

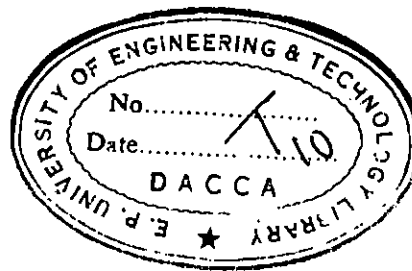
To
Dr. Jack D. Gray

POST GLACIAL DEPOSITS AS FOUNDATION MEDIA
IN LOWER ALLUVIAL VALLEYS AND DELTAIC AREAS

A Dissertation

By

MD. HOSSAIN SEKANDAR HAYAT KHAN EUSUFZAI



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May 1965

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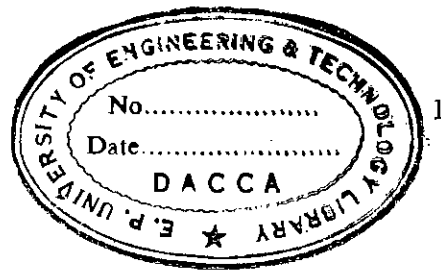
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CHAPTER I

INTRODUCTION

The development of Pakistan, by the people themselves, began immediately after independence in 1947. Prior to this event the country was underdeveloped by modern standards and had inherited little from its former rulers. Subsequent to this event, great effort has been put forth by our leaders and people to develop the natural resources, agriculture, industry, commerce and communications to meet existing and future needs. This dissertation treats a means of overcoming some aspects of the problems.

The awareness of the need and urgency for the economic development of the country demands a very critical study of the problems and the possible sound and economic methods of their solution. A tremendous amount of effort in attacking the problems in their true perspective is necessary. Planners must see the needs of today and tomorrow and formulate their recommendations with foresightedness and practicality.

It is easily seen that the prerequisite for any economic growth of a country is the development of its communication and transportation systems, as well as of the construction industry for their creation and expansion.

The first things needed for the development of industry, commerce, or agriculture are good communication and transportation

systems, well-planned ports, and other like facilities. Pakistan is striving hard to provide these. Regarding communication, transportation and other construction industries, East Pakistan is in a difficult situation. The cost of construction is at least 30% greater today in East Pakistan than in West Pakistan, primarily because of the lack of desirable systems of communication and transportation and limited development of manufacturing concerns to produce construction materials. The province of East Pakistan is criss-crossed by many rivers, creeks and marshes thus making transportation and communication very difficult. In many instances, one must travel a whole day from a city to reach places in the province only 40 or 50 miles away. In case of emergency, supplies have to be airlifted, a practice which is not only costly, but is sometimes impractical. The recent introduction of helicopter service in the province has reduced the travel time from Dacca, the provincial capital city, to the district headquarters of Faridpur to about 30 minutes. Presently the fastest land transportation between these centers, by railway and ferry across the River Ganges, requires about 18 hours. Movement of bulk goods by helicopter, however, is out of the question.

At present East Pakistan has one port at Chittagong in the eastern part of the province, while another is being built in Chalna, in the southern deltaic region, to serve the western part of the

province. Communication and transportation from Chalna to other parts of the province is not, at present, conducive to proper growth of commerce and industry. At present, the port at Chalna is not served by rail or highway systems. The country has been feeling the need of constructing highways, railways and waterways connecting all parts of the province to facilitate rapid economic growth. A direct route between Chalna and Dacca will not only reduce the cost and time of transport by many times, but it will help general prosperity flourish.

The Brahmaputra River divides the province into two almost equal parts. A highway and railway bridge across the river will facilitate the movement of goods and people between the two parts and thus help commerce and industry grow faster. Another bridge across the Sitalakhya River connecting the capital city of Dacca with the area on the south will bring similar benefits to this area. Cases may be multiplied to show that all round economic development of these areas, everywhere in the province, is a necessity. This development must precede any other plan for the development of the province and of the whole country.

Planners and designers for such projects first of all must know the potential foundation media in the area and its ability to support various structures.

The major part of the land area of the province is of deltaic

origin, as shown by Figure 1. The area has been formed by the two great rivers, the Ganges and the Brahmaputra. A good knowledge of the geology of the area, including the history and the stages of the development of the rivers and creeks is important; also knowledge of the kind, condition and state of foundation materials must be available before railways, communication systems and structures to serve the areas are planned, designed, or constructed.

The primary cause of many hazardous failures of structures throughout the world, in the past, has been due to lack of knowledge of foundation media. We should not continue to repeat the mistakes of the past.

Knowledge gained from other areas having similar geology to that in East Pakistan is inferred to be applicable to the situations there.

The apparent geological history and formations in the alluvial valley and deltaic regions of the Brahmaputra River System in East Pakistan are strikingly similar to the geological history and formations of such regions of the Lower Mississippi River System. These similarities are used in this study.

CHAPTER II

PHYSICAL GEOLOGY AND THE RIVERS OF EAST PAKISTAN

Physical Geology

East Pakistan is situated between latitude $20^{\circ} 45' N$ and $26^{\circ} 45' N$ and longitude $88^{\circ} 2' E$ and $92^{\circ} 45' E$. It is principally bordered by India on three sides and by the Bay of Bengal on the south, as may be noted from Figure 2. The province of East Pakistan can be physically and geologically divided into three broad divisions: (a) The gently rolling hilly part in the southeastern region comprising of the districts of Chittagong and Chittagong Hill Tracts and a small part of relatively high land in the northern part. (b) The alluvial valley of the rivers the Brahmaputra and the Ganges, and (c) The recent deltaic area in the southern part. The three divisions are shown in Figure 1.

An examination of the few elevations recorded on the map, Figure 1, shows a very flat slope for both the alluvial and the deltaic areas. An elevation of 54 feet at the mouth of the alluvial plain, which is approximately 225 miles from the shoreline in the south, gives an average slope of three inches per mile for the rivers (1). This very flat slope suggests that the portion, which now appears as the alluvial plain, was possibly a deltaic area originally. As a result of the extension of the delta formation into the sea, the shoreline is moving further south and gradually this portion has become the alluvial plain of the rivers, the Brahmaputra and the Ganges.

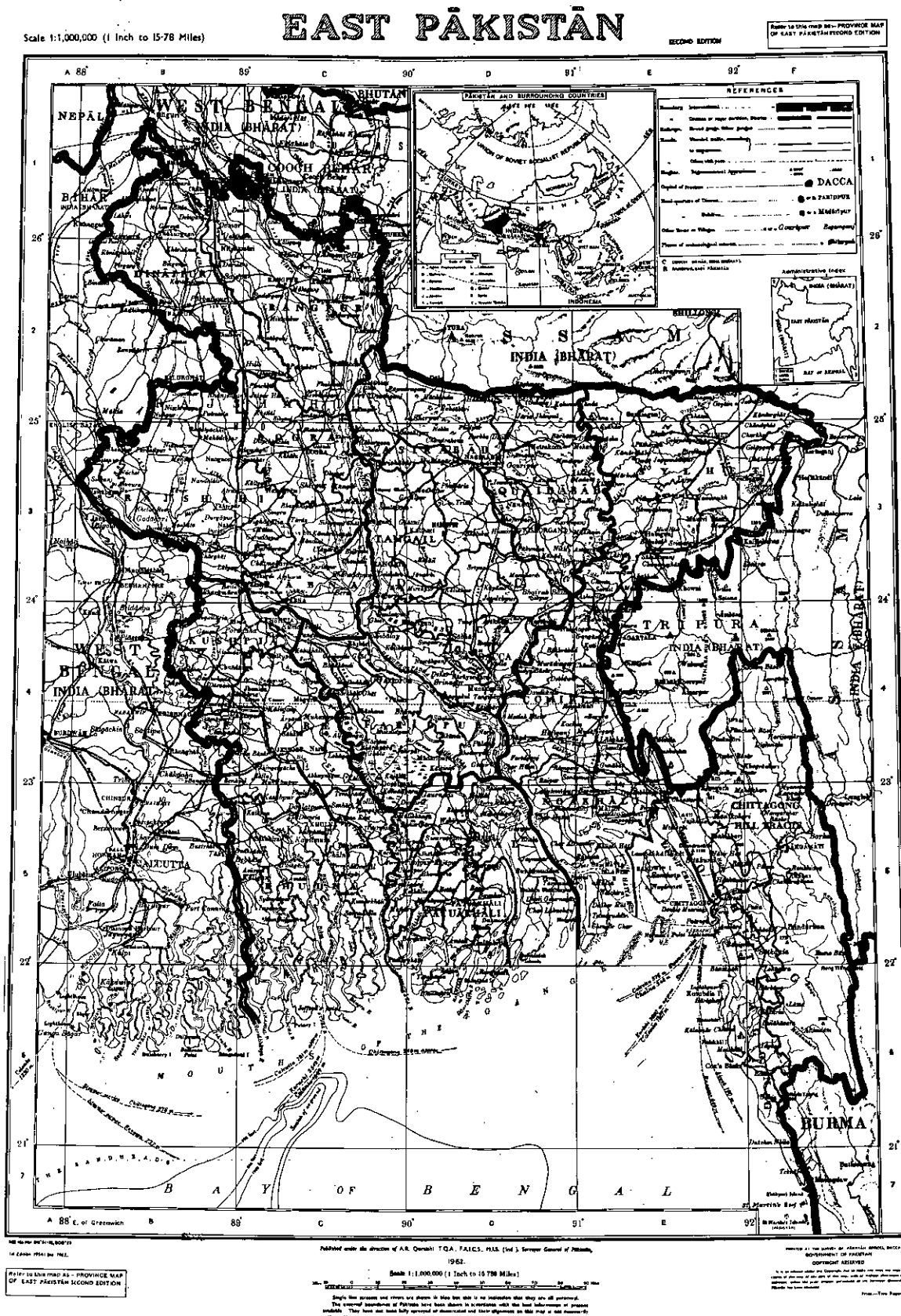


Figure 2. Map of East Pakistan (general).

With time, the present deltaic area may also one day appear as an alluvial plain from a geographical point of view.

The highland, occupying parts of the districts of Mymensingh and Dacca, including the city of Dacca, is a ridge comparable to the Crowley Ridge in the Mississippi alluvial plain. These ridges contain soil of residual nature.

During the Pleistocene Epoch, when the last great ice caps and sheets were in existence, the sea was four to five hundred feet below its present level. The two mighty rivers, the Brahmaputra and the Ganges eroded deep gorges in their valleys that extended into the Bay of Bengal (2, 3). As the ice, which occurred north of the Himalayan Mountain Range in Central Asia, melted and the glaciers receded, the melt waters raised the oceanic level to complete the glacial cycle. During this period of rising sea level, stream gradients, stream loads, and channel characteristics all experienced transitional changes. The recent alluvium, which fills the buried valley system, should also be characterized by a sequence of deposits grading upward from gravels and sands to finer materials.

Following the wastage of the ice sheets and the consequent rise of the level of the sea, the gradients of the rivers were greatly reduced resulting in a corresponding reduction in both their ability to transport materials and in the size of particles moved. Meandering of river channels occurred as the gradients were reduced so that

broad alluvial valleys developed and vast deltaic formations spread across the entire width of the Bay. The unique combination of deltas is shown by Figure 1.

Due to the gradual aggradation of the immediate valley floor, the Ganges lately shifted its course towards the east, possibly after the 15th century (4), and joined the mighty Brahmaputra River. The process of building of alluvial valleys and extension of the deltas into the sea are continuing.

The Rivers of East Pakistan

The Brahmaputra River. The Brahmaputra River originates in western Tibet and drains the enormous basin north of the Himalayan Mountains. It flows in a generally easterly direction, paralleling the mountain ranges, to a location where a natural gap exists. The course through the gap is in a southerly direction but changes to a westerly direction through the hilly upland province of Assam of India to the northern boundary of East Pakistan. It is noteworthy that in flowing through the province of Assam the valley of the river is bounded on the north by the Himalaya and on the south by smaller mountains where elevations are of the order of 4500 feet. Thence the course changes to the southerly direction and continues to the Bay of Bengal. The overall length of the river is approximately 1800 miles of which about the last 300 miles lie in East Pakistan. It

is thus evident that the gradient of the river is relatively steep from the source to the western limit of the province of Assam and then becomes relatively flat from that point to the Bay of Bengal. A reference elevation of 54 feet, mean sea level, has been found for a point approximately 60 miles south of the common border of Assam and East Pakistan. This serves to confirm the indication of the flatness of the gradient through the major portion of the course through East Pakistan. The steep gradient of the river through the major portion of its course can be taken, with reason, as a strong indication of the capacity to transport large quantities of detrital materials during the heavy outwash period, to the area occupied by East Pakistan. Subsequent to the outwash period the Brahmaputra River has deposited vast quantities of bed and suspended loads of materials over the broad deltaic plain to build up the present alluvial plain and extend its vast delta into the Bay of Bengal.

TABLE 1

THE BRAHMAPUTRA RIVERSummary of Basic Information

Total drainage area	361,200 sq. miles
Length	1,800 miles
Maximum discharge (approximately).	2,000,000 cfs
Average discharge.	428,000 cfs
Minimum discharge	15,000 cfs

The Ganges River. The Ganges originates from the Gongotri Glacier in the Himalayas and flows in an easterly direction until it meets the Brahmaputra River in East Pakistan. The river is about 1600 miles long. The nature of the terrain and length of the river flowing through each type has been approximately estimated as follows:

- (a) steep sloped mountainous terrain - 100 miles
- (b) alluvial terrain of northern India - 1250 miles
- (c) deltaic terrain of East Pakistan - 250 miles

In the lower valley the river has changed its course many times. It is said that the Ganges, after entering the deltaic region, originally flowed to the Bay of Bengal through the Bhagirathi River by the side of Calcutta. It is also said to be the principal builder of the area now known as West Bengal and southwestern East Pakistan.

Slowly, with the passage of time, and due to gradual aggradation, the river shifted towards the east and is now contributing to form and extend the deltaic region into the Bay of Bengal. Some basic data about the river, which may be of interest, are presented in Table 2 that follows (5).

TABLE 2

THE GANGES RIVERSummary of Basic Information

(a) Maximum flood discharge at Faracca (about 100 miles upstream from its entrance to East Pakistan	2,125,000 cfs
(b) Mean annual discharge	498,000 cfs
(c) Minimum discharge	61,500 cfs
(d) Maximum silt content at Faracca (Oct., 1948-May, 1949)	0.1%
(e) Minimum silt content	0.005%

CHAPTER III

THE PROBLEM

In civil engineering a structure is only as good as its foundation. Study of the remaining structures of antiquity shows that their long life is attributed in part to the stability and satisfactory behavior of the materials on which they are founded.

It has been accepted for ages that very unstable foundation media are characteristic of deltaic and young alluvial areas. However, with modern engineering tools that are now available, it is being found that through proper examination and intelligent design, satisfactory and economical solutions of problems for all types of structures can be developed for such areas. Raft substructures inducing low intensity stresses may be used in the upper unconsolidated media. Friction piles offer a potential solution, or long piles penetrating into the stable and dense post-glacial valley floors may be used, depending upon the circumstances. It remains therefore, for the young engineers of Pakistan to learn of the modern engineering tools and processes of design so as to accomplish the solutions of tomorrow.

The thoughts advanced in the preceding chapter and confirmed by factual information latter suggest that stable strata, within reasonable and economical depths, may be expected to exist in the alluvial and deltaic areas of East Pakistan. This remains only to be discovered and used. Further, the application of alternate solutions need to be

considered for situations where they may be applied.

This dissertation will present the results of studies of similar conditions, their solutions and applications to problems existing in East Pakistan. It will also investigate the problem of establishing the kind, condition and state of existing foundation media.

Scarcity of Data

To form sound conclusions and solutions concerning foundations, one needs authentic and adequate data for analysis. Such information should include the geological history of the area of interest, the stratigraphy as established by exploration, and the properties of the materials involved. In addition, it is desirable to have a record of behavior of structures in the area observed through a reasonably long period of time. For East Pakistan, no such data is known to be available. No geological studies are known to have been made, no records of the behavior of heavy or complicated structures are at hand because such structures have not been built, and no comprehensive tests and experimental studies have been made. It is noteworthy that no bridge has ever been built in East Pakistan across the main course of the Brahmaputra River, which extends throughout the length of the province. There is only one bridge across the Ganges River in this area. This may be attributed in part to the prevailing thought that suitable foundation medium may not be available.

Recently, however, due to urgent needs, buildings up to about 10 stories high have been built in Dacca, which is not in the alluvial valley. It is understood that very heavy and possibly uneconomical foundation designs may have been used which might have been avoided had sufficient pertinent data been available.

General knowledge of physical geology and related matters on East Pakistan is available. Advantage can be taken of this knowledge by comparing it with that of a similar region, about which both specific and general knowledge is available. The successful solutions used for the known areas may then be applied to the areas about which there is no specific knowledge.

CHAPTER IV

THE VALLEYS, THE DELTAS AND THE DEPOSITS OF THE RIVER SYSTEMS

Careful study and analysis show that the alluvial valley and deltaic regions of the Lower Mississippi River, about which much specific knowledge is at hand, offers a basis for the comparison desired. The points of the similarities and comparisons of features existing in the state of Louisiana, United States, and those in East Pakistan are presented in the sub-paragraphs that follow.

a. Contributory areas. The major rivers feeding the two areas being compared are the Mississippi River in the State of Louisiana in the United States, and the Brahmaputra and Ganges Rivers in East Pakistan. The Mississippi River was the major outlet for a vast glaciated region of North America. The Brahmaputra River was the major outlet for its vast glaciated basin north of the Himalayan Mountains.

b. Nature of post-glacial materials. In both cases the bed or transported loads of the gigantic glaciers or ice caps were enormous in quantity. They were by their nature, predominantly detrital, ranging from boulders to rock flour in particle size. The bed load materials encased within the body of the ice had little or no opportunity to experience chemical weathering, thus eliminating the possibility of their transformation into fine grained cohesive materials

of varied properties. Thus, the geological history of recent materials filling the bottom of the post glacially formed gorges and underlying the surface alluvial formations in which a foundation engineer may be mainly interested, should be the same.

c. Transportability of the rivers. The ability of a stream to transport suspended and bed load materials varies as the square of the velocity of the water and the velocity varies as the one-half power of the slope of the bed of the stream. The Brahmaputra River traverses mountainous terrain for 1500 miles and flows over flat area for a relatively short distance through East Pakistan.

The Mississippi River flows through a much longer flat area before entering the alluvial plain in Louisiana. This situation suggests that coarser materials may be expected to have been transported further down into the alluvial and deltaic areas of East Pakistan than in the Louisiana area.

d. History of delta formations. The swamps and marshes making up the lowlands of coastal Louisiana include a 200-mile wide deltaic plain and the 100 mile wide Chenier plain. The recent coastal area covers an area of approximately 14,000 square miles. It has developed in the last few thousand years in response to the shifting channels of the Mississippi River.

In East Pakistan the deltaic area occupies the major portion of the area of the province and is formed mainly due to the shifting

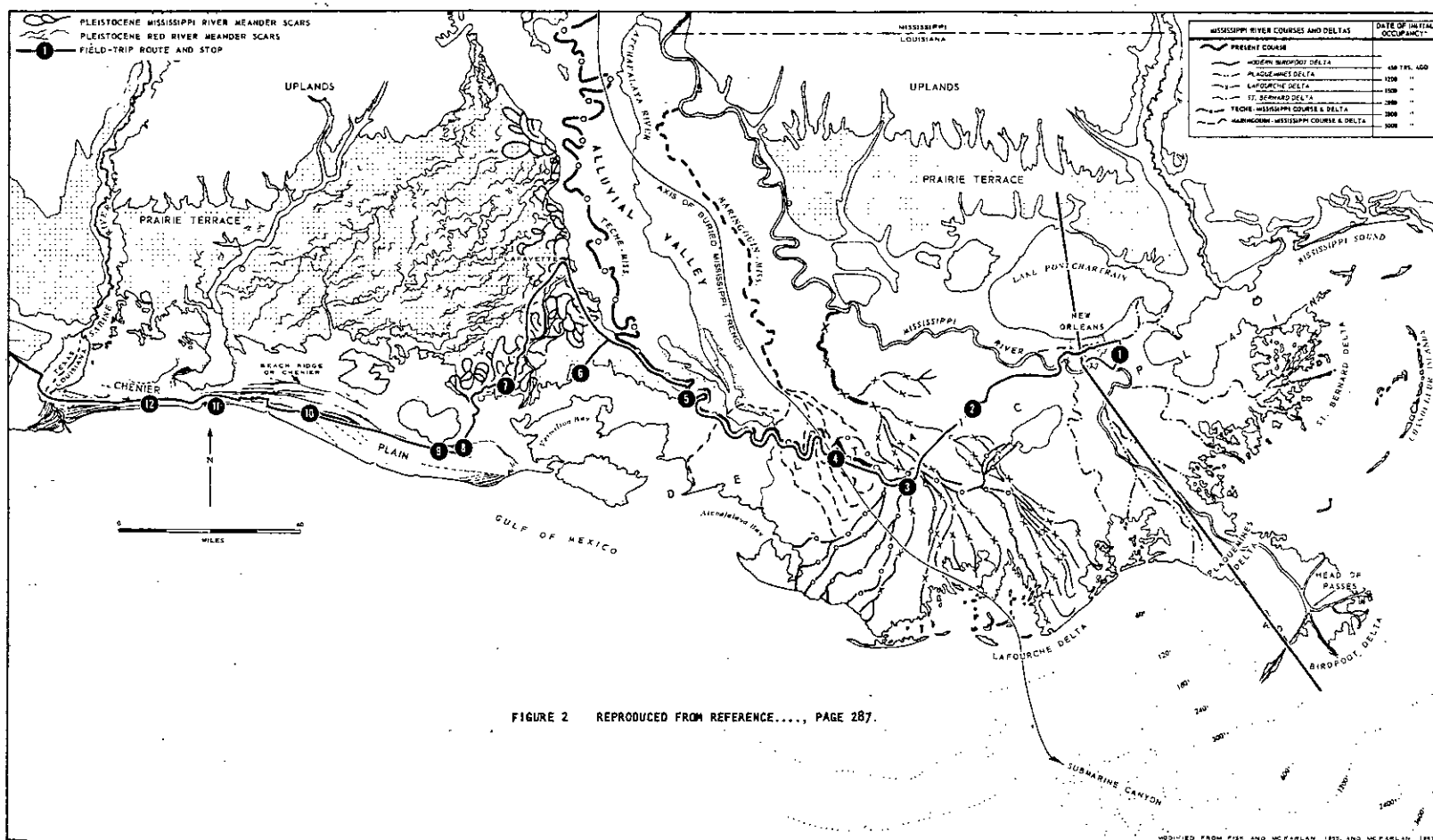


Figure 3. Physiographic Map, Central Gulf Coastal Plain,
Mississippi River Alluvial Valley

courses of the Ganges and the Brahmaputra Rivers. Comparison of the maps of the two areas, Figures 1, 2 and 3, show these similarities.

"In constructing the deltaic plain, the Mississippi River occupied and abandoned two courses prior to establishing its present course. The sequence of courses and delta developments has been determined largely from physiographic evidence. The oldest delta, the Maneingoin-Mississippi, began to develop about 5,000 years ago. It was abandoned in favor of the Teche-Mississippi delta when the river shifted to its Teche course about 3,800 years ago. Deltas associated with the present course and the modern birdfoot delta were initiated about 450 years ago" (6).

The river Ganges formerly flowed through West Bengal of India by the side of Calcutta. In the 15th century it changed its course to the present position, thus extending its delta-forming activity to the present site of southern East Pakistan.

In the Louisiana coastal area each of the deltas, except the modern birdfoot delta, were built forward onto the shallow inner margin of the continental shelf. These modern deltas were characterized by many outlet channels. As each of the deltas continued its seaward growth, many of the channels were abandoned and filled as more favorable outlets enlarged or as new ones developed. These abandoned channels extend several tens of feet below sea level and are filled with clean fine sand at the base, grading upward into sandy

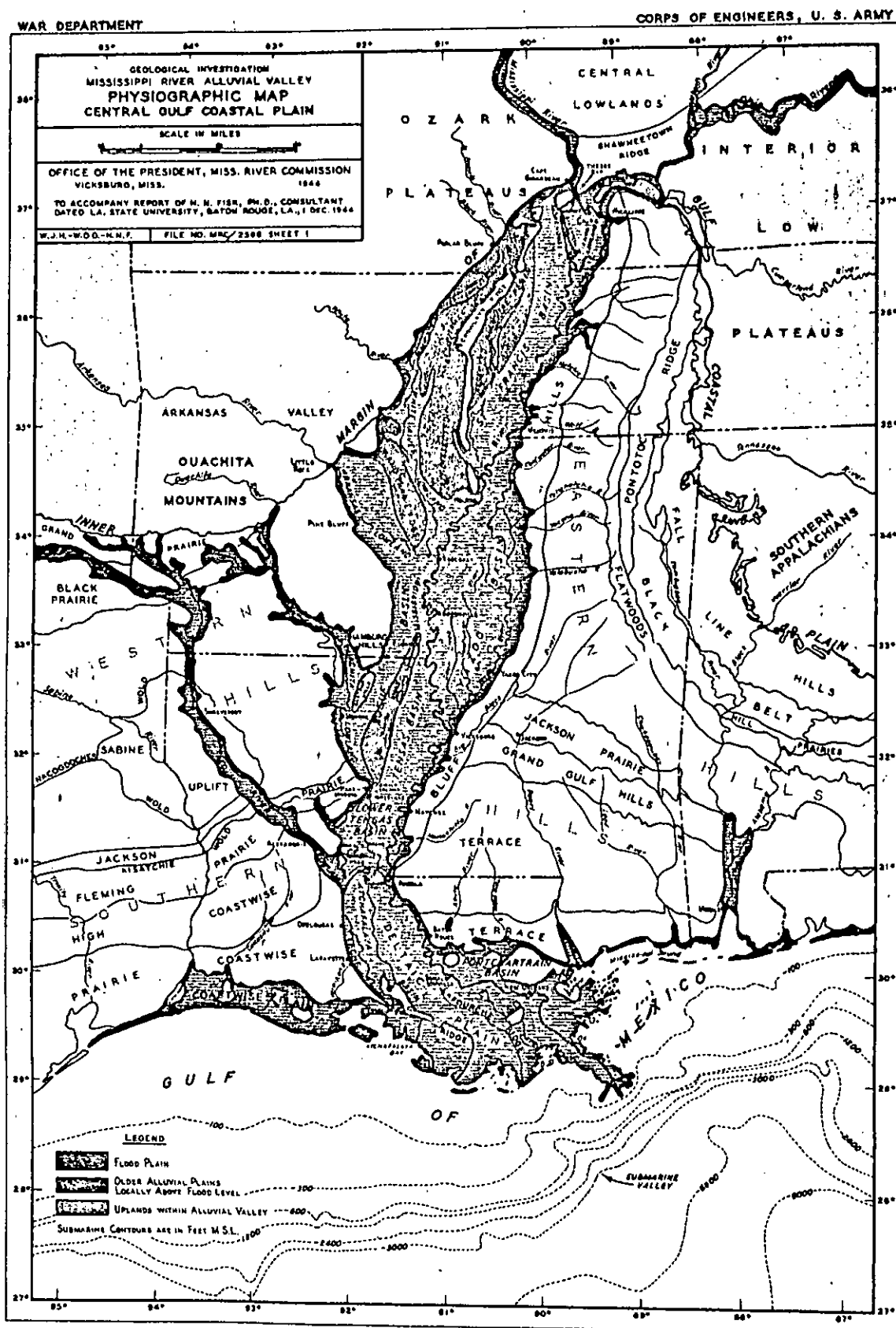


Figure 4. Mississippi River Course and Deltas.

silts and marsh deposits. In the East Pakistan deltas, the river Ganges and its outlet channels abandoned and filled their former courses through West Bengal and found their outlets through their present courses about 500 years ago.

e. Organic deposits. Peat deposits occur beneath the landward section of the Chenier plain in the Lower Mississippi Valley, at depths ranging up to more than 20 feet. This provides evidence of older marshland surfaces that developed during the last stages in a rising sea level. A widespread two-foot thick peat layer in the eastern section of the plain, dated as 4,000 years ago, and lying 12 feet below sea level overlaps buried natural levees. In East Pakistan peat deposits at shallow depths have recently been discovered in the swampy area of the district of Faridpur, an area similar to the Chenier plain.

f. Off-shore gorges. The off-shore gorges of both river systems bear a striking similarity, further confirming the proposition that the two areas are alike in nature. These may be noted from the maps of the areas, Figures 2 and 4. For East Pakistan, in the Bay of Bengal and a little south of the land area, the contours for the sea bottom rapidly converge in an area designated as "swatch of no ground," which virtually means that the area is bottomless. In Figure 4 for the Coastal Mississippi area the "Axis of Buried Mississippi Trench" is shown by a firm line ultimately pointing, by

an arrow, to the submarine canyon of very great depth in the sea.

This striking similarity clearly shows the existence of buried trenches in both the areas carved during the period when the sea level was low and the slope of the rivers was extremely steep. The "swatch of no ground" is evidently the final run of the submarine canyon of both the Ganges and Brahmaputra rivers.

The foregoing similarities serve to substantiate the proposition that the two areas are geologically nearly identical. They serve to encourage the study of the solutions of problems encountered in the Lower Mississippi area for use and guidance in solving those of the East Pakistan area.

The Alluvial Valley of the Lower Mississippi River

Fisk (7) prepared an excellent and very valuable report on the geology of the alluvial valley of the Lower Mississippi River. The report is thorough and extensive. It is based on field exploration works and reports from many established organizations working in this field. According to Fisk, "During the last glacial age, when the sea level was 450 feet lower than at present, the Mississippi River Valley system became deeply incised within the coastal plain sediments." This suggests that, possibly in addition to the formations of the Tertiary Period, there may have been erosion of valley fill deposits related to the glacial ages prior to the Wisconsin

glacial age.

Fisk divides the alluvial plain into two parts: (a) The flood plain, subject to seasonal flooding, and (b) the dissected plain, not completely covered by flood waters. The flood plain, which is of recent origin has a total area of about 35,000 square miles, including the near sea level deltaic plain shown by Figure 3.

In 1881 deep borings made by the Mississippi River Commission disclosed that the alluvium extended far below the maximum depth of the modern river. The existence of a buried valley system, underlying the Mississippi Alluvial Plain, was recognized. The details of the valley system, as revealed by the logs of several thousand borings, are similar to those of a normal stream system. The buried system consists of several major tributary trenches, separated by divides. The trenches follow closely the position of the Mississippi courses.

The bedrock deposits of the Cenozoic Era, forming the floor of the buried Mississippi valley system, vary in age and nature. The width, depth and slope of the trenches varies accordingly, depending on the hardness of the bedrock. For harder bedrocks the width and depth are small, while in less resistant bedrocks they are large. Scattered borings and geophysical data show an abrupt increase in depth of the entrenched valley from 350 feet at Houma to over 600 feet near the present shoreline.

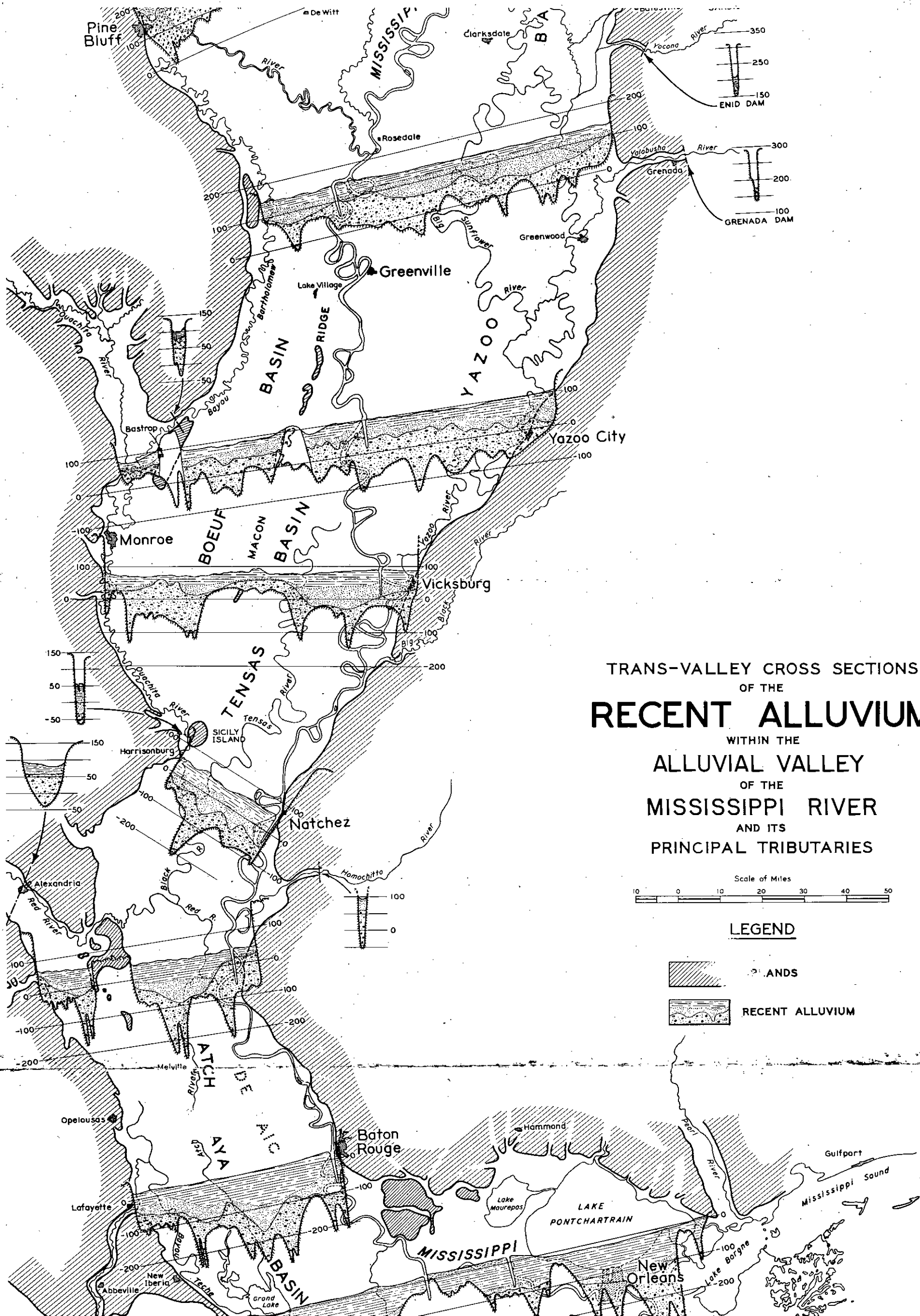
General Feature of the Mississippi River Alluvium

"The principal event of the late geologic history is a cycle of fluctuation in sea level which accompanied the formation and melting of the continental ice sheets during, and subsequent to, the last Glacial Age (Pleistocene)" (8). About 50,000 to 60,000 years ago the ice sheets of the late Wisconsin glacial stage began to develop. The sea level fell to a minimum level, which was reached when the maximum development of the sheets occurred about 25,000 to 30,000 years ago. During the period of recession of the ice sheets the oceanic level gradually rose to a level 5 to 8 feet higher than its present level about 4,000 years ago. Since that time it has fallen to its present level (9).

This cycle of fall and rise of sea level influenced the slope and transportational characteristics of the streams draining into the oceans.

"The Recent alluvium which fills the buried trenches is everywhere characterized by a sequence of deposits grading upward from coarse sands and gravels, through clean non-graveliferous sands, into the fine grained silty, clayey and sandy sediments of the alluvial plain."

"The alluvial sequence with its upward decrease in particle size, results from the progressive decrease in slope brought about by rising sea level and consequent filling of the valley. The deposits



provide proof of a gradational reduction in the carrying capacity of the streams."

The maximum thickness of the total alluvium in the northern part of the valley is slightly over 200 feet, and in the southern part it is more than 350 feet. Of this total thickness, the earlier post glacial graveliferous deposits vary from 75 feet in the northern part to 250 feet in the southern part. These deposits are in turn overlain by the recent alluvial materials that vary in thickness from 125 feet in the north to 138 feet in the south.

Ready understanding of the alluvium in the Lower Mississippi River and its deltas may be gained from observation of the excellent transvalley profiles reported by Fisk. They are reproduced herein as Figure 5 and show the various basins in the valley, the deltaic plain and present shoreline. It is particularly noteworthy to observe the progressive decline in the elevation of the bottom of the valley gorge from the latitude of Greenville, Mississippi, averaging about elevation 0, to that at the latitude of New Orleans, Louisiana, where the bottom of the gorge lies at about elevation -300 feet. This deepening of the gorge occurs in about 250 miles along the center of the valley. The minor influence of small tributaries, such as the Red River, the Ouachita and Homochitto Rivers, are shown by Figure 5. It is of special interest to observe the vertical arrangement of the alluvium in the valley. The recent alluvium is divided into two distinct

parts; namely, the graveliferous and the non-graveliferous deposits. The former, represented by the appropriate symbol and being the coarse post glacial materials, was deposited over the floor of the gorge to significant depths. The latter or the non-graveliferous materials, represented by the horizontal and broken hatched lines, was deposited by the Mississippi River after the sea had returned to its present position. It is to be noted that the bottom of the non-graveliferous materials lies at about elevation -100 feet at New Orleans and becomes approximately elevation 0 at near the midpoint of the lower valley. The environment for its deposition is in sharp contrast to that for the underlying graveliferous deposits. A brief discussion of each of the two basic materials is presented under the following individual headings.

The Graveliferous Deposits

The deep underlying graveliferous portion accounts for 45 percent of the total of the alluvium, with 25 per cent of the indicated amount being gravels and the remainder being coarse sands. Detailed study of samples of the materials obtained from closely spaced borings in local areas indicates that the gravels and sands are intermixed. The largest of the gravels are naturally concentrated in the lower part of the deposit. It has been reported that the maximum size of the particles are over one foot in diameter. The gravels in

the southern areas decrease in size from the reported maximum down to the range of pebbles. Some gravels beneath marsh areas encountered in the entrenched valley exceed one inch in diameter, but the bulk of them are much smaller. The distribution of the graveliferous portion shows that its upper surface slopes away from the mouths of each tributary valley. The thickness of this portion depends on the depth of the entrenched valley which the gravels fill. The gravel mass is thicker, however, as is expected, in the northern area than in the southern area.

The Non-graveliferous Deposits

The deposits forming the upper portion of the alluvial section are sands, silts and clays. This section thickens generally southward and reaches a maximum of over 250 feet near the shoreline, south of Houma. Pervious sands of this section have a greater thickness than the overlying relatively impervious surface stratum.

The relatively impervious surface stratum consists of combinations of sands, silts and clays. These deposits constitute most of the visible surface layer except for the sand bar areas along the present stream channels. This surface layer is, in general, thought to be very unconsolidated and to constitute an unstable and unreliable foundation medium. However, when a foundation engineer is aware of the existence of stable dense layers of coarse materials beneath

this surface layer, he can devise structures that will be safe and secure. A knowledge of the geology of the area assures him of the existence of stable materials for his planning and designs. He can safely transfer the load of his structures down to the lower stable stratum by means of properly designed piles or piers. Therefore, this apparently unsuitable foundation medium does not pose a great threat to his structure.

The materials in the natural levees and sand bar ridges are relatively coarse and constitute good foundation materials. Surface strata of these categories do not present much of a problem to the foundation engineer. A third type of materials, the finer materials, filling the depressions, which do cause foundation problems are discussed next. These consist of backswamp deposits and deltaic plain soils which are varieties of the non-graveliferous deposits.

Backswamp soils. The most extensive top stratum deposits of the flood plain are those laid down in the flood basins beyond the natural levees. These deposits consist of silty clays and clays with high organic content. The backswamp deposits encase logs, stumps and roots of trees, as well as thin layers of carbonized leaves and other vegetative matter. These deposits are slightly oxidized and vary in thickness from paper thin streaks to about 40 feet. This type of material, deposited in swamps, is the most dangerous for the foundation engineer. They are soft and cannot

satisfactorily support heavy loads. These are problem areas, so far as founding of structures over them is concerned. Discussions of the types of foundations suitable for such areas will be made later in Chapter VI.

Deltaic plain soils. In the deltaic plain region the backswamp deposits merge with the top stratum of the deltaic plain and form an extensive clay mass. This mass thickens gulfward from 60 feet on the north to about 150 feet on the south.

The surface stratum in the deltaic plain includes a mass of fine grained deposits laid down during the most recent period of valley filling. Silty and sandy sediments mark the natural levees and channel deposits of the rivers. The marshes generally have a superficial layer of muck, which is very loose and not compacted and has a maximum thickness of 20 feet. This layer is very high in organic content. In most places this muck is very soft and cannot support any significant load. This muck generally becomes compacted with depth, and in some places a few feet below surface shows stratification. Organic and vegetative matter occurs as peaty, clay layers.

The deltaic plain of the Mississippi River lies wholly in Louisiana and includes three types of land: natural levee ridges, which are slightly raised above surrounding levels and formed of relatively coarse particles; the forested swamps; and, coastal marshes whose vegetation is chiefly grass. In East Pakistan the deltaic region can

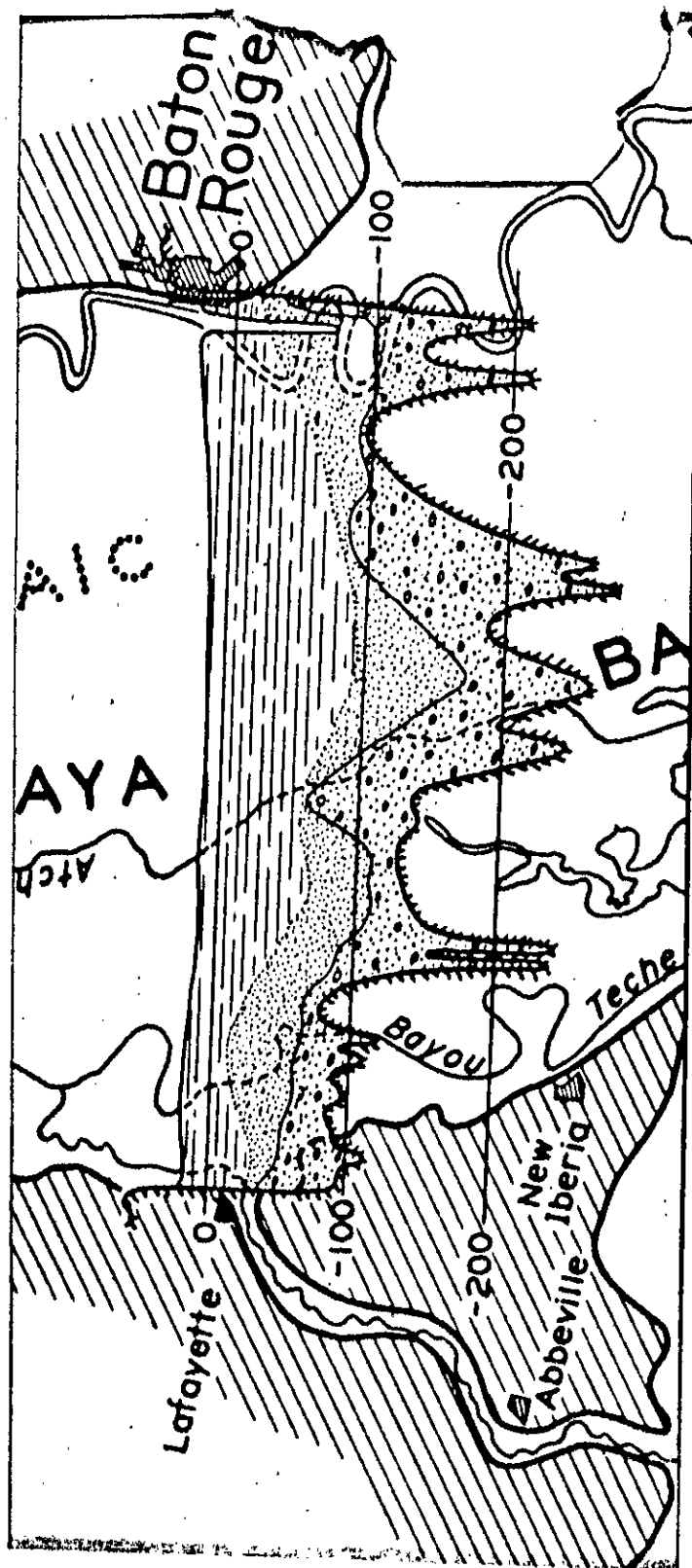
be divided similarly into these three categories: The natural levee ridges along the banks of the rivers and streams; the forested swamps, known as the "Sunder ban" in the southern district of Khulna; and the marshy areas distributed all over the area with the biggest one in the district of Faridpur, shown by Figure 2.

CHAPTER V

DETAILED STUDY OF A SPECIFIC TYPICAL AREA

The excellent report of the geology of the lower valley and deltaic plain of the Mississippi River by Fisk, as reviewed in the preceding chapter, forms the basis for specific engineering examination of the kind, condition and engineering properties of the alluvium. Study of the transvalley profiles, shown by Figure 5, resulted in the selection of the profile at the latitude of Lafayette - Baton Rouge, Louisiana, for detailed examination. This profile has been enlarged to show the general features and is presented as Figure 6. It is to be noted that a major part of this profile crosses the undeveloped low lying marsh land adjacent to the Atachafalaya River. This area is considered to be similar to the most difficult terrain existing in the southern region of East Pakistan, and therefore should represent similar problems to that area. Further, this profile lies near the upper limit of the deltaic plain of the Mississippi River. A corresponding transvalley profile of the valley of the Brahmaputra River is estimated to lie approximately 25 miles south of Dacca.

The transvalley profile selected, shown by Figure 6, crosses an area where the water level is either slightly above or below the ground surface. Thus the area would represent a currently aggrading delta. Further, the dense vegetation, vegetative debris



and loose unconsolidated near-surface soils represent the most undesired foundation conditions that could be visualized. Therefore, solutions for foundation problems in such an area would represent the most adverse circumstances anticipated in East Pakistan.

Fortunately, deep exploration and examination of a transvalley profile nearby and paralleling the one reported by Fisk at this latitude was being made during the summer of 1964. It was for a segment of a new Interstate Highway between the cities of Lafayette and Baton Rouge. The part of the profile examined was approximately 18 miles long extending from about 3 miles west to 15 miles east of the Atachafalaya River, as shown by Figure 5. The deep examination was achieved by three-inch diameter borings extending through the non-graveliferous and into the graveliferous deposits. Undisturbed samples of the non-graveliferous deposits were obtained with the standard three-inch Shelby Tube sampler, extruded and sealed in wax in the field for delivery to the laboratory. Standard penetration samples of the latter graveliferous deposits were secured and sealed in the conventional glass jars. The sampling devices and techniques used are described in reference 10, Art. 44.

Three widely spaced borings were used for this study. It is realized that it is desirable to examine the foundation media across an entire transvalley profile; however, such a program would far exceed the scope of this dissertation. The basic

objective is to study critically the pertinent factors and principles involved in the problem of foundations on unstable materials. Therefore, the borings were selected to satisfy this objective. The borings and their relative locations are enumerated as follows: (a) Boring S1A - 24 was located near the west end, (b) Boring S1A - 91 near the mid length, and (c) Boring S1A - 156 near the east end of the line examined. The depths of the borings were 125 feet. It may be noted from the profile, prepared by Fisk, that the thickness of the non-graveliferous deposits are shown to be at variance to some extent with the depths measured by the borings. This is to be expected due to the undulations normally found at the contacts between deposits of this nature.

The borings used for obtaining the soil samples were made by the rotary method. The drill rigs were mounted on "pull boats" that are capable of operating in shallow marsh areas as well as on low lying lands. Each rig was equipped with hydraulically powered rams for pushing the undisturbed samplers into the foundation materials. In addition, they were equipped for driving the standard penetration samplers into the cohesionless graveliferous materials.

The Soil Profile of Study Area

The results of the exploration showed that the non-graveliferous deposit extended to depths ranging from approximately 80 to 95 feet.

The near-surface materials, extending to an average depth of 20 feet, are very soft unconsolidated fine-grained cohesive clay containing a large proportion of decaying vegetative matter. The remaining portion was found to be highly plastic inorganic clay, described in detail later in this chapter. The graveliferous deposit was encountered at depths ranging from 80 to 95 feet and was explored to an average total depth of about 125 feet. It was found to grade with depth from silty sand, upon first being encountered, into medium to coarse sands at the limiting depth examined. The sands were found to be in a very dense condition, as evidenced by the results of standard penetration tests made in place. The driving records for the tests averaged 50 blows or more of the hammer for one foot of penetration into the coarse sand.

In order to determine the kind, condition and the engineering properties of the foundation media, selected samples of it were subjected to detailed laboratory tests and examinations. To establish the kind of the materials, the Unified Soil Classification System was used. The Plasticity Chart of that system coupled with the Atterberg limits of the materials, presented in Table 3, shows the non-graveliferous deposits to be highly plastic inorganic clay designated by the symbol CH. Visual examination revealed that the graveliferous deposits ranged from very fine to medium to coarse sand. To establish the approximate condition of the materials, other physical

TABLE 3.
Summary of Soil Constants

Boring No.	Depth	L. L.	P. L.	P. I.	m. c. %	Wet Unit Weight pcf	Dry Unit Weight pcf
S1A-24	20'-22'	111.2	35.8	75.4	53.1	106	69
	40'-42'	80.0	27.2	52.8	39.9	113	81.5
	60'-62'	82.2	31.4	50.8	53.4	104	68
	80'-82'	88.5	31.8	56.7	61.75	101	62.5
	90'-92'	56.6	24.3	32.3	47.7	110.4	70
S1A-91	30'-32'	89.8	30.8	59.0	41.7	114.6	81
	50'-52'	80.7	29.9	50.8	57.2	102	65
	75'-77'	63.4	21.6	35.8	36.4	116.4	85
S1A-156	20'-22'	102.4	26.1	76.3	58.6	102	64.5
	50'-52'	78.4	28.3	50.1	51.2	108.2	71.5
	80'-82'	53.2	21.4	31.8	38.3	119	86

properties, such as moisture content and dry unit weight were determined. The results of these tests are shown in Table 3.

To establish the engineering properties of the materials, standard consolidation tests were performed on the representative undisturbed samples of the clay soils. The result of these tests are used to determine the state of consolidation of the material in situ, the amount and rate of settlement of structures that may be founded thereon. The shear strength characteristics of the materials were established by the unconsolidated-quick tri-axial shear tests. The results of the shear tests are used to establish the stability of sub-structural elements such as footings, rafts, and piles for supporting structures founded on the deposits.

All the tests reported were performed in accordance with the procedure set forth in Soil Testing for Engineers by T. W. Lambe (John Wiley & Sons, 1962). Detailed explanation of the tests and results are presented under headings that follow.

Discussion of the Results of Tests

Soil constants. The purpose of soil constants is to establish the kind and general condition of the materials under consideration. The constants are defined as the Atterberg Limits, moisture contents and unit wet weight-in-place of the soil mass. The values determined are also shown graphically by Figures 17a, 18a, and

19a.

The relation between the liquid limits and the plasticity indices with respect to depth are shown in the above enumerated figures.

For example, the values of the liquid limits decrease from the range of 90 to 110, near the surface, down to the values of 53 to 63 at the lower portions of the clay layer. For the plasticity indices the range is from 50 to 75 at the surface to 32 to 35 at the bottom.

It is known that if all the other factors remain the same, the shear strength of a soil mass is inversely proportional to its liquid limit.

This means that as depth increases the strength of this layer should progressively increase, unless influenced by some other factors.

It will, however, be seen later that this clay layer has apparently been preconsolidated, possibly by desiccation, and as a result, the strength does not actually increase to the extent expected from the foregoing relationships.

The decreasing values of liquid limits and plasticity indices with increase in depth confirm another fact that the particle size of the clay also increases with depth. The finest and the colloidal particles have been deposited at the top while the larger particles possessing lower plasticity occur at the lower levels of this layer.

The sequence of materials in this section suggests that this was originally a deltaic area. At the later stage of the rising sea level, when the ground surface was about 20 feet below its present level,

this area became a backswamp. As a result, the soil in the top 20 feet is very soft and contains decomposed vegetative matter. It is evident that for greater depths, the soils are inorganic and thus were not part of a former backswamp. It is possible the portion of the non-graveliferous deposit between the depths of 20 to 80 feet were deposited under water in a shallow bay whose width was about equal to the width of the valley at that time.

Study of the exploration records shows that the non-graveliferous deposit is not interbedded with lenses or strata of sands or silts but is mainly clay throughout, as described previously. Masters (11) found the materials in a transvalley profile, approximately 10 to 15 miles north of the profile under study, to be of the same nature; that is, without silts or sand lenses. Study of an unpublished report (12) on the deposits at a location about 12 miles south also showed another similar profile. Thus, the reported finding of Fisk, that non-graveliferous deposits are composed of interbedded sand, silt and clay, is apparently not applicable for the upstream area of the deltaic plain. The results reported are considered noteworthy because they serve to extend our knowledge of the non-graveliferous deposits of major rivers.

Moisture Contents. Moisture content is one of the easiest properties

of soil to obtain. It is also one of the most useful in establishing the approximate condition of a stratum or deposit when compared with the liquid and plastic limits of the materials. For example, it is an accepted fact that the moisture content corresponding to the plastic limit is approximately equal to that of the optimum for the standard proctor process of compaction which represents a relatively stable state. Further, when the in-situ moisture content corresponds to the liquid limit, it can be considered to indicate a state of minimum stability or strength. The higher the moisture content of a soil mass, the lower is generally the shear strength.

It may be noticed from the above referenced graphs that the moisture contents of the samples follow a general trend for all the borings. The inorganic clay layer ranges from a depth of about 20 feet from ground level to depths of about 80 to 95 feet. The moisture contents for soils from the top of this layer approach the plastic limit, at the middle they are almost midway between the liquid limit and the plastic limit and at the bottom they approach the liquid limits. This is of significance and indicates that, at the top, the soil is preconsolidated to a pressure greater than its present overburden whereas at the bottom it is normally consolidated. The moisture contents at about mid depth of this layer are the maximum. According to Terzaghi's theory of consolidation, this is to be expected. The moisture contents from samples at 20 feet from both

borings S1A - 24 and S1A - 156, however, show relatively higher values of 53.1 and 58.6 per cent compared to 53.4 and 57.2 per cent respectively at mid depth of these borings. This is explained by the fact that the values of liquid limits of these two samples are quite high, 111.2 and 102.4, respectively, compared to those for samples from other points. The general trend of the values of moisture contents is confirmed from the results of consolidation tests reported later in this chapter.

The unit weight. The wet unit weights of the soil from boring S1A - 91 are 114.6 pcf and 116.4 pcf at the top and bottom respectively, and 102 pcf at the middle of the layer. It may be noted from Table 3 that the moisture contents for the samples vary inversely as their unit wet weights, as expected.

The data obtained from samples of borings S1A - 24 and S1A - 156 show the same general trend, as discussed above, except that at a depth of 20 feet, samples from both these borings show lower values than would normally be expected. It has been mentioned earlier that the samples from this depth showed comparatively higher values for liquid limits which suggested that the particle sizes of these soils were much smaller. A soil mass composed of small particles will have a high void ratio and will naturally possess a low unit weight. The moisture contents of these samples were also found to be higher, 53.1% and 58.6% respectively, compared to

41.7% for the sample from a depth of 30 feet from boring S1A - 91.

This general trend that the unit weights are lower at the middle and higher at the top and bottom is also depicted by the dry unit weights shown in Table 3 with the two exceptions mentioned above.

Consolidation Characteristics

From the point of view of the state of consolidation, a clay layer can be in one of the three conditions: underconsolidated; normally consolidated; and, preconsolidated or overconsolidated. A clay layer is termed to be underconsolidated when all the primary consolidation has not taken place under the present overburden pressure. When all the primary consolidation has taken place the layer is said to be normally consolidated. If the layer has been consolidated under a pressure greater than its present overburden, the layer is pre- or overconsolidated. Overconsolidation is possible when either part or whole of the previous overburden load has been removed by some action like erosion, or the layer had been subjected to desiccation.

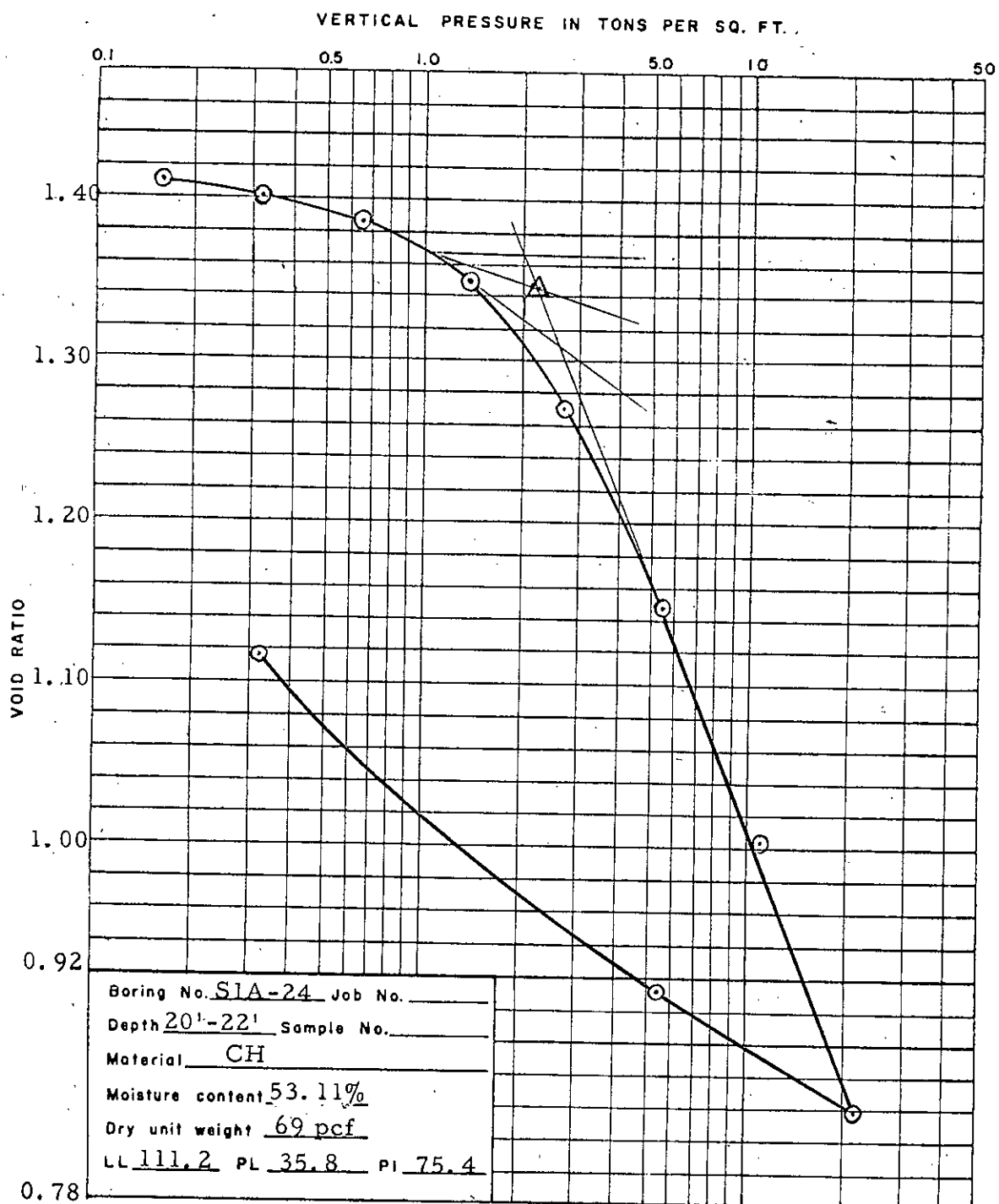
The consolidation characteristics of the non-graveliferous portion of the alluvium were established by use of the Casagrande fixed-ring type of consolidometer. Undisturbed specimens 2.50 inches in diameter and 0.722 inch high were employed. Drainage of the specimen was provided at both the top and bottom surfaces

of the specimens. Results of these tests are shown graphically in the form of e -log p curves in Figures 7 through 16.

Adopting the method, developed by Cassagrande, the preconsolidation pressures existing at different depths for all the three sites were calculated. These values along with the values of the present overburden pressure are recorded in Table 4 and are also shown graphically in Figures 7 through 16.

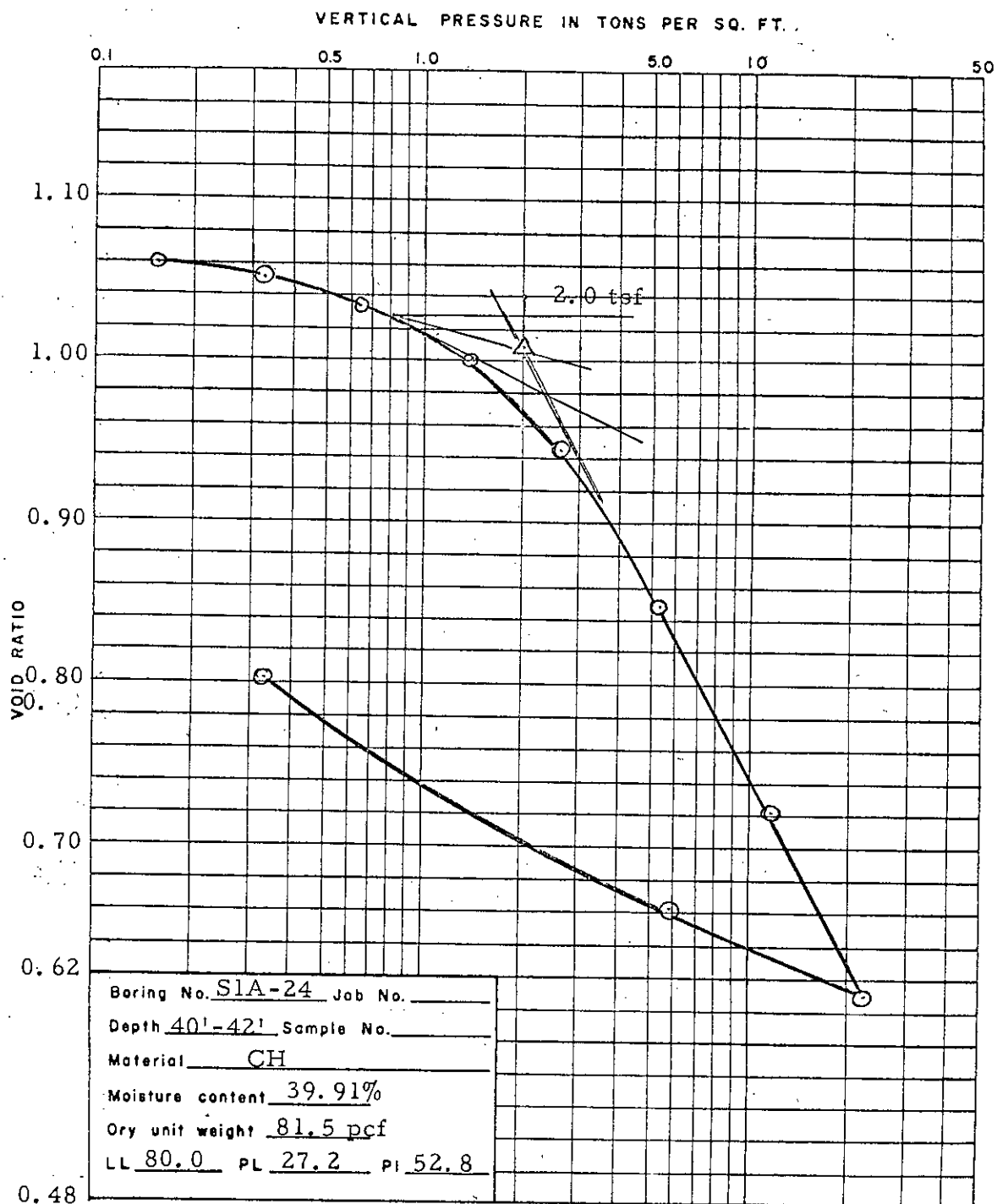
Examination of these graphs for materials from boring S1A - 24 shows that the soil from the depth of 20 feet to about 60 feet is preconsolidated. The degree of preconsolidation decreases with increase in depth. Below the depth of 60 feet the soil appears to be normally consolidated at this site, thus indicating that the soil below the indicated depth has never been subjected to overburden pressure greater than exists at present. The foregoing is significant in that, as reported from the results of geological investigation, which describes the near surface portion of the deposit to be of most recent origin. However, the preconsolidated state for the portion of the mass from about 20 feet down to 60 feet is unusual and has been found in only one previous instance. The only logical explanation that can be offered for this finding is that through some unusual circumstance the deposit experienced desiccation, the effect of which extended to a depth of about 60 feet.

The difference between the two curves showing the preconsolidation



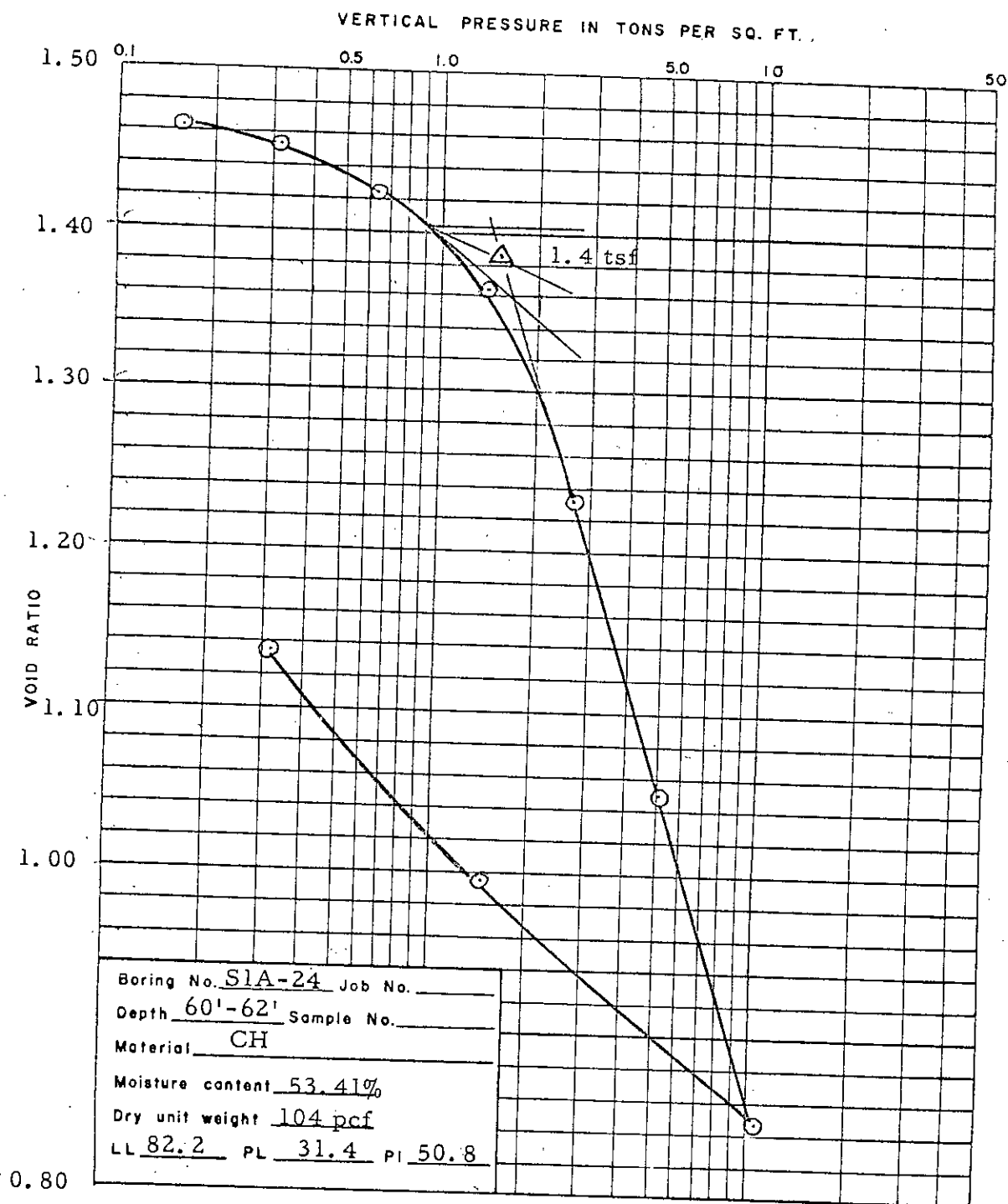
VOID RATIO VS LOG PRESSURE CURVE

Figure 7



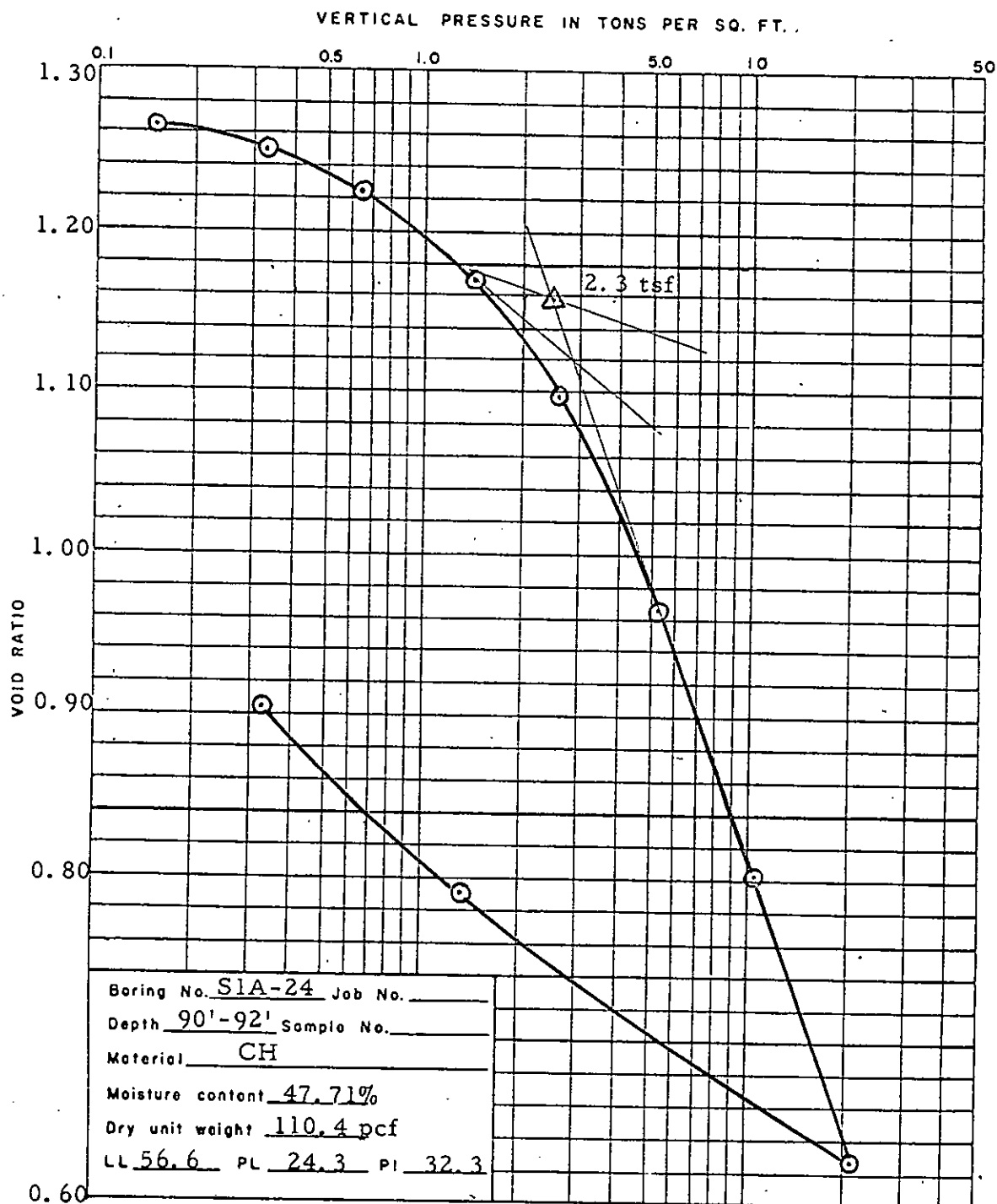
VOID RATIO VS LOG PRESSURE CURVE

Figure 8



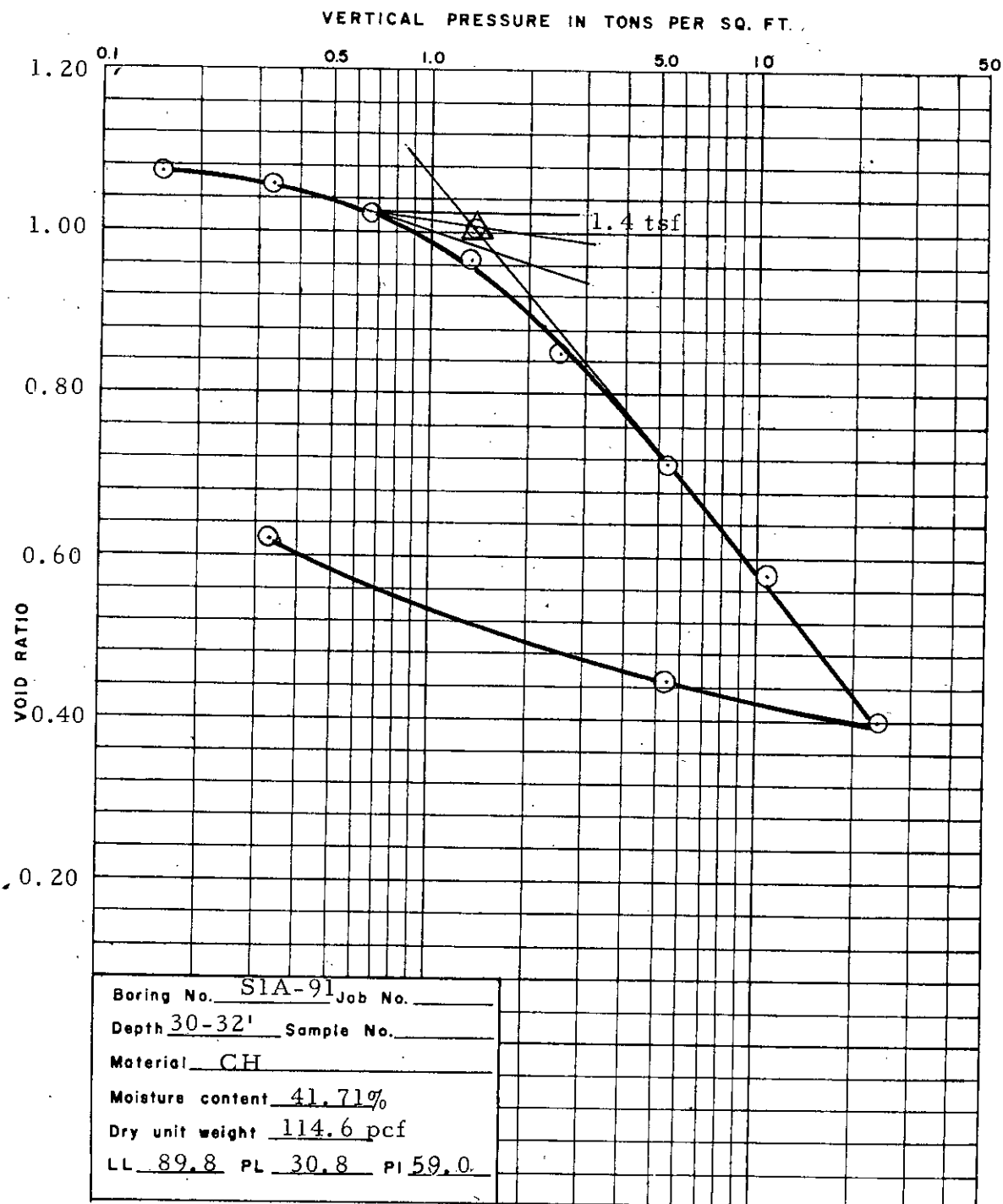
VOID RATIO VS LOG PRESSURE CURVE

Figure 9



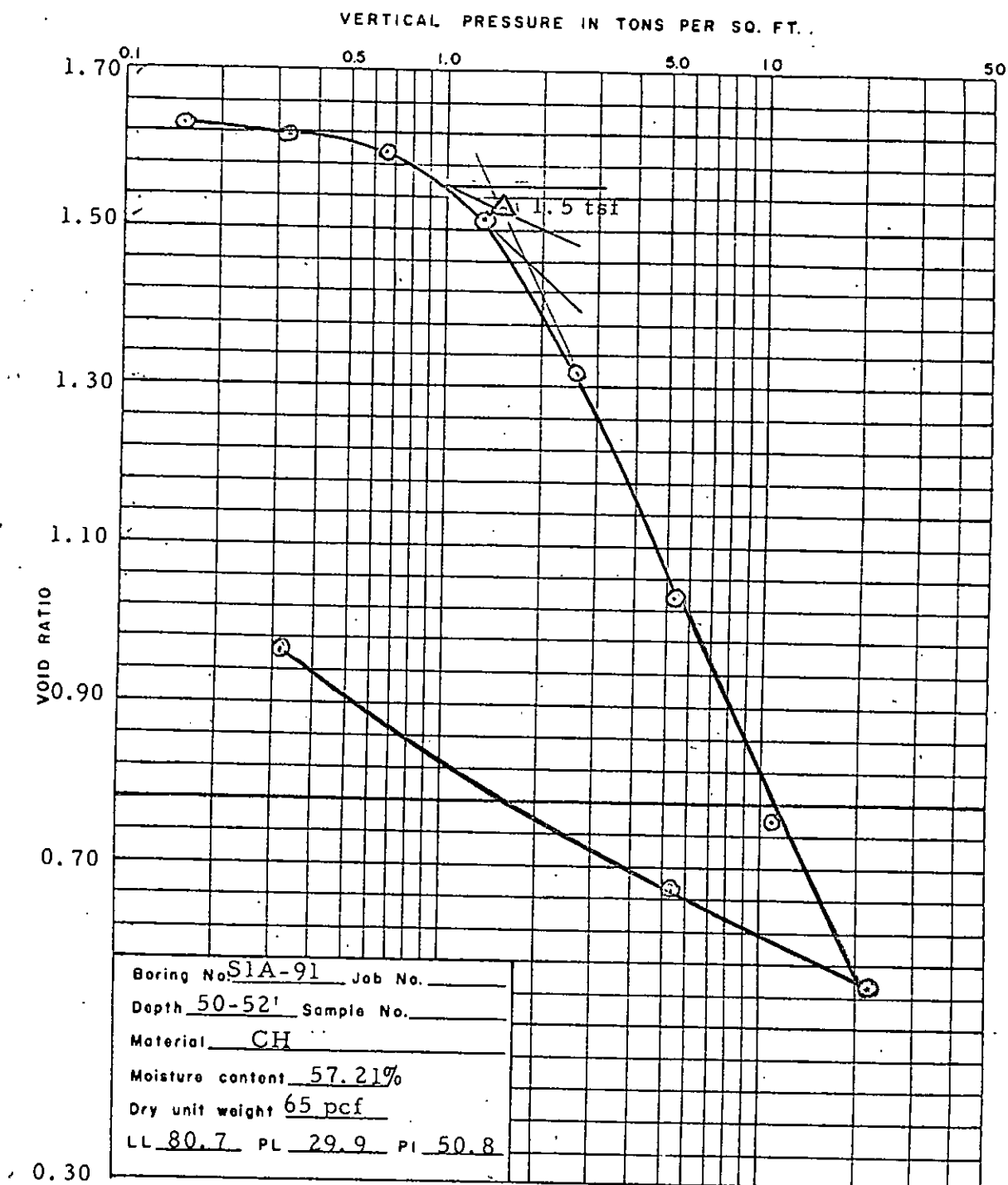
VOID RATIO VS LOG PRESSURE CURVE

Figure 10



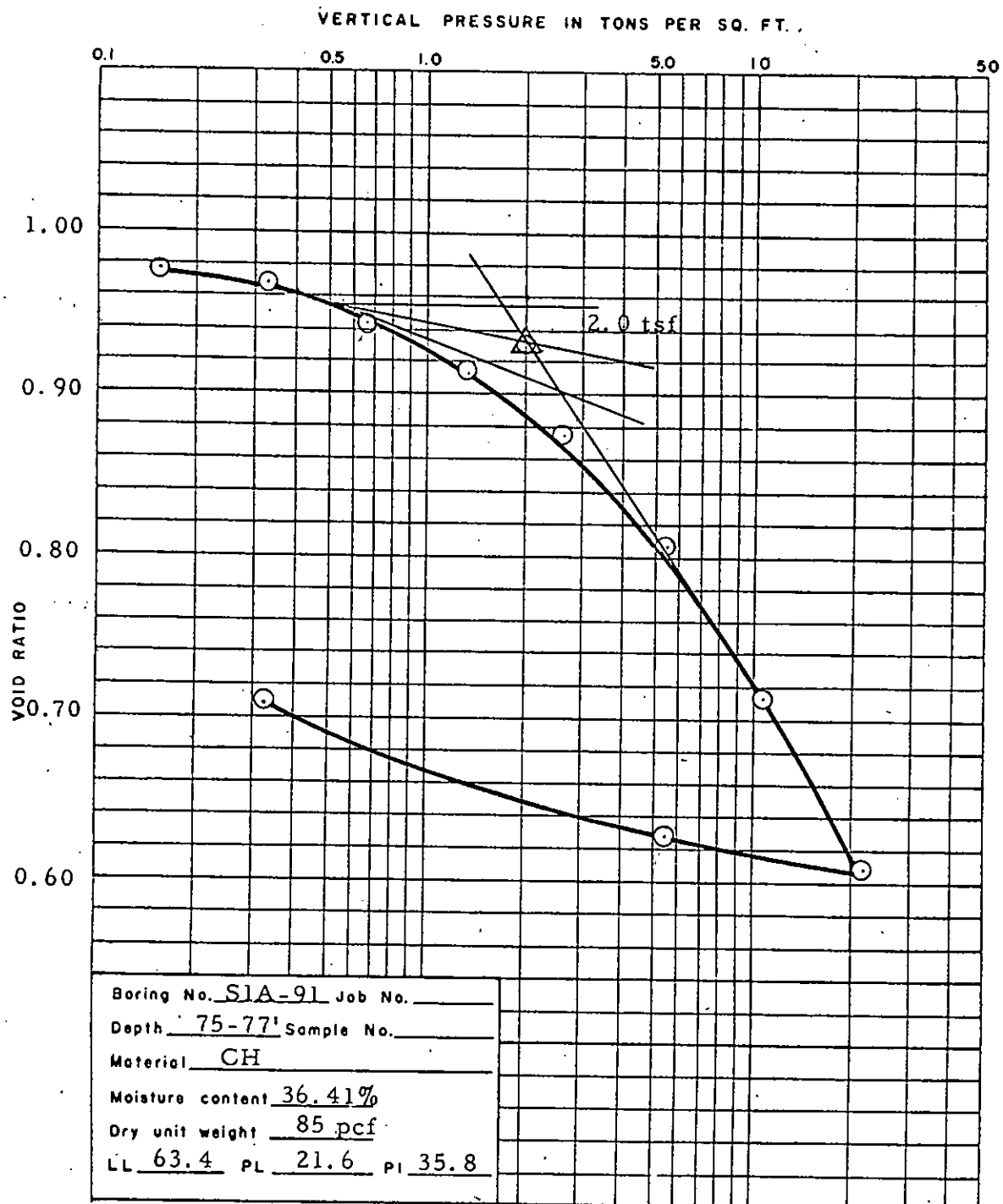
VOID RATIO VS LOG PRESSURE CURVE

Figure 11



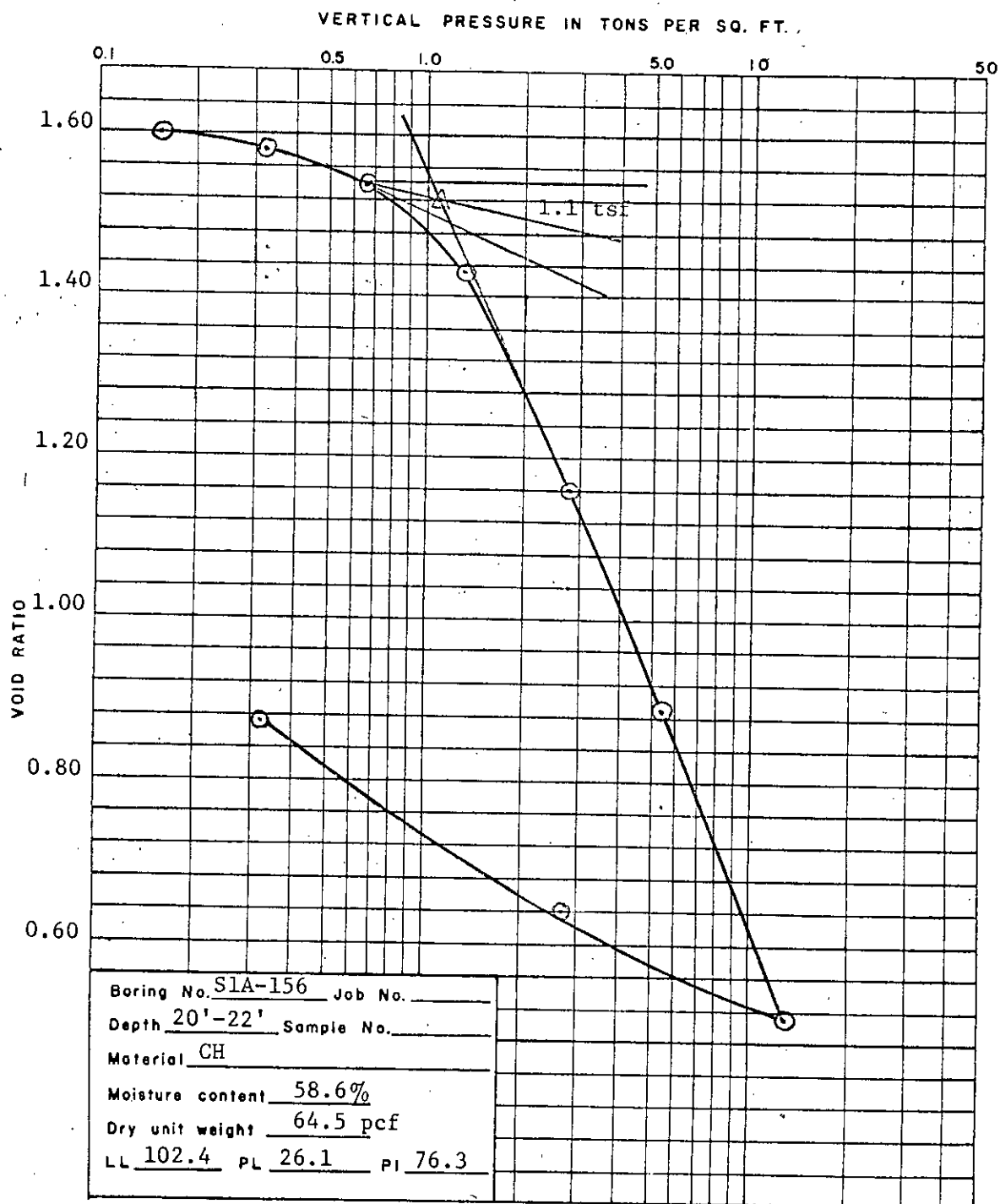
VOID RATIO VS LOG PRESSURE CURVE

Figure 12



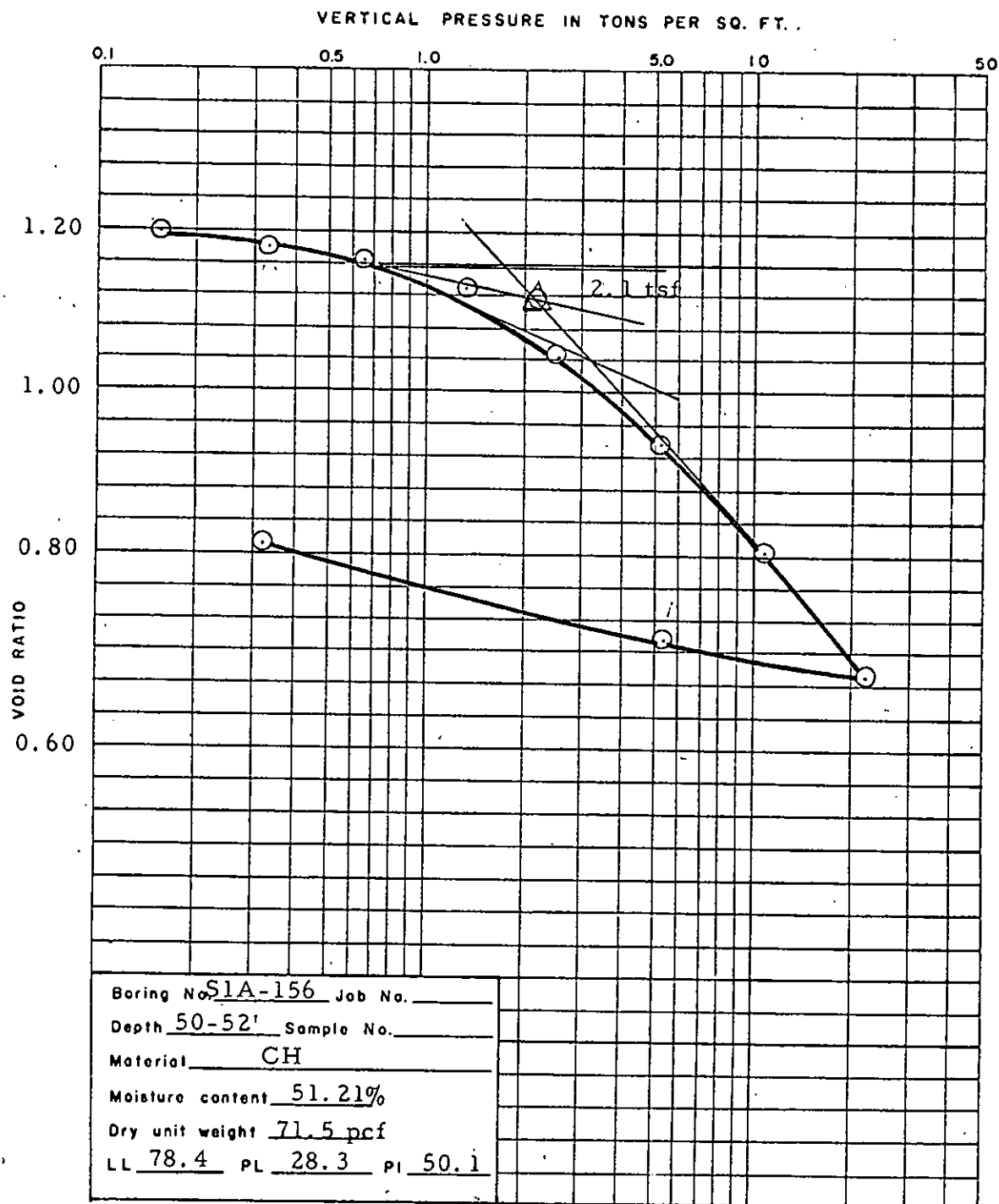
VOID RATIO VS LOG PRESSURE CURVE

Figure 13



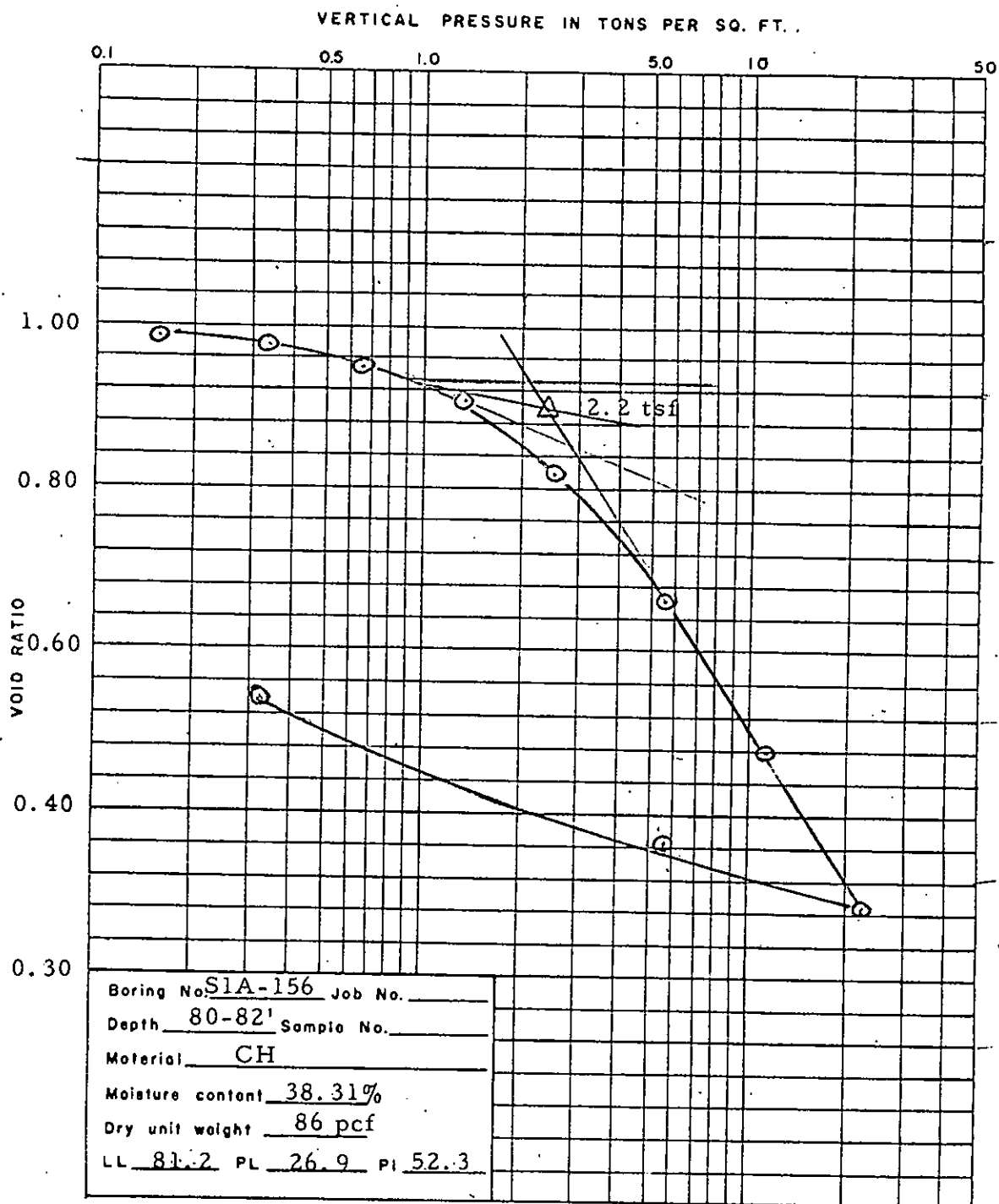
VOID RATIO VS LOG PRESSURE CURVE

Figure 14



VOID RATIO VS LOG PRESSURE CURVE

Figure 15



VOID RATIO VS LOG PRESSURE CURVE

Figure 16

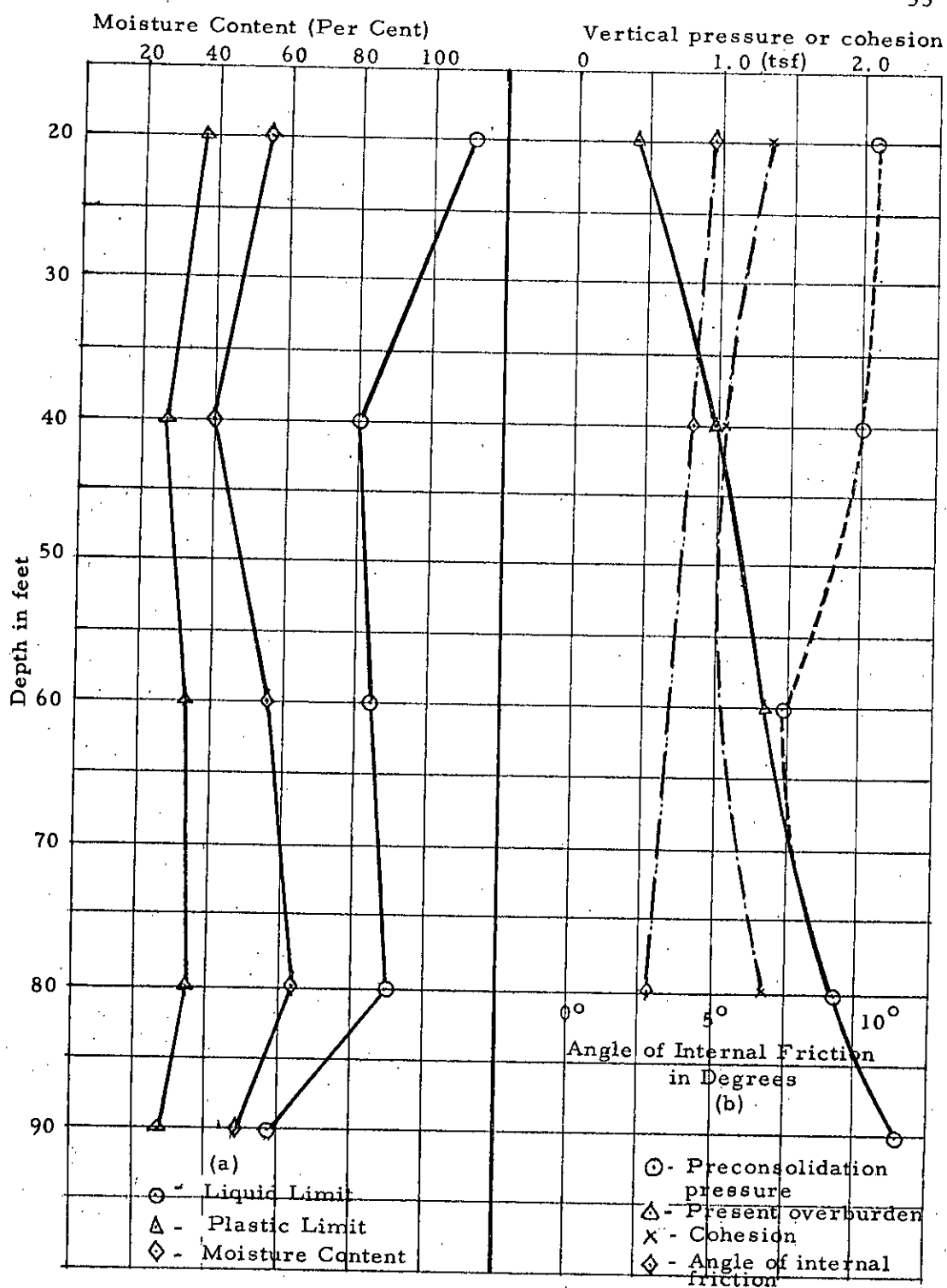


Figure 17. Soil Constants and Consolidation and Shear Characteristics of Soil Sample S1A-24.

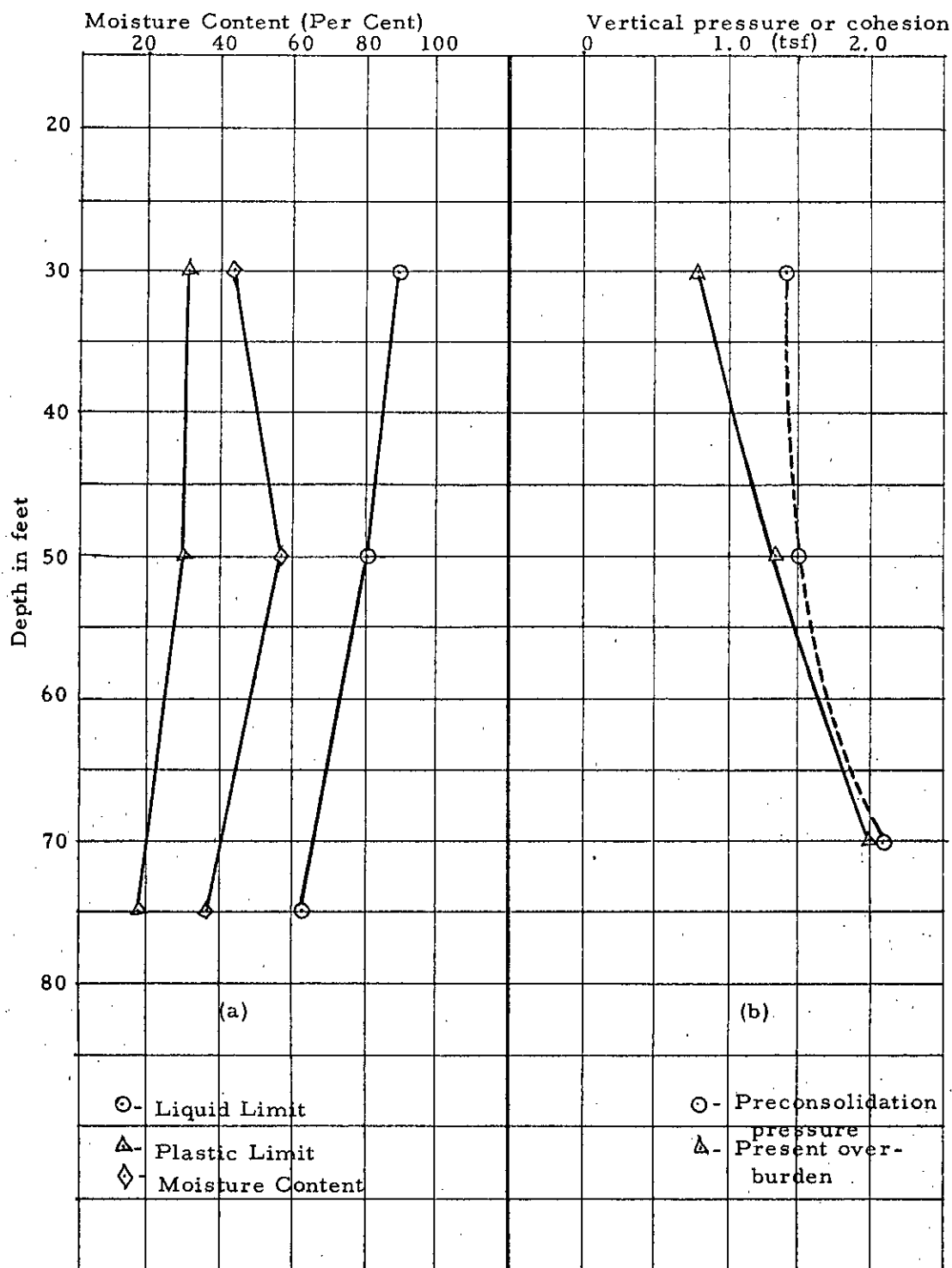


Figure 18. Soil Constants and Consolidation and Shear Characteristics of Soil Sample S1A-91.

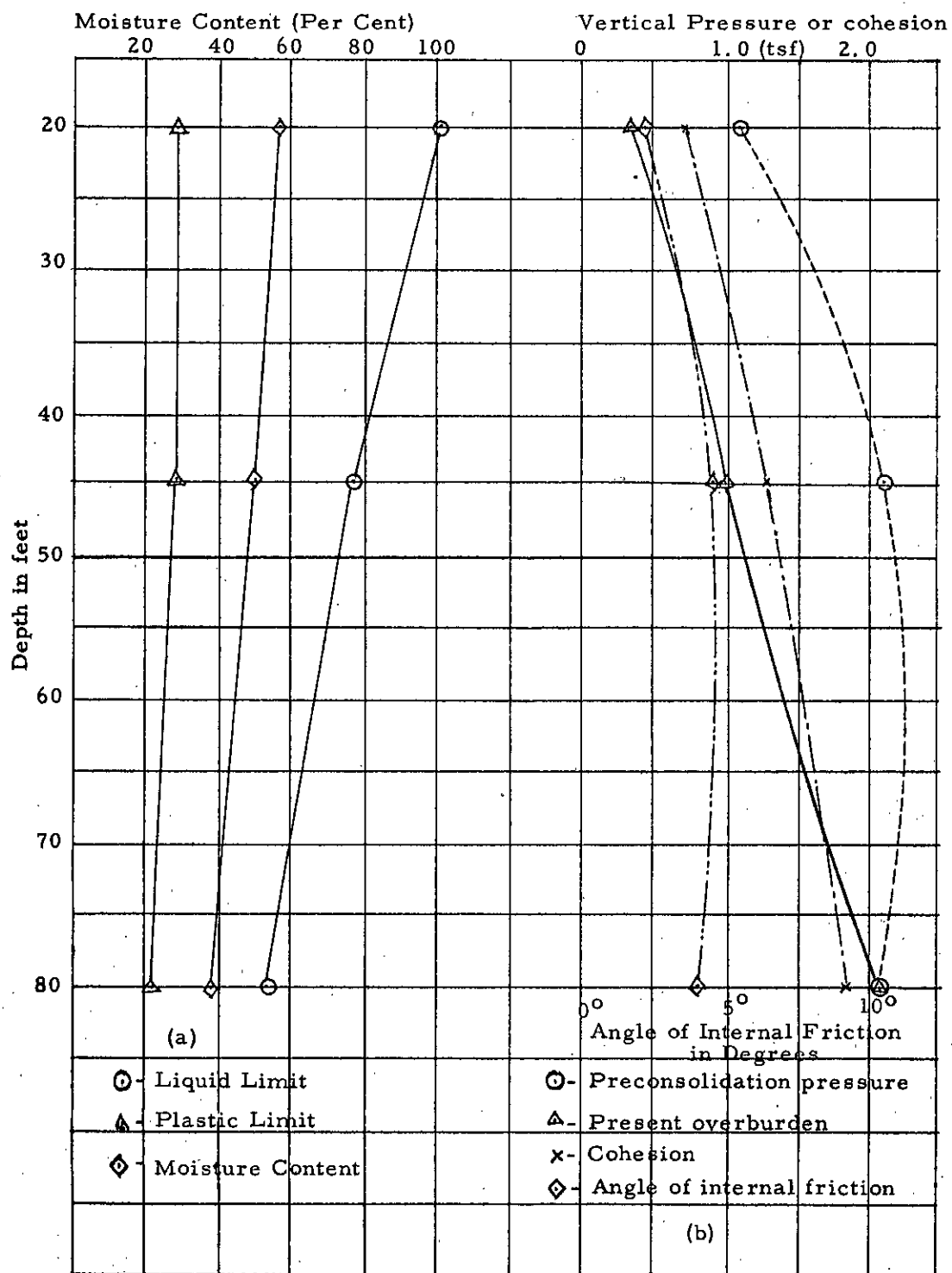


Figure 19. Soil Constants and Consolidation and Shear Characteristics of Soil Sample S1A-156.

pressure and the present overburden pressure, as recorded, can be noted from Figure 17b. This difference can be taken to be the additional vertical stress the material may withstand without experiencing any appreciable settlement. For example, at depth 20 feet of site S1A - 24, the preconsolidation pressure is 2.1 tsf whereas the present overburden pressure is only 0.45 tsf. Therefore, at this level an additional vertical stress of 1.65 tsf can safely be applied, so far as settlement is concerned, without consequence. Though the difference between the two above mentioned curves decreases with increase of depth, it may be remembered that the vertical stress, due to the application of loads at the surface level, also decreases with increase of depth. Advantage of this situation must be taken at the time of design.

Similar information for materials from borings S1A - 91 and S1A -156 is shown by Figures 18b and 19b. It may be noticed from these figures that the degree of preconsolidation is less for materials from boring S1A - 91 than for materials from boring S1A - 24. Also, the soil appears normally consolidated at a depth of about 50 feet as against 60 feet for boring S1A - 24.

At site S1A - 156, Figure 19b, the preconsolidation load is slightly higher than at site S1A - 91 but lower than at site S1A - 24. The degree of preconsolidation is uniform from 20 feet down to about 50 feet and then it decreases and becomes equal to the present

overburden pressure at the interface between the clay layer and the underlying sand layer at a depth of about 80 feet. The test results obtained for all the three sites appear to be consistent within the limits which may be expected in such cases.

The foregoing discussion of the consolidation characteristics shows that the non-graveliferous portion of the alluvium in the upper reaches of a deltaic plain may be in a more advanced state than has been considered by geologists and engineers. The soil from the entire section of 18 miles, examined and reported here, appears to have gained strength and stability through desiccation. However, this will allow this layer to withstand a considerable amount of additional vertical stress. This information is most significant and may be utilized to advantage in designing foundations of structures.

Investigation in the case of Dow Chemical Company's Plant at Plaquemine, Louisiana, a place 12 miles south of Baton Rouge, also disclosed that the upper 20 to 40 feet of soil there was pre-consolidated, probably due to desiccation. This is reported in the next chapter in detail.

Tri-axial Shear Test Results

The shear strength of a cohesive soil is made up of two components: the cohesion, c , and internal friction, as in cohesionless soil. The exact nature of surface forces which cause cohesion is

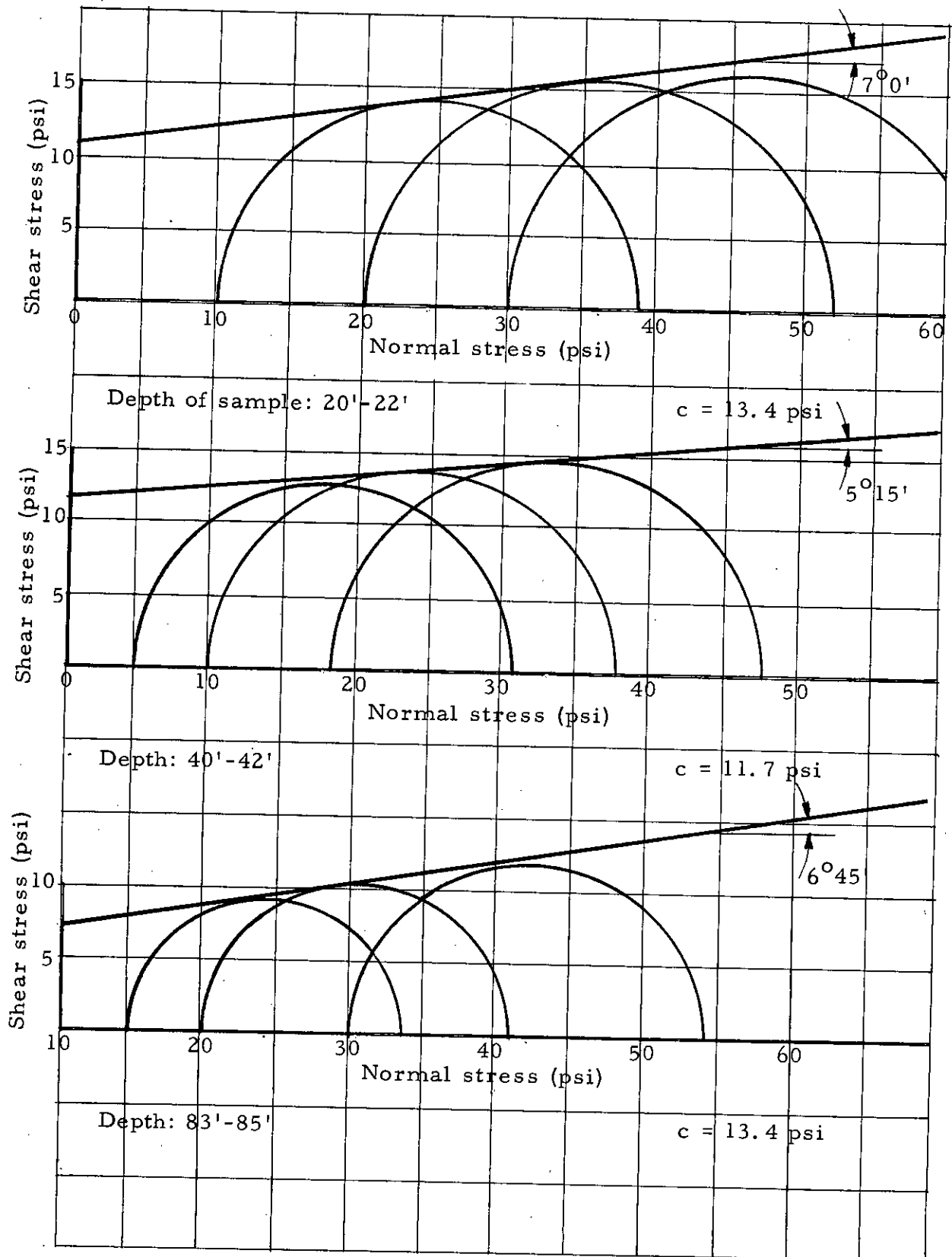


Figure 20. Mohr's Diagrams for Samples S1A-24.

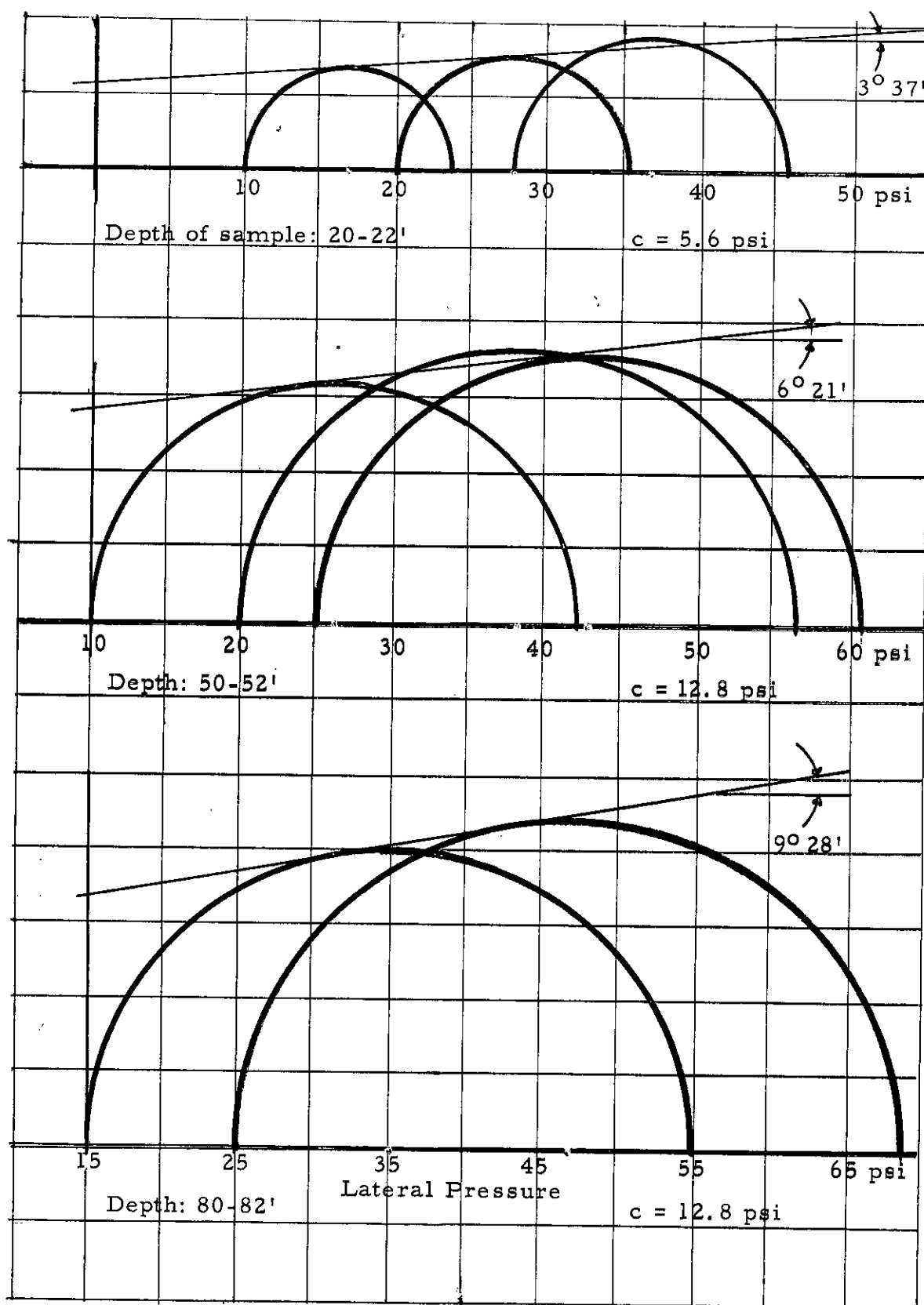


Figure 21. Mohr's Diagrams for Samples S1A-156.

not known. The cohesion of a soil is not a constant soil property but is a function of the load carried by the soil structure (13) and the ad-sorbed water complex.

In order to measure the values of c , the cohesion, and ϕ , the angle of internal friction, which combined give a measure of the shear strength of a soil, tri-axial shear tests were performed. To determine the in-situ strength, the unconsolidated-undrained, known as the Q-test, was used. Samples used in the tests were 1.25 inches in diameter and 2.50 inches high. Lateral pressure was applied by compressed air. Materials from borings S1A - 24 and S1A - 156 only were used.

The results of the tests are plotted in the form of Mohr's diagrams and the values of c and ϕ determined from the plots, Figures 20 and 21. These values are graphically shown in Figures 17b and 19b. The maximum value of c was obtained as 13.4 psi for the sample from a depth of 20 feet for boring S1A - 24. This sample was found to be preconsolidated under pressure of 2.1 tsf. The values of c gradually decreased to 7 psi at a depth of 80 feet. The values of ϕ ranged between $5^{\circ} 16'$ and $6^{\circ} 58'$.

The value of c for the sample from 20 feet depth for boring S1A - 156 was 5.6 psi. It may be noted that the preconsolidation pressure for this sample was found to be only 1.1 tsf compared to 2.1 tsf for boring S1A - 24. The maximum value of c recorded for

TABLE 4

Summary of Results of Consolidation and Triaxial

Tests

Boring No.	Depth	Preconsolidation pressure tsf	Present overburden pressure tsf	State of consolidation	Value of c psi	Value of ϕ
S1A-24	20'-22'	2.10	0.45	Preconsolidated	13.4	7° 0'
	40'-42'	2.00	1.00	Preconsolidated	11.7	5° 15'
	60'-62'	1.40	1.35	Normally Consolidated	- - -	- - -
	80'-82'	1.80	1.80	"	7.0	6° 45'
	90'-92'	2.30	2.30	"	- - -	- - -
S1A-91	30'-32'	1.40	0.80	Preconsolidated	- - -	- - -
	50'-52'	1.50	1.20	"	- - -	- - -
	75'-77'	2.00	2.00	Normally Consolidated	- - -	- - -
S1A-156	20'-22'	1.10	0.40	Preconsolidated	5.6	3° 30'
	50'-52'	2.10	1.00	"	12.8	6° 15'
	80'-82'	2.20	2.10	Normally Consolidated	11.4	9° 30'

the samples from this boring was, however, 12.8 psi. The values of ϕ increased from $3^{\circ} 37'$ at a depth of 20 feet to $9^{\circ} 28'$ at a depth of 80-82 feet. These values of c and ϕ appear consistent with the results of consolidation tests.

The Standard Penetration Test Results

The results of the standard penetration test performed on the graveliferous deposits at the three sites are shown in Table 5. From this table it can be seen that, with increase in depth into the sand deposit, the density changes from a medium to a dense to a very dense state within the range of a few feet. For boring S1A - 24, the deposit appears very dense within 10 feet from its top surface. For the other two sites also, the sand becomes very dense at about the same depth of about 110 feet below ground level. This indicates that a very firm foundation medium is available at this depth. It should be capable of supporting heavy loads through long piles or piers founded on it. The confining pressure, which adds to its bearing capacity, is also great at this depth.

Mineralogical Studies of the Sand Samples

In order to determine the mineralogical composition of the sand samples, X-ray diffraction studies were made on samples from all the three borings. The X-ray diffraction curves, obtained, are reproduced in Figure 22. The principal constituents of the materials

TABLE 5
Results of Standard Penetration Test

Boring No.	Depth	No. of Blows per foot
S1A-24	105'	42
	110'	50
	115'	50 for 10"
	120'	37
	125'	54
S1A-91	80'	18
	85'	21
	90'	14
	95'	19
	100'	15
	105'	18
	110'	50 for 8"
	115'	50 for 9"
	120'	50 for 10"
S1A-156	90'	21
	95'	21
	100'	22
	105'	32
	110'	50 for 9"
	115'	50 for 11"
	120'	50 for 10.5"
	125'	46.

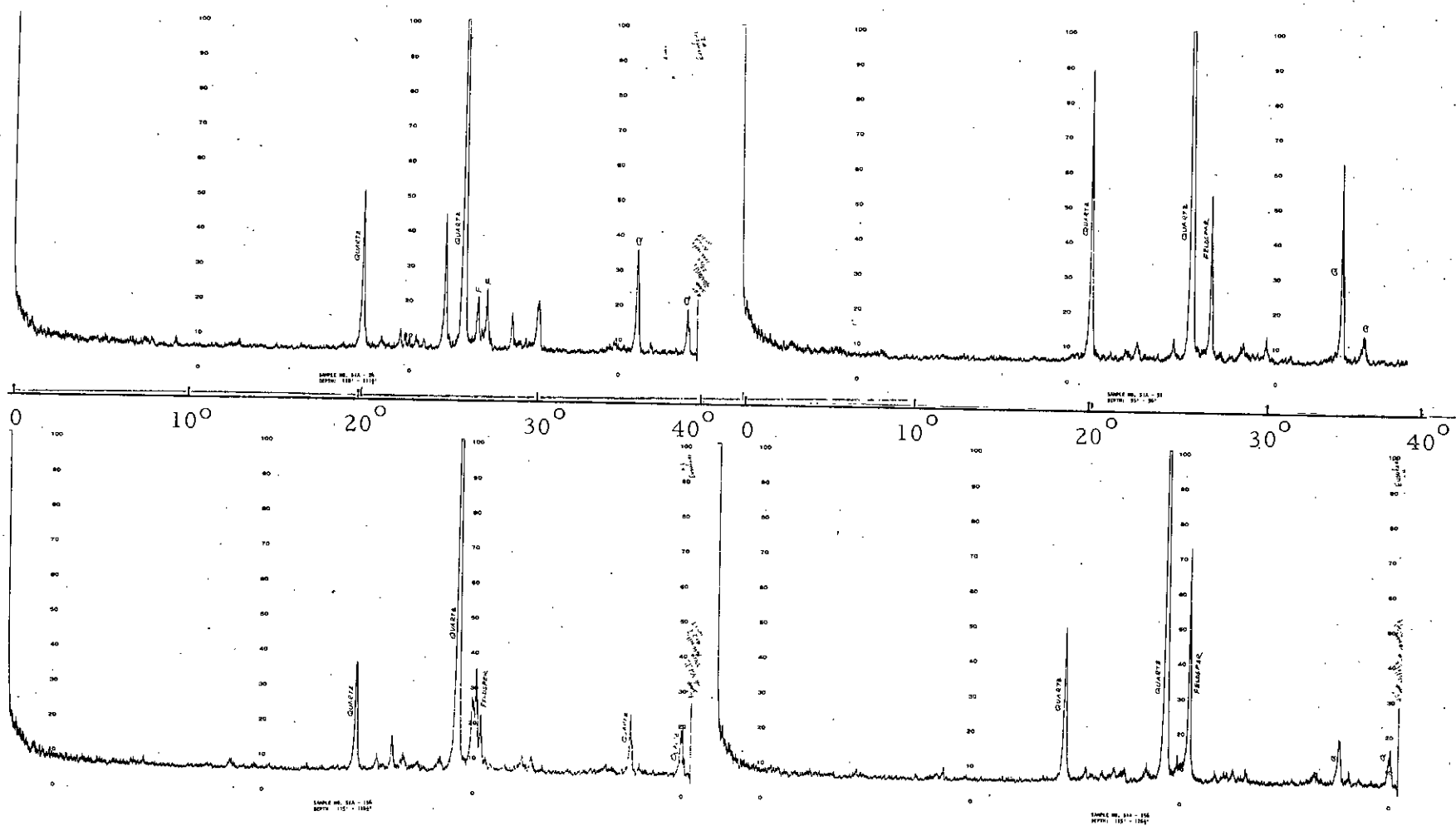


Figure 22. X-ray Diffraction Curves for Sand Samples.

appear to be quartz and feldspar. The mineralogical composition of such coarse materials depends, however, on the composition of the parent rocks from where these had been mechanically weathered.

CHAPTER VI

SELECTION OF FOUNDATION SUBSTRUCTURE FOR DIFFERENT TYPES OF FOUNDATION MEDIA

Factors affecting the type of foundation. The major factors influencing the selection of the type of foundation for a structure are as follows: (a) The subsurface condition and the state of the materials therein, (b) the type and functions of the structure itself and the load it imposes, and (c) the relative cost of the different types of foundation substructures.

Because of this interplay of several factors, there are usually several acceptable solutions to a foundation problem. The function of the foundation or the substructure is to adequately transfer the load of the superstructure to the underlying foundation media in such a manner as to assure the normal functioning of the structure.

To serve this purpose, the substructure must be structurally sound and the foundation medium must satisfy two separate criteria. The first criterion is the bearing capacity of the supporting soil, which is a function of the shear strength of the soil, the second is the consolidation characteristics of the soil. A soil mass may have sufficient shear strength to support the load of a structure, but the amount of settlement, which may occur during a certain period of time, may be detrimental to the superstructure. Unlike the superstructure, which is normally considered safe after it stands the first

application of the design load, a foundation media must, in addition, stand through a longer period of time without undergoing settlement that may damage the structure.

The permissible settlement depends upon the type and functions of the structure, while the amount of anticipated settlement is a function of the state and properties of the foundation media. A statically determinate structure can withstand several inches of settlement, whereas a statically indeterminate structure, such as a rigid frame or a continuous structure can tolerate only very small differential settlement. A concrete frame can tolerate less differential settlement than a similar steel frame. The load transmitted to the soil at the base of a bridge pier may be largely due to the self weight of the pier and the settlement may amount to a few inches. However, if the settlement occurs during the construction of the pier, it does not present any problem of significance. On the other hand for a continuous structure, a differential settlement of the order of 0.75 inch between adjacent supports may not be tolerable. For an embankment, the amount of settlement, which may be quite high, is of little importance, provided all of it takes place before any concrete roadway or other rigid structure is built over it, and bearing capacity failure does not take place. Irregular or erratic settlement is more harmful to a structure of any type than uniformly distributed settlement.

Types of Substructures

The different types of substructures, which are normally used on alluvial soil are (a) spread and combined footings; (b) rafts or mats; (c) piles and (d) piers or drilled footings.

Individual or continuous spread footings are principally used to support single concentrated or wall loads at relatively shallow depths on uniform foundation media possessing acceptable stability as regards shear and settlement. In the instances of nonuniform foundation media, resulting in significant differential settlement between footings or within the length of a continuous footing, this type of substructure is not desired. In view of the highly compressible and near surface alluvium found in the deltaic plains of East Pakistan, this type of substructure would probably not be desirable.

A raft or a mat substructure is in effect a large spread footing that covers the entire area beneath a structure and supports all walls and columns. It serves to distribute the applied loads to result in a uniform intensity of low magnitude. For a plastic clay foundation medium, a reinforced concrete raft minimizes the differential settlement and also reduces the total absolute settlement by spreading the load to a greater area.

The previously discussed surface stratum in the alluvial and deltaic areas normally consists of highly plastic and highly compressible clay materials in the upper layer. If a firm layer of coarser materials, such as sands and gravels, is not available at

a depth for the economical use of piles, a properly designed reinforced raft may prove suitable. When a structure must be located over a very low strength and highly compressible deposit, the reinforced concrete raft is sometimes established at a depth such that the weight of the excavated earth fully compensates for the load of the entire structure including the raft. In such a case, the foundation is termed a 'floating foundation.'

When the immediate supporting media is very soft, the load to be supported is very heavy and a firm layer of material is available within a reasonable depth, piles may be used to transfer the load of the structure down to the firm layer. The origin of pile foundation is lost in antiquity. Examples of the use of timber piles below permanent water table have been found in many places. Piles made of timber, steel and concrete (reinforced, prestressed, or cast-in-place), are in common use.

Piles are commonly divided into two categories, the point bearing and the friction piles. A friction pile derives its supporting power principally from the surrounding soil through the shearing resistance between the soil and the pile. A point bearing pile derives its support capacity principally from its bottom tip bearing in dense sand, gravel or rock. Many piles derive their strength from both the sources simultaneously.

The capacity of a point bearing pile depends almost entirely on

the capacity of the material upon which the point finds its bearing and the degree to which the point of the pile has a satisfactory seat on the bearing material (14).

The support capacity of friction piles depends upon the characteristics of the surrounding materials. As a general rule, the structural strength of a pile itself is more than adequate. The size selected is determined by soil characteristics. If the piles are driven in soft clay for which the frictional resistance is small, the difference in support capacity of parallel sided and tapered piles is negligible. For soils possessing appreciable frictional resistance such as sands and silts, the wedge action of a tapered pile increases the lateral pressure and increases the shearing resistance. Thus a tapered pile is likely to be advantageous under such circumstances.

Depending upon the amount of load to be transferred, piles may be used singly or in groups. The capacity of a group of friction piles is taken equal to the sum of the capacities of single piles multiplied by a reduction factor that depends on the number involved as described by Chellis (15). Another practice to determine the capacity of such a group is to consider the friction around the periphery of the group for a depth equal to the length of the piles plus bearing at the base plan of the group. The capacity of a group of end bearing piles is derived as explained, except no reduction factor is used. Through suitable choice of spacing of piles in a group, maximum benefit can be derived for its

capacity.

A pier is defined to be a support, usually of concrete or masonry, used for the support of the superstructure of a bridge. The base of a pier shaft may rest directly on a firm stratum, or it may be supported on groups of piles. Its purpose is to transfer a highly concentrated load, such as from a bridge pier, through weak or erodible strata to a deep firm stratum.

Illustrations of the application of the types of substructures in the deltaic plain of the Mississippi River are presented under the following subheadings.

I. Reinforced Concrete Raft for the Houma Gas Products Plant

A report (16) on the foundation investigation and analysis of the Houma Gas Products Plant in the southern area of the alluvial plain was made available to the author by the designers through the courtesy of Spencer J. Buchanan and Associates, Inc., Consulting Engineers, Bryan, Texas. This site was typical of the lower deltaic plain of the Mississippi River, in that the thickness of the non-graveliferous deposits in this area are more than 100 feet. In this lower portion of the deltaic plain, the deposits were found to be in a relatively unconsolidated state.

For preliminary investigation, four three-inch diameter continuously sampled core borings suitably located in the area were

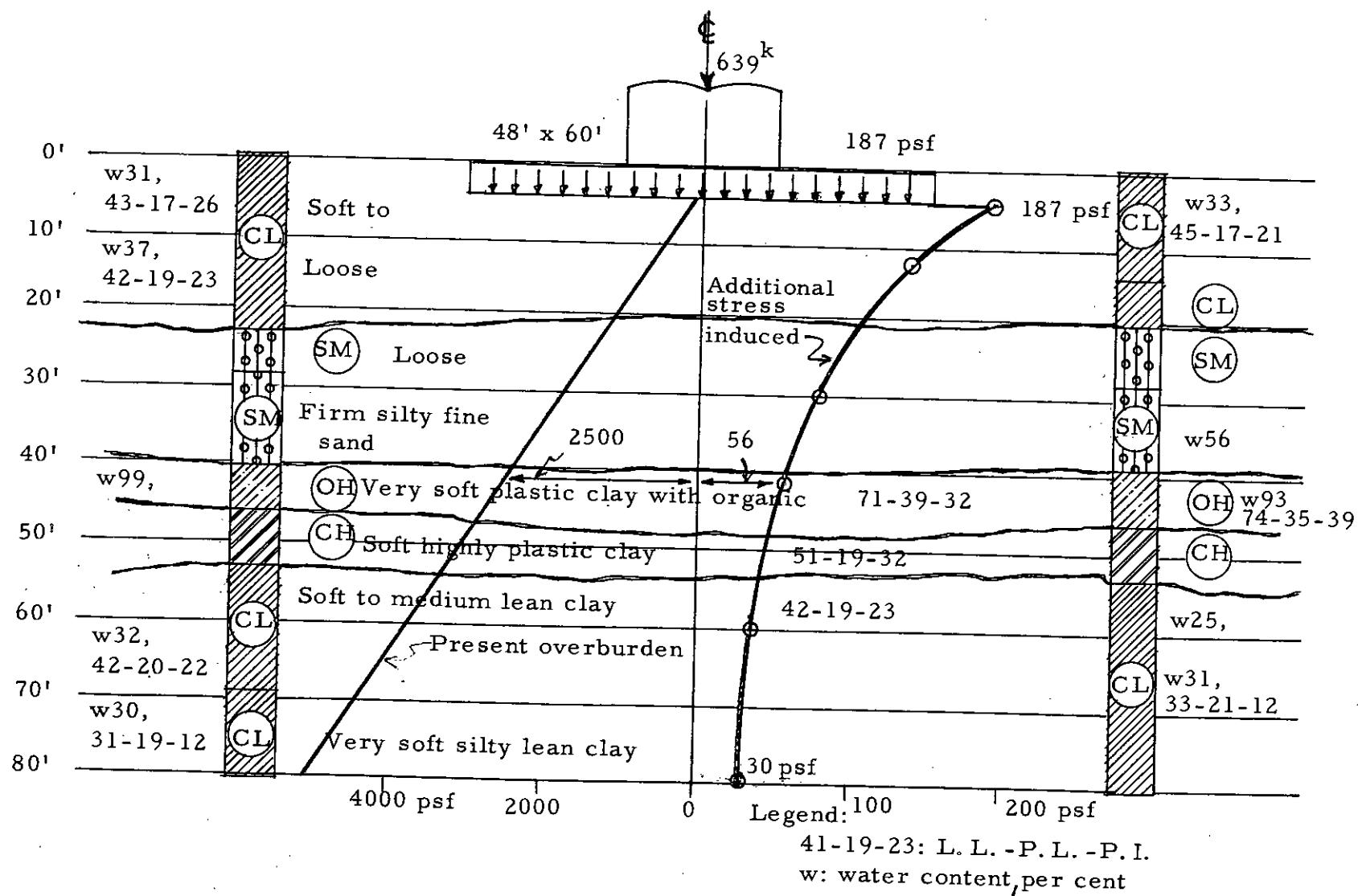


Figure 23. Soil Profile at Houma Gas Products Plant Site, Houma, Louisiana.

made, three to a depth of 40 feet and the other to 10 feet. Results of this exploration indicated a highly organic stratum at a depth of 40 feet. Accordingly, it was necessary to extend three of the borings to a depth of 80 feet in order to determine the extent of the organic stratum and to seek a stable stratum for founding long piles, if such would be required. The soil profile is reproduced here as Figure 23. An examination of the soil profile shows that even at 80 feet from ground level the materials are CL and ML. According to the discussions presented in Chapter IV and Figure 5, however, a firm layer of sand or gravel was not expected at this depth. The presence of the organic layer presents a special difficulty in handling this problem.

A gas products plant involves use of relatively heavy vessels and towers with small bases, thus resulting in concentrated loads over small areas. For example, one vessel weighed 639 kips and another 493 kips, had base areas of about 20 feet in diameter. The limitation of allowable settlement was of the order of one inch.

Two alternate solutions appeared to be open for consideration. The first solution involved possible use of long piles to transfer the loads down to the graveliferous deposits. The second solution involved the use of reinforced concrete raft substructure whose area could be designed to distribute the heavy loads to reduce sufficiently the unit stress applied to the foundation to within the available shear

strength of the supporting medium and to cause settlement within the allowable limit. This was achieved with the resulting unit stress amounting to 137 and 134 psf respectively. The computed settlement was kept within the one inch limitation.

In Figure 23 also shown are the stresses induced at different depths due to the tank producing the maximum stress. It may be noticed from there that the stress of 187 psf at the bottom of the 48' x 60' raft is reduced to only 30 psf at a depth of 80 feet. The diagram also includes a curve showing present overburden pressure. The present overburden at the center of the OH layer is approximately 2500 psf whereas the additional stress induced due to the tank load is only 56 psf which is negligible compared to overburden pressure. At depth 80 feet these values are 4800 psf and 30 psf respectively.

This example illustrates how through the choice of the appropriate type of foundation element and size, significant gain can be achieved in the individual cases of special nature.

II. Timber Friction Piles for Morganza Floodway in Louisiana

Short frictions piles are often used in deltaic and alluvial deposits, when stable sandy strata or other such firm materials suitable for the support of heavy concentrated loads are not available at a reasonable depth. These piles are embedded in the upper silt and

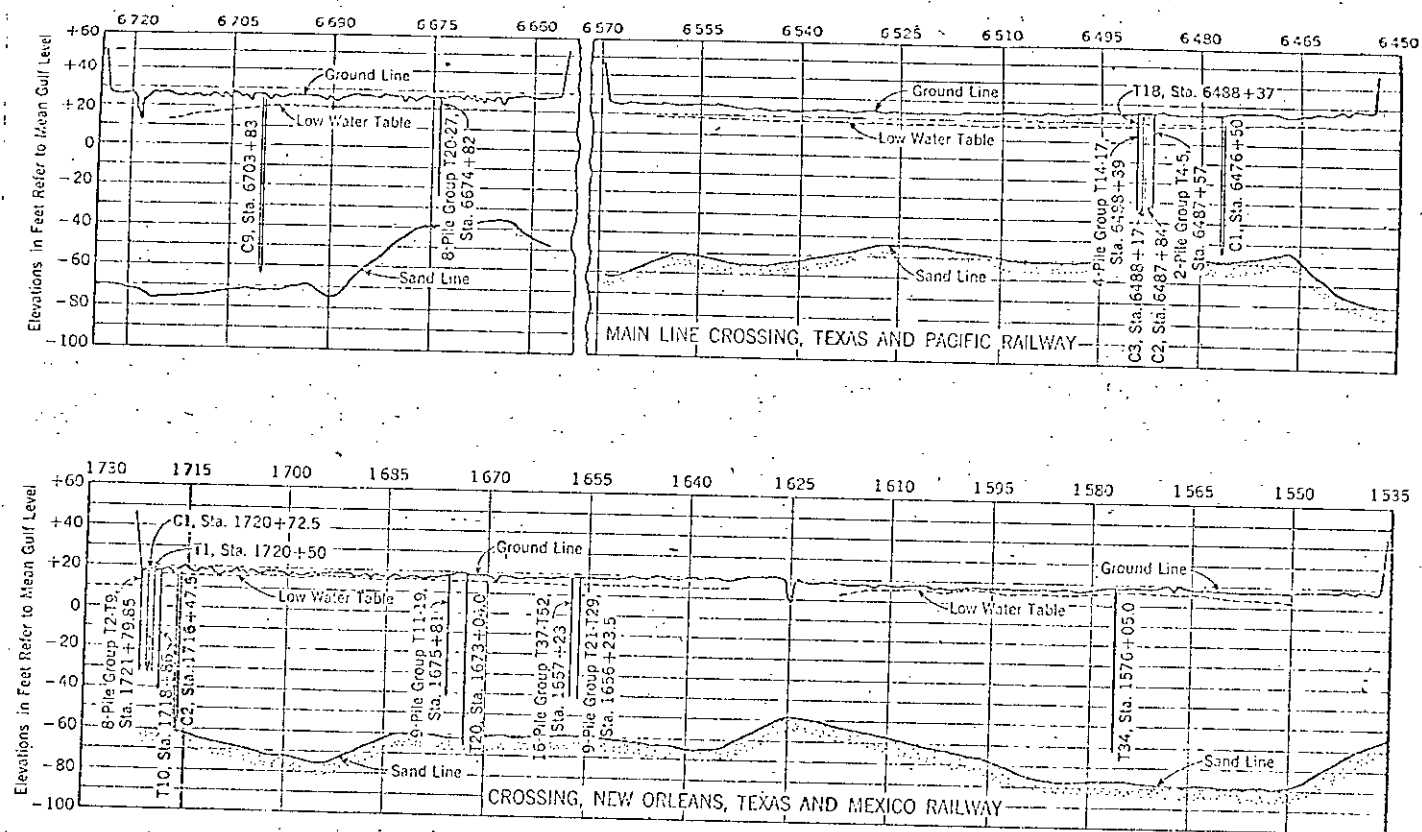


Figure 24. Profiles of Railway Crossings of the Morganza Floodway.

Reproduced from Reference 11, page 117.

clay materials and they derive their entire strength from the frictional resistance between the pile surface area and the soil surrounding it. Masters (11) presents the case of the Morganza Floodway where such short piles made of wood have been successfully used.

The Morganza Floodway in Louisiana is located in the lower Mississippi Valley. The U. S. Engineer Corps was required to construct railway and highway structures across this floodway. Cross-sections of soil profiles shown in Figure 1 of reference 11 are reproduced here as Figure 24. It can be seen from these profiles that the sand layer which is a good foundation medium was available at depths from approximately 80 to 100 feet below the ground level. The materials above the sand lines were of approximately uniform quality along the floodway, consisting of successive strata of loam, silt and clay and admixtures of these materials.

The loads required to be transferred to the foundation medium were low and use of long piles down to the firm sand layer were calculated to be uneconomical. The use of spread footings was precluded because of high amount of anticipated settlement over the unconsolidated upper layer. Use of short piles embeded within this layer was investigated. Fortunately, the water table was available within a few feet from the ground level. This permitted the use of short timber piles which are remarkably durable below water table. The loads transfered to the piles were of the order of 25^k to 30^k

only per pile. Under these conditions the use of short timber piles offered the most suitable method of transferring the loads to the foundation media.

Examination of figure will further show that the lengths of the piles required to develop the desired friction varied normally between 50 and 60 feet. Piles were used both singly and in groups. The support capacities of the piles determined from load tests were found to be amazingly close to the values predicted by Masters on the basis of his soils analysis. The maximum difference between these values was within 5 per cent.

This example, which is more fully treated in the referenced article shows how short friction piles can successfully be used in unconsolidated foundation media of deltaic areas.

III. Long Pile Substructures for Dow Chemical Company's Plant, Plaquemine, Louisiana.

This investigation was made in order to determine the foundation condition and its effect on substructure design for the Dow Chemical Company's Plant at Plaquemine, Louisiana, by Spencer J. Buchanan and Associates, Inc., Bryan, Texas, in 1956. The site of the plant is about 12 miles south of Baton Rouge, Louisiana, and lies on the west bank of the Mississippi River. It is in the present alluvial former deltaic plains. The shearing strength of these

materials were found normally to be within the range of 0.5 to 1.00 tsf. It is interesting to note that the materials for which test results have been reported in Chapter V of this dissertation were also found to be preconsolidated by desiccation and the degrees of preconsolidation were found to decrease with increase of depth. At about 50 feet the clay approached normally consolidated condition. Also the shear strength, determined for a limited number of samples, showed a maximum value of 0.96 tsf and a minimum value of 0.36 tsf.

At depths below 100 to 110 feet the foundation medium was found to be very dense sand typical of graveliferous deposits. It consisted of a silty fine sand at its surface and graded, with depth, to a clean medium-to-coarse sand and then to a gravelly sand, as was expected in this area.

In order to investigate the suitability of different types of foundations for this area the consulting firm carried out a number of load tests on individual spread footings and short friction piles for light loads and long piles and drilled-in-place piers for heavy loads.

On the basis of the load tests made on square and circular spread footings, whose maximum dimensions did not exceed 10 feet, a bearing capacity of 985 psf was recommended for areas with softer materials and 1280 psf for areas of higher soil strength.

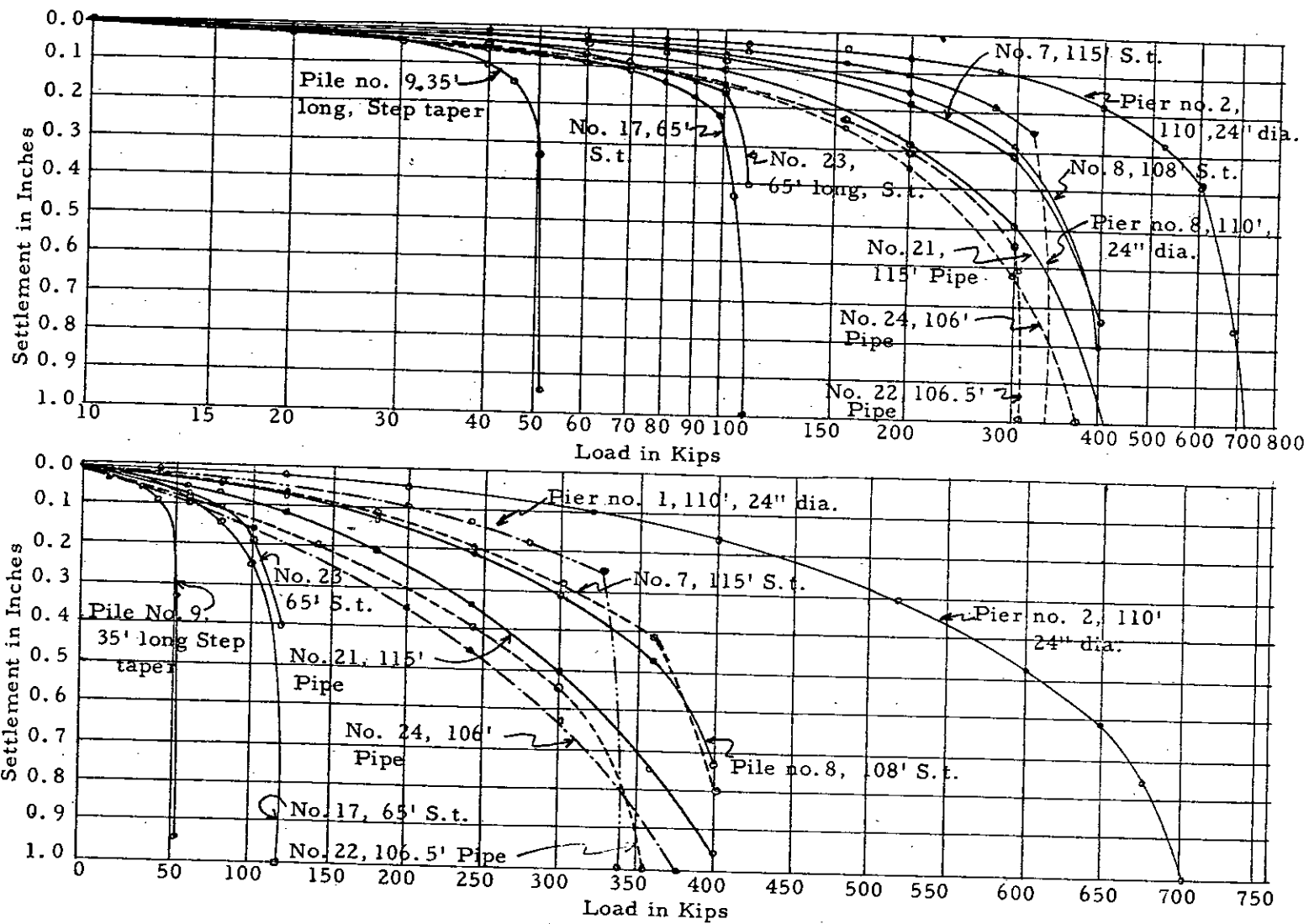


Figure 25. Load-deformation Curves for Piles in the Dow Chemical Company's Plant at Plaquemine, Louisiana.

Short length piles for support of light loads were precluded because spread footings were found to be less expensive and provided a greater factor of safety. Further, they were subjected to predictable ultimate settlement whereas it was difficult to estimate the ultimate settlement of the short friction piles.

To support the heavy loads, long piles penetrating a few feet into the dense sand strata were driven and load tested. Raymond step-taper piles, closed-end steel pipe piles and Raymond pipe step-taper piles were investigated. The load deformation curves for the piles tested are reproduced in Figure 25 from reference 12. The plots are shown both in arithmetic and semi-log scales. They show that definite points, which may be considered to indicate the ultimate load capacity of the piles are not readily established in the plot in arithmetic scale. It is, however, interesting to note that the plots, using the semi-log scale, show the behavior of the piles more clearly and the ultimate loads could be predicted more confidently. The points of maximum curvature were considered from the semi-log plots to indicate the point of ultimate capacity of the piles under consideration. This approach appears more rational and removes the difficulty of selecting the right point from the curves on arithmetic scale.

The ultimate loads, total settlement under the loads, design loads using a factor of safety of 2 and settlement under the design

TABLE 6

Test Loads and Recorded Settlements for the Piles
 Tested at Dow Chemical Company's Plant at
 Plaquemine, Louisiana

Pile No.	Length	Type	P _{ult} kips	Settlement inch	Settlement at design load, inch
9	35'	Raymond Step taper	42	0.12	0.03
17	65'	S. t.	95	0.21	0.055
23	65'	S. t.	100	0.16	0.055
24	106'	pipe	230	0.46	0.12
22	106'	pipe	300 Sudden Failure	0.53	0.21
21	115'	S. t.	260	0.40	0.16
7	115'	S. t.	280	0.27	0.09
8	108'	S. t.	290	0.27	0.08
1	110'	24" dia.	310 Sudden Failure	0.24	0.08
2	110'	24" dia.	510	0.30	0.08

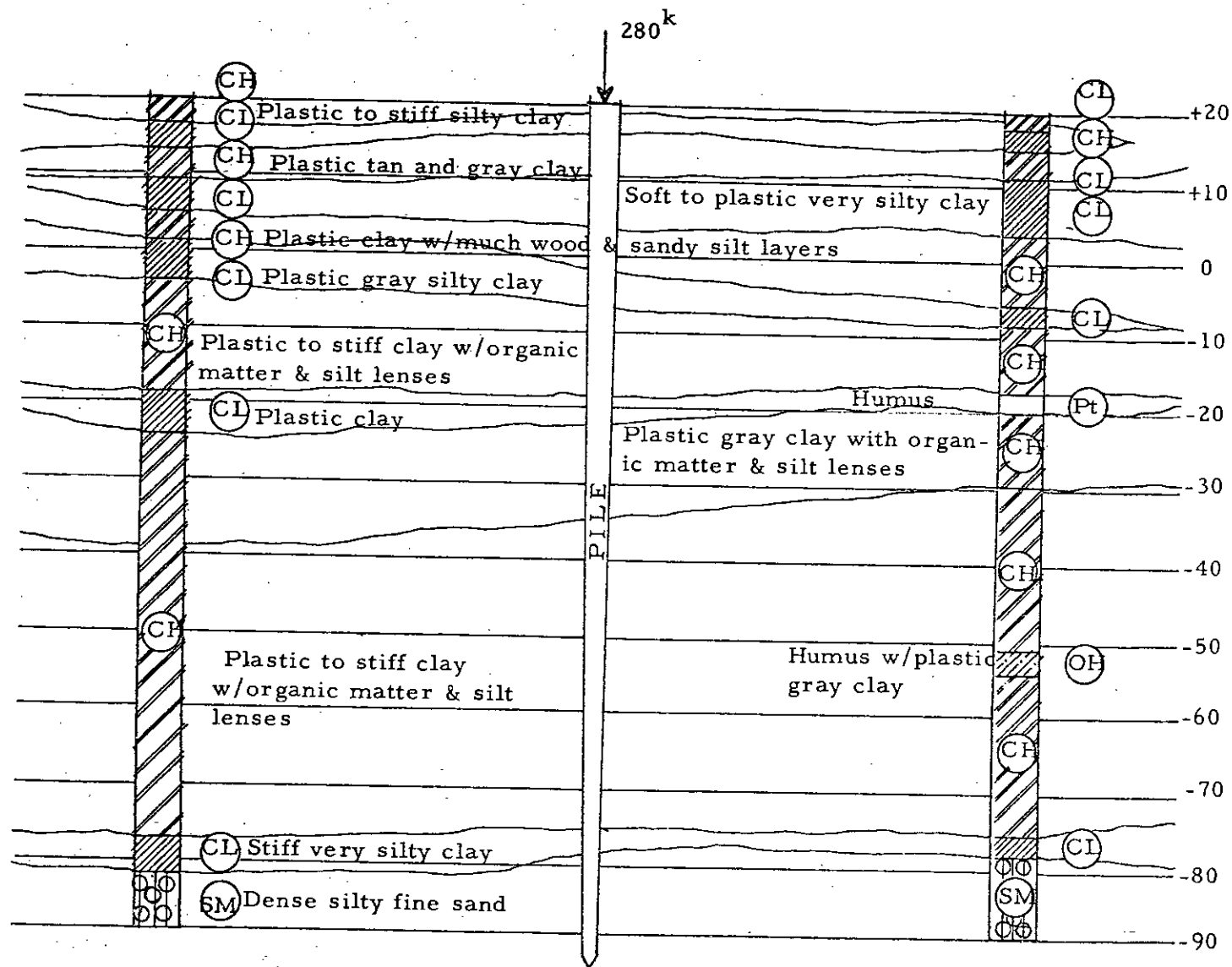


Figure 26. Typical Soil Profile of Plaquemine Site.

loads are shown by Table 6. It can be seen from the plots and this table that Raymond step-taper piles were found to be the most effective for this site.

Figure 26 is presented to show a soil profile typical of this site. In the figure also shown is a pile driven into the firm sand layer and its capacity as recorded from test results.

The maximum settlement experienced in case of Pile No. 21 was 0.40 inch under the ultimate load of 260^k and the settlement under the design load was only 0.16 inch. The settlement in case of other long piles penetrating into the sand strata was normally less than 0.10 inch under the design loads. It may also be emphasized that point bearing into the sand strata increased the capacity of the piles considerably. Comparison of the piles of different lengths will reveal this. The 35 foot long step-taper pile showed an ultimate load of 42^k while the two 65 foot long piles showed an average value of 97.5^k for the ultimate load. On the basis of this, a pile about 110 feet long should have carried an ultimate load of $160^k - 170^k$ due to frictional resistance. But such piles, Pile No. 7, 8 and 21, showed an average ultimate load of about 280^k . The difference of about 100^k was attributed to point bearing.

Drilled-in-place piers were also considered for supporting heavy concentrated loads. In consideration of construction difficulties and minimum practical size, these were not preferred for

this site.

IV. Highway Embankment on Unconsolidated Combinations of Decayed Vegetation, Roots, Logs and Highly Plastic Clays of Deltaic Deposits

The design of highway embankments across the topstratum of alluvial and deltaic areas, consisting of very loose unconsolidated materials with decayed vegetation, roots, logs, and highly plastic clays and silts, is a challenging problem to foundation engineers. Removal of the loose materials and filling the site with stable ones is a time-consuming and costly operation and process. The older method of letting an embankment settle and consolidate for ten or more years before any roadway can be built over it just cannot be tolerated in this rapidly progressing world. It has been mentioned earlier that the growing economy of East Pakistan demands the development of its communication system over its alluvial and deltaic areas as rapidly and effectively as modern knowledge and tools will permit.

Dodd (17) presents a new method for both the design and means of controlling the construction of embankments over such materials in the lower flood plain of the Neches River adjacent to Beaumont, Texas, USA. The immediate foundation medium of the embankment consisted of typical lower river deltaic plain topstratum which

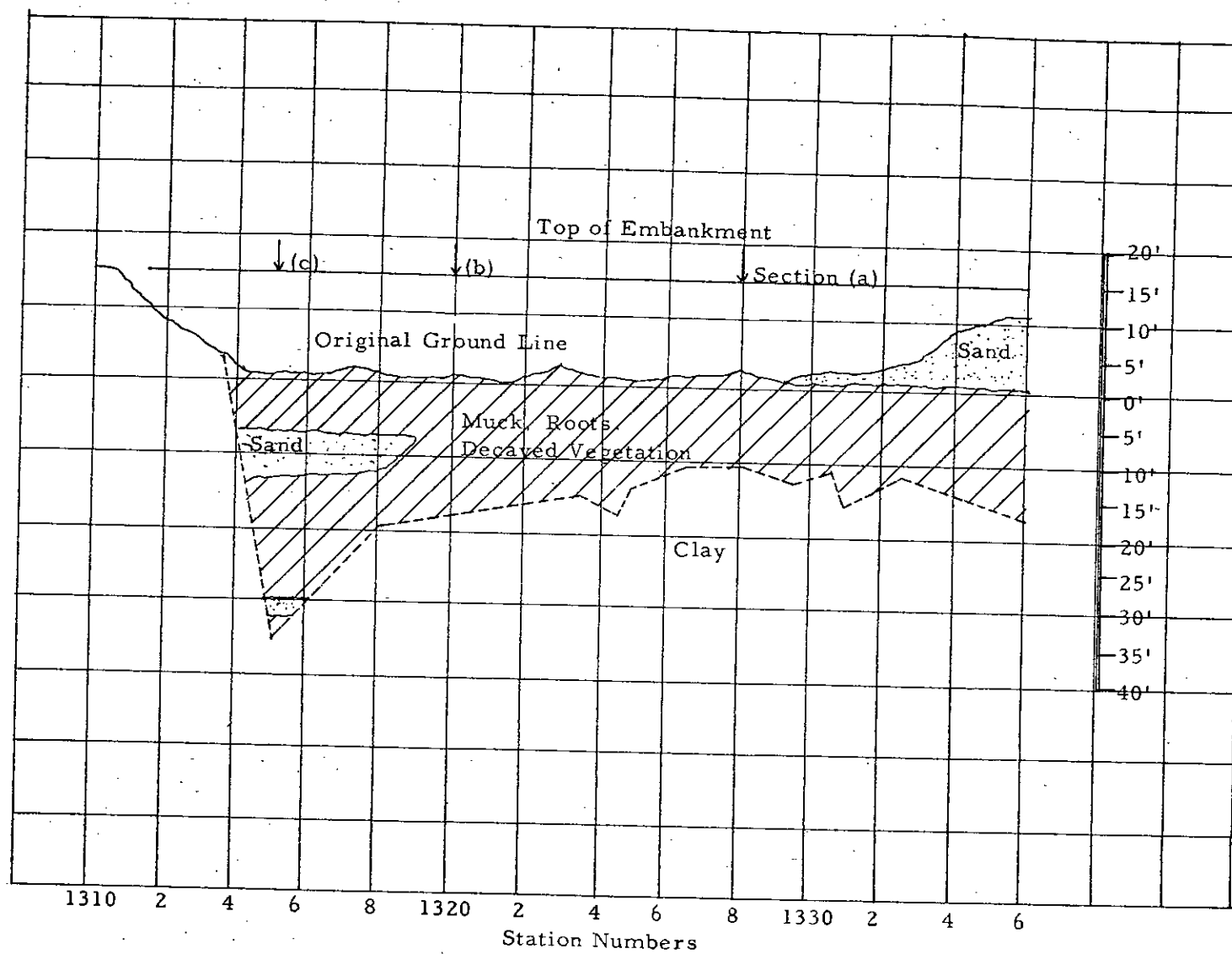


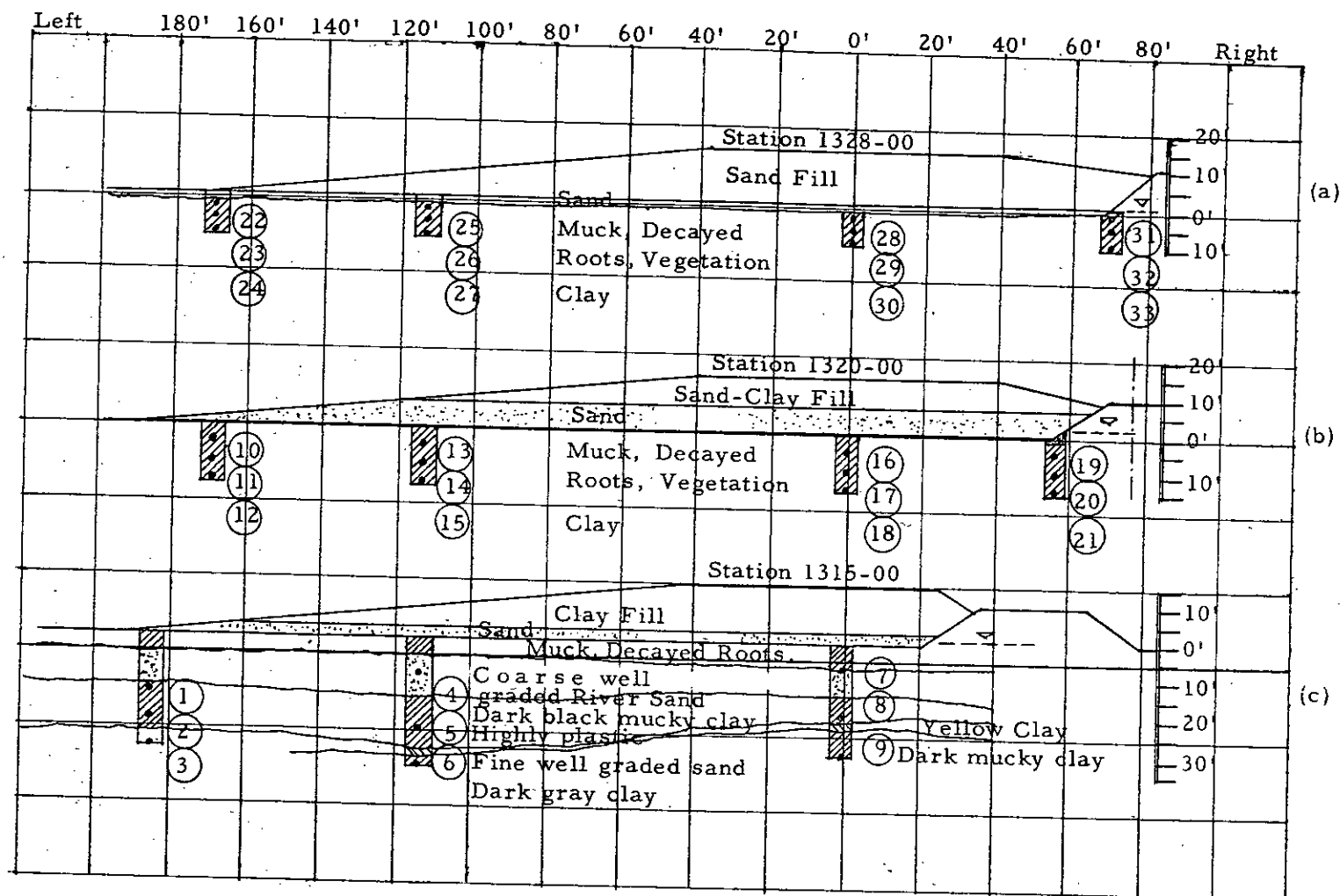
Figure 27. Longitudinal Soil Profile Along the Embankment (Test Section).

consisted of saturated and unconsolidated materials of very low strength, and of a very unstable nature. The method described by Dodd was adopted and used successfully by the Texas Highway Department in the construction of this embankment in the Beaumont area. The site of the project was in Orange County, Texas, in the alluvial valley of the Neches River near Beaumont, Texas.

The Neches River in its passage to the Gulf, eroded a very broad channel through the area and redeposited alluvial soils in this channel to form a flood plain. The alluvial materials covering the flood plain range in thickness from 5 to 45 feet and are composed of decayed vegetation, roots, logs, very soft plastic clays and admixture of silts and clays, which are typical of swamp deposits. This material is similar to that found in the near surface 20 to 30 feet of the area from Baton Rouge to Lafayette, reported in Chapter IV of this dissertation. A cross-section of the soil profile along the embankment is shown in Figure 27.

The soil constants, Atterberg limits, as well as moisture contents of these materials showed the clay layer to be normally loaded. The liquid limits of the samples tested were near or a little over 100, whereas the plastic limits ranged normally in the twenties. The moisture content varied from 85 to 94 per cent.

Since the material immediately below the embankment was too soft for collection and determination of its shear strength in



(a) Eastern End, (b) Western End, (c) Between Station 1329 and 1335.

Figure 28. Cross-Sections Showing Soil Profiles and Piezometer Tips of the Embankment of Beaumont, Texas.

its undisturbed state, the conventional method of design involving the balancing of stress with available strength could not be adopted. Rather, means of observing the strains and the pore water pressure influencing the shear strength of the material had to be resorted to in restricting the elements that produce failure in a foundation medium. Cross-sections showing the locations of the piezometers are shown in Figure 28.

In order to assure the maximum possible gain in strength in the minimum time, provision was made in the design for the surface of the natural ground or foundation medium to be blanketed by a pervious river sand. The blanket was three feet thick and covered the entire base of the embankment. This served as an excellent drainage medium for the foundation. The remaining height of the embankment was formed of clay dredged from the river bank. The maximum rate of consolidation of the embankment material was assured because of the underlying drainage medium provided in the design.

From the theory and behavior of a consolidating stratum of clay, it is known that with the progress of consolidation the soil mass gains strength. The load initially applied on a saturated mass of soil is immediately carried by the void water in the soil mass. It is known as the excess hydrostatic pressure or the pore water pressure. With the passage of time this pressure is relieved due to drainage of water out of the soil mass. The applied stress is

transferred as pressure from grain to grain of the soil mass.

This pressure, known as the effective pressure, increases the shear strength of the soil according to the well-known equation $s = c + \bar{p} \tan \phi$, and where $\bar{p}$ is the effective pressure. As the process of consolidation progresses, more and more of the pore water pressure is relieved and the value of $\bar{p}$ increases gradually, which means gain of shear strength, s , for the consolidating medium.

The general approach to a problem of embankment design across a deltaic area involves, first, the determination of the stresses created in the foundation media due to the weight of the overburden, and, second, the available strength of the foundation material to stand these stresses. If the pace of construction can be controlled so that the stresses induced in the media are always within an acceptable limit of its immediate strength, the foundation would not fail. This principle was adopted in the case cited, and an arbitrarily fixed relation of the percentage of the pore water pressure to the consolidating load was found a complete success. The magnitude of the excess hydrostatic pressure indicates the state of consolidation and is a measure of the effective pressure, which is responsible for the gain of shear strength of the medium. The rate of construction was thus controlled to limit the induced stresses within the available strength of the foundation media by observing the pore

water pressure.

It may be recalled that the usual method involving the determination of consolidation characteristics of soil is to test undisturbed samples of these in the laboratory. But in this case the muck in the area was so soft that such tests could not be made, and also the presence of excessive amount of organic matter in a layer which serves as avenues of escape of void water precludes the application of the theory of consolidation. This situation compelled the working engineers to take recourse to measuring the pore water pressure during the construction of the embankment, in order to determine the gain in shear strength of the material and the state of primary consolidation. For want of any information or literature on the subject on which a criterion could be established for the control of construction, the consulting engineer, Professor Spencer J. Buchanan, Texas A&M University, established the following parameters, based on broad experience with similar projects, for the control of the construction.

"During construction the relation of the excess hydrostatic pressure in regard to the applied structural stresses should not be allowed to exceed one hundred per cent for the area between the mid point of the slope and the center line of the embankment. For the area between the mid point of the embankment slope and the planned toe-of-slope, which is the most critical area, the relation

should not exceed ninety per cent. The percentage referred to is the per cent of structural stress is of excess hydrostatic pressure."

This recommendation proved ample and satisfactory. From measurement, it was found that the entire excess hydrostatic pressure was relieved in six months and there has not been more than a quarter of an inch of settlement during the last 15 years. Pavements constructed over the embankments have not been subjected to any distress.

Discussion on the Cases Enumerated

The four typical cases enumerated in this chapter demonstrate how different types of foundation structures can profitably be used to suit different conditions normally encountered in alluvial and deltaic areas.

Case I, The Reinforced Concrete Mat for the Houma Gas Products Plant, shows how a type of foundation structure can be adjusted to the conditions available in the foundation media. In this case no firm founding medium was available within a depth of 80 feet and a layer with decayed vegetation was detected at a depth of 40 feet. To meet this situation a raft was used. This reduced the unit pressure on the foundation media by distributing it over a larger area to within safe limits both from the point of view of bearing capacity and settlement.

Case II, Timber Friction Piles for Morganza Floodway, demonstrates the use of short piles for a problem where the loads were not heavy and the foundation media consisted of silts, clays and loam and their admixture typical of deltaic deposits.

Case III, Long Piles for Dow Chemical Company's Plant, demonstrates the use of long point bearing piles for transferring heavy concentrated loads into the graveliferous stable materials. In this instance the upper 100 feet of foundation medium was silt and clay of Recent Age that did not have enough strength to support the piles by skin friction. Other types of substructures were not suitable.

Case IV, Highway Embankments on Unconsolidated Deltaic Deposits, is an excellent case to demonstrate how by proper use of a drainage medium and by controlling the rate of construction, the strength and stability of a foundation medium can gradually be developed to withstand the stress applied over it. The materials over which the embankment had to be built were very soft and possessed little shear strength in situ. Embankment loads were added in such increments that the extra load placed over the foundation medium produced stress lower than the gain in shear strength due to consolidation of the foundation media. And, in the absence of any other suitable method to determine this gain in strength, measurement of pore water pressure was made to know

the rate of consolidation. This also demonstrates how the principles of soil mechanics can be gainfully employed in various direct and indirect ways to solve intricate problems.

CHAPTER VII

CONCLUSIONS

a. The systems of communication and transportation, as well as the construction industry, must first be developed at a pace at least equal to that of the economic growth of East Pakistan to assure the latter. The Brahmaputra River divides the province into two approximately equal parts, but not a single bridge exists over the river. These circumstances dictate the urgent need for a broad study of the general geology and the condition of the foundation media in the province.

b. Study of the history of glaciation in the northern hemisphere during the Pleistocene Ice Age discloses similarity between the ice caps within the limits of the Mississippi River system in North America and of the Brahmaputra River system in Central Asia.

c. Study of literature indicates a striking similarity in reexistence of the deep offshore gorges and other geological features formed by both the Mississippi and Brahmaputra rivers during and after the last period of glaciation.

d. Firm graveliferous deposits similar to those present in the lower alluvial valley and deltaic areas of the Mississippi River, in the state of Louisiana in the USA, are believed to exist also in the lower alluvial valley and deltaic areas of the Brahmaputra River in East Pakistan.

e. The dense sand layers in the upper portions of the graveliferous deposits in the lower Mississippi Valley have been used and served as excellent foundation media for heavy structures of all types. Similar deposits are anticipated in the lower valley of the Brahmaputra River in East Pakistan and should serve as good foundation media.

f. Examination and study of the recently deposited non-graveliferous deposits have shown that through proper and intelligent application of engineering knowledge they are capable of supporting industrial plants and their heavy concentrated loads without experiencing significant settlement. The existence of similar situations in the East Pakistan area seems probable.

g. Through the proper use of the principles of soil mechanics, highway embankments 20 - 30 feet in height may be designed and constructed economically and satisfactorily in deltaic plains with non-graveliferous under-consolidated highly plastic materials.

h. Non-graveliferous deposits in the Deltaic areas of East Pakistan are expected to be shallower in depth than in the Mississippi Deltaic area.

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