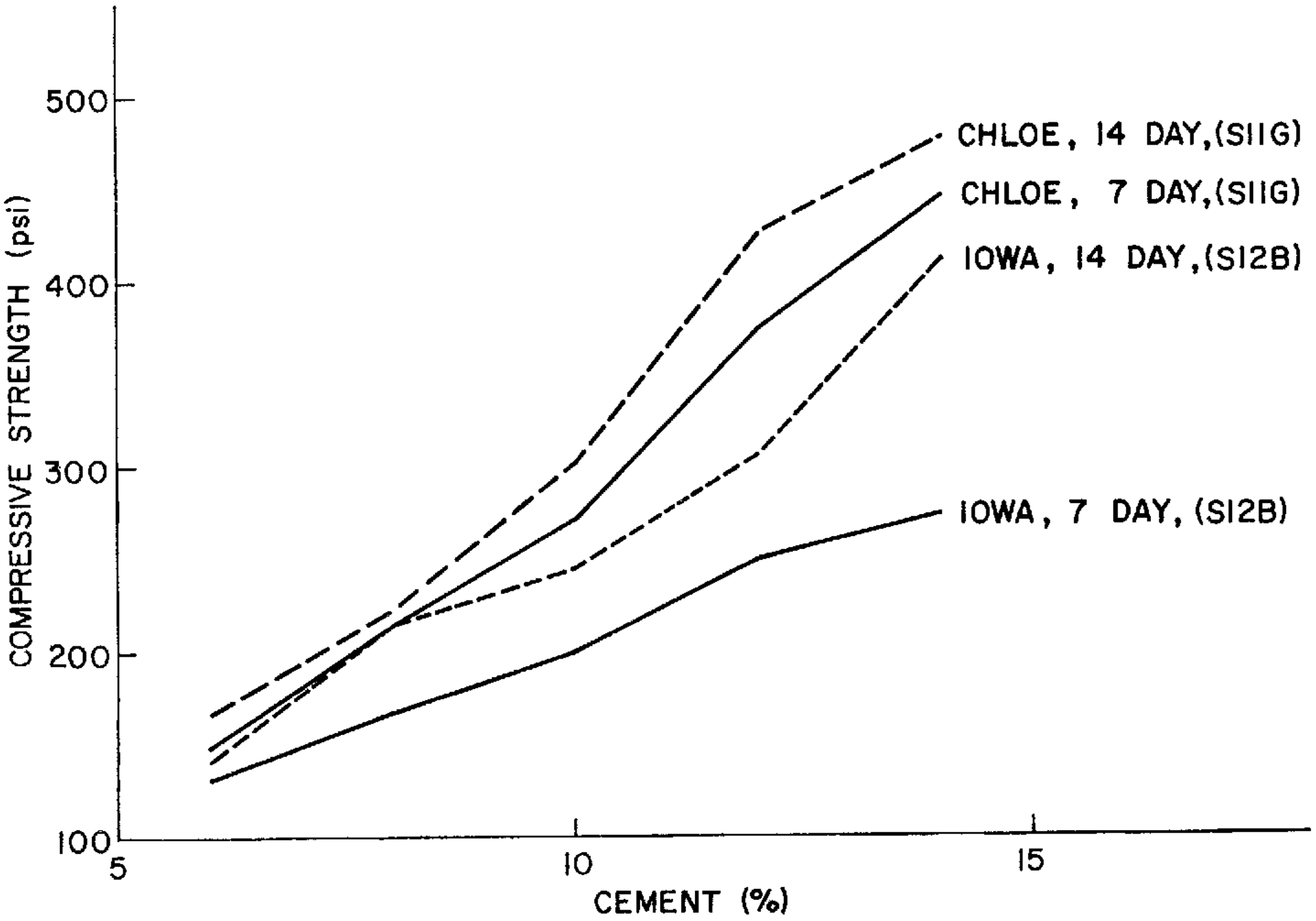
Samples of S12B containing 6% to 14% of Portland cement by weight developed lower unconfined compressive strengths after 7- and 14-day curing periods than specimens of S11G with the same cement content. The mixtures were cured in a 100%-humidity room for 7 days and 14 days, measured and weighed, soaked in water for 4 hours, then broken in compression. Each point on Figure 38 (a plot of compressive strength versus cement content for S11G and S12B) is the average strength of three tests with 10% cement content and a 7-day cure, samples of S12B resisted 200 psi, while S11G samples broke at 270 psi.

Durability, as measured by the brush test, was within the acceptable tolerances of the Portland Cement Association for both S11G and S12B; i.e., no



EFFECT OF CALGON WASHING — NEW IBERIA

FIGURE 37



CEMENT STABILIZATION

FIGURE 38

specimen lost over 10% of its weight (Table 12). Figure 39 shows weight loss versus cement content for two test specimens each of S11G and S12B.

Lime Stabilization

The compressive strength of S11G and S12B samples containing 4% lime and cured for 14 days did not exceed 23 psi (Table 13). Each point on Figure 40 (compressive strength at 3, 7, and 14 days) is an average of three compressive specimens. Sample S11G was slightly stronger than S12B, but the strengths of both were within the range of 18 to 23 psi.

Lime in S11G and S12B had no short-time effect on their liquid limit, but increased their plasticity slightly. (Atterberg limits were determined for samples containing 1, 2, 3, and 4% lime by weight, with curing periods of 2, 3, and 7 days.) Table 14 summarizes the effect of lime on the Atterberg limits (ASTM D 423 and D 424).

The pH of S11G-lime and S12B-lime mixtures (LDH TR 430-67) did not vary with time. However, it increased with the lime content to a maximum pH of 12.2. Table 15 lists pH values for S11G and S12B at 0 to 4% lime content for curing times of one hour to seven days. The pH of samples S11G and S12B increased to 11.7 when 1% lime was added and to 12.2 when 2% or more lime was added.

The addition of lime to S11G and S12B had little effect on standard compaction results. The mixtures, with moisture contents 5% below those for the plastic limit, were cured for periods of 2, 3, and 7 days (Table 16). Small variations in the maximum unit weight may have occurred as a result of faulty testing technique.

Lime in samples S11G and S12B increased the number of sand-size particles and reduced the numbers of both the clay- and silt-size particles. Table 17 lists the grain sizes of samples S11G and S12B for several lime contents and curing times. It should be noted that at 4% lime content, the sand-size particles comprise 40% of the sample; the clay-size particles have been reduced from 15% to less than 4%. Increases in curing time reduce the amount of clay-size particles when lime is added.

TABLE 12

WEIGHT LOSS FROM BRUSH TEST OF SOIL-CEMENT MIXES

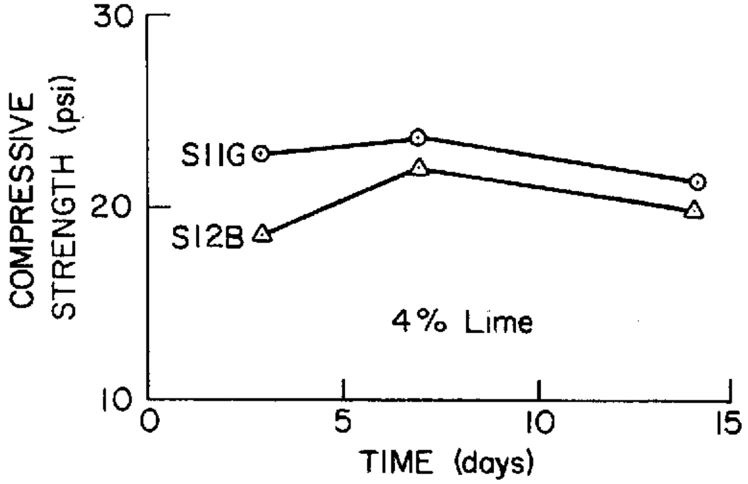
	Cement Content				
Sample	6%	8%	10%	12%	14%
	Percent Weight Loss				
S12B	5.7	5.7	6.2	3.8	1.8
S12B	5.9	5.9	3.0	1.8	1.8
S11G	9.5	7.4	3.1	1.9	1.7
S11G	6.7	3.6	3.9	2.2	3.1

TABLE 13

COMPRESSIVE STRENGTH* OF SOIL-LIME MIXTURES IN PSI (4% LIME)

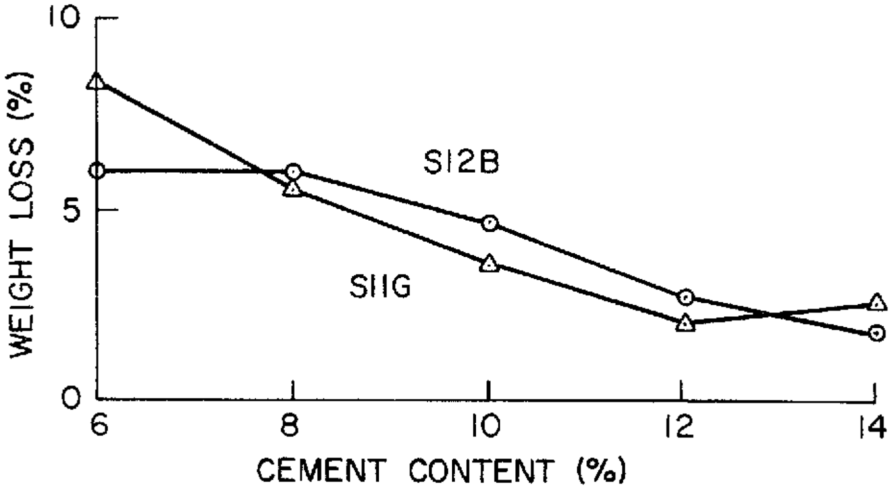
	Curing Time		
Sample	72 Hours	7 Days	14 Days
S11G (Chloe)	22.3	23.1	20.7
S12B (Iowa)	18.0	21.5	19.4

*Average of three specimens per data point



LIME STABILIZATION

FIGURE 40



BRUSH TEST

FIGURE 39

TABLE 14

ATTERBERG LIMITS OF SOIL-LIME MIXTURES

	48 Hours		72 Hours		7 Days	
Lime Content	Liquid Limit	Plasticity Index	Liquid Limit	Plasticity Index	Liquid Limit	Plasticity Index
Sample S11G (Chloe)						
0%	20	0	20	0	20	0
1%	19	1	19	1	19	2
2%	20	1	20	3	21	4
3%	19	2	20	2	21	3
4%	20	3	22	2	19	0
Sample S12B (Iowa)						
0%	23	0	23	0	23	0
1%	21	1	22	3	22	3
2%	20	0	23	3	24	4
3%	21	1			23	4
4%	21	2	23	4	23	4

TABLE 15

pH* OF SOIL-LIME MIXTURES

Sample S11G (Chloe Silt)

Lime Content	Lapsed Time								
	1 Hr	2 Hr	3 Hr	4 Hr	5 Hr	6 Hr	2 Days	3 Days	7 Days
0%	6.5	6.1	6.3	6.2	6.1	6.1	6.1	6.1	6.1
1%	11.8	11.8	11.7	11.7	11.5	11.5	12.1	11.5	11.4
2%	12.2	12.2	12.1	12.1	12.1	12.1	12.1	12.2	12.1
3%	12.2	12.2	12.1	12.2	12.2	12.2	12.1	12.2	12.2
4%	12.2	12.2	12.2	12.2	12.2	12.2	11.6	12.2	12.2

Sample S12B (Iowa Silt)

Lime Content	1 Hr	2 Hr	3 Hr	4 Hr	5 Hr	6 Hr	2 Days	3 Days	7 Days
0%	7.3	7.3	7.3	7.4	7.3	7.3	7.3	7.3	7.3
1%	11.9	11.8	11.8	11.7	11.7	11.7	12.1	11.8	11.6
2%	12.2	12.2	12.1	12.1	12.1	12.1	12.1	12.2	12.1
3%	12.2	12.1	12.2	12.2	12.2	12.2	12.1	12.2	12.2
4%	12.2	12.2	12.2	12.2	12.2	12.2	11.7	12.2	12.2

*Soil to water ratio 1:5 by weight

TABLE 16

COMPACTION RESULTS WITH LIME

Lime Content	Slaking Time 48 Hours lbs/cu ft	48 Hours w %	72 Hours lbs/cu ft	72 Hours w %	7 Days lbs/cu ft	7 Days w %
Sample S11G (Chloe)						
0	106.4	14.3	106.4	14.3	106.4	14.3
1%	106.2	14.1	105.5	14.9	106.4	14.5
2%	105.4	15.0	104.6	15.0	105.8	15.2
3%	106.2	14.1	104.8	14.4	104.7	14.9
4%	104.7	15.2	102.9	15.6	102.6	16.1
Sample S12B (Iowa)						
0	102.6	15.4	102.6	15.4	102.6	15.4
1%	104.5	14.8	105.2	14.6	101.7	18.1
2%	104.5	15.3	105.2	14.6	102.8	15.6
3%	104.0	15.7	104.5	14.7	102.9	16.0
4%	101.3	16.9	104.3	15.9	103.4	16.0

TABLE 17

GRAIN SIZE ANALYSES OF SOIL-LIME MIXTURES

Sample S11G (Chloe)

	Slaking Time								
	48 Hours			72 Hours			7 Days		
Lime Content	% Sand	% Silt	% Clay	% Sand	% Silt	% Clay	% Sand	% Silt	% Clay
0	19	68	13	19	68	13	19	68	13
1%	28	59	13	21	69	10	23	65	12
2%	28	66	6	40	59	1	38	59	3
3%	34	60	6	42	58	0	36	64	0
4%	41	56	3	43	55	2	40	60	0
Sample S12B (Iowa)									
0	14	71	15	14	71	15	14	71	15
1%	23	64	13	26	66	8	26	63	11
2%	26	67	7	35	63	2	32	67	1
3%	40	55	5	35	63	2	43	55	2
4%	37	59	4	39	57	4	45	52	3

VI. DISCUSSION OF TEST RESULTS

Suspended Moisture

Early permeability and drainage tests showed that silts hold moisture suspended in their pores. Tests with polar and non-polar fluids indicated that electro-chemical charges may be responsible. Sampling disturbance further reduced the permeability. The effect was best demonstrated in the permeameter--(2-meter head) after glycol (a polar liquid) was mixed with sample S12B. Disturbance of the soil brings more polar liquid into contact with the electro-chemical forces, thus allowing it to seal the pores.

Loose Structure

In-place dry unit weights were less for unstable silts than for stable silts. Thus the unstable silts have a looser structure with greater potential for subsidence. Clevenger (1956, p. 2) reported similar weights and grain sizes for collapsible soils.

The surcharge compression tests (Figure 17) indicated that sample S11G (stable silt) became denser than S10B and S12B at all applied stresses after remolding. The unit weights of S10B, S11G, and S12B, respectively (perhaps by coincidence), are the same as their in-place unit weights (Table 5). Sample S11G is more stable than S10B and S12B because its structure is denser. An increase of surcharge stress by 300% (from 400 to 1,200 lbs per sq ft) increased the density of the samples less than 5%, or 4 lbs per cu ft.

The tendency for the unstable silts to retain their low relative densities (large void ratios) explains why they remain a construction problem even after reworking by farming or construction activity. Most criteria for collapsible soils (Clevenger, Denison, Gibbs and Bara, Soviet Code, Feda, Gibbs, Kassiff, and Henkin) are based on, or include, a measure of the voids.

Standard compaction tests proved, most outstandingly, the difference in mechanical properties of the remolded silts. The higher densities obtained with S11G confirm that this soil, which was used as fill material on Interstate 10 Highway, east of Lake Charles, is more stable than the others. The maximum densities obtained by standard compaction occur with higher moisture contents in the unstable silts. The higher optimum moisture contents obtained from standard compaction tests indicate that moisture-holding forces are responsible for the looser soil structure. The strength of these forces may be responsible for the linear relationships found between density and moisture content of the Lafayette and New Iberia soils (Figure 33 and 34).

The Calgon Reaction

The Calgon test proved to be very practical and important because of its easy application for identifying unstable silts. In preparation for the hydrometer analysis (a routine soil grain size test), the color of the Calgon-soaked soil can be noted before it is placed in a 1000 ml graduate.

The analyses of the Calgon liquor did not conclusively identify the material in it. Infrared spectrographs indicated the presence of lignin, but more and better defined peaks are necessary for positive identification. Better infrared spectrographs could not be obtained because the black liquid reduced the light transmission. No reports of X-ray diffraction or differential thermal analysis of lignin could be found in the literature. Results of these tests are included in Appendix E in the hope that some future researcher can identify the material.

In samples containing little organic material--such as S12B, and S10B--the liquid limit was lower after the Calgon-Washed test and approached the liquid limit of stable sample S11G (Table 11). The high organic content in all samples probably obscured this effect because the organic material is known to cause an increase in the liquid limit. The liquid limit increased linearly with organic content in the New Iberia samples (Figure 37).

The Role of Clay Content

The presence of the swelling clay mineral, montmorillonite, in unstable samples S12B and S10B explains in part their undesirable engineering characteristics. Stable sample S11G contains vermiculite, also a swelling clay mineral, though not so active. The lattice structure of montmorillonite expands to 100Å or more in water, whereas vermiculite only expands through the thickness of a few molecular layers (about 10Å) (Grim, 1968, p. 252). The difference in the swelling properties of their clay minerals is one reason unstable silts collapse.

Collapse Test

The collapse test proved to be a useful indicator of the relative stability of a silt. Maximum values of collapse (.236 and .145 in per in for S12B and S11G, respectively, under 16 tons per sq ft pressure (Figure 30) show that unstable silts may subside twice as much as stable ones. The maximum collapse values should be representative of field conditions because of sample disturbance.

Electron Micrographs

Electron micrographs of samples S11G and S12B (Figures 19, 20, and 21) indicate that both stable and unstable silts were formed by the same geologic processes (similar particle shapes, the presence of microdiatoms, or plant structure, and agglomerated soil particles). This conclusion is also seen from their massive structures, (no bedding planes or ordered arrangement of soil particles) and by their occurrence in the former Red and Mississippi River floodplains.

Leaching Tests

The chemical analyses of samples from the soil profile failed to show that a soil binder such as calcium carbonate had been leached out of the unstable silts and redeposited in lower horizons. Leaching of a binder out of the unstable silts that had remained in the stable silt would have provided a reason for the difference in density. Contrarily, it was found that plant-extractable minerals increased with depth more rapidly in stable sample S11G than in unstable samples

S12B, S10B, TS7, and TS17. This fact indicates that the electro-chemical forces have better protected the unstable silts from weathering. The authors hypothesize that leaching out of lignin and other minerals from the unstable silts has not occurred because their moisture does not flow freely.

Identification of Other Unstable Silt Deposits

The usefulness of the identification tests developed during the study was proved by the finding of other unstable silt deposits in the Iowa (Figure 29) and New Iberia (Figure 13) areas. The identification of these deposits in turn confirmed the previously-found differences indicated by tests on disturbed samples (Calgon test and standard compaction) and undisturbed samples (in-place unit weight and collapse). Because of the data reinforcement, collapse criteria for Louisiana silts were developed with more confidence.

Collapse Criteria for Louisiana Silts

On the basis of all the data and analysis, any combination of two of the following common characteristics of collapsible silts can be used for their identification:

1. The in-place unit weight of the undisturbed silt is less than 80 lbs per cu ft
2. The maximum density obtained by standard compaction (ASTM D 698) is less than 104 lbs per cu ft
3. The supernatant liquor of the silt in the stock solution of Calgon used for hydrometer analyses is black after the solids have settled (See Appendix A, "Calgon Test")
4. The sample compresses at least 15% in height under pressure increments from 0 to 16 tons per sq ft in the collapse test

Comparison With Other Criteria

Samples S12B and S11G, selected as the most representative of unstable and stable silts, respectively, were classified according to collapse criteria posited by seven other researchers. Sample S12B would be considered as collapsible by all; sample S11G is classified as collapsible by six.

According to the in-place unit weight criteria (Clevenger, 1956, p. 60), S12B is collapsible; S11G is not. Undisturbed in-place unit weights (dry basis) are less than 80 lbs per cu ft at Iowa and 90 lbs per cu ft at Chloe. Moisture content, which must exceed 20% according to Holtz and Hilf (1961, p. 677), varies between 18% and 26%.

Both S12B and S11G silts are collapsible soils according to Denison's coefficient of subsidence (Sultan, 1969, p. 6). This coefficient, obtained by dividing the void ratio at the liquid limit by the natural void ratio, indicates a highly collapsible soil if its value lies between 0.5 and 0.75. The coefficient of subsidence was 0.60 at Iowa and 0.66 at Chloe.

The voids must be sufficient to contain the moisture of the soil at its liquid limit, according to Gibbs and Bara (1962, p. 234). The void ratio required to contain the average liquid limit of S12B (23.3%) would be 0.60. The actual void ratio is near 1.00. For sample S11G, the required void ratio at the liquid limit (20.0%) would be 0.52. The natural void ratio is 0.79.

Both S12B and S11G samples meet the collapse criterion for loessial soils in the Soviet Building Code (Sultan, 1969, p. 9) because these criteria are less rigid than those discussed above. (Both the Soviet Code and Gibbs and Bara are concerned only with the void ratios at the liquid limit and in the natural state.)

According to Feda (1966, p. 213), granular soils are subject to collapse if their natural porosity exceeds 40%. The porosity of S12B (50%), is well above the 40% limit and thus would be collapsible. S11G, however, has a porosity of only 44%, a borderline case.

The collapse ratio developed by Gibbs (Sultan, 1969, p. 10) indicates that both S12B and S11G are collapsible. (A ratio greater than unity indicates collapse.) S12B has a collapse ratio of 1.65; S11G has a collapse ratio of 1.69.

The criterion proposed by Kassiff and Henkin (1967, p. 16) also indicates that S12B and S11G are both collapsible. Their indicator, the product of dry density and moisture content, must exceed 15. However, it applies only to loess and is sensitive to the moisture content.

Table 18 summarizes the application of collapse criteria to samples S10B, S11G, S12B, and Iowa samples TS1-TS19. The criterion that the undisturbed, in-place unit weight be below 80 lbs per cu ft, proposed by Clevenger, most clearly fits one of the authors' criteria. The problem with it is that undisturbed surface deposits are becoming hard to find. The New Iberia samples, for example, have been disturbed by farming operations.

Stabilization

Sample S12B required more Portland cement than sample S11G to gain the same compressive strength in soil-cement mixtures. The additional cement was required in S12B because lignin is deleterious to cement hydration. Durability was good for both silts. Portland cement, then, is an acceptable stabilizing agent for these soils if it can be incorporated economically into the soil with normal construction equipment.

With lime added as a stabilizing agent, the compressive strengths did not exceed 23 psi in 7 days. Lime, however, continues to react with soil for years--the strength increasing with time. Recent reports from highway engineers indicate that lime was used successfully to stabilize the unstable silts along U. S. Highway 90 (New Iberia soils).

The added lime in samples S12B and S11G had no effect on the liquid limit. Therefore, there was little pozzolanic reaction, or that reaction was slow. Slight increases in the plasticity of these soils are the result of the unreacted lime acting as fines.

The effective grain size of soil-lime mixtures increased with increased curing time, indicating a slow pozzolanic reaction. The increase in grain size accounts for the higher densities in the standard compaction of sample S12B.

TABLE 18

COLLAPSE CRITERIA APPLIED TO LOUISIANA SILTS

S - Stable C - Collapsible N - Not Applicable

Sample	Thornton & Arman	Clevenger	Denison	Gibbs & Bara	Soviet Code	Feda	Gibbs	Kassiff & Henkin
S11G	S	S	C	C	C	C	C	C
S12B	C	C	C	C	C	C	C	C
S10B	C	C	C	C	C	C	C	C
TS 1	S	S	N	S	S	C	C	N
TS 2	S	S	N	C	C	S	C	N
TS 3	S	S	N	C	C	C	C	N
TS 4	S	S	N	S	C	S	S	N
TS 5	S	S	N	C	C	S	C	N
TS 6	S	S	N	C	C	S	C	N
TS 7	S	S	N	C	C	S	C	N
TS 8	S	S	N	C	C	S	C	N
TS 9	S	S	N	C	C	S	C	N
TS10	S	S	N	C	C	S	C	N
TS11	C	S	N	C	C	C	C	N
TS12	S	S	N	C	C	S	C	N
TS13	S	S	N	N	N	N	N	N
TS14	S	S	N	N	N	S	N	N
TS15	S	S	N	C	C	S	C	N
TS16	S	S	N	C	C	S	C	N
TS17	C	C	N	C	C	C	C	N
TS18	C	S	N	C	C	C	C	N
TS19	C	C	N	C	C	C	C	N

VII. CONCLUSIONS

Collapsible silts exist in a band across southwest Louisiana from Lafayette and New Iberia to the Texas border that includes the coastal prairies, the loessal hills, and the Mississippi terraces.

The loose flocculated structure found in the collapsible silts of Louisiana is due to electrochemical forces that attract and hold water to the soil particles. This moisture makes the soil less permeable, harder to drain, and therefore more resistant to weathering.

The requirement that two of the following four criteria be met for identification of a collapsible silt in Louisiana is more applicable than earlier criteria because they are based on engineering experience in Louisiana. Three of them also have the advantage of not requiring undisturbed samples.

1. The in situ unit weight of the undisturbed silt is less than 80 lbs/ft^3
2. The maximum dry density (standard compaction, ASTM D 698) is less than 104 lbs/ft^3
3. After the solids of the suspected soil have settled out in a 3% solution of sodium hexametaphosphate (used in routine hydrometer analyses), the supernatant liquid is black
4. In a collapse test (modified consolidation test of an oven-dried sample that has been saturated under pressure), a total strain of at least 15 percent occurs at the end of the 16 $tons/ft^2$ loading

Collapsible silts in Louisiana can be stabilized with either lime or cement, provided construction equipment can incorporate them into the soil. Additional research is needed on stabilization to determine the long-term effects of lime and to relate the moisture content of the soil to equipment trafficability.

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APPENDIX A

TESTING PROCEDURES

A-1 Calgon Test

A-2 In-Place Unit Weight Determination--Nuclear Method
(La. Dept. of Highways TR:401-67)

A-3 Determination of the pH Value of Soil
(La. Dept. of Highways TR:430-67)

A-4 Determination of Soil Reaction (pH)
(LSU Agronomy Laboratory)

A-5 Soil-Lime Stabilization Tests

CALGON TEST

The "Calgon Test" is a color test developed in this study to distinguish between stable and collapsible silts in southwest Louisiana.

1. Place 100 grams of prepared silt in a 500 ml beaker.
2. Add 300 ml of distilled water containing 9 grams of Calgon (brand name for sodium hexametaphosphate).
3. Observe the color of the supernatant liquid after allowing the sample to settle overnight.

A black color in the liquor (as shown in Figure 18B, p. 44) indicates that the soil is collapsible.

IN-PLACE UNIT WEIGHT DETERMINATION

Nuclear Method

La. Dept. of Hwys. TR:401-67

Scope

This method of testing is intended to determine the density and moisture content of soils, aggregates, stabilized soils, and stabilized aggregates in the natural state or after compaction by counting a proportional number of events occurring as a consequence of the interaction of a radioactive substance with the material to be tested.

Apparatus

1. Scaler--Troxler Model 200B with a maximum counting rate of 25 kilocycles per second, equivalent to a resolution time of 40 microseconds, and associated electronic equipment.
2. Surface Moisture Gauge--Troxler Model 117 (w/source) and Reference Standard. Any Louisiana Department of Highways accepted standard method of determining moisture content may be used in lieu of the surface moisture gauge.
3. Surface Density Gauge--Troxler Model SC--120 (w/source) and Reference Standard.

Procedure

After selection of the test location, an area approximately 30 inches square shall be carefully leveled and smoothed. If necessary, a very thin (1/8 inch or less) sand blanket may be applied to reduce any large air voids.

The density device shall be connected to the scaler and allowed to count continuously for a minimum of three minutes in order to assure a sufficient warm-up of the density device and the scaler. A standard count shall then be run on the reference standard. (A standard count shall consist of the sum of three one minute counts, none of which shall deviate from the average by more than 2.0 times the square root of the average. For subsequent tests on the same job on the same day a one minute count should be run on the standard. If this count is within 2.0 times the square root of the average of the original standard count then the original standard count may be used for the test and no additional warm-up is required.)

A vertical hole shall then be drilled into the test location to the required depth using the modified auger (the steel spike is used where it is difficult or impossible to use the brace and bit). The density device shall be placed on the test location and the probe lowered to the desired depth. The device shall be pulled against the side of the hole firmly seated. Three one minute readings will be taken, none of which shall deviate from the average by more than 2.0 times the square root of the average. The sum of these readings shall be divided by the standard count to give the percent of standard which is then plotted on the calibration chart to obtain the in-place wet density of the material. The density device shall then be disconnected and removed at least 25 feet from the test location.

The moisture device shall then be connected to the scaler and after a three minute warm-up a standard count as described for the density unit shall be obtained on the polyethylene standard. The moisture device is then firmly seated at the test location and three one minute counts are obtained, none of which shall deviate from the average by more than 2.0 times the square root of the average. The sum of the three one minute readings is divided by the standard count, as previously determined, to get the percent of standard. The percent of standard is plotted on the calibration chart and the water content in pounds per cubic foot is read on the ordinate.

DETERMINATION OF THE pH VALUE OF SOIL

La. Dept. of Hwys. TR:430-67

Procedure

1. Prepare the soil in accordance with L.D.H. TR:411, Method A, (Mechanical Analysis).

2. Weight approximately 10.0 ± 0.1 gram of soil into 2 oz. waxed cup or glass beaker.

3. Add 50.0 ± 5 cc of distilled or demineralized water to the soil sample.

4. Stir the sample vigorously and disperse soil well in water. Stir the sample every 15 minutes in order to disperse the soil and make sure all soluble material is in solution.

5. Allow sample to stand for a period of one hour after addition of water. Record the temperature of the mixture and adjust pH meter.

6. Immediately before immersing electrodes into sample, stir and remove glass stirring rod. Place electrodes into solution and gently swirl so as to make good contact between the solution and the electrodes.

7. Allow electrodes to stand for 15 seconds in sample before reading.

8. Read and record the pH value to the nearest tenth of a whole number.

9. Rinse electrodes well while wiping lightly with a soft cloth to remove any film formed on the electrodes.

DETERMINATION OF SOIL REACTION (pH)

L.S.U. AGRONOMY LABORATORIES

Procedure

Approximately 35 grams of air-dried soil are placed into a 4 1/4 oz. paper cup and 35 ml. of distilled water are added with a filamatic vial filler. The force of delivery is sufficient to thoroughly mix the soil and water.

After 24 hours the suspension is agitated with an electric stirrer and the soil reaction (pH) is determined by inserting the electrodes into the suspension. A Leeds and Northrup No. 7401 pH-meter has proved to be very satisfactory for soil analysis. The pH-meter is standardized by using buffer solution adjusted to pH 4.0 and pH 7.0.

SOIL-LIME STABILIZATION TESTS

Unconfined Compression

Samples were molded according to ASTM D 559-57 at a moisture content determined from the standard compaction test. Specimens were cured at 100% humidity for 48 hours, 72 hours, 7 days, and 14 days then tested in compression at a strain rate of .05 inches per minute.

pH of Soil-Lime Mixtures

In this test 10g of soil-lime mix is stirred in a beaker containing 50cc of demineralized water. The pH readings were taken every hour for 6 hours. A second set of pH tests were run on soil-lime mixtures cured at water contents 5% below the plastic limit for 48 hours, 72 hours, and 7 days because this moisture content was near the optimum moisture content for the standard compaction effort and therefore is representative of field conditions. After one hour, a pH test was run on the second set of samples as outlined above.

Grain Size

The effect of lime on grain size was studied by using a modified ASTM test (D 442). Four percent lime by weight was mixed with soil and cured at a water content five percent below the plastic limit for 48 hours, 72 hours, and 7 days. After curing hydrometer analyses were run two ways. First, 100g of soil-lime mixture was corrected for moisture content, and then run according to the ASTM procedure. Second, the cured soil-lime mixture was dried at 110°F then reprepared to pass a 10 mesh screen before running the ASTM test.

APPENDIX B

PIMPLE MOUNDS

PIMPLE MOUNDS

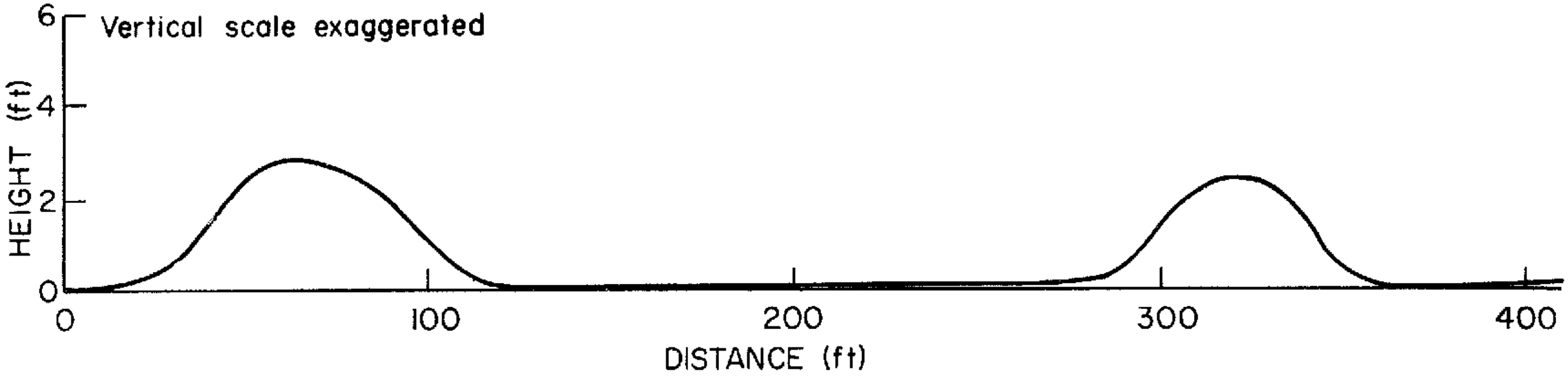
Samples from pimple mounds, small hummocks of silt twenty to eighty feet in diameter and one to three feet high, showed a sudden decrease in volume (collapse) up to 20% under 0-16 tons per sq ft load. Undisturbed and disturbed samples were taken along a cross section (Figure 41) from a site 4 miles east of Lake Charles just off U. S. Highway 90. Pimple mound sites are shown in Figure 29.

Results from the collapse test, standard proctor density, mechanical analysis, Calgon color test, and Calgon washed test are given in Table 19. Collapse tests indicate that the pimple mounds contain collapsing soils, because all six samples subsided between 11.8 and 20.2% of their height when subjected to a 16 ton per sq ft load. The standard proctor results, however, were substantially higher than those of the silts examined previously--the lowest standard compaction density being 106.8 lbs per cu ft. The samples contained 24% to 42% sand, substantially more than the samples examined previously, which contained as little as 1%, but no more than 20% sand. Four of the six samples showed a black color when mixed in a Calgon/water solution. Calgon washing had no effect on samples with low organic content but, unlike other unstable samples, the liquid limit of samples containing 4% organic material was reduced.

TABLE 19

SUMMARY OF TEST RESULTS OF SOILS FROM PIMPLE MOUNDS

	Collapse Test		Std. Compaction		Mech. Analysis			Calgon	Calgon Washed Test			
	% Strain	in/in						Test	Liquid Limit	Liquid Limit		Organic
Sample	0-16T	8T-8R	lbs/cu ft	w%	Sand	Silt	Clay	Color	Before Washing	After Washing	Change	Content
PM 1	20.2	11.3	107.0	14.6	39	52	9	Black	18	18	0	2.4
PM 2	15.0	6.4	106.8	13.6	42	52	6	Black	18	18	0	2.7
PM 3	19.9	15.7	112.4	13.5	33	51	15	D. Gray	17	17	0	1.0
PM 4	11.8	0.7	108.2	13.6	35	55	9	Black	19	17	-2	2.2
PM 5	14.5	5.6	114.2	12.0	24	61	15	Black	21	18	-3	13.5
PM 6	14.4	8.3	114.8	13.0	26	53	21	Gray	21	16	-5	5.0



CROSS SECTION OF PIMPLE MOUNDS

FIGURE 41

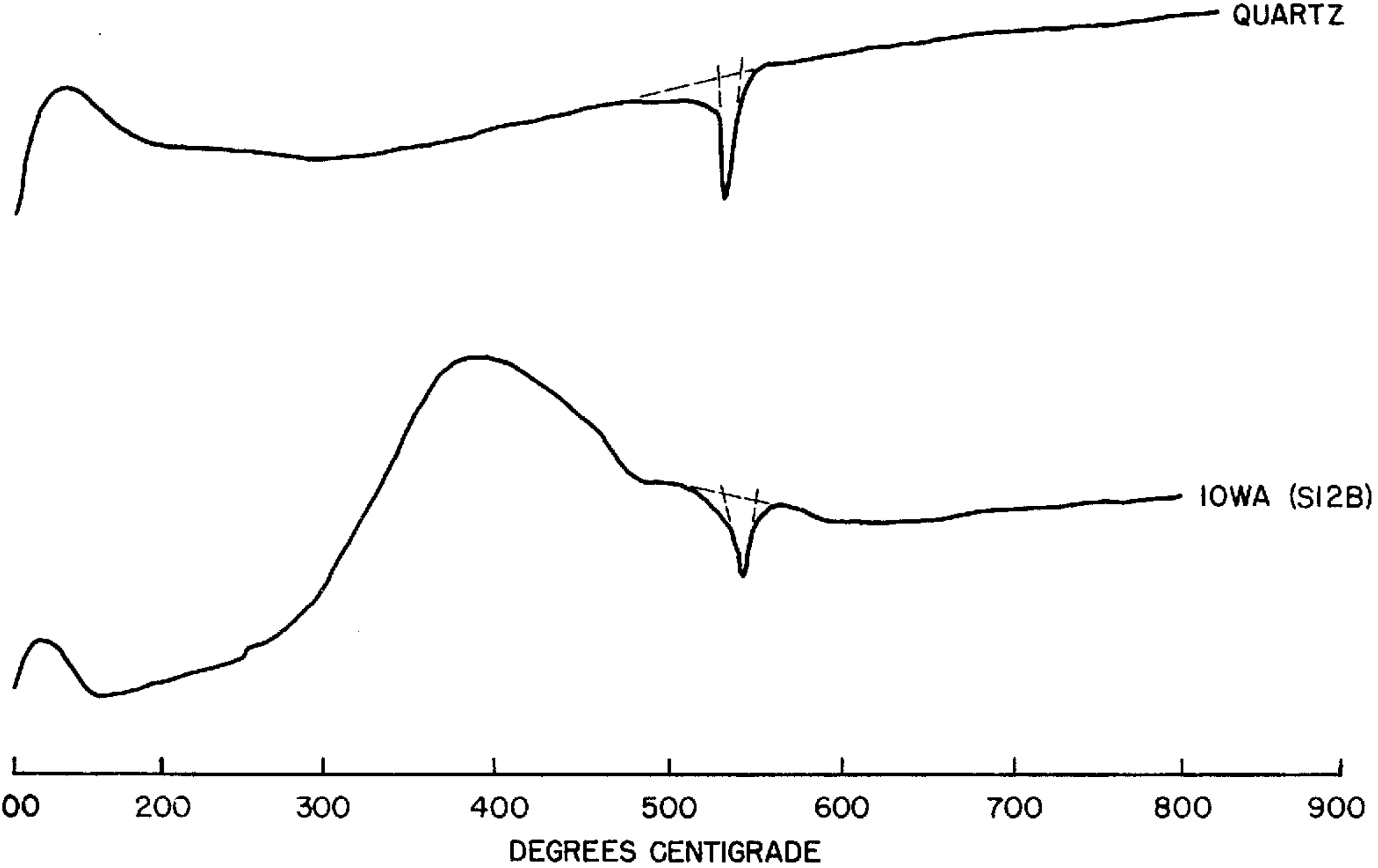
APPENDIX C

QUARTZ CONTENT DETERMINATION

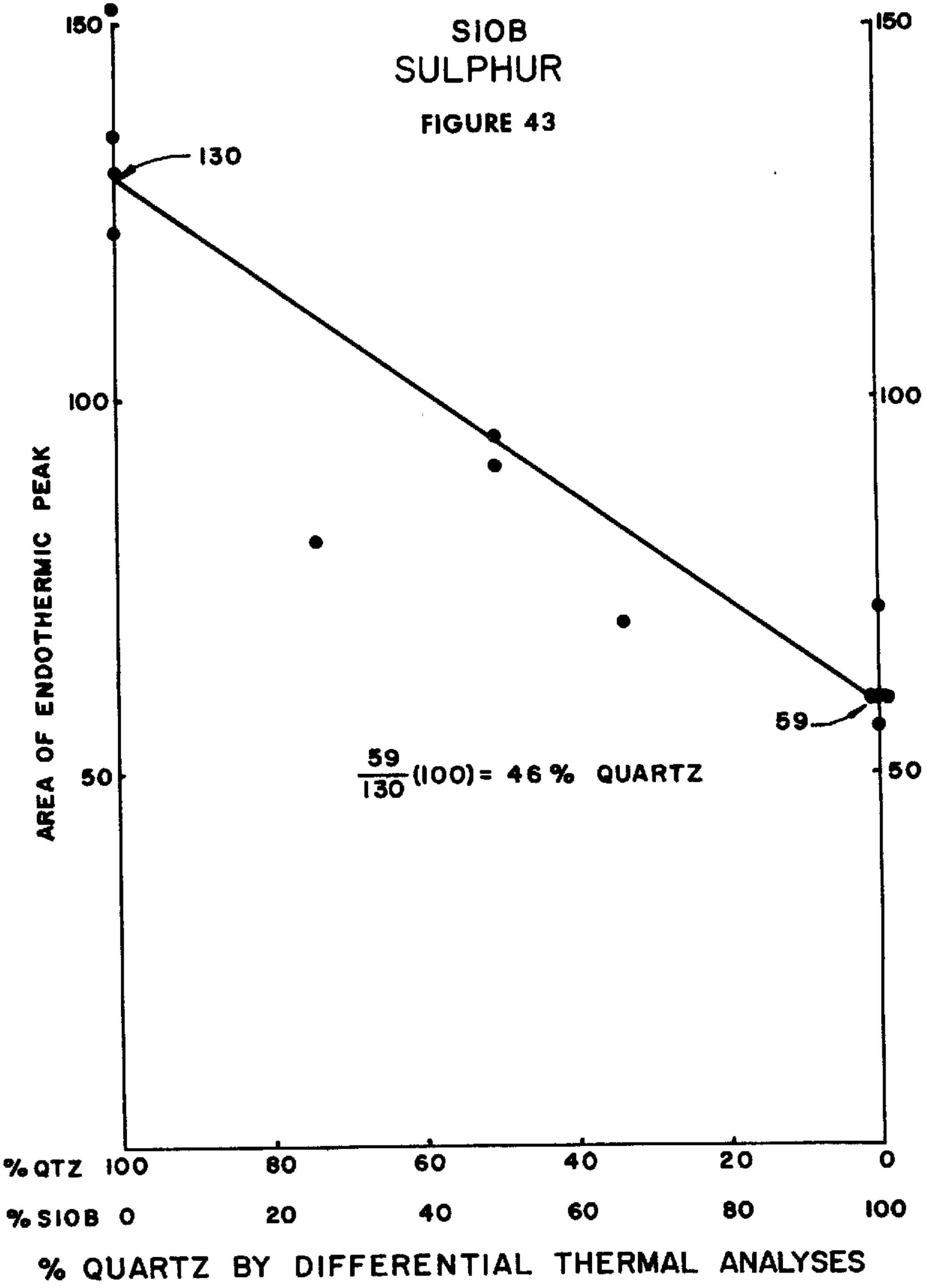
Quantitative analysis of quartz, a stable mineral which is highly resistant to weathering, was made by differential thermal analysis (DTA) using equipment manufactured by Robert L. Stone Co. Areas of endothermic peaks due to alpha quartz inversion were correlated (Smothers and Chiang, 1966, p. 125) for 100% quartz, mixtures of quartz, and 100% silt. A typical DTA curve showing the quartz endothermic peak is shown in Figure 42.

Iowa, Sulphur, and Chloe silts all contain a high percentage of quartz. The unstable silts at Iowa, S12B, and Sulphur, S10B, contain 76% and 46% quartz, respectively. Results of the Iowa (S12B) and Sulphur (S10G) analyses are shown in Figure 43 and 45. Chloe silt (S11G) contains 60% quartz as shown in Figure 44.

FIGURE 42

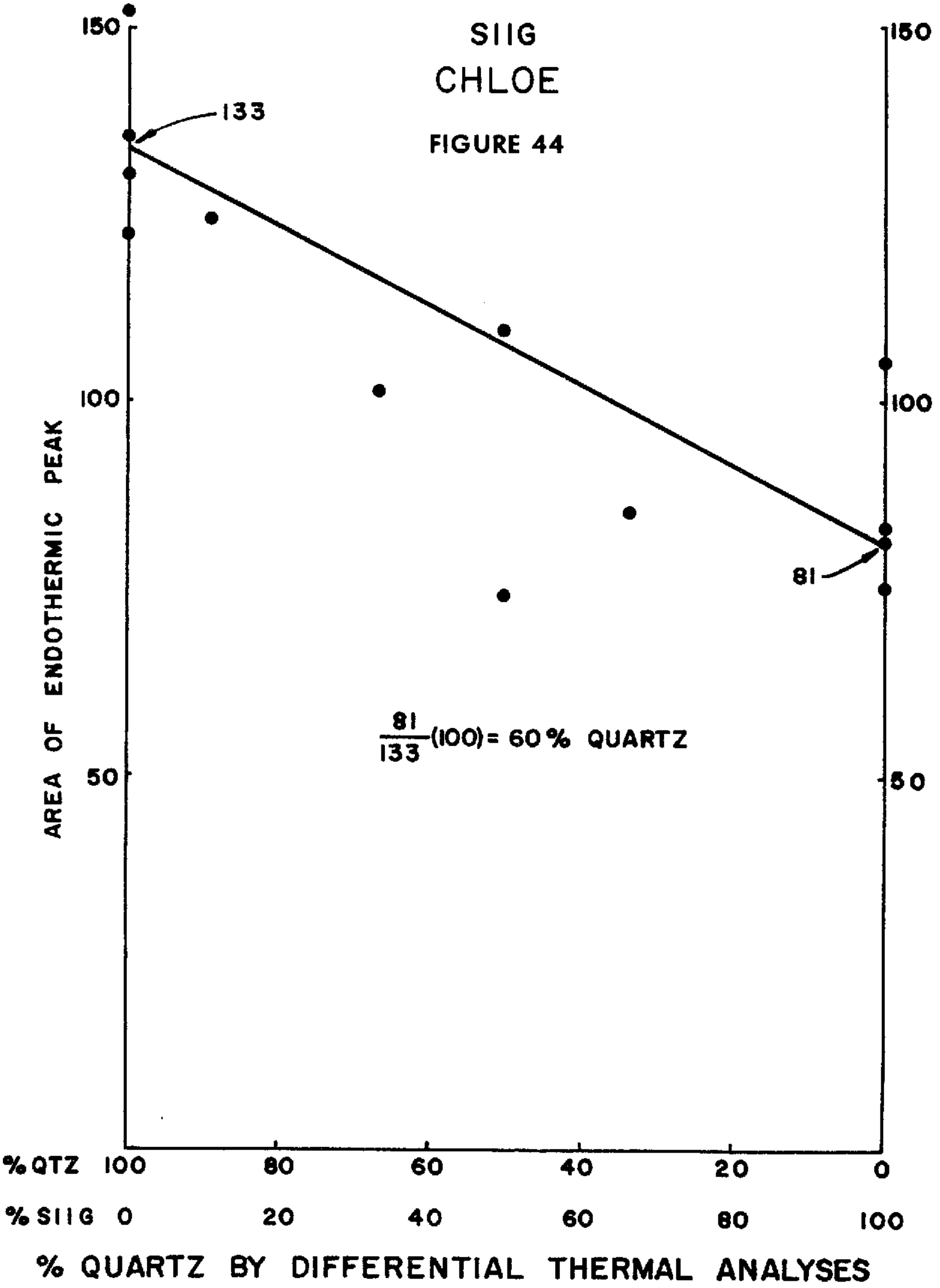


DIFFERENTIAL THERMAL ANALYSIS



SIOB
SULPHUR

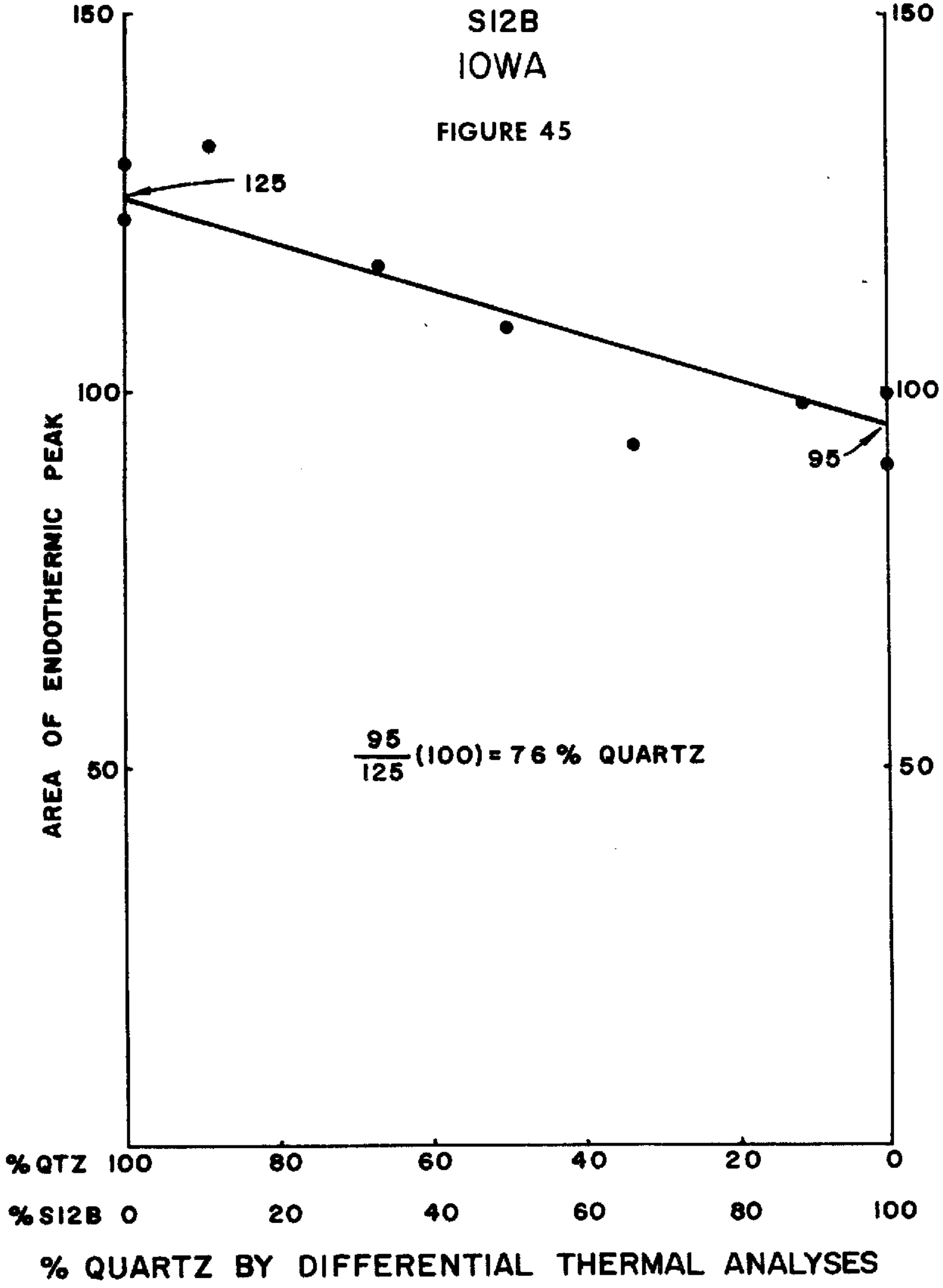
FIGURE 43



S11G

CHLOE

FIGURE 44

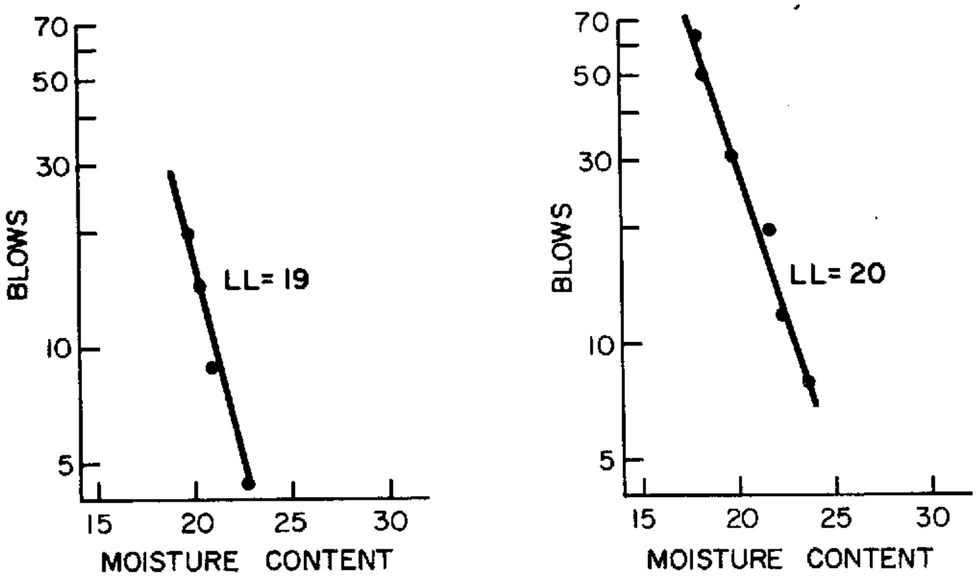


S12B
IOWA

FIGURE 45

APPENDIX D

LIQUID LIMIT PLOTS FOR IOWA, LOUISIANA SURVEY



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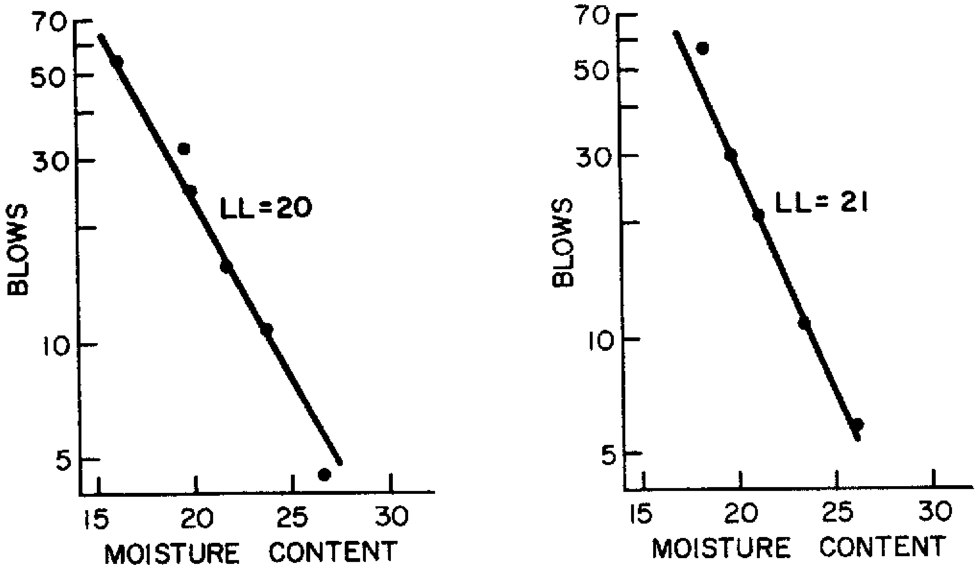
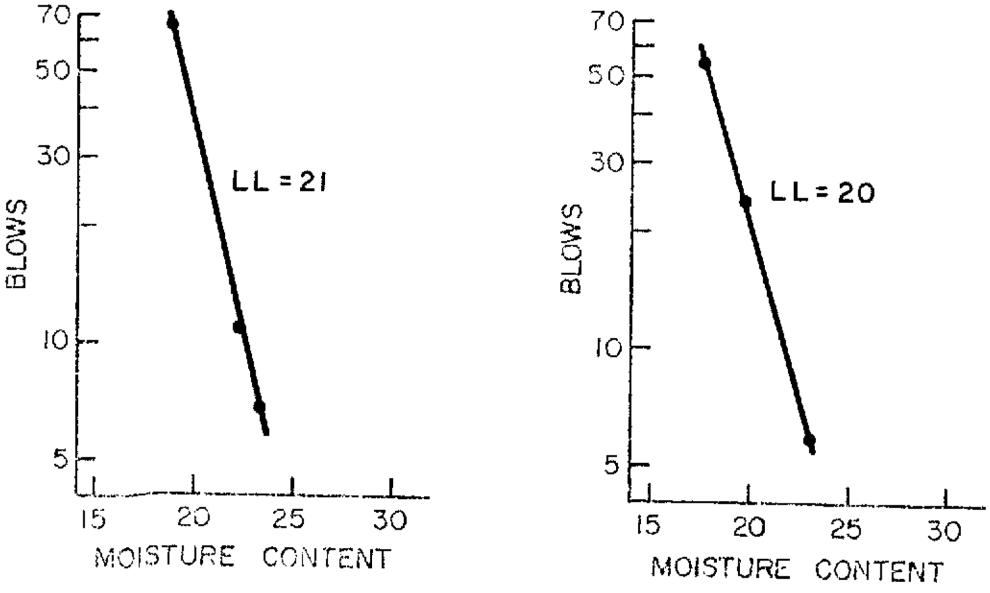
TEST SITE 2



TEST SITE 3

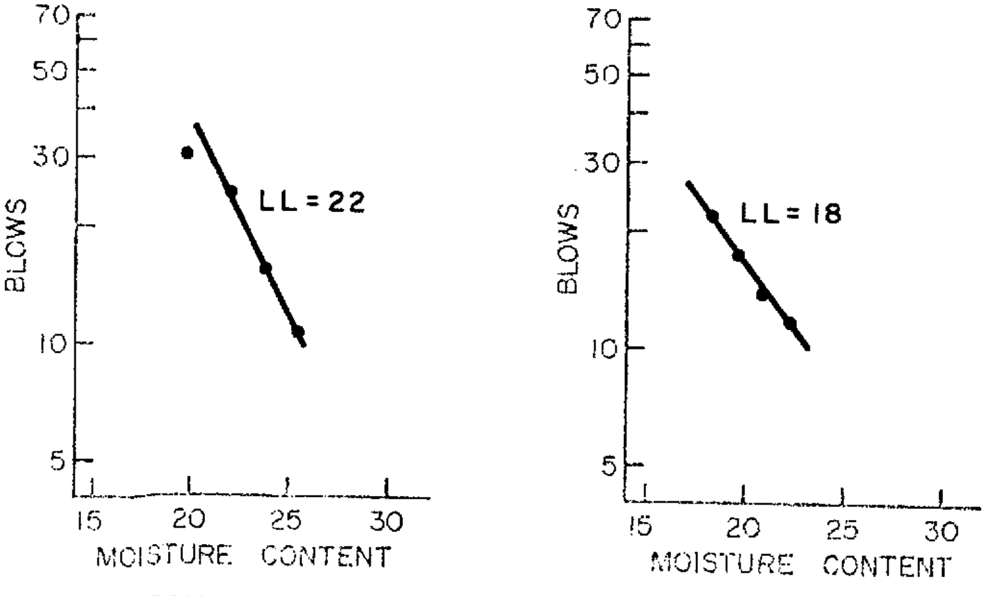
TEST SITE 4

FIGURE 46



TEST SITE 5

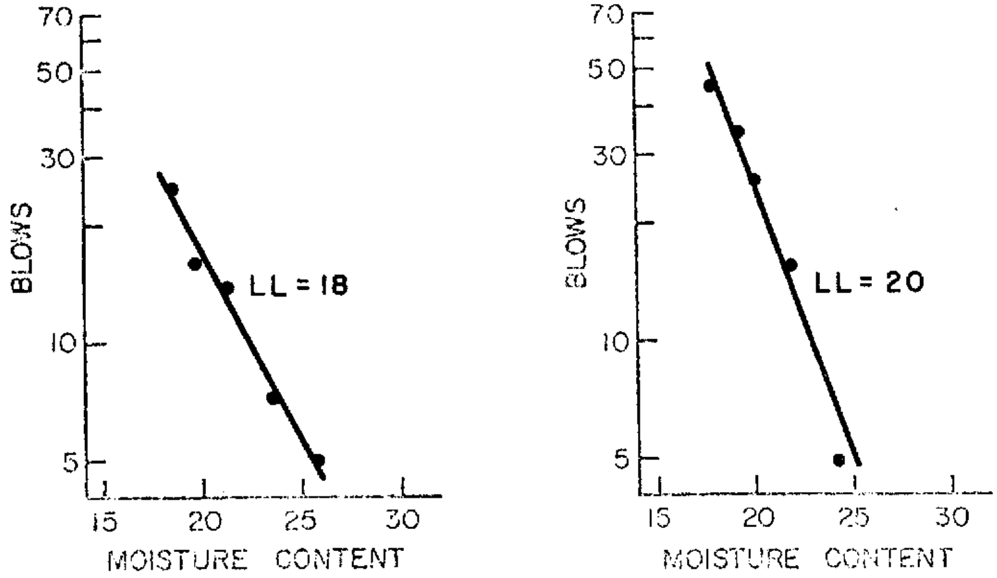
TEST SITE 6



TEST SITE 7

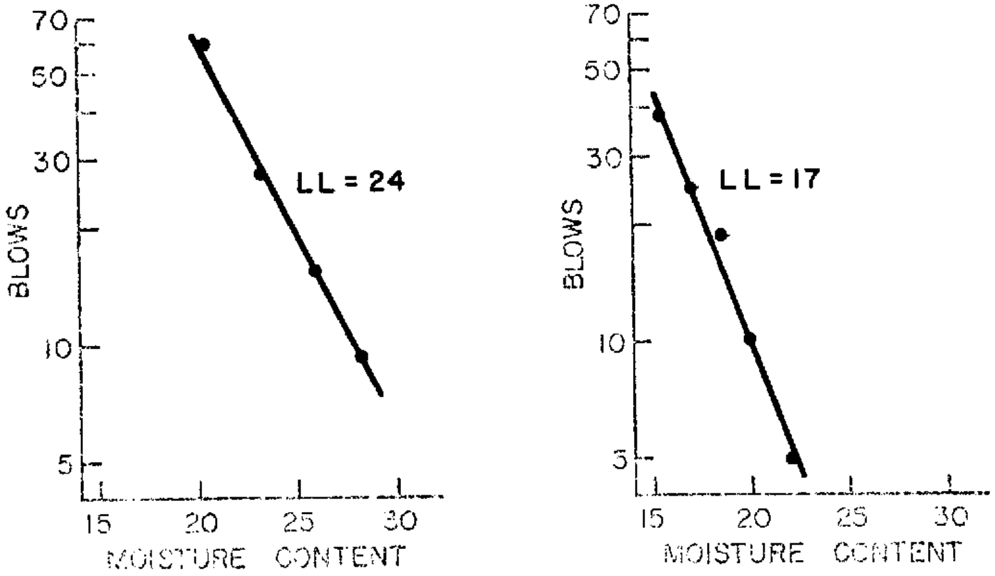
TEST SITE 8

FIGURE 47



TEST SITE 9

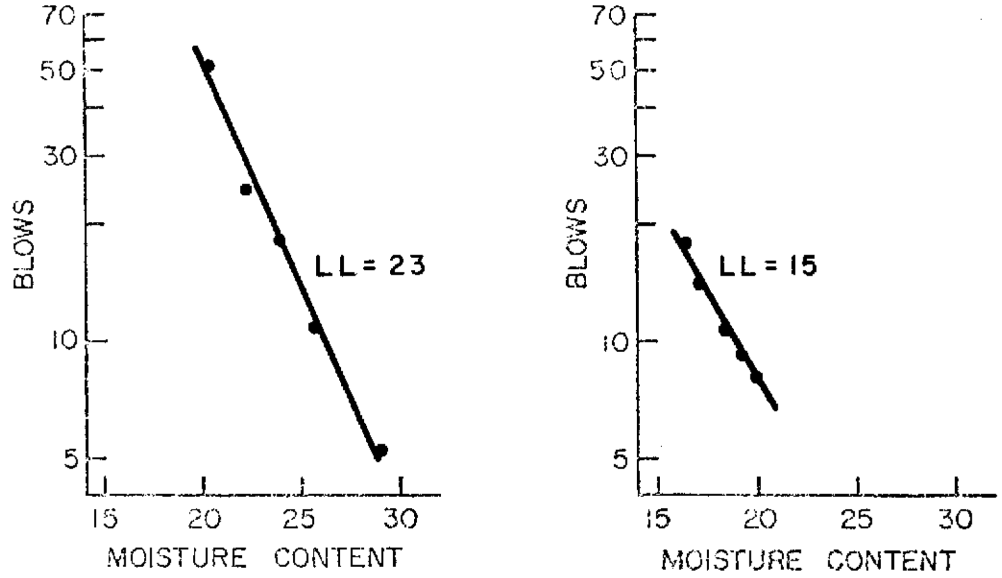
TEST SITE 10



TEST SITE 11

TEST SITE 12

FIGURE 48



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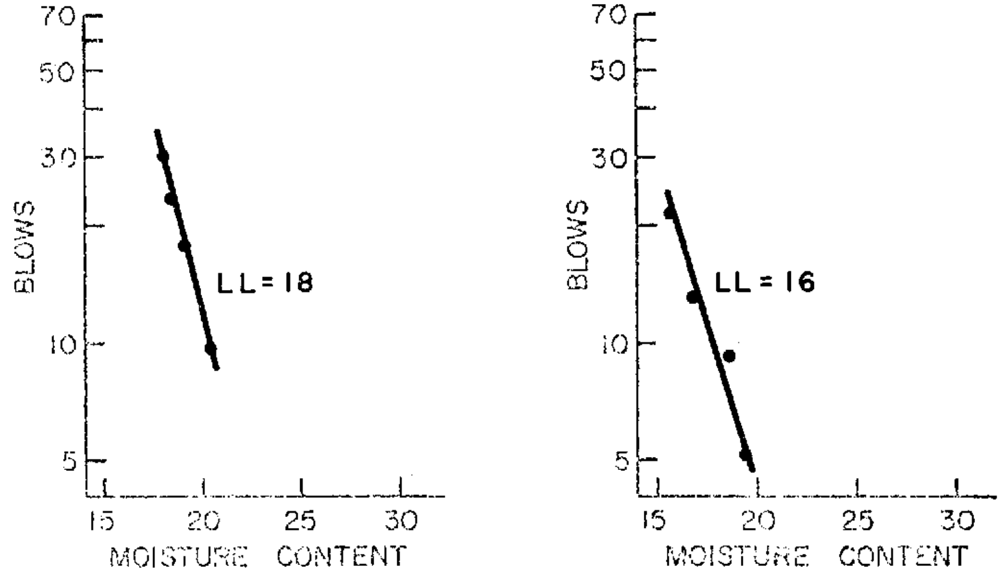
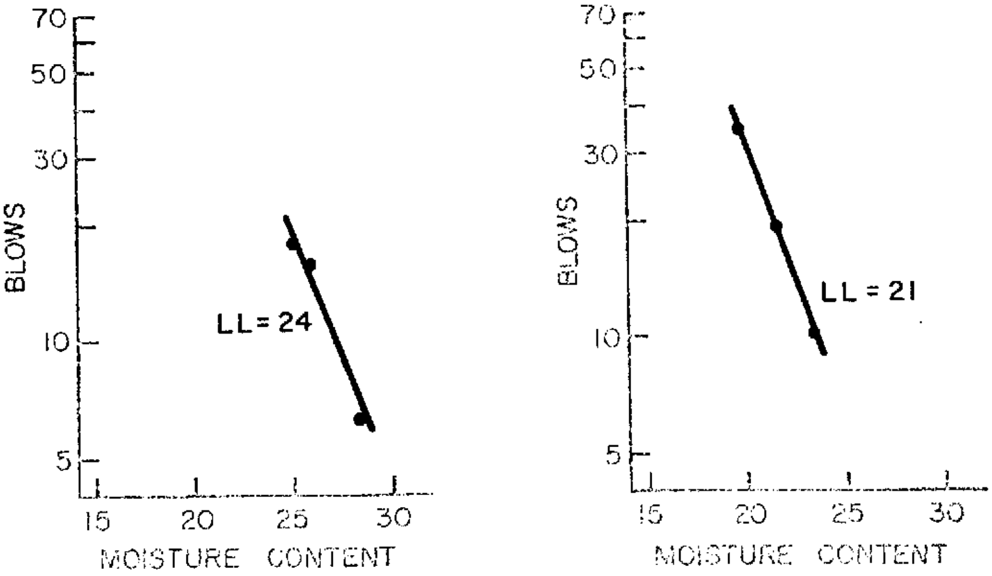
TEST SITE 14



TEST SITE 15

TEST SITE 16

FIGURE 49



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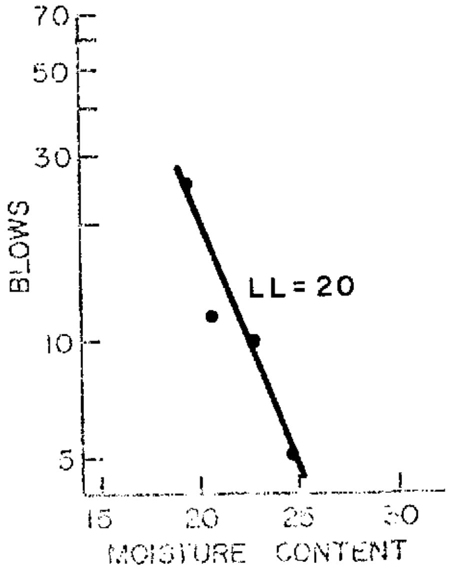
TEST SITE 18



TEST SITE 19

FIGURE 50

APPENDIX E

IOWA, LOUISIANA (S12B)
CALGON LIQUOR DATA

This appendix contains a total chemical analysis (Table 20), X-ray diffraction pattern (Figure 51), and differential thermal curve for Iowa, Louisiana (S12B) Calgon liquor.

TABLE 20

TOTAL CHEMICAL ANALYSIS -- S12B CALGON LIQUOR

Element	Percent
SiO_2	11
Al_2O_3	6
CaO	3
Fe_2O_3	3
MgO	1
P_2O_5 (approx.)	30
Na (10%)	20 (estimate)
C	5
Unknown Substance	21

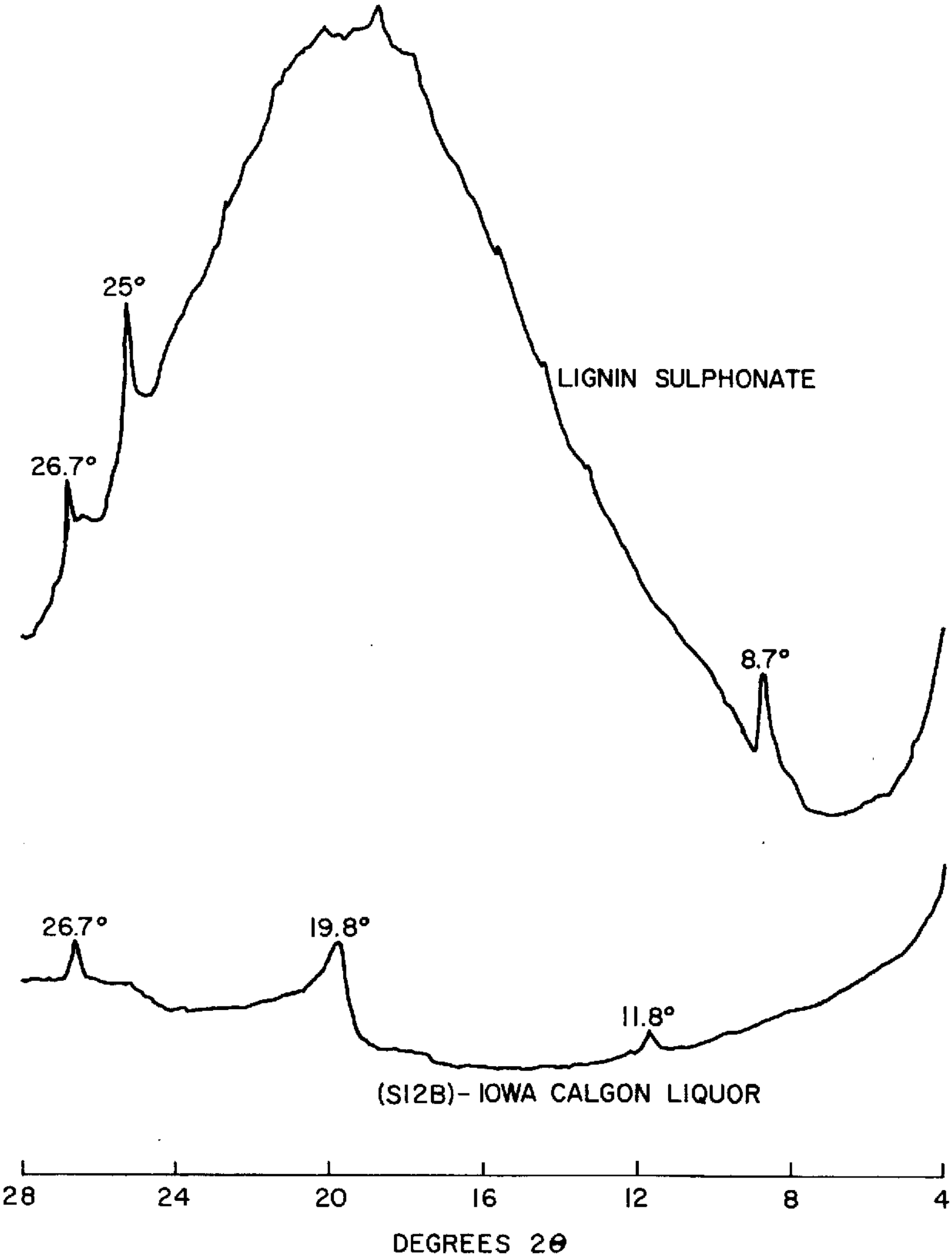
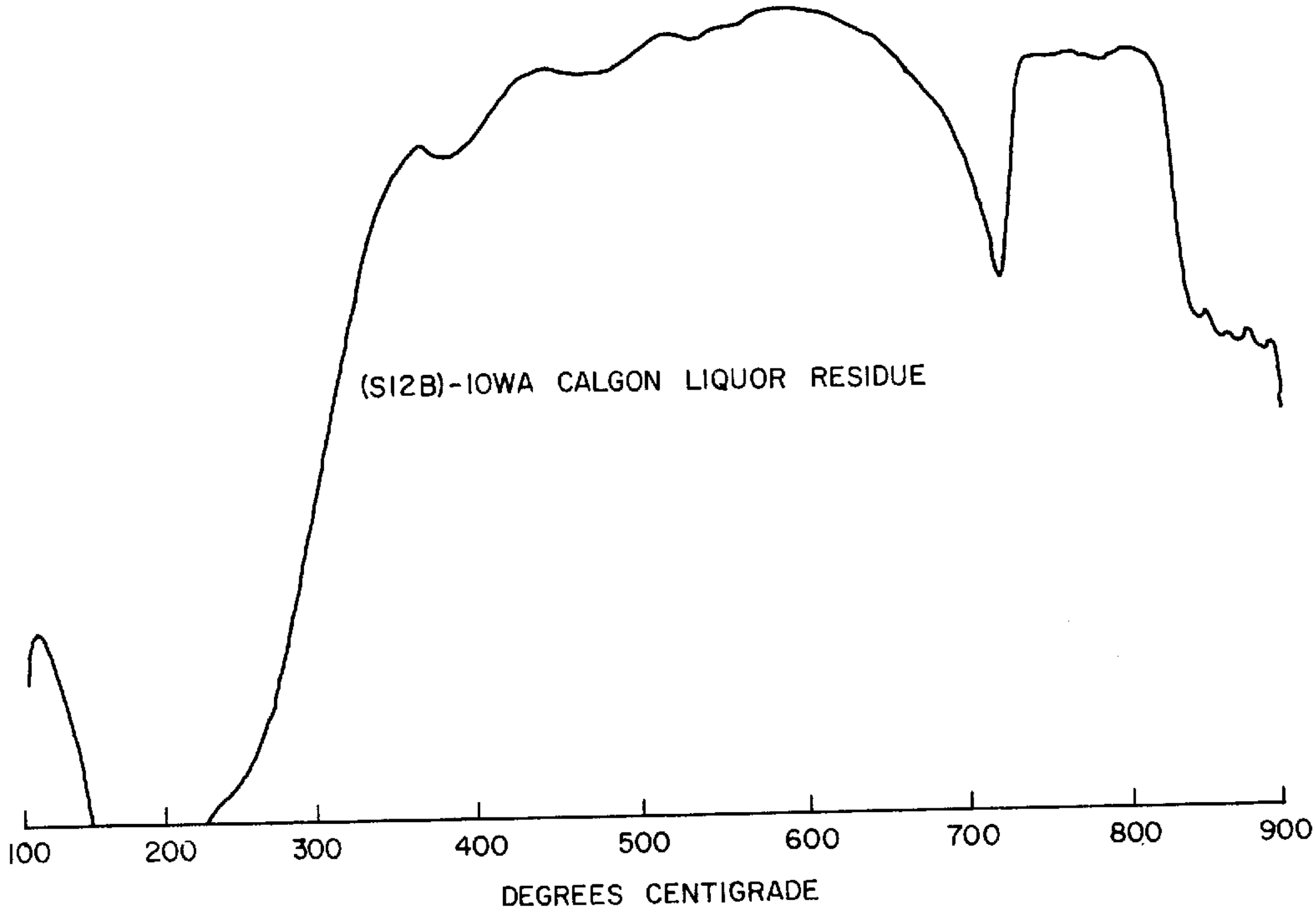

X-RAY DIFFRACTION

FIGURE 51



DIFFERENTIAL THERMAL ANALYSIS
FIGURE 52

RETURN TO:

DIVISION OF ENGINEERING RESEARCH
LOUISIANA STATE UNIVERSITY
BATON ROUGE, LOUISIANA 70803

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