INDUSTRIAL DEVELOPMENT OF MARGINAL LAND:

A CASE STUDY

by

BUFORD KENT MERRITT

Bachelor of Science in Civil Engineering

University of Oklahoma

Norman, Oklahoma

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To my wife, Lavina, and my children,
Randy, Dusty, and Brenton, for their
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INTRODUCTION

General

As our nation's economy becomes increasingly dependent on worldwide sources of raw materials, especially crude petroleum and natural gas, land transportation of these raw materials to processing and refining facilities becomes an increasingly complex problem. One viable solution to this problem is the increasing densification of the coastal industries which rely on these water transported materials, thus eliminating the need for overland transportation of the stock. This direction has been followed by the petroleum industry along with related and dependent industries along the Gulf Coast with predictable ramifications. Much of the coastal and near-coastal lands which are suitable for industrial development from the standpoint of maximum economic return are previously occupied.

Undeveloped industrial sites along the upper Texas Gulf Coast that are within economic proximity of water and land transportation routes, utilities, potential labor forces, <u>and</u> exhibit good site or subsurface conditions are nearly depleted. However, there still exists much marginal land, i.e., marshes, swamps, and flood plains, which offer most of these desirable characteristics with the exception of easy development. Philip C. Rutledge (26) in his practice along the east coast has done much in bringing about a marriage of the planning and engineering professions in the task of development of lands previously thought to be undesirable because of site and subsurface conditions. His work in this area has pioneered a reevaluation by industry of many of these marginal lands, previously passed over as undevelopable.

Along the upper Texas Coast these marginal lands often provide ready access to navigable waters because of their geologic evolution. Because of tectonic activities during the Quaternary Era, primarily inland uplift coupled with subsidence of the coastal plain, channel cutting in a general westward direction has occured. This, in turn, has left the eastern abandoned channels subject to "back swamp" deposition. The Neches River in southeast Texas is a classic example of these marginal lands with characteristic swamps bordering the eastern bank from many miles north of Beaumont, Texas to its coastal outflow into the Gulf of Mexico via Sabine Lake.

In late 1973, the Georgetown Texas Steel Corporation selected a site with near-surface stratigraphy characteristically of Recent depositional origin, directly across from Beaumont, Texas on the east bank of the Neches River. This site was selected for construction of one of the "ministeel mills" for which the parent Korf Corporation is internationally known. The site consists of approximately 560 acres of marsh and swamp land. For direct offsite transportation links the site is bounded on the north by Interstate Highway 10 and the Kansas City Southern mainline and on the south has a 4,700 foot frontage on the navigable Neches River. The site has direct access to all of the necessary utilities and is nestled in the Golden Triangle, a highly developed industrial and petrochemical area of Southeast Texas which provides a very active labor market. A map indicating the site location is provided as Figure 1.

The ministeel mill, now complete and in operation, consists of nearly 600,000 square feet of area under roof. The main plant building houses two-100 ton furnaces, ladle turrets, casting machines and other process devices which

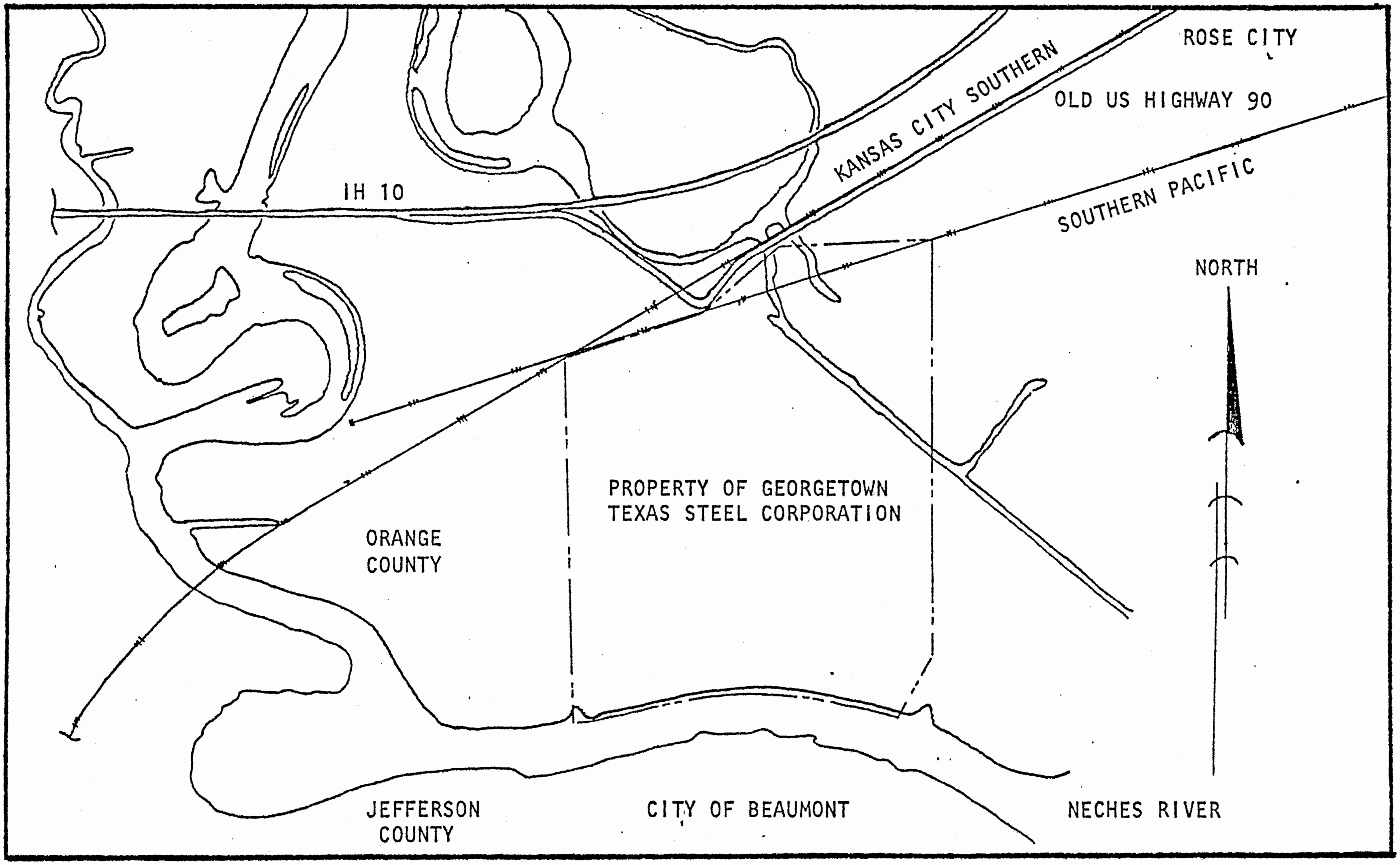


Figure 1. Location Map - Georgetown Texas Steel Corporation Property

represent very heavy concentrated loads. General floor loading ranges up to 6,000 pounds per square foot in areas where billets are stored. Incidental facilities include a scrap-storage yard 70 foot wide and 1,700 foot long with ground loading up to 2,000 pounds per square foot, secondary structures such as an office-warehouse building (Building 8), oxide pellet silos, a power substation, and nearly eight miles of rail spurs and roads. Rated capacity of the mill is over 500,000 tons of round-stock, wire, and reinforcing bars per year.

Prior to development, the northern two-thirds of the site selected by the Georgetown Texas Steel Corporation was a cypress swamp with an average elevation of +2 to + 6 feet (elevation above mean sea level). The entire site had been clear cut during the early part of this century to provide timber for the extensive lumber industry of that period which was centered directly across the river in Beaumont. This northern two-thirds of the site was sparsely covered with "second growth" cypress trees at the time of development.

The southern one-third of the site had been used as a spoil area for the past 30 years and had accumulated in some areas as much as 25 feet of hydraulically placed spoil as a result of channel dredging in the Neches River. Surface topography in this area exhibited a relatively large degree of relief with surface elevation of the dredged spoil ranging from about +6 MSL at some locations near the river to about +25 MSL at the top of a few spoil hills, however, the average elevation in this area was approximately +12 MSL.

Purpose and Scope of Study

The purposes of this paper are twofold: 1) to recount the physical investigation and application of engineering principals in the development of concepts, elimination of alternatives, and design of this project, and 2) to examine the construction and later performance of the various structures which, in a large part, demonstrate the adequacy and/or acceptability of the design assumptions and concepts necessary for the economic development of this facility.

Emphasis is placed on the techniques developed to analyze and make predictions of the accelerated settlements experienced with the root-penetrated soft clays and the control and densification of the hydraulically placed sand backfill integral to the economic development of this site.

CHAPTER II

AREA AND SITE GEOLOGY

The general area along the upper Texas Coast of which this site is a part is typified by deltaic and interdeltaic deposition of the Quaternary Period. This area, known as the Quaternary coastal plain of Southeast Texas, forms a belt paralleling the Gulf shoreline and contacting Tertiary outcrops, unconformably, 70 to 90 miles inland. The coastal plain is transected by seven major river systems.

The majority of this Quaternary belt consists of Pleistocene sediment. These Pleistocene sediments are subdivided into four coastwise depositional surfaces: the Beaumont (youngest), an unnamed second terrace (locally referred to as the Montgomery Formation), the Lissie, and the Willis (oldest). Each of these formations are correlated with major glacial and interglacial stages and each lies unconformably above the eroded and weathered surface of the next older formation (18) (28).

The youngest portion of the Quaternary plain comprises the Recent (post-glacial) depositional surface which occurs principally along the coast but extends inland along the major drainage systems. These inland extensions of Recent deposition are characterized by relatively narrow alluvial flood plains. Often these Recent flood plains are laced by abandoned channels of the river system and result in relatively thick deposits of underconsolidated to slightly preconsolidated materials, usually very organic in nature.

The site of the Georgetown Texas Steel Project is situated on one such Recent deposit. During the Recent Epoch the Neches River has cut into the older Pleistocene soils of the Beaumont Formation. Later redeposition in the abandoned portion of the channel occured in a backswamp environment characterized by slack waters and deposition of very fine grained materials on the perimeter of the main stream velocity. Much of the deposition is probably a result of periodic overbank flooding of this low plain during seasonal runoff.

The result of the Recent deposition at this site is a stratum of very soft to soft normally consolidated to slightly preconsolidated clay, as much as 30 feet thick. The average undrained cohesion of this stratum is approximately 300 pounds per square foot, however, the range of these values varies from substantially less than 100 pounds per square foot to around 500 pounds per square foot. Evidence from a number of borings advanced through this stratum indicates a nearly continuous growth of surface vegetation during the period of deposition. As a result this stratum is saturated with partially decomposed organic matter.

A feature of this Recent stratum, of significance to the development of this site, is that the stratum more or less continuously nurtured the growth of cypress trees during and after its deposition, and consequently is penetrated with a high density of randomly oriented roots, ranging in size from a fraction of an inch to over three inches in diameter.

The stratum of soft, compressible clay is underlain at most locations by a stratum of firm (medium dense to very dense) quartz sand deposited during the Peorian Interglacial Stage (depositional period of the Beaumont Formation). Other strata encountered below the Pleistocene sand, throughout the depth of exploration, are characteristic of the Beaumont Formation.

The Beaumont (Clay) Formation is a deltaic, primarily non-marine, heterogeneous deposit containing thick interbedded layers of clay, sand, and silt. The clay portion is tan, red, and gray in color, with the mineralogical components differing only slightly between clays of different color. The clay fraction is primarily composed of montmorillonite, illite, kaolinite, and finely ground quartz, approximately in order of prevalance. The presence of the montmorillonite contributes greatly to the swelling potential of the soil, particularly when present with sodium as its exchangeable cation. The sands and silts, which vary in compactness from loose to very dense, are composed of quartz, the feldspars, large particles of kaolinite, calcite, and occasionally hornblende. In the clay strata of the Beaumont Formation large calcareous nodules are often observed within the clay matrix.

The clay present in the formation has been preconsolidaed by a process of desiccation, with the indicated preconsolidation pressure generally being about two to four tons-per-square foot. Numerous wetting and drying cycles have also produced a network of randomly oriented and closely-spaced joints, which are sometimes slickensided, that is, have a shiny appearance when exposed. The joint pattern strongly influences the engineeering behavior of the soil and is often the controlling factor in the design of near surface spread and shallow drilled footings.

Typical stratigraphy below the southern one-third of the site (portion of the site covered by hydraulic spoil) is shown on Figure 3. Note the shaded area representing the soft, compressible, root-penetrated clay stratum.

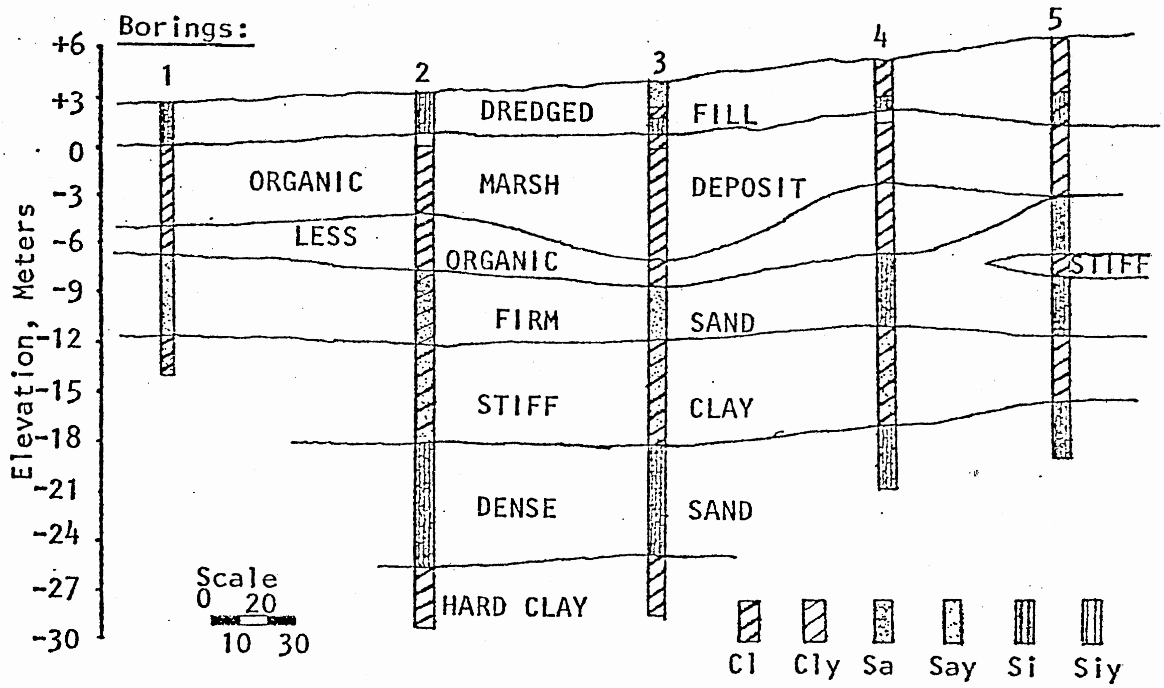


Figure 2. Typical Subsurface Profile (North - South)

CHAPTER III

PRELIMINARY DESIGN

General

In December, 1973, planning for development of this site began when the Geotechnical Engineering Division of Southwestern Laboratories, working in close cooperation with Lockwood Greene Engineers, Inc., prime engineering consultants, conducted a preliminary subsurface investigation of the entire site (1). This preliminary investigation consisted of over 1,500 linear feet of borings with boring depths ranging from 50 to 200 feet. Due to the relatively firm surface afforded by the hydraulic spoil along the southern one-third of the site, initial thoughts were for development of the mill in this southern corridor. Consequently, approximately three-fourths of the borings were concentrated in this southern corridor. Immediate design and construction problems identified by this preliminary investigation were the very poor foundation support to be afforded by the soft clay stratum and the very large long term settlement to be expected under even small imposed loads.

Initial concepts envisioned the location of the mill parallel to the Neches River in the area surcharged, randomly, by dredged spoil. In this concept all major concentrated loads (structural frames, melting-casting devices, heavy floor loads, etc.) would necessarily have to be supported by deep foundation elements which would transmit loads to one of the major sand strata below the compressible stratum. By selective placement of the various mill components a general cutting (lowering of the undeveloped site elevations) could be affected.

By this concept if the general floor elevation was placed at +8 MSL and the various components were placed in "cut" areas, a compensated condition (31) would result, leading to general floor loads which would cause no net increase or net decrease in the effective stress acting in the soft, compressible stratum.

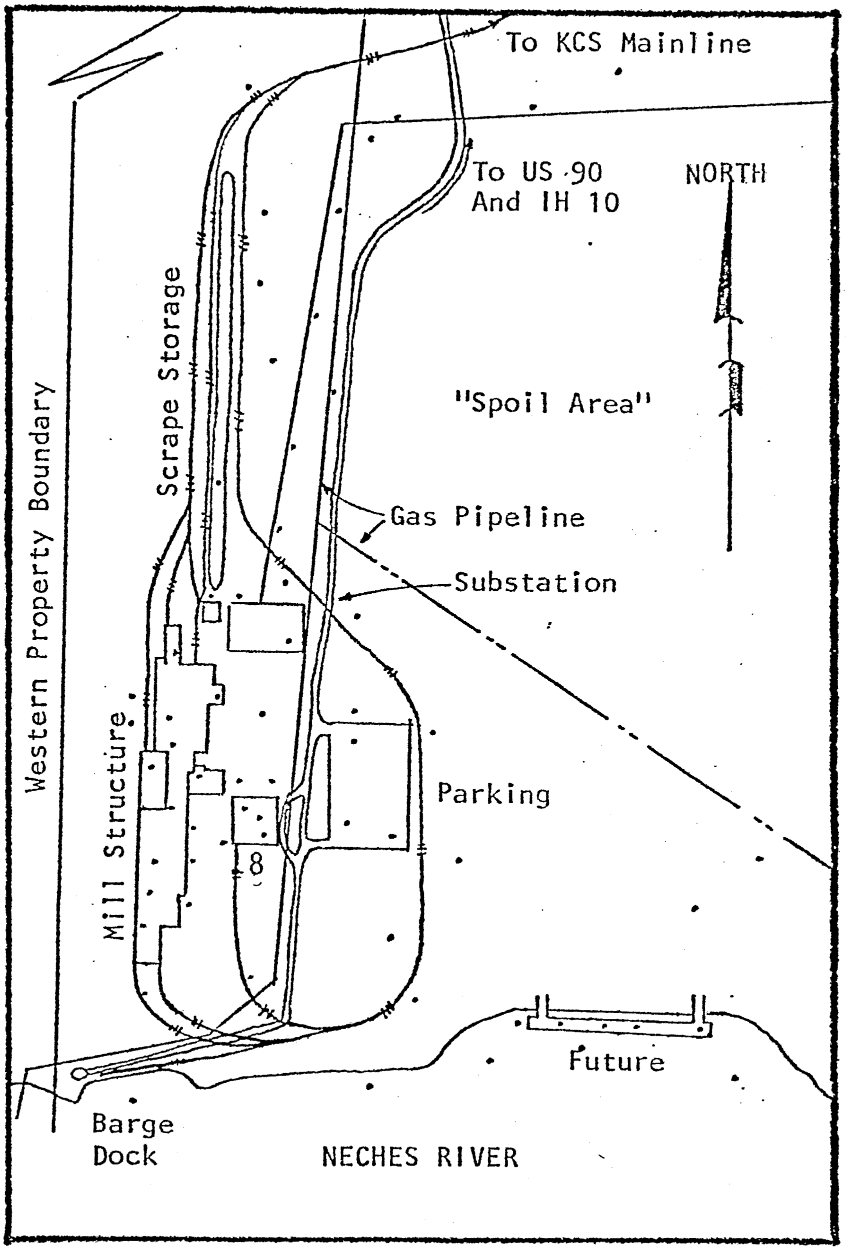
This concept, however, proved impractical because of flood insurance requirements which established the site grade at elevation +14 MSL. With the general site grade raised six feet, insufficient cutting could be accomplished to provide a compensated condition for control of settlement under general floor loading. At this time it also became apparent that the initial development of this site must be situated in such a way as to maximize the initially undeveloped areas for future development. This design constraint favored location of the mill perpendicular to the river along a corridor on either the east or west side of the property.

A further consideration in locating the plant was the existence of a high-pressure natural gas transmission line which crossed the site, generally diagonally from the southeast corner to the central portion of the northern site boundary. Situation of the plant axis perpendicular to the river would necessarily restrict construction to a corridor on the western-most part of the site to avoid relocation of the gas line and minimize land usage. Location of the mill in this corridor, however, provided very poor surface conditions with surface elevations varying from approximately +9 MSL in the hydraulic spoil along the river frontage to +4 MSL in the northern two-thirds of the site. The elevations precluded the concept of developing a compensated condition for general floor loads. The elevation also dictated extensive filling and thus large long-term settlements under the weight of the fill alone, if the finished site grade of +14 MSL was to be accomplished.

In all, seventeen alternate locations and orientations were studied, but when all of the design and operations constraints were considered, the plant location was established with the plant axis perpendicular to the river in a corridor 4,600 feet long and approximately 1,400 feet wide along the western property boundary. This location minimized initial land usage, precluded the necessity of relocating the existing high pressure gas line, and provided certain operational advantages in materials handling. With this plant orientation steel scrap and other materials could be delivered by truck or rail to the north (receiving) end of the plant, flow through the plant processes, and the finished product be delivered directly to the docking facilities at the south end. The final plan location of the mill is shown on Figure 3.

With location and orientation of the mill established, the next concern was to develop concepts and designs for plant roadways and railroad spurs. Construction scheduling called for these transportation routes to be constructed along with initial site clearing. The elevation of +10 MSL was selected for the roads and railroads to provide drainage and to maximize protection against flooding during the expected life of the roadway. Two alternatives were immediately available for design of these facilities.

The first concept would require "mucking out" of the soft, compressible clay to the surface of the underlying sand, backfilling with a superior material and construction of an embankment to establish final subgrade or subballest elevation. This concept would provide protection against rotational shear failure of the embankment and limit long term settlements. Disadvantages of this concept were the obvious expenses associated with large scale mucking and backfilling operations and the excessive time required to accomplish this operation.



• Boring Locations Scale: 1" = 850'

Figure 3. Final Design Layout

The second alternative was to construct floating embankments directly on the soft clay foundation, employing counterweight berms to provide stability. This concept indicated a distinct economic advantage if low theoretical factors of safety with respect to embankment stability could be tolerated. The large potential savings afforded by construction of roads and railroads on floating embankments was therefore offset by the proportionally higher risk which must be accepted.

Test Embankment

In order to assess the feasibility of floating embankments, site trafficability, and rate of settlement, a small instrumented test embankment was constructed near the northern point of access to the site (3). Also, in conjunction with the test embankment, a small test pit was excavated to a depth of twelve feet.

Prior to construction of the small test embankment, samples of the soft clay stratum were obtained with open-type, thin-wall tubes with inward-tapered cutting edges and area ratios of less than ten percent. Considerable sampling difficulties were encountered because of the presence of the tree roots. Many of the recovered samples contained small roots and were visually disturbed rendering them inappropriate for consolidation testing. Of the samples recovered at this location approximately one-fourth to one-third were considered of sufficient quality to yield reliable information as to the rate and magnitude of compressibility of this stratum.

On opening the small test pit it became apparent that the upper two to three feet of the soft compressible clay stratum consisted of a veneer composed of very organic clay, peat, and petroleum waste. Observation of the walls of the test pit indicated a relatively high density and random orientation of the roots included in the clay matrix. Free water flowing from the exposed roots was an indication of the high root permeability. Careful observation of the pit walls did not show any evidence of sand or silt seams which could provide additional internal drainage of the compressible stratum. However, based on the high density and apparently high permeability of the included roots, it was hypothesised that settlement due to consolidation of this stratum might occur significantly faster than would be indicated by predictions based on conventional theory.

Consolidation parameters and index properties of undisturbed samples obtained from the soft clay stratum at the location of the proposed test embankment were determined in the laboratory. Samples selected for consolidation testing were visually free of roots. The relationship between the decrease in void ratio and the change in effective stress (e-log P relationship) was determined experimentally by means of the one-dimensional consolidation test using a Lever-O-Matic odometer. The coefficient of consolidation, c_v, was computed by the square root of time fitting method suggested by Taylor (29). The results of these consolidation and index property tests are presented on Figure 4. The relatively low average values of the initial void ratio and Atterberg limits of the samples is indicative of the minor influence of any included organic matter exclusive of the partially decomposed roots. Consequently all samples were classified as either a CL or CH in the "Classification of Soils for Engineering Purposes" (32).

Computation of preconsolidation pressure from the Skempton-Bjerrum correlation (10) between the $c_u/\bar{p}_n$ and the plasticity index of the soil results in

	W_L	I_P	W_N
Range	39 - 58	25 - 42	27 - 35
Average	48	33	32

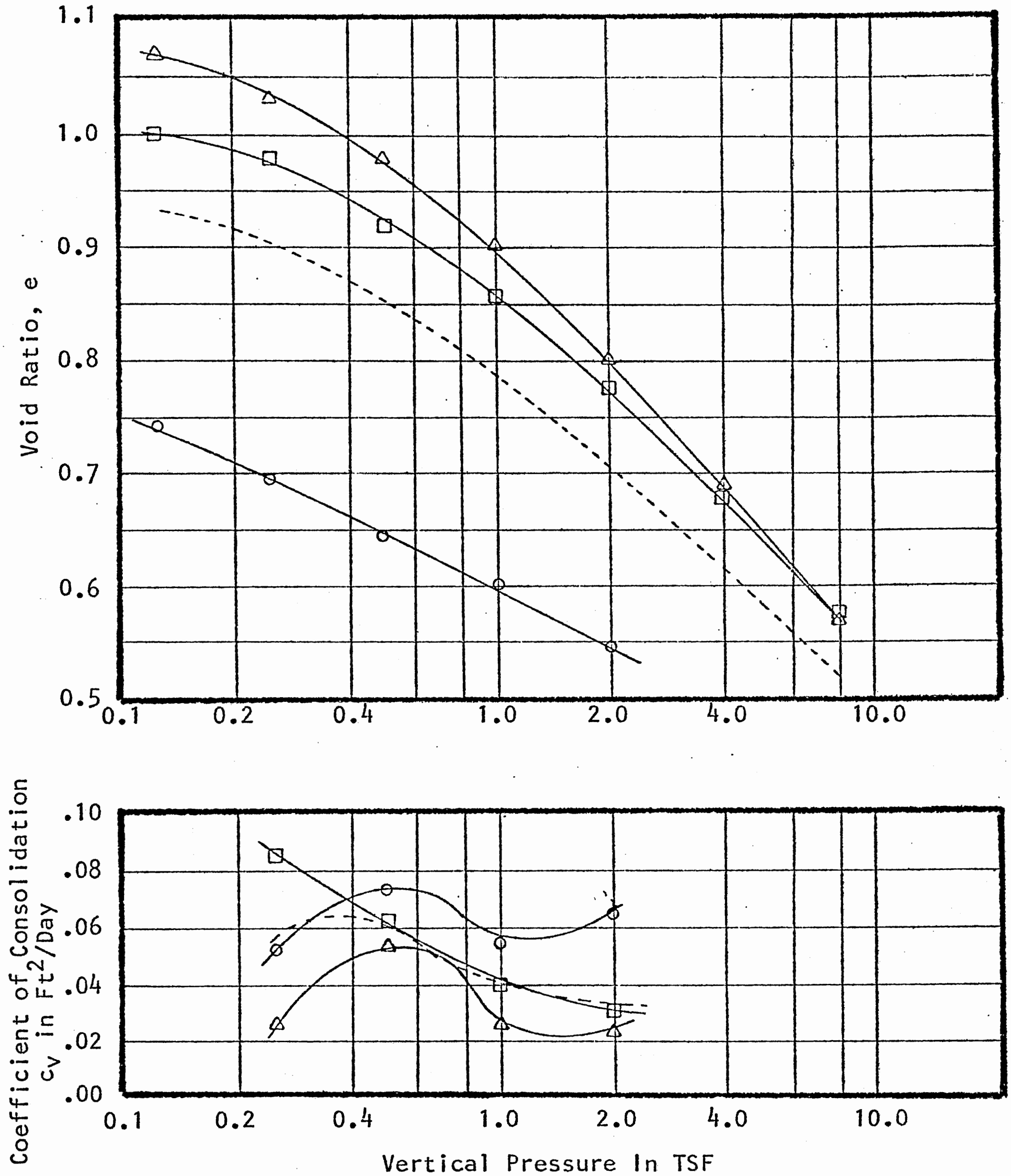


Figure 4. Consolidation Test Results - Test Embankment

values slightly higher than the present effective overburden pressure, indicating this clay stratum to be slightly preconsolidated. During consolidation testing, load increments were allowed to remain on the specimen for periods of time greater by an order of magnitude than the time required to achieve complete primary consolidation (U=100%). Resulting ratios of primary to secondary compression generally exceeded 0.7 at levels of effective stress greater than the calculated overburden pressure. Consequently the clay matrix would be expected to consolidate chiefly in a mode predictable by the theory of consolidation introduced by Terzaghi (30).

The soft clay stratum, including the veneer of very organic material, at the test embankment location was approximately nine feet thick and was underlain by a stratum of medium dense gray silty fine sand. Consequently, free drainage (compared to the permeability of the clay matrix) was available at both surfaces of the compressible stratum. Rate computations based on the experimentally determined coefficient of consolidation of the clay matrix (disregarding any anticipated internal drainage due to the included permeable roots) and considering the half thickness of the stratum yielded times of 100 days and 429 days corresponding to t_{50} and t_{90}, respectively. These times were computed using equation 1.

$$t_i = \frac{T_i h^2}{c_{vavg}} \qquad \text{(1)}$$

The test embankment was instrumented with four settlement plates placed at the top of the compressible stratum and two piezometers placed half way between the upper surface and the mid-depth of the compressible stratum. The northern portion of the test embankment location was covered with a random weave filter fabric. The embankment was placed by progressively "end-dumping" truck loads of the tan silty sand while spreading and compaction was attained by a medium-weight dozer. Considerable "mudwaving" of the subgrade was observed when fill was placed over that portion of the subgrade not protected by filter fabric. The entire embankment placement required approximatley four hours. Figures 5 and 6 present the plan and profile of the test embankment, respectively.

Settlement was monitored by measuring the change in elevation of the settlement plates relative to a permanent bench mark outside of the area affected by the embankment for a period of sixty days. Figure 7 is a plot of the observed settlement versus the square root of time. Computed values of time required to achieve fifty and ninty percent of the settlement due to primary compression are indicated on the figure. In comparing these times to those computed from laboratory data based on the parameters associated with on the clay matrix, it was observed that the field settlement occured well over an order of magnitude more rapidly than conventional predictions would indicate. Internal drainage of the consolidating stratum aided by the included roots was considered to be the only rational explanation of this phenomonon. This was further substantiated by the erratic readings obtained from the two piezometers which indicated large local variations in excess pore pressure produced by the presence of the roots.

The presence of the filter fabric covering the subgrade under a portion of the embankment retarded mudwaving (plastic failure of the upper portion of the compressible stratum). The magnitude of consolidation settlement was, however,

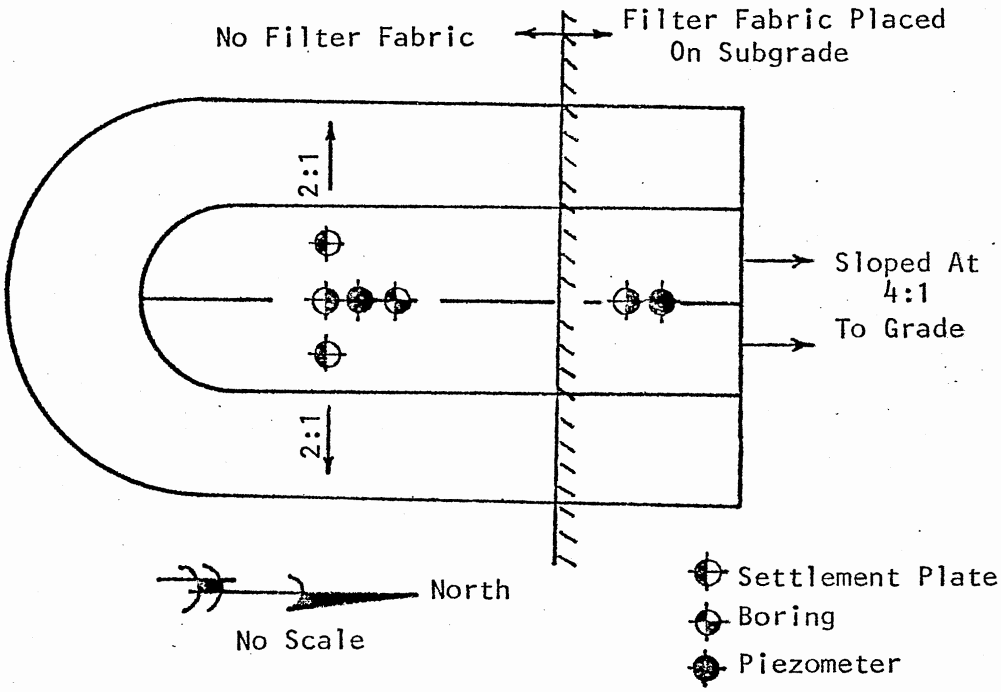


Figure 5. Plan of Test Embankment

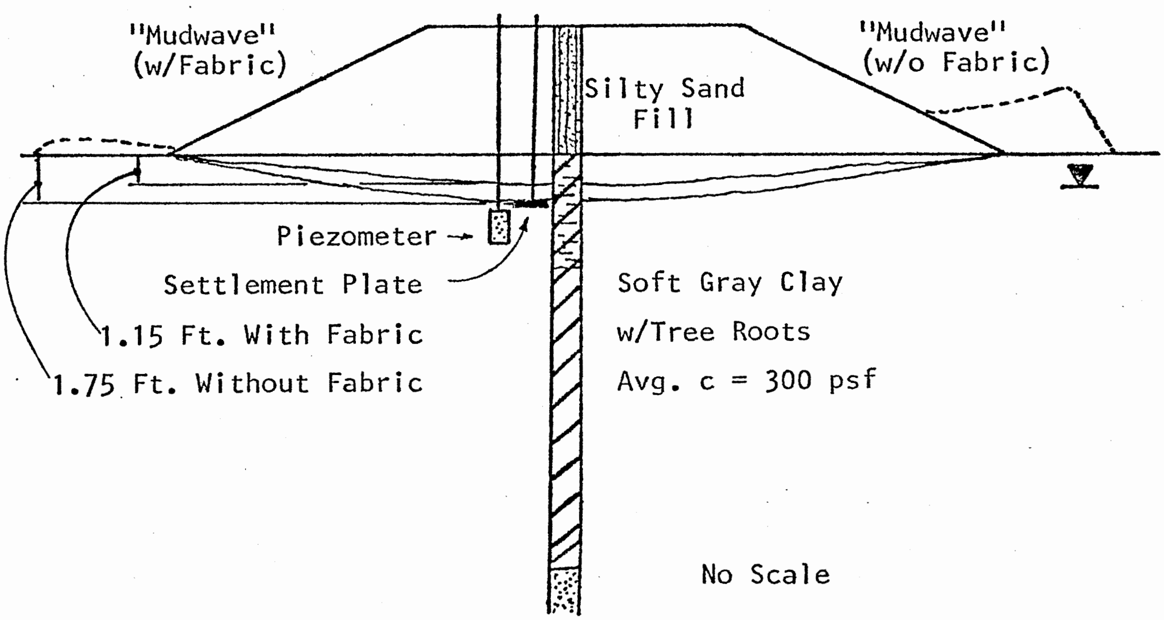


Figure 6. Profile Of Test Embankment

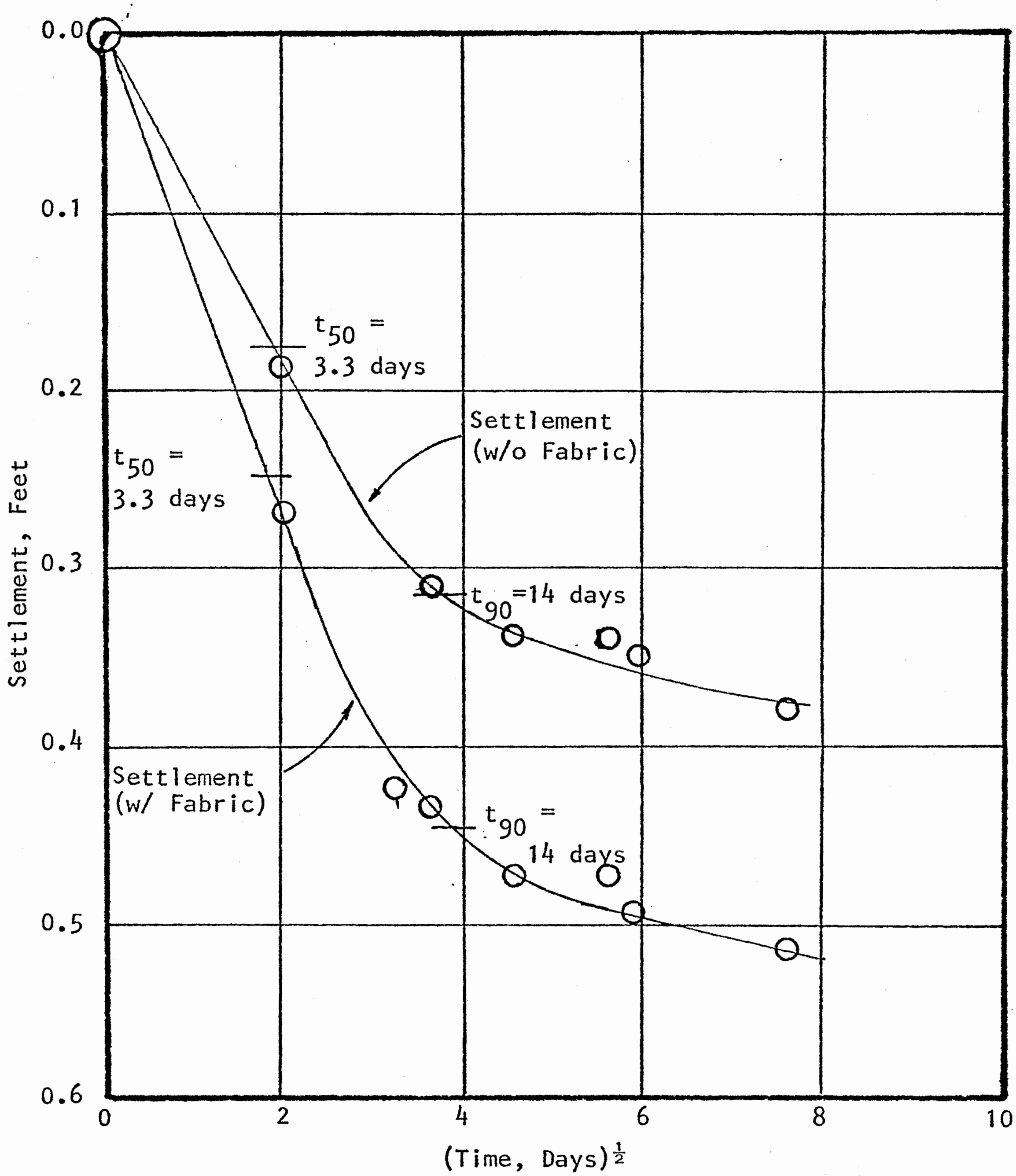


Figure 7. Measured Test Embankment Settlement

observed to be greater during the period in which the settlement was monitored. This fact prompted the conclusion that ultimate vertical deformation of the embankment, with or without the filter fabric was roughly the same. The filter fabric would, however, promote larger vertical strains (consolidation) after embankment placement, increasing the potential for rotational and/or translational failure of the embankment. Based on this consideration, the further use of filter fabric over the subgrade for future embankment construction was abandoned.

A review of the relatively large number of test borings advanced during the preliminary geotechnical study of this site indicated the compressible clay stratum to be present, although of variable thickness, under the entire site. Of significance, where the soft clay stratum was encountered in the borings, in all cases boring information indicated the relatively high density of included roots. With evidence from the test embankment that settlement would be greatly accelerated due to the presence of the roots, major consideration was given to the use of preload fills to modify the troublesome strata prior to site development.

Prior to the development of a designed preload scheme for site development, information obtained from the test embankment was further analyzed in order to extrapolate findings at the test embankment location to predictions of preload preformance at other locations on the site. This was accomplished by development of a "characteristic drainage parameter" (3).

Based on theory applicable to instantaneous loading and using laboratory determined values of c_v and values of t_{50} and t_{90} measured at the test embankment, equation 1 was solved for h. A value of h = 1.02 feet resulted for both t_{50} and t_{90} measured at the test embankment. Consequently this value of h, now designated the characteristic drainage parameter, h', was obtained.

Other information obtained from the test pit and test embankment provided a basis for the preliminary design of floating road and railroad embankments, should this method of constructing these facilities be selected. Based on the excessive mudwaving that occured during placement of the test embankment which appeared to be limited to the veneer of organic materials, it was decided that any embankment designs where fill material was to be placed directly on the subgrade would include provisions for removal or "mucking out" of this veneer.

CHAPTER IV

DESIGN

With the plant location and orientation selected, an intensive geotechnical investigation of the plant corridor was initiated (2) (4). In all, this design subsurface investigation incorporated over 4,000 linear feet of test borings at sixty-three locations within the plant corridor. After reducing and organizing the subsurface information available from this and the preliminary investigation, three classes of design problems were recognized. These were:

1. Roads and Railroads
2. Incidental and Secondary Structures
3. Main Plant Structure

Overall construction scheduling required that the design for the first two classes must be complete and ready for contract at the same time as the contract was to be let for clearing, grubbing, and general site work, therefore, these two classes received much of the initial design effort.

Roads and Railroads

The established design elevation of the roads and railroads was to be at +10 MSL and fill would be required for these facilties over much of the site. Consequently, both stability and settlement of the resulting embankments was of major concern. Two primary concepts were considered for the design of these facilties.

A "conventional" approach to constructing embankments over similar stratigraphy, where long term settlements and potential instability would be catastrophic would be by extensive excavation and replacement with superior backfill to provide a stable foundation for the proposed embankment. At this site excavation to a minimum depth of ten feet and subsequent backfilling would represent a major undertaking, both economically and from the standpoint of construction time.

A second approach to the problem of constructing transportation routes on this site was the consideration of using "floating embankments" constructed over the soft clay subgrade and employing counterweight berms to provide stability. Approximately two-thirds of the road and railroad embankments traverse the northern flats (northern two-thirds of the site with a typcial elevation of +4 to +6 MSL). A preliminary analysis of the embankments raised four to six feet above the soft subgrade indicated intrinsic instability.

Excavation of the very soft organic veneer and stipulated replacement with a sand or silty sand backfill to re-establish the original grade aided analitically in improving the stability of the embankments. The sand backfill improved the strength of the subgrade with respect to both rotational and translational failure and aided in precluding the propogation of tension cracks upward from the base of the embankment which would likely otherwise result from stiffness incompatability between the embankment and the subgrade. This later condition, i.e., tension cracking of the base of a relatively stiff embankment when placed over a soft subgrade due to excessive elastic and/or plastic deformation of the subgrade has been recognized qualitatively for a number of years. Not until very recently (post-dating this design study), however, has the phenomenom been studied quantitatively. In 1976, Chirapunta and Duncan (11) reported extensive work on this problem using nonlinear finite element techniques. Rechecking the original design analysis and recommendations in light of their study and findings, verified the necessity of replacing the organic veneer for these embankments.

Even after strengthening the subgrade, the stability of these embankments when loaded with the anticipated rail or rubber tired traffic was suspect. Computed factors of safety for this condition equaled 1.0 ± 0.1. In order to provide additional weight on the resisting side of the center of the critical failure arcs, counterweight berms were designed (33). In order to conserve offsite borrow materials, recommendations for stockpiling of the excavated organic veneer was proposed. By the time the embankments had been raised to the final elevation, the stockpiled veneer would have desiccated sufficiently to provide stable berm material which could be drawn up on the sides of the embankments. Incorporating these design considerations, the theoretical factors of safety for these loaded embankments was increased to 1.15 ± 0.05. A design of this nature represented a relatively high initial risk, however, economic considerations were heavily in its favor. Another consideration in favor of this approach was the fact that all design analysis of the embankment was done for undrained of Q- conditions and represented a condition immediately after construction. However, with time, as settlement of the compressible stratum occured the subgrade would gain strength with a resulting increase in the factor of safety.

In order to account for the relatively large settlements anticipated with this embankment design, slight overheight construction was recommended in order that the final height would comply with desired elevation requirements. Based on the accelerated rate of settlement as demonstrated at the test embankment, it was believed that a large majority of the detremental settlement would occur in a relatively short period of time, but that final surfacing should be deferred until measured rates of settlement indicated that additional detrimental settlements would not occur. Typical design recommendations for the roadway and railroad embankments are presented on Figure 8 and Figure 9, respectively.

Design for Incidental and Secondary Structures

Incidental and secondary structures for this project included such items as an electrical substation, dust collectors and scrubbers, a gas substation, water and sewage treatment facilities, oxide pellet silos, a slag disposal area, barge docking facilities, an employee parking lot and a maintenance shop-warehouse building. Many of these secondary structures were in the proximity of and dependent on selection of the substructure design scheme for the main plant facilities.

Others, such as the employee parking lot were located within that portion of the site covered with the essentially granular portion of the hydraulic spoil. This facility was located in such a way that a general cut was required to establish the final subgrade elevation. Thus, presented with an essentially sandy subgrade, even though the parking lot area was to be used as a "marshalling yard" during superstructure construction, it was believed that an eight inch thickness of crushed limestone directly over the subgrade would be sufficient to provide all weather trafficability.

Secondary structures, such as the oxide pellet silos and the electrical substation, had sufficiently heavy loads or such critical settlement tolerances as to preclude any foundation concept other than deep foundations (driven piling or drilled piers). Still others such as the barge docking facility were contracted on a design-construct basis and are beyond the scope of this paper.

Of particular interest among the incidental and secondary structures on this project was the maintenance shop-warehouse building (Building 8). Construction scheduling dictated that Building 8 be completed and available to

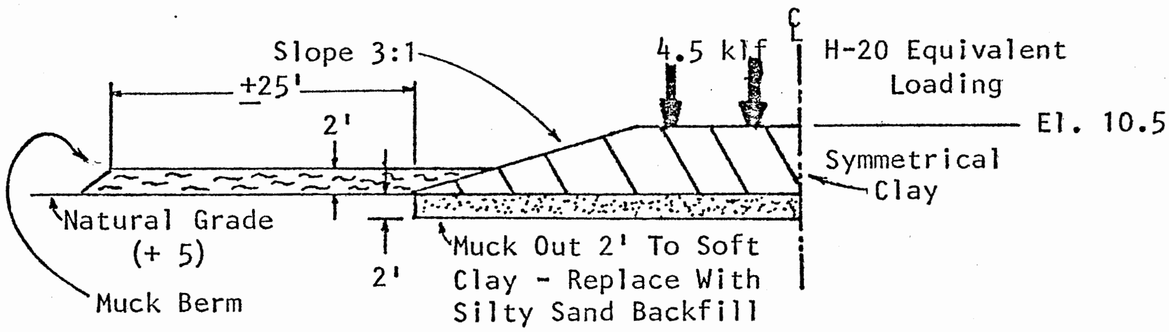


Figure 8. Roadway Embankment - Northern Flats

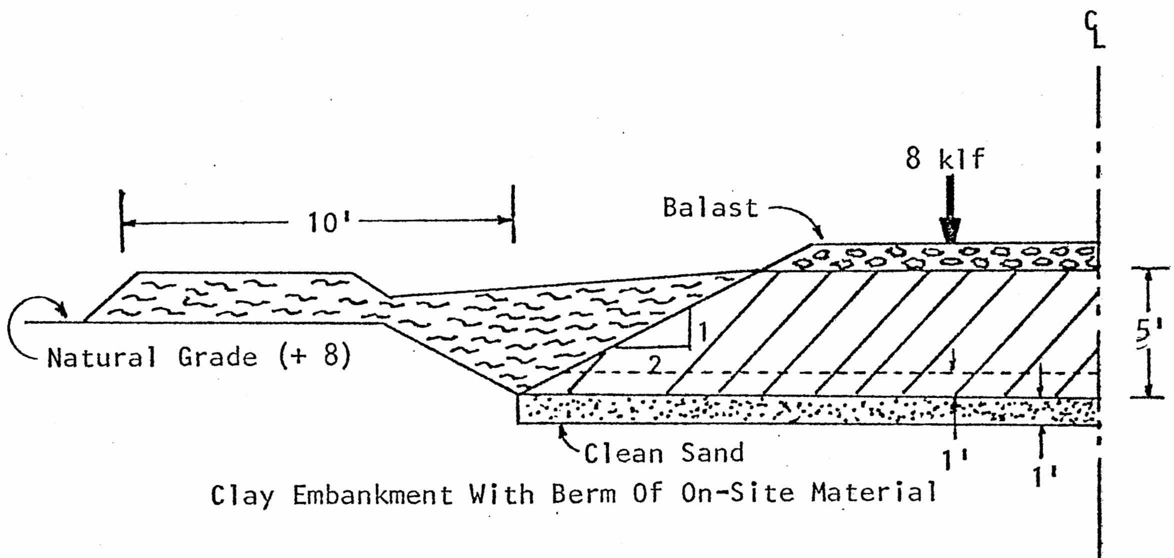


Figure 9. Railroad Embankment - Main Plant Area

warehouse weather-degradable materials and components well in advance of construction of the main plant superstructure. Consequently the completion of this structure represented a critical event in the construction schedule. With the design phase under way in the second quarter of 1974, the planned completion of Building 8 was designated as September 1975 or slightly more than a year after initial site work was begun.

Building 8 was to be a single-story maintenance shop-warehouse building, approximately 200 feet square in plan. The building was prefabricated metal with concrete block partitions and partial concrete block exterior walls. Column loads were anticipated to be in the order of forty kips with a design floor loading of 500 pounds per square foot. The stratigraphy at this location consisted of approximately eighteen feet of soft, root-penetrated clay overlain by hydraulically placed, uncompacted sandy silt dredgings from the adjacent Neches River. The dredged material varied in thickness from approxiately four feet on the western portion of the location to approximately nineteen feet on the east. The silt, which had been desiccated in the upper few feet, was soft in consistency.

As with the main plant, flood insurance requirements placed the finished floor at elevation +14 MSL. Consequently, in order to establish the subgrade elevation at approximately +14 MSL, cutting would be required on the east side while filling would be necessary on the western portion of the site. The average increase in effective stress at the center of the compressible stratum if the building were situated on a surface foundation system would be about 500 pounds per square foot. Calculated settlements under the western portion of the structure, due to the imposed fill required to establish the subgrade elevation and the weight of the building, were in excess of one foot. However, due to the removal of some of the pre-existing dredged fill, creating a preconsolidated condition on the east side of the structure, essentially no settlement was expected for this portion of the structure.

Soil borings in the area of the planned location of Building 8 revealed the existence of a dense, randomly oriented root system throughout the soft clay stratum, very similar to that observed at the test embankment-test pit location. With this preliminary indication of the probability of greatly accelerated consolidation resulting from internal drainage of the clay stratum by the included roots, modification of the subsurface soils became a viable alternative for this structure. However, some concern was noted that the effective stress resulting from a surcharge fill for this structure being considerably larger than those resulting from the test embankment, might impede or completely close off internal drainage through the included network of roots. Also, the index properties and initial void ratio of the clay matrix under this location were considerably different than the same properties from under the test embankment. Consolidation test results and index properties of the clay matrix at the Building 8 location are presented on Figure 10.

Considering the time constraint for foundation and superstructure construction, three concepts were evaluated for this facility. These concepts were:

1. Preload the site of Building 8 with a surcharge fill designed to accomplish, within six months, settlements equal to or exceeding that which would be expected to occur under the weight of the permanent fill plus the structure. This would allow construction of Building 8 with shallow spread footings and slab-on-fill construction after removal of the surcharge fill.
2. Use a surcharge fill to reduce or eliminate settlement and allow slab-on-fill construction, however, the structural frame would be supported on driven piling.

	W_L	I_P	W_N
Range	27 - 113	10 - 79	26 - 82
Average	71	44	52

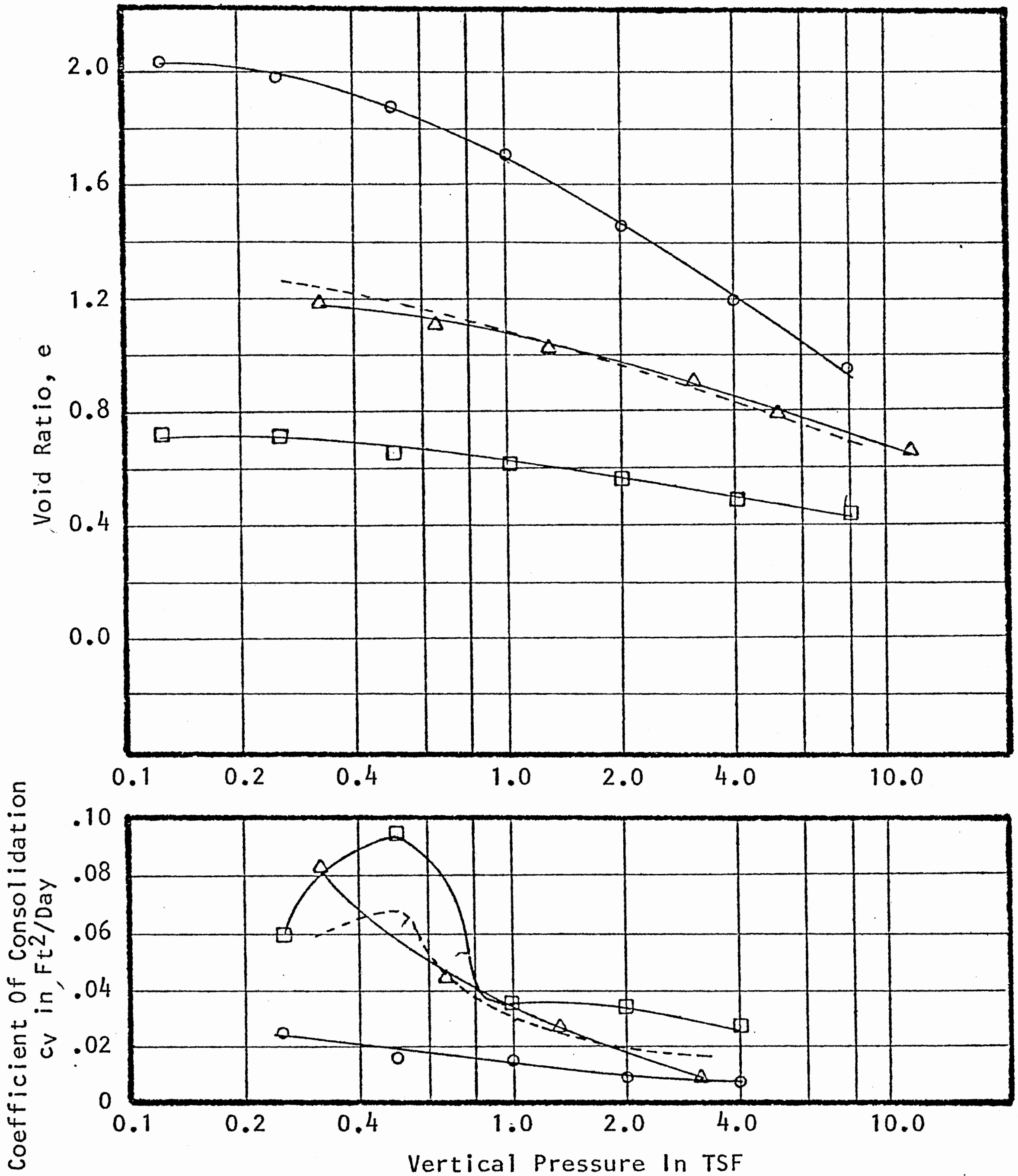


Figure 10. Consolidation Test Results - Building 8

3. Use of driven piling for support of the structural frame and support of a structurally suspended floor system.

If concepts 1 or 2 were selected, the hypothesis of greatly accelerated settlement of the root-penetrated clay would be hedged by budgeting contingent monies for the installation of sand drains through the surcharge fill and the compressible stratum if settlements did not occur at a rate consistent with the time limitations for this portion of the construction. Based on this consideration, the potential time delays involved with the surcharge fill was accepted and design proceeded in order to implement concept 1. Projected savings by utilizing concept 1, even if the installation of sand drains was ultimately required, was in the order of \$ 90,000 or one-third of the cost of the next closest alternative.

In the design of the surcharge fill it was believed that the uncompacted silt dredgings would provide an effective drainage layer between the imposed fill and the compressible stratum, therefore, removal of the silt and replacment with a more pervious material was not planned. Drainage at the bottom of the compressible stratum would be provided by a naturally occuring relatively clean sand stratum. In order to insure that the magnitude of settlement in a relatively short period of time would equal or exceed setlement resulting from the permanent fill and structure, a surcharge height of ten feet was selected (surface of the surcharge fill initially at elevation +24 MSL). This would result in an effective stress acting at the mid-depth of the compressible stratum in the order of 2500 pounds-per-square foot and 2850 pounds-per-square foot under the eastern and western portions of the surcharge fill, respectively. These surcharge stresses compare to 1700 pounds-per-square foot and 2050 pounds-per-square foot which would result from the permenant portion of the fill and the imposed structure after removal of the surcharge.

The preload fill would be instrumented with several settlement plates as depicted in Figures 11 and 12. Due to the unfavorable experience with piezometers at the test embankment location which was attributed to large random variations in excess pore pressure resulting from the included roots, similar installation was not planned for this surcharge.

With a surcharge fill designed in this manner, calculations of the magnitude and rate of settlement based on conventional theory, but using the characteristic drainage parameter of 1.02 feet, developed from performance measurements at the test embankment, were made. The magnitude of the settlement due to primary consolidation for the east and west sides of the surcharge fill were calculated to be 7 inches and 19.5 inches, respectively. The rate of consolidation as described by times calculated to correspond to 50 percent and 90 percent primary consolidation, respectively, and using the full half-thickness of the compressible stratum (nine feet) were 532 days and 2290 days, respectively, under the east side and 798 and 3434 days, respectively, under the west side of the fill. However, if the characteristic drainage parameter of 1.02 feet is substituted in these calculations for the half-thickness of the stratum, these times are reduced to 7 days and 30 days for the east side and 10 days and 44 days for the west side of the surcharge fill, respectively. Consequently, if the included roots would again be effective as internal drains under the imposed surcharge load, the desired preconsolidation of the soft clay could be accomplished well within the alloted time.

It is believed that not only the settlement due to primary consolidation must occur under the preload, but, also substantial consolidation due to secondary compression should occur. It has been demonstrated by Johnson, et. al., (16) that if secondary compression occurs for a period of time under a given

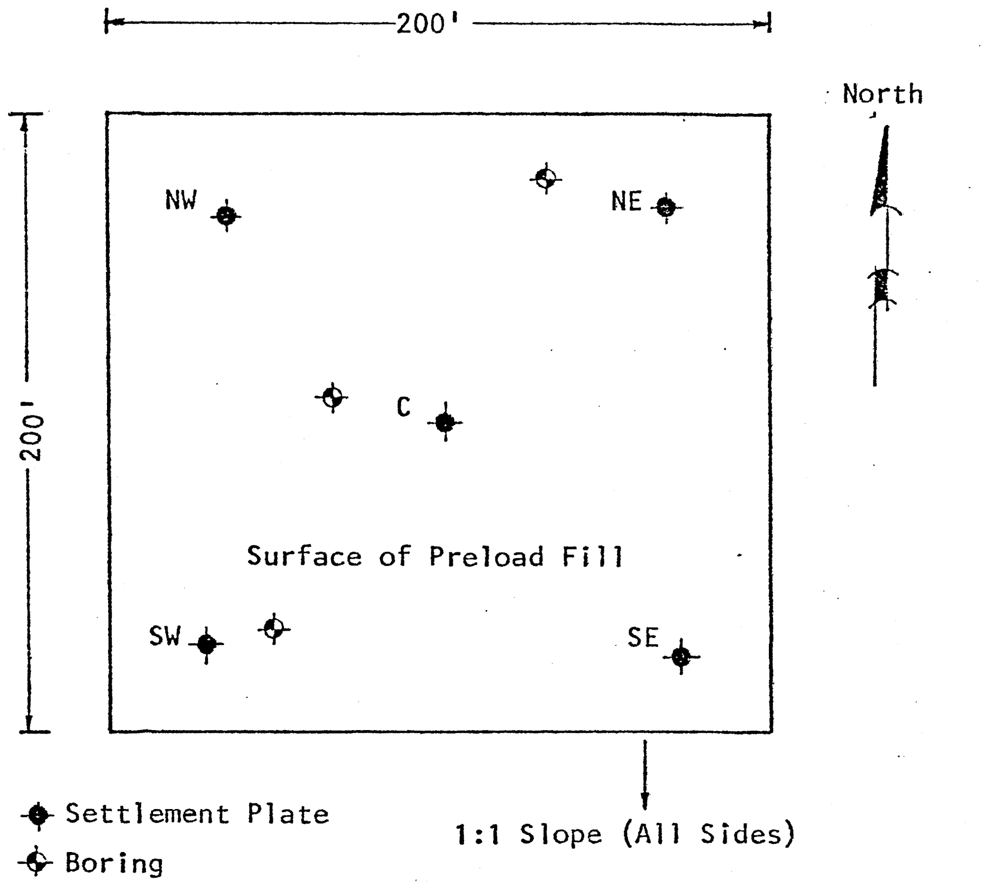


Figure 11. Plan Of Building 8 Preload Fill

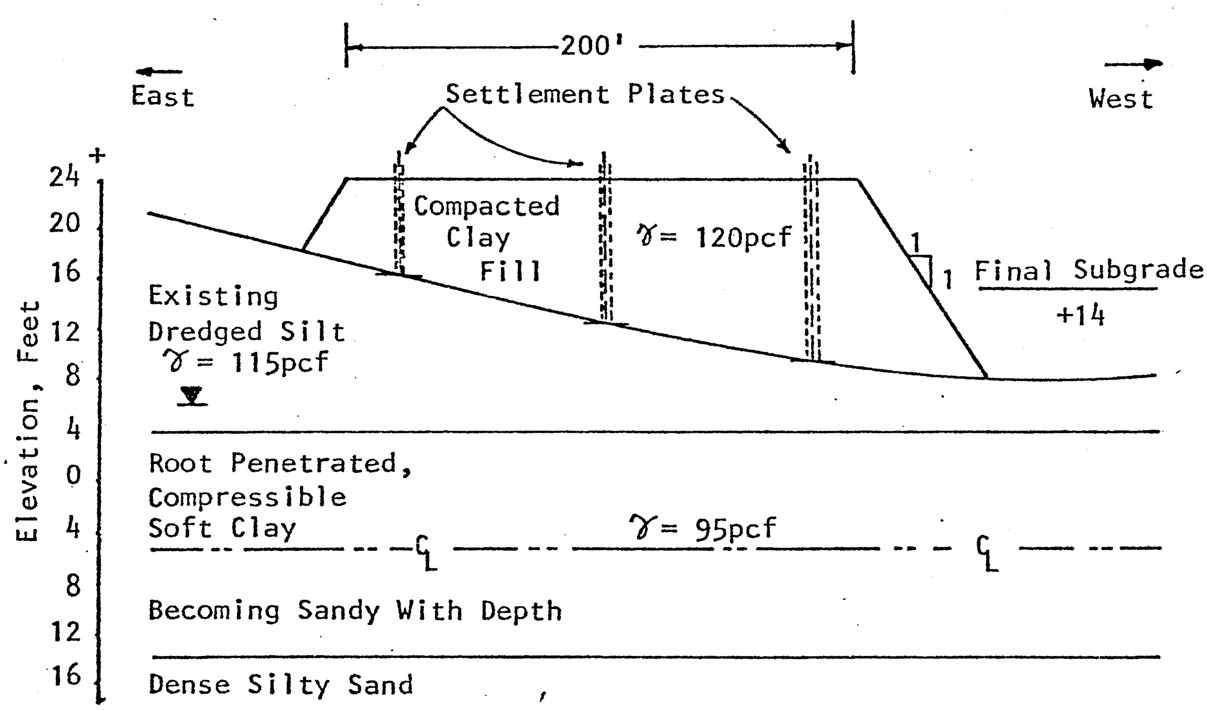


Figure 12. Profile Of Building 8 Preload Fill

effective stress which is subsequently reduced, then secondary compression will be suppressed for a substantial period of time. This phenomen is not well understood and little quantitative information is available for its prediction. It is, however, believed that if the primary consolidation occured at the predicted accelerated rate and the surcharge remained effective long after primary consolidation had ceased, the subsequent structure would not be subjected to secondary consolidation throughout its design life.

Design Considerations - Main Plant

The main plant, consisting of the furnace area (melt shop), casting aisles, billet storage area, reheating furnaces, a rolling mill, and a new product storage area, all arranged in a linear array, would be housed in a structure approximately 2000 feet long and 300 feet wide. Because of the extremely heavy concentrated loads associated with the production of steel such as those resulting from melting furnaces, turret ladles, casting machines and reheater furnaces, and their very small tolerance for either total or differential settlement, no alternatives other than deep foundation elements were ever considered for these components. There were, however, a number of lighter loads such as most of the structural framing, the rolling mill (which occupied over half of the total structure), and the billet and new product storage area, which did not require extremely close control of settlement and were subject to the consideration of alternative sub-surface schemes.

As a result of the intensive subsurface investigation of the corridor in which the main plant was to be situated, the continuous existence of the soft clay stratum under the main plant location was verified. These and previous borings indicated the thickness of the soft clay stratum to vary from about 20 to

30 feet, generally becoming thicker toward the river. As a result of this investigation four alternate construction concepts were developed which appeared to be realistically within the construction time-frame and economic feasibility of the project. These concepts were:

1. Preload the entire main plant location as with the Building 8 surcharge. Successful preloading of the area would reduce settlement of incidental components to within tolerable limits when founded with a near surface system. Deep foundation elements would still be required for very heavy concentrated loads and/or very settlement sensitive components.
2. Utilize deep foundation elements for the support of all structures and components. This would provide for a minimum of general site development.
3. Excavation of the soft clay stratum in the main plant area. The excavation would be dewatered and backfilled with clean clay and/or silty sand from off-site borrow areas. All operations including compaction would be accomplished using conventional earthwork equipment and procedures. Shallow spread footings and/or drilled, underreamed footings would be used for all but the heaviest structural framing and component loads which would be supported by deep foundation elements.
4. This concept would be very similar to concept 3., however, excavation and backfilling would be accomplished by hydraulic removal and placement of sand selectively dredged from the river bed. Densification of the sand backfill would be accomplished by deep compaction techniques such as Terra-

Probe or Vibroflotation. After site preparation foundation selection would be as anticipated for concept 3.

Preliminary engineer's estimates indicated the areal surcharge fill would be unfeasible from the standpoint of either economics or construction time. From a construction time standpoint, even if settlement under the influence of the surcharge occured very rapidly, as predicted, the amount of time required to place and remove the surcharge would greatly exceed time allocations.

With Concept 1. eliminated, a close economic comparison of the remaining three alternates was conducted. The most costly of these alternatives by at least three million dollars was the concept in which the soft clay would be excavated and the area backfilled by conventional earthwork techniques. Other than the economic considerations, sufficient sources of available and suitable backfill material were questionable within economic hauling distances of the site. Also, the amount required to accomplish this large excavation and backfilling operation would be pushing the tight construction schedule for this project. Concept 2., the support of all components and structural frames on deep foundations, had some very undesirable characteristics although it was less costly than Concept 3. Primarily because of the absence of general site development with this concept very strict limitations would be imposed by making very costly the modification of any of the designs during the construction phase or later operation of this plant. Any modification, relocation, or addition of loads would require the costly addition of new deep elements.

The least costly and most promising of the alternatives was concept 4. in which dredged removal of the soft clay and backfilling by hydraulic placement was specified. This alternative would allow for the extensive use of relatively high capacity shallow footings for all but the heaviest of structural framing and component loads and by developing the entire general plant area would provide great flexibility for design alterations or future additions. Engineer's estimates indicated this alternate to be considerably cheaper than concept 3., largely because of the more favorable unit cost of excavation and backfilling by hydraulic means as compared to conventional earthwork methods.

Later bids verified the economic comparison of these alternates based on the engineer's estimates. For example, in a bid comparison of development of the main plant area, the soft clay stratum (and overlying hydraulic spoil) could be removed by dredging, the dredged material spoiled, the resulting excavation hydraulically backfilled and the backfill densified at a unit cost of between $1.55 and $1.95 per cubic yard. This unit cost is compared to a cost of between $12.00 and $16.00 per cubic yard for conventional land techniques.

In considering the dredging and hydraulic backfilling alternative, many problems had to be resolved. The first problem in design of the dredging alternate was to develop a plan for disposal of the resulting dredged spoil. The plan had to meet applcable requirements of various federal and state agencies such as the U.S. Army Corps of Engineers and the Texas Water Quality Board. Working within constraints imposed by the site and the various agencies, it was decided to spoil the dredged material on the undeveloped portion of the site belonging to the Georgetown Texas Steel Corporation. Since the southern portion of the site was already covered with reasonably competent hydraulic spoil, discharge of additional spoil would be initated in the area immediately behind the existing spoil hills, thus utilizing the pre-existing spoil as a natural levee. Since the granular portion of the discharge would settle out first, this discharge location would have a second advantage of advancing the granular spoil hill landward (to the north).

In order to prevent a backflow of the fine-grained particles suspended in the dredging water back into the river, a system of low dikes was designed to enclose three settling ponds on the eastern two-thirds of the site. The discharge water could then be stilled as it moved slowly from one pond to another allowing sedimentation of the fine particles. The discharge water would ultimately be released into an old logging canal and returned to the river. A plan design of the spoil area is shown on Figure 13.

Integral to this concept was the location of a sufficient quantity of sand within economic pumping distance of the site to provide backfill for the excavation. Several abandoned channels and oxbows were explored by borings, probes, and a depth finder, both upstream and downstream from the site. An oxbow on the north side of the river immediately downstream from the site, locally know as Star Bayou, was selected because of its proximity and the quality of the available sand (5). Contrary to most of the late Pleistocene sand encountered in this area, which are very fine-grained, often grading into a silt, a stratum of medium coarse sand was identified in this oxbow. The surface of the sand stratum was generally encountered around elevation - 40 MSL and was overlain by ten to thirty feet of organic silt.

The limits of the excavation were defined and the problem of stable submarine slopes for the excavation were addressed. Because the slopes would be cut primarily in the soft clay stratum overlain in some locations by pre-existing hydraulic spoil, relatively flat slopes were needed. Depending on the surface elevation of the hydraulic spoil, theoretical safe dredged slopes (F.S. = 1.2) ranged from one vertical to three horizontal to one vertical to five horizontal or flatter.

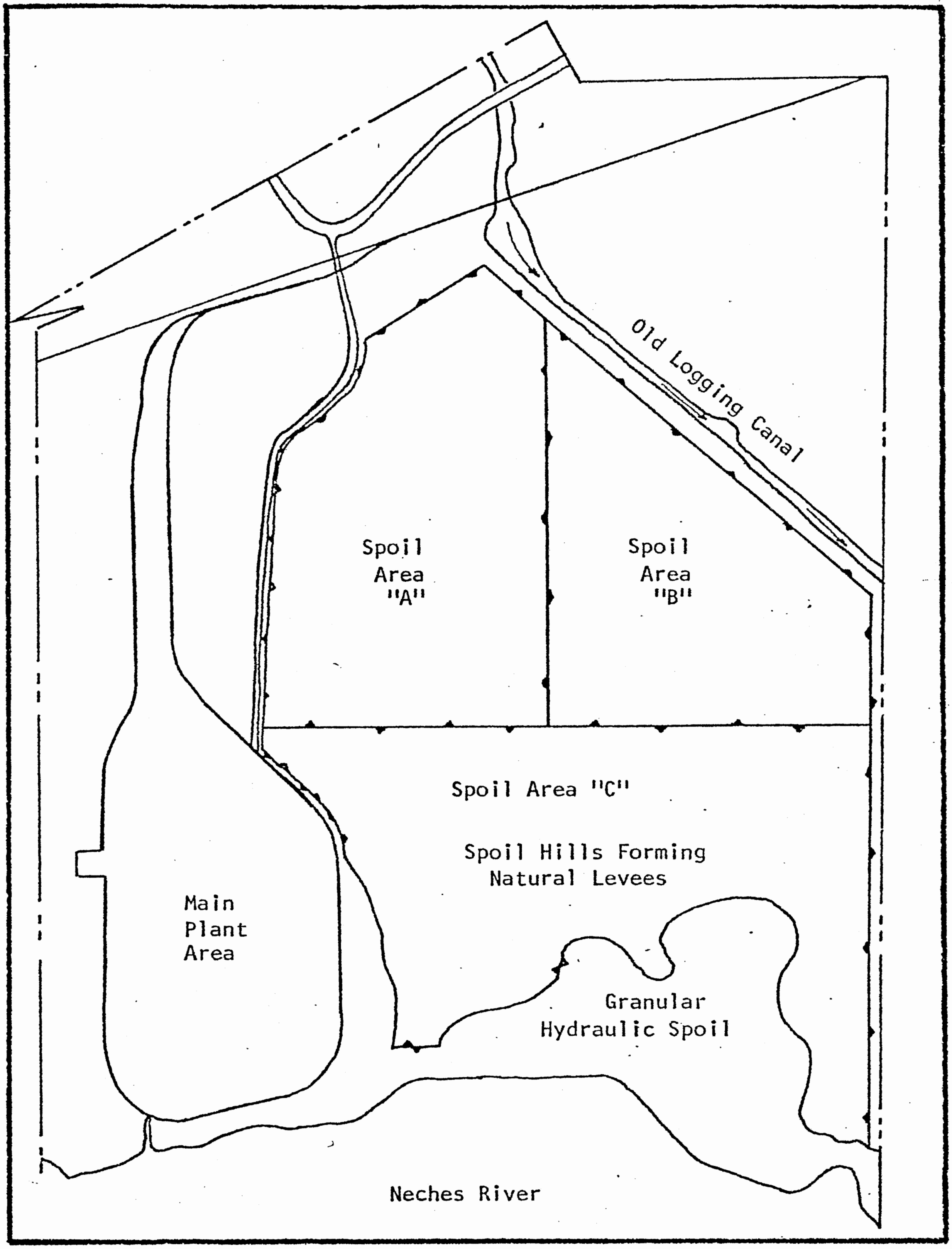


Figure 13. Plan Of Dredged Spoil Areas

Again, the presence of the relatively dense root system assisted in the site development. It was believed that the root system would act as "earth reinforcement" and allow somewhat steeper slopes. The minor risk of steeper (than theoretically safe) slopes was accepted and all slopes in the main plant excavation were designed to be dredged to one vertical to three horizontal. Later dredging demonstrated the reinforcement provided by the root system with slopes as steep as one vertical to two horizontal being common.

The bottom of the soft clay stratum (top of the medium dense sand stratum) varied from elevation - 12 MSL to - 22 MSL within the limits of the main plant area. All dredging was to be carried to the top of the sand stratum. A design plan of the main plant excavation is shown in Figure 14.

After dredging in the main plant area was complete the dredge would move downstream to the borrow channel. Dredging schedules called for the dredge to strip the organic silt overburden from the deposit of sand borrow, which would in turn be discharged into the onsite spoil area. The sand would then be dredged for discharge in the main plant excavation.

It was imperative that the sand backfill deposited in the main plant excavation be very clean, i.e., particles finer than the Number 200 U.S. Standard Sieve less than ten percent, in order for densification of the sand backfill to be accomplished. In order to optimize this means of site development, relatively deep densification of the sand backfill (and lower natural sand) had to be accomplished to allow the use of near surface foundations designed with settlement as the controlling factor.

To accomplish the backfill control necessary to optimize this scheme a methodical backfilling plan was devised. All backfilling would progress from the head of the dredged excavation back toward the confluence of the dredging

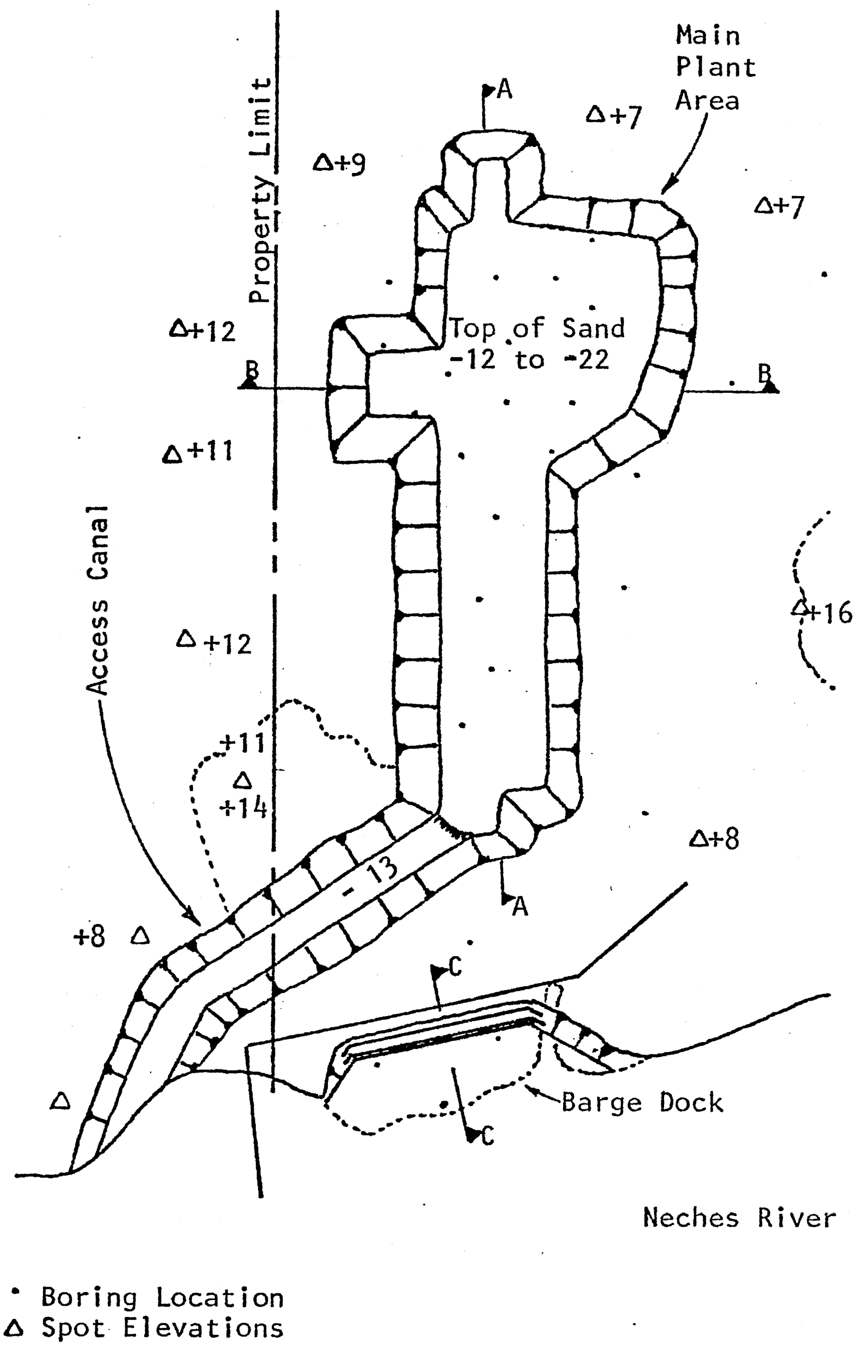


Figure 14. Plan Of Dredged Excavation For Main Plant

channel and the river. When necessary the downstream face of the deposited sand backfill would be dressed to maintain high discharge water velocities, thus allowing the deposition of only coarse-grained material. All fine material would remain in suspension until they were carried well past the limits of the "structural" backfill (into the dredging access channel). As an environmental consideration the main channel of the Neches River was to be protected by a "silt screen" positioned across the mouth of the access channel. Consequently, most of the fine-grained sediment would be trapped downstream from the structural backfill, but before reaching the main navigation channel.

All additional site filling (in non-excavated areas) would be accomplished by hydraulic methods. This included such periferal areas as the substation area, the scrape storage area, etc. This would provide substational savings when compared to conventional offsite truck hauled borrow.

In order to limit the volume of fine-grained particles reaching the backfill area, a valved "Y" was to be installed in the discharge line. From this "Y", one pipe would lead to the excavation for discharge backfilling and the other branch would lead back to the designated spoil area. With this branching, any deleterious materials being pumped from the dredge, downstream, could be immediately diverted to the spoil area, thus avoiding contamination of the backfill. As with the access channel, all outlets from the borrow channel and all distributaries through which the discharge water could find its way back to the river were to be separated from the river by silt screens.

After completion of dredging and backfilling operations in the main plant area, the dredging schedule called for excavation in the barge docking area on the perimeter of the main navigation channel. To complete the dredging parcel of the site development, materials dredged from the barge docking area would be deposited in the main plant access channel until the original grade was re-established. Additional dredged spoil resulting from the barge slip would be discharged in the designated spoil area.

The remaining element of this design alternate was densification of the backfill. Essentially two concepts were evaluated. The first concept would be the utilization of "pressure injected footings" as developed by the Franki Foundation Company. These relatively shallow footings, installed at elevation 0 to - 10 MSL in the sand backfill would combine the densification and footing installation phases of this construction. The second concept would utilize conventional viboration techniques as developed by the Vibroflotation Corporation or by the use of Terra-Prob equipment.

Economic estimates eliminated the pressure injected footings from consideration. These estimates indicated that either of the vibrational techniques would be competitive. In order to provide the desired density, namely, a minimum of eighty percent relative density under all foundation elements, with a sixty-five percent requirement for the general fill, a ten foot triangular insertion pattern was specified.

For design purposes close control of the backfilling operation would be assumed to produce a relatively clean sand backfill (Finer than No. 200 Sieve 10%). With the clean sand it was further assumed that minimum eighty percent relative densities were attainable by deep vibration techniques. Using a correlation developed by Gibbs and Holtz (14):

$$\phi = 28^{\circ} + 15\ (D_r\ /\ 100^{\circ}) \text{ ---------------- } (2)$$

where: D_r is the relative density in percent, an indicated angle of internal friction of 40 degrees would be available. Because of clearance requirements a minimum founding elevation of + 6 MSL (8 feet below the finished floor elevation) was considered in design of spread footings. Using these geometric and strength parameters and considering the prevailing design parameter which limited elastic settlements to 0.75 inches, the design of shallow foundation elements would proceed.

Based on Meyerhof's (21) emperical formula limiting the settlement of foundation elements in a sand stratigraphy:

$$P_a = \frac{NS_a}{12} \left(\frac{B + 1}{B}\right)^2 \text{ ------------------- } (3)$$

where; P_a is the allowable bearing capacity in tons-per-square foot, N is the average result of standard penetration tests to a depth of 2B, S_a is the settlement in inches, and B is the least dimension of the footing, the footings could be designed for limiting elastic settlement. It was believed that any consolidation settlement which occured in lower clay strata would be uniform in nature due to the great thickness of sand above the clay strata. An "SPT" value of 50 blows per foot was assumed (corresponding to the 80 percent relative density requirement).

As a second check on settlement and bearing capacity values determined by use of equation (3), bearing capacities were also computed using the Terzaghi Bearing Capacity Formula (30) and considering bearing capacity factors for local-shear conditions:

$$q_{ult.} = 1.3cN_c + q\,N_q + 0.4\gamma BN_\gamma \text{ ---------- } (4)$$

where; N_c, N_q, and N_γ are the bearing capacity factors based on the Terzaghi failure theory, c is the average cohesion of the soil to a depth of B below the foundation element, q is the effective overburden at the founding depth, $\overline{\sigma}$ is the average effective unit weight of the soil, and B is as defined above.

In considering a common design load of 500 kips and a unit weight of the sand backfill of 100 pounds-per-cubic foot (to account for possible fluctuations in the water table), it can be readily seen that an allowable bearing capacity of 8,000 pounds-per-square foot (F.S. = 3) was available at the eight foot depth. According to the work by Meyerhof and Terzaghi this allowable value was justified for both settlement and local shear criteria.

Where concentrated loads were substantially larger than 500 kips and/or settlement limitations were more extreme, deep foundation elements became desirable. In these cases driven piling was selected and design parameters were developed according to local experience. Typical profile drawings of the main plant area showing these foundation concepts are presented on Figure 15. Cut section designations correspond to cut lines shown on Figure 14.

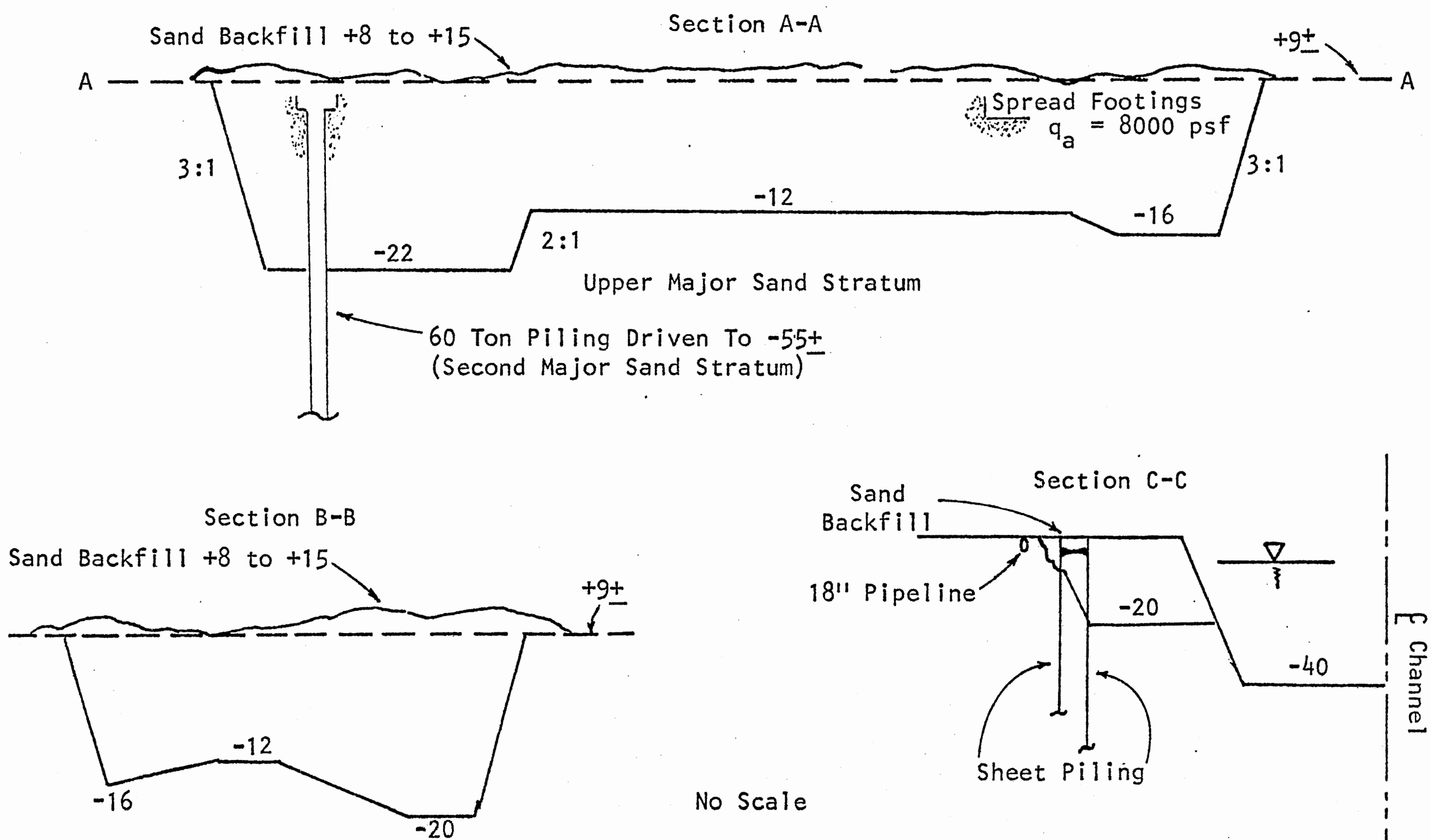


Figure 15. Typical Profile Of Dredged And Backfilled Areas For Main Plant

CHAPTER V

ANALYSIS OF BUILDING 8 PRELOAD

Construction

With a design committment for the use of a surcharge fill to affect a preconsolidated condition of the soft clay prior to the construction of Building 8, the necessary earthwork contract was awarded. After clearing and grubbing operations, actual placement of the permanent and surcharge fill began on September 25, 1974 (7). Materials used for the fill were borrowed from adjacent spoil hills and consisted of a heterogeneous mixture of red, tan, light gray, and gray clay and silty sand. The heterogeneous fill was placed over the desiccated silt crust which was present over the plan location of the surcharge and densification was accomplished by repeated dozer passes during the placement and shaping operations.

The fill was placed in quadrants, with the construction sequence consisting of bringing one quadrant of the fill essentially to final grade, before beginning placement in the next quadrant. Approximately two days were required to bring the fill to grade in each quadrant. No appreciable settlement was evident in the unfilled quadrants, as indicated by reading of the settlement plates, as a result of filling operations in adjacent quadrants. Consequently the initial or reference time for each settlement plate was taken as the time filling began in that quadrant. The access and reference pipe for the center plate was struck during fill placement and subsequent readings proved to be unreliable.

The surcharge portion of the fill was allowed to remain for a period of five months, after which the preload portion of the fill was removed, thus establishing the final subgrade at approximately elevation + 13.3 MSL. A one month delay between removal of the surcharge and beginning of construction was allowed in order that settlement resulting from residual excess pore pressures might cease. Dug, spread footings to found the structural framing for Building 8 were placed on a bearing surface in the fill at an approximate elevation of + 9.5 MSL. All footings were dimensioned for an allowable bearing pressure of 500 pounds-per-square foot. This allowable bearing pressure was subjectively selected in order to provide a factor of safety of at least three for total loads as a further protection against undesirable settlement.

A three inch sand cushion (capillary barrier) was placed over the building subgrade prior to placement of a six-inch, reinforced concrete floor system. In a fourteen month period after removal of the surcharge fill maximum settlements had not exceeded 0.55 inches with maximum differential settlement of less than one-fourth of an inch. Since that time no quantitative settlement measurements have been made, however, periodic inspection of the building has indicated no additional distress which could be related directly to continuing settlement.

Analysis

In an unpublished study, Olson defines a parameter, T_c, the construction time factor, as:

$$T_c = \frac{c_v t_c}{h^2} \quad \text{-------------------------} \quad (5)$$

and has determined that if T_c is less than 0.1, good field correlation with instantaneous loading theory is expected for values of the Percent of Consolidation, U, equal to or greater than 50 percent.

If t_c, the time required to place the fill over a given plate, is taken as two days, the upper value of the coefficient of consolidation, c_v, is taken as 0.038 Feet^2/Day, and if the characteristic drainage parameter of 1.02 Feet (developed by analysis of the rate of settlement of the test embankment) is substituted for the stratum half-thickness, the resulting value of T_c is substantially less than 0.1. Consequently, settlements of the surcharge fill were analyzed using instantaneous loading theory.

As each quadrant was filled, periodic reading of the corresponding settlement plate was begun. The readings for each settlement plate were referenced to a permanent bench mark approximately 500 feet away from an construction. All plate elevations were read and recorded every two to four days for the first three weeks after construction and approximately weekly, thereafter. Settlement was plotted vs. the square root of time and Taylor's procedures were used to determine settlement times corresponding to 50 percent and 90 percent primary consolidation. The settlement of each of the plates as it corresponds to the square root of time is shown on Figures 16 and 17.

From the expression relating time to percent of primary consolidation, Equation 1, the characteristic drainage parameter, denoted h', can be calculated for observed settlement of the preload fill. Table 1 presents the various values obtained for h' corresponding to t_{50} and t_{90} and using an average value for c_v of 0.03 Feet^2/Day corresponding to 1.2 Tons per Square Foot, from the preload observation. These values of h' corresponding to t_{50} and t_{90} for settlement of the preload fill (and especially the average of these values) compare very favorably to corresponding values of the characteristic drainage parameter, 1.02

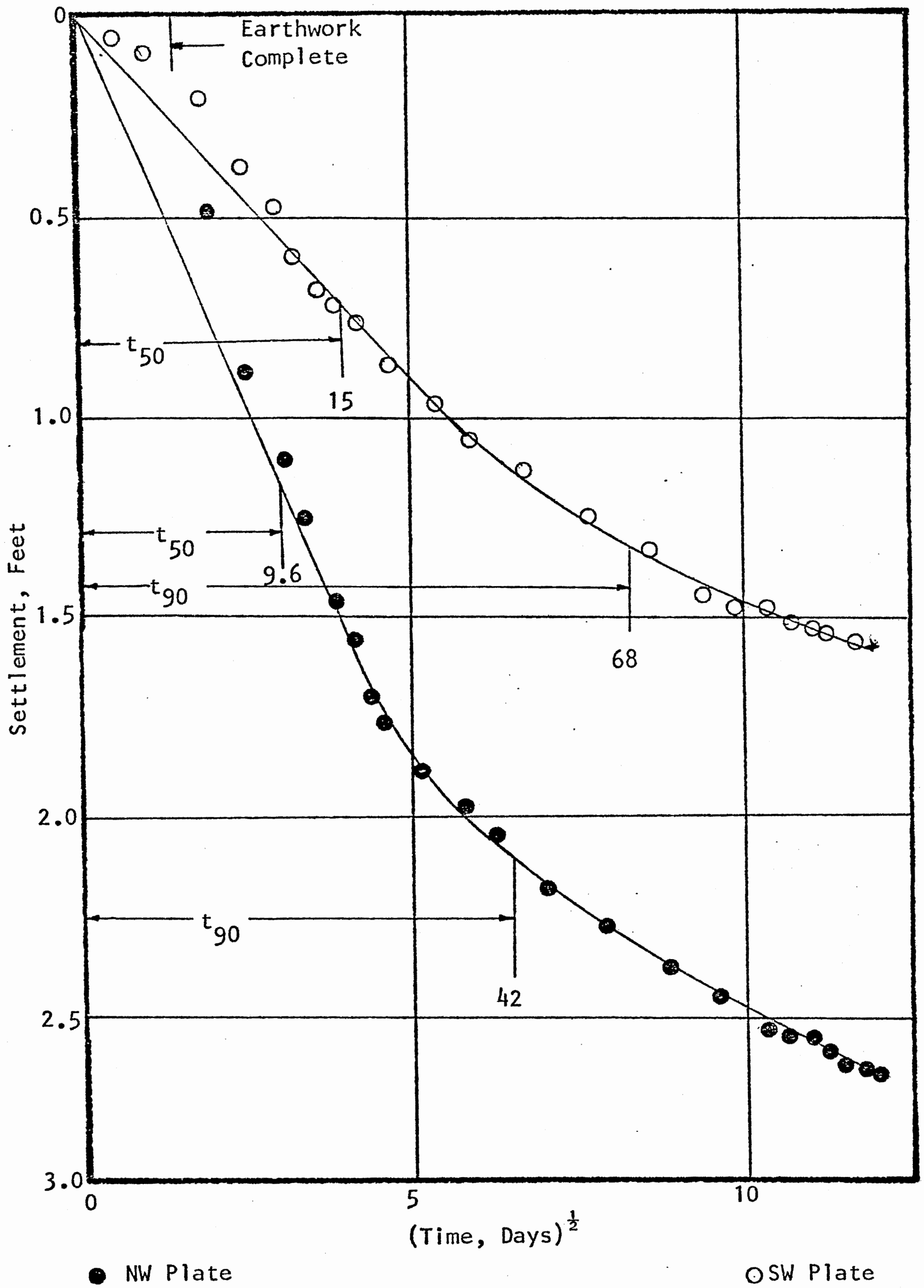


Figure 16. Rate Of Settlement - Western Plates

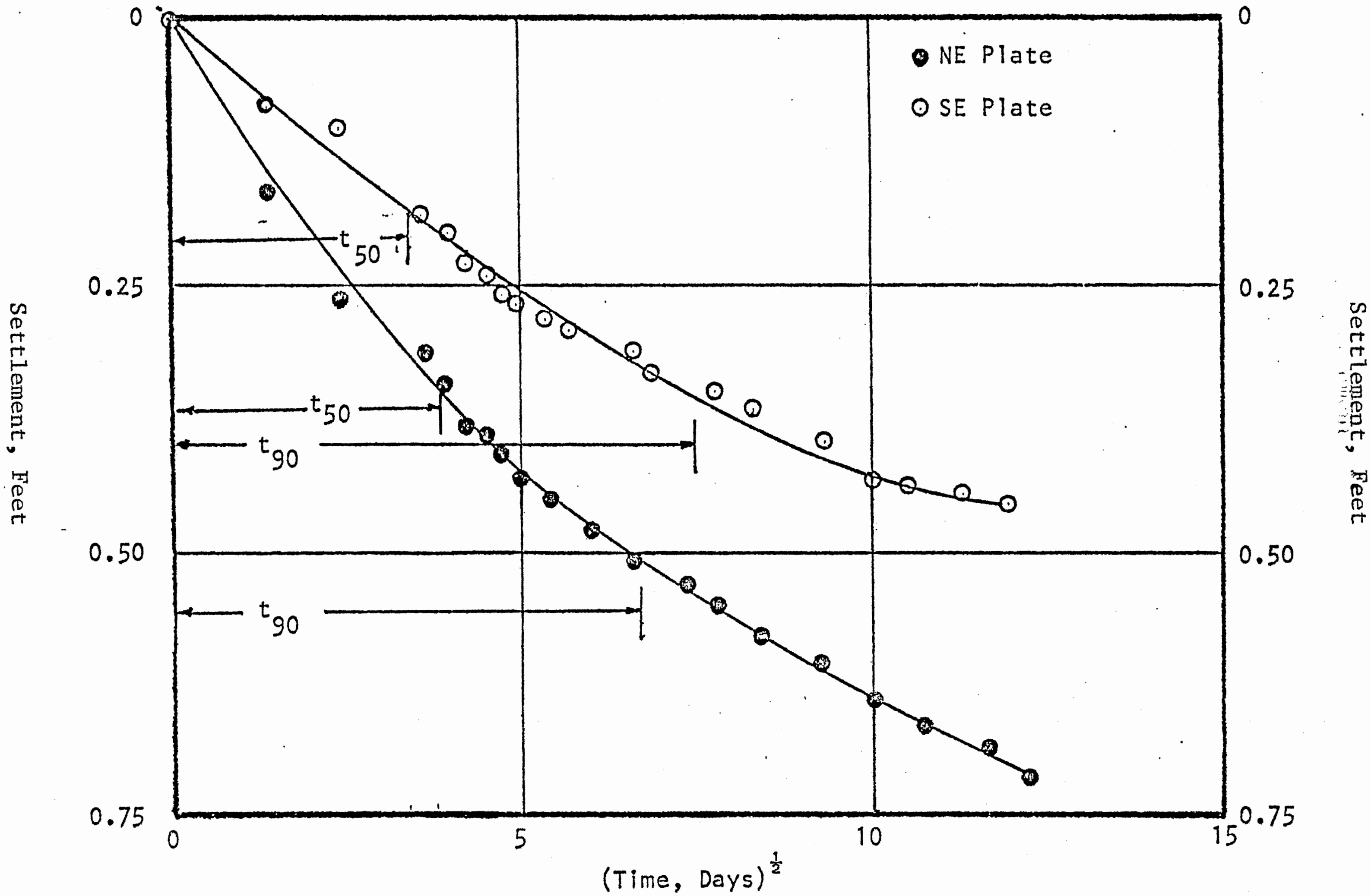


Figure 17. Rate Of Settlement - Eastern Plates

feet, calculated for settlement of the test embankment. This comparison is substantial evidence supporting the hypothesis of internal drainage by the root mass.

TABLE I

VALUES OF THE CHARACTERISTIC DRAINAGE PARAMETER CALCULATED FOR SETTLEMENT OF THE PRELOAD FILL

Settlement Plate	t_{50}	h'	t_{90}	h'
Northeast	10.2	1.25	42.3	1.22
Southeast	4.0	0.78	19.4	0.76
Northwest	9.6	1.21	42.0	1.22
Southwest	15.0	1.51	68.0	1.55
Average		1.19		1.21

It must be considered in evaluating this comparison that: (1) the index properties of the clay matrix were significantly different at the two locations on the site, i. e., average W_L at the test embankment was 48 compared to the value of the corresponding property at the preload fill of 71, and (2) the thickness of the compressible stratum at the two locations was substantially different. A subjective evaluation of included root frequency and size indicated, however, the vegetative environment during deposition to have been very similar at both locations. The included roots in the compressible stratum, therefore, remains the only obvious similarity between the two locations.

Sample disturbance (during sampling and handling) and its effect on the measured value of the coefficient of consolidation, particularly at these inherent low values of effective stress could be used to explain part, but by no means all, of the discrepancy between conventional rate predictions and measured rates at these two sites. Analysis of typical consolidation tests presented by the Navy (33) indicates that extreme sampling disturbance can cause measured values of c_v to be as much as three to five times lower than corresponding in-situ values within the range of effective stresses considered in this project. However, considering that extreme care was exercised in sample recovery and selection for consolidation testing, it is believed that the laboratory values for c_v are much closer to in-situ values than the extremes just described. Even assuming that the laboratory value of c_v was only one-fifth of the in-situ value, the major cause of the accelerated settlement must be related to a drainage distance shorter than the half thickness of the consolidating stratum.

Gibson and Shefford (15) have found that a potential drainage layer in a two-dimensional flow field must be at least 3×10^4 times more permeable than the clay being drained for it to be effective. In order to verify the effectiveness of the root system as drains, samples of the cypress roots were collected and were subjected to laboratory falling head permeability tests with an initial hydraulic gradient of 30. The results of these tests indicated coefficients of permeability (parallel to the grain) of 3 Feet/Day to 10 Feet/Day. When these values are compared with the average value of 3×10^{-5} Feet/Day which was similarily determined for the clay matrix, the effectiveness of the roots as drains is verified.

In an attempt to improve the predictibility of settlement considering the macrostructure and properties of this compressible stratum the problem was modeled considering several different theories. As a result of visual observations and a review of a number of laboratory consolidation tests a random variation of the coefficient of consolidation throughout the thickness of the compressible stratum was detected. Of particular interest, the compressible layer at the Building 8 location was much more plastic near its surface than the layer at the test embankment. Also, at the Building 8 location, the compressible stratum became progressively less compressible (decreasing m_v) with depth while exhibiting an inverse relationship for c_v. Consequently, it was thought that a variation in some of the additional stratum properties affecting settlement may improve the method of prediction.

Three specific models were considered in an attempt to improve settlement predictability for this problem. The first of these models considers a modification of Terzaghi Consolidation Theory developed by Schiffman and Gibson (27) to account for continuous variation in c_v and m_v throughout the thickness of the compressible stratum. The model developed by Schiffman and Gibson, designated Case 2k, which is applicable to this problem allows the coefficient of consolidation to increase as the volume compressibility decreases with depth in the stratum. Also, in this case, the permeability of the stratum remains constant with depth. The variation in physical soil properties with depth for this model is shown on Figure 18.

The relation between settlement and time for this model can be expressed in a form analogous to Equation 1, specifically:

$$t_{2k} = \frac{T_{2k}h^2}{c_{vo}} \quad \text{----------------------------} \quad (6)$$

The time factor, T_{2k}, for this method is shown along with the unmodified Terzaghi factor for constant initial excess pore pressure in Figure 19. The

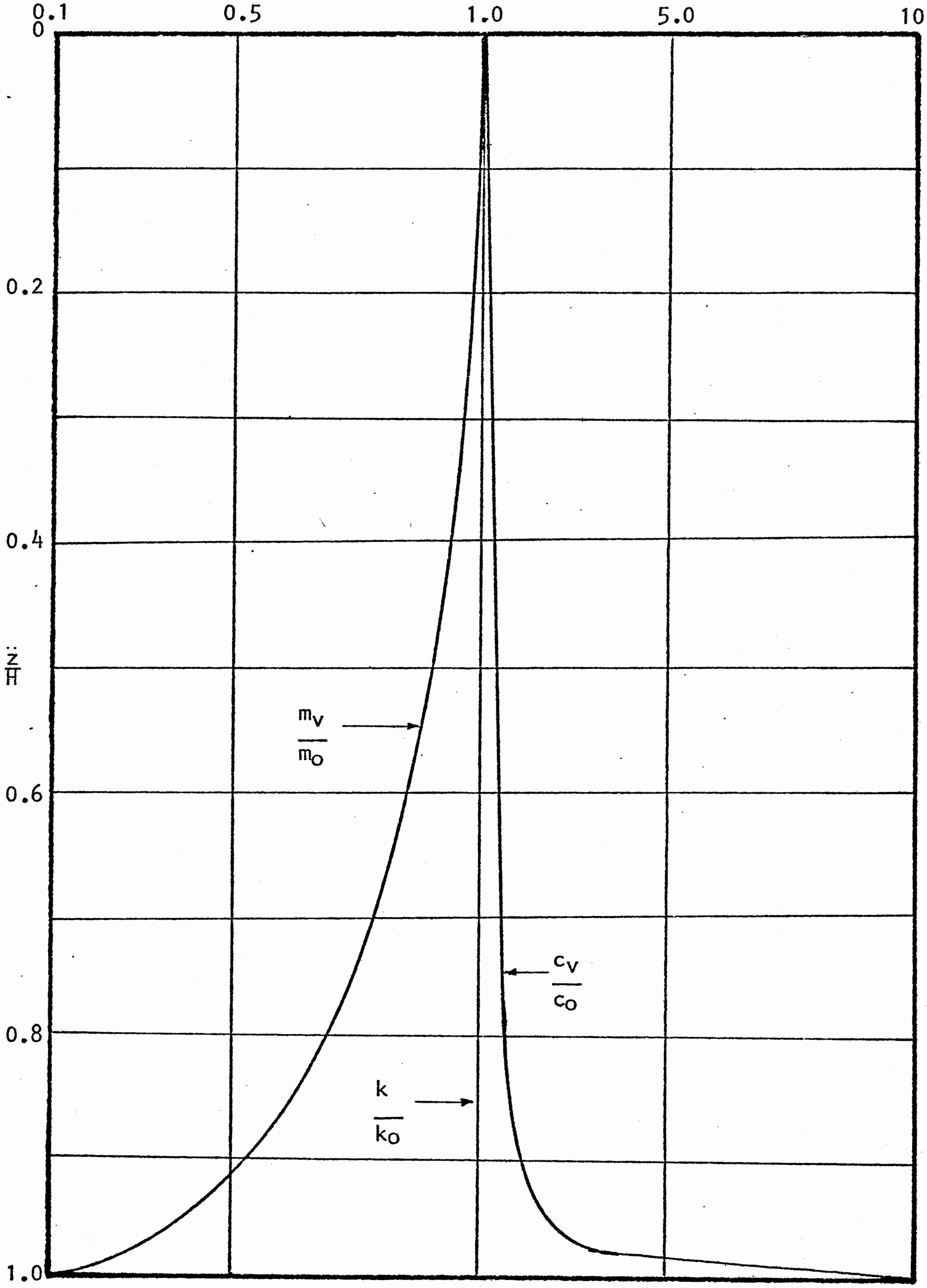


Figure 18. Variation In Soil Properties For Case: 2k

quantity c_{vo} is the coefficient of consolidation at the top of the compressible stratum.

If it is assumed that c_v, m_v, and k_v were essentially constant throughout the thinner compressible stratum at the site of the test embankment (which from a review of laboratory results at the test embankment, appears to be a valid assumption) the characteristic drainage parameter of h' = 1.02 Feet, may be used in predicting preload settlement with this model. From Figure 19, $T_{2k50} = 0.045$ and $T_{2k90} = 0.20$. The final effective stress (due to the preload) at the top of the compressible stratum at the Building 8 location would be approximately 1.2 tons-per-square foot, and from Figure 10, the corresponding value of c_{vo} is 0.011 Feet^2/Day. Then with this model, $t_{50} = 4.3$ days and $t_{90} = 18.9$ days.

Another method of modeling this problem is by the use of unmodified Terzaghi theory applied to the compressible stratum which has been subdivided into sublayers, each 2h' thick and separated by an infinitely thin drainage layer. Each sublayer could then be assigned an independent value of c_v, while m_v and k_v remained constant. This model was applied to the problem of predicting Building 8 settlements for two different variations of c_v, namely: (1) a step-wise, linear increase in c_v with depth, and (12) a step-wise, nonlinear variation in c_v as depicted in Figure 18. Again from Figure 10, a value of c_v = 0.011 Feet ^{2}/Day corresponding to 1.2 tons-per-square foot at the top of the stratum and c_v = 0.036 Feet^2/Day corresponding to 1.45 tons-per-square foot for the bottom of the stratum, established the limits for the variation of c_v, for both cases. From this model the resulting predictions were: t_{50} = 8.7 and 17 days, and t_{90} = 45 and 73 days, for cases 1 and 2, respectively.

The time required to achieve 50 percent and 90 percent of primary consolidation at Building 8, by each of these models is plotted on Figure 20.

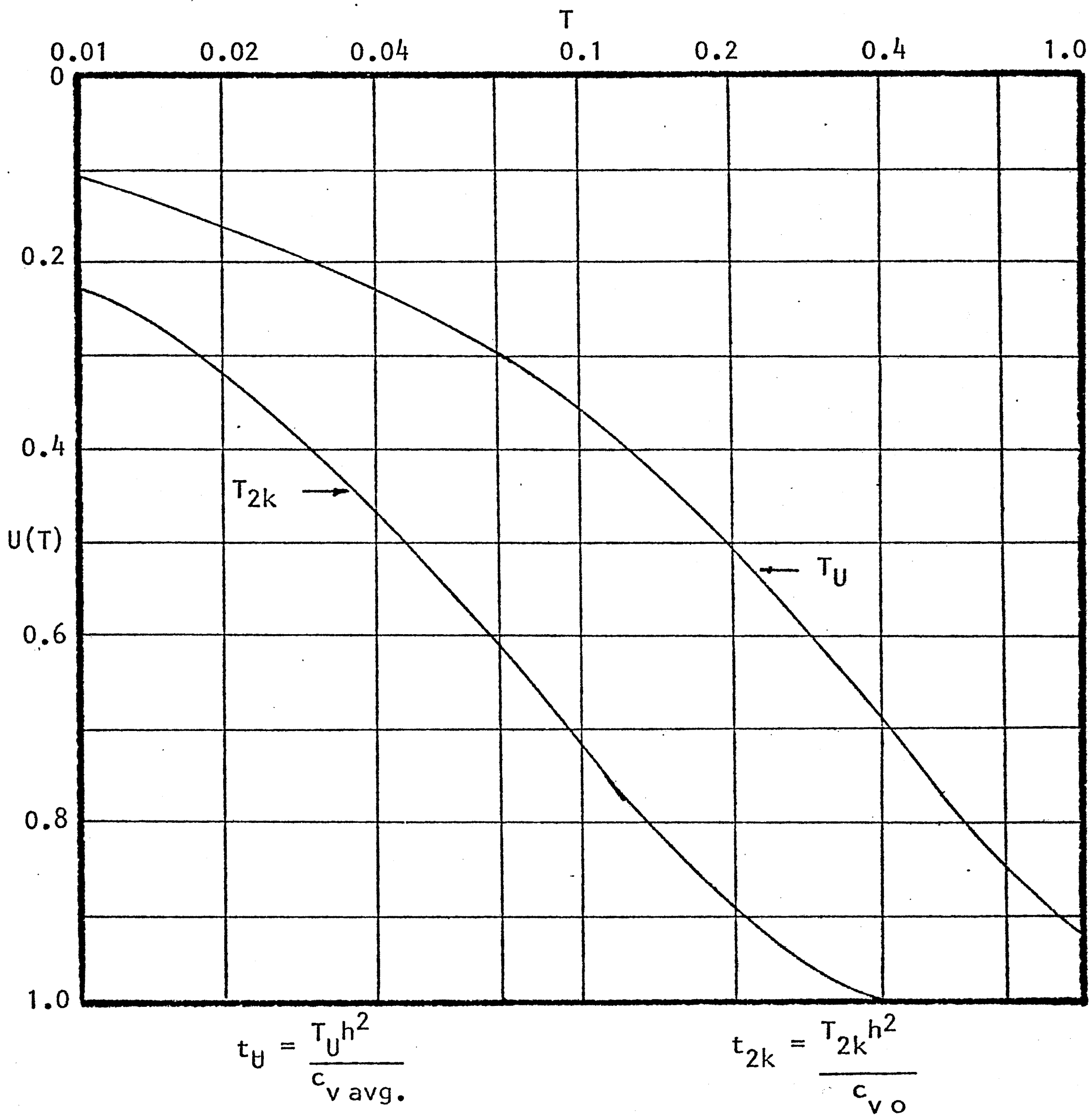
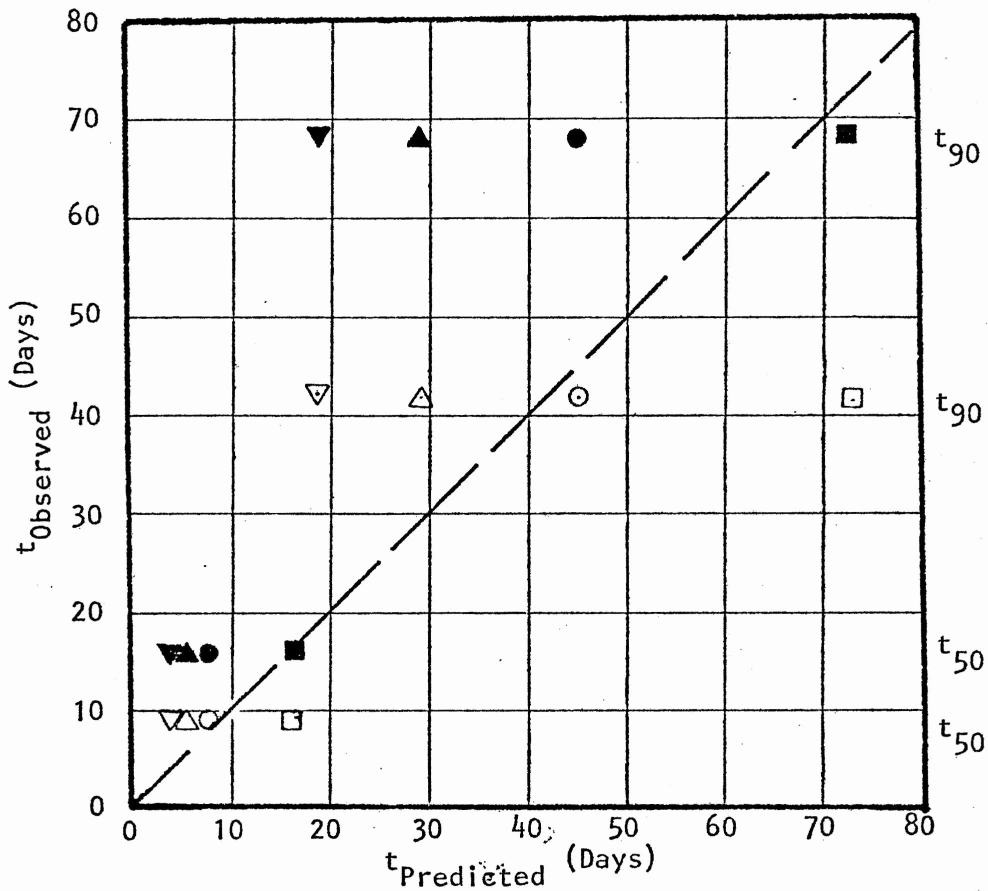


Figure 19. Time Factors For Case: 2k

These plots represent the extremes of correlation for this study, the best results being obtained from the northwest settlement plate and generally the poorest resulting from the southwest plate.

Of these analytical models the one incorporating Shiffman's and Gibson's modification of the Terzaghi theory yielded the poorest predictions. This is probably due to the assumption of the continuity inherent in the modified theory which is violated by the physical conditions at this site. Refering again to Figure 20, it can be seen that the unmodified Terzaghi theory with a linear variation of c_v gave the best prediction of settlement rate according to data measured at the northwest settlement plate and unmodified Terzaghi theory with a nonlinear variation of c_v gave superior predictions at the southwest plate.

It is uncertain whether the difference in "superior" prediction models for the two plates were a result of differences in the root pattern, properties of the clay matrix, or both. Both models, however, exhibited errors which are probably smaller than those produced by inaccuracies resulting from our inability to measure, precisely, the necessary input parameters. Figure 20 does indicate, however, that the unmodified Terzaghi theory considering some type of increase in c_v with depth did improve the rate predictions when compared to the simplist prediction model in which c_v, m_v, and k_v were considered constant throughout the stratum.



▽ Modified Terzaghi

△ Unmodified Terzaghi, c_v, m_v, & k_v constant

○ Unmodified Terzaghi, linear variation of c_v

□ Unmodified Terzaghi, c_v varies as in Figure 18.

Solid Symbols - SW Plate
Open Symbols - NW Plate

Figure 20. Comparison Of Predicted And Measured Settlement

CHAPTER VI

MAIN PLANT SUBSURFACE PREPARATION AND SUBSTRUCTURE CONSTRUCTION

Dredging and Hydraulic Backfilling

As discussed in the chapter on design, three alternative foundation schemes appeared feasible for the main plant foundation, i.e., total dependence on deep foundation elements, conventional excavation, backfilling, and compaction with a minimum of deep foundations, or dredging, hydraulic backfilling, and vibratory compaction with minimized deep foundations. Plans and a bid package considering these three alternates were advertised with a sucessful response. Although exact bid information is confidential for this project, a savings of from three and one-half to seven million dollars was realized by utilizing the dredging/hydraulic backfilling alternate. The most costly alternate by three to four million dollars was that scheme which utilized conventional land-operated excavation and backfilling techniques.

After awarding the contract for the dredging/hydraulic backfilling alternate, draglines operating on mats immediately began to construct levees around the designated spoil areas (Figure 13). The method of levee construction consisted of drawing the highly organic surface veneer and underlying root-penetrated clay up to form the levees. Some concern was voiced about using these on-site materials to construct the levees due to the relatively high permeability inherent in the included roots. Subsequent falling head permeability tests conducted with two and one-half inch diameter "undisturbed" samples taken from the levees indicated the "micro-permeability" of the levee material to be less than 10^{-8} centimeters per second (6). To further invalidate the concern, no "leakage" problems were encountered when the levees were pressed into service to impound the dredged spoil.

The impermeability of the levees which were constructed from materials, which in an undisturbed (natural) state demonstrated a relatively high "macro-permeability" is attributed to several factors:

1. Remolding of the clay matrix probably resulted in a significant decrease in its horizontal permeability characteristics,
2. Complete remolding of the mass probably served to disrupt inter-root communication, and,
3. The remolding process may have created "smear zones' around the roots impeding their ability to receive and communicate water.

Actual dredging of the main plant location began in March, 1975. In order to avoid an eighteen inch pipeline which crossed the river and came ashore immediately south of the main plant location (most direct path of access), a temporary easement was obtained on the adjacent (west) property and dredging of the access canal was begun (See Figure 14). The dredging was accomplished with a twenty-seven inch diameter discharge, self-contained dredge. The dredge boat, the FRITZ JAHNCKE, was approximately 280 feet long and operated an eighty-five foot "stringer".

The soft clay stratum and overlying hydraulic spoil was completely removed from beneath the main plant location resulting in a typical dredging depth of twenty-eight feet. The volume of material dredged from the main plant area and spoiled on the eastern (undeveloped) portion of the property was 825,000 cubic yards. After completing the main plant dredging an additional 122,000 cubic yards was dredged from the barge dock area.

As a result of the dredging, stable side-slopes of two horizontal to one vertical were common. This was considered evidence of the reinforcing effect of the root-mass inherent in the soft clay stratum. No attempt was made to quantitify this phenomenon for subsequent design purposes.

On completion of dredging operations, the dredge boat moved downstream to Star Bayou, Figure 21, to begin the hydraulic backfilling of the excavation. As previously stated, a valved "Y" was installed in the discharge pipe to allow diversion of the dredged backfill into the spoil area should deliterious materials be encountered. The first operation of the dredge in the borrow channel was to strip the layer of organic silt from atop the selected sand stratum. Ultimately some 525,000 cubic yards of silt was stripped and spoiled before backfilling could begin.

After removal of the organic silt, dredging of the sand backfill was very closely controlled. Any deleterious materials encountered in the dredging operations, such as isolated pockets and lenses of silt and clay or organic silt overburden which occasionally slumped onto the working face, was immediately diverted to the spoil area by means of the "Y". Also, high initial discharge velocities were maintained at the head of the backfill to limit the deposition of fines (particles smaller than 74 microns) in the backfill. Figure 22 shows the recommended grading limits for material which can be readily densified by the vibro-densification techniques. Also shown on Figure 22 are the natural grading limits of the sand existing in the borrow channel and the average gradation of the

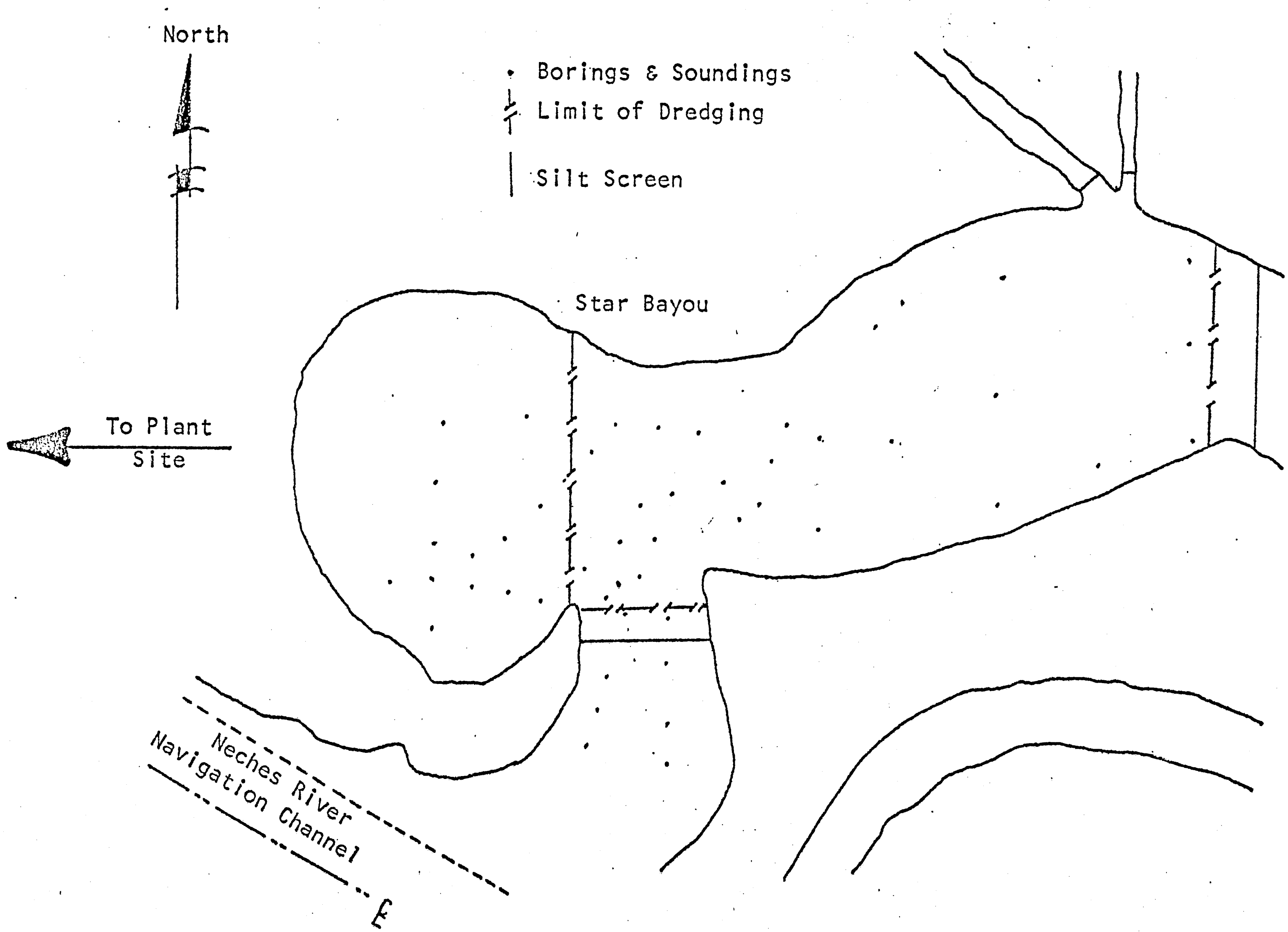


Figure 21. Plan Of Sand Borrow Channel

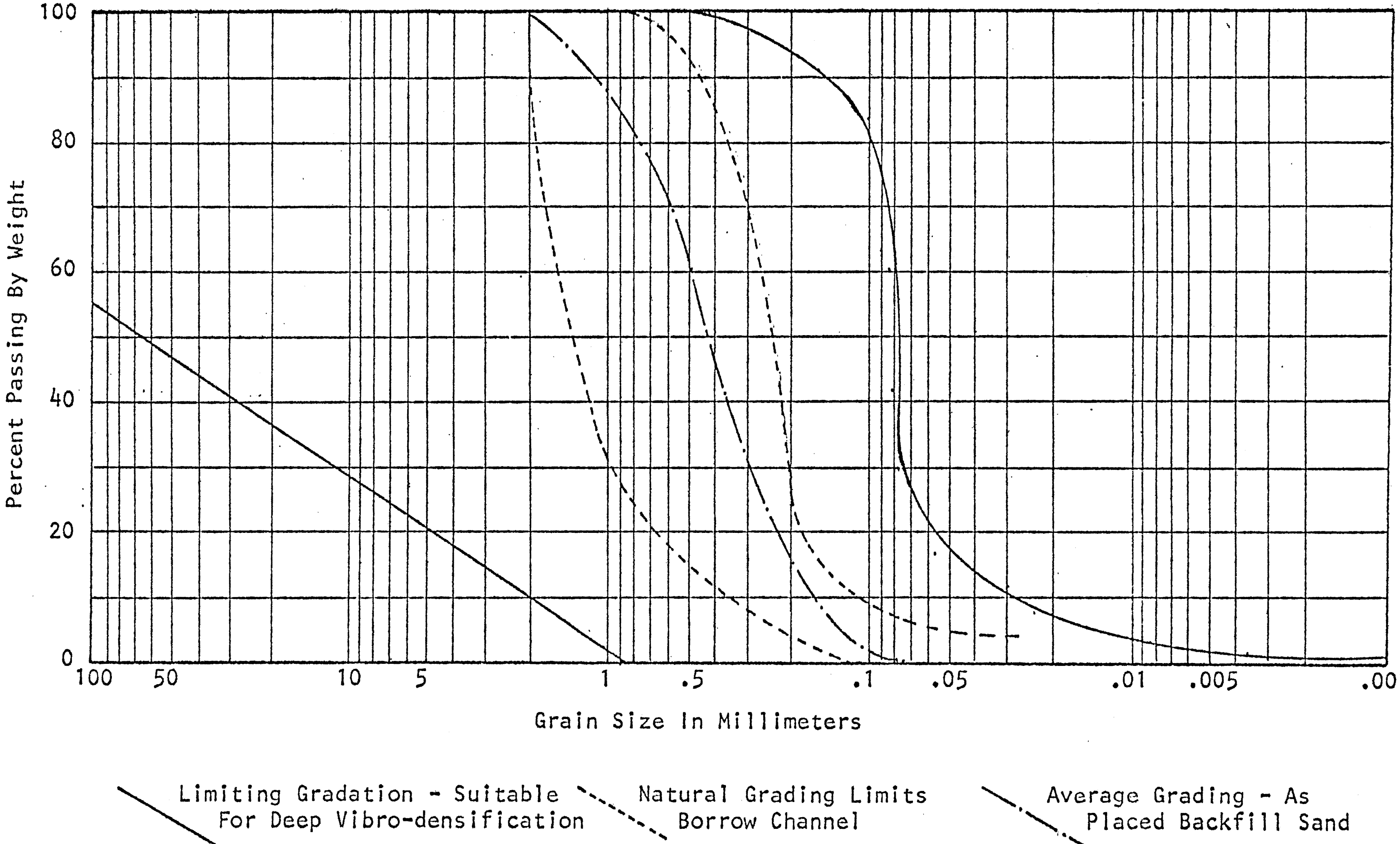


Figure 22. Grading For Vibro-densification

sand as deposited in the excavation. From this figure it can be seen that steps taken to control the gradation of the hydraulic sand backfill were effective. The gradation of the backfill, in-place, was well within the limits subject to vibro-densification.

In addition to backfilling of the main plant area to a required grade of +14 MSL, filling to grade of areas which were not dredged, including the scrap storage area, the substation area, and locations of auxilliary structures, was accomplished hydraulically. The total quantity of hydraulically placed sand fill was 1,425,000 cubic yards.

The last operation preformed by the dredge was to excavate the area in front of the barge dock to elevation -18 MSL. Materials dredged from this area were used to backfill the access canal to the main plant area.

Backfill Densification

In order to make maximum use of the sand backfill on which the plant was to be placed, a relatively high, uniform relative density of the backfill was necessary. This would allow the use of relatively shallow spread footings as specified in the design for structural framing, many interior equipment loads, and billet and product storage. As a result of competetive bid selection, the Vibroflotation Corporation was selected to accomplish densification of the backfill. In order to satisify the foundation requirements for the mill a general fill relative density of sixty-five percent had been specified. In addition to this general density requirement, a minimum relative density of eighty percent was required below all "shallow" foundation elements.

A ten foot triangular pattern was specified for the vibroflot probe insertions in the contract document, however, provisions were also made for additional payment or credit should a different spacing be necessary. In order to establish the in-place density of the sand after backfilling and to measure the effectiveness of the vibroflotation techniques the Standard Penetration Test (ASTM D-1586) was selected.

In order to standardize the results of the STP tests and exclude the effects of overburden pressure, a correction for the overburden pressure developed by Gibbs and Holtz (25):

$$C_N = 0.77 \log \frac{20}{\bar{p}} \quad \text{------------------------------(7)}$$

was applied to the STP test results. In this relationship; C_N is the overburden correction factor to be applied to the STP blow count, and $\bar{p}$ is the effective vertical overburden pressure in tons-per-square foot at the elevation of the penetration test. This relationship is considered valid for values of $\bar{p} > 0.25$ tsf, however, to uniformly evaluate the densification program, the overburden correction was assumed to be valid for any value of $\bar{p}$. Since the sand backfill and underlying natural sand contained a very small percentage of fines, no gradation correction was necessary for STP tests conducted below the water table. In order to further standardize these measurements a moist unit weight of 125 pounds per cubic foot was assumed for the sand above the water table. The water table was taken to occur at a depth of five feet below the backfill surface.

In order to verify the effectiveness of the vibroflotation technique and to select the proper spacing for the probe insertions, three test panels were located on the surface of the backfill. Standard Penetration Tests were conducted a five foot intervals to a depth of fifty feet (well into the lower natural sand) at three locations in each test panel. The sand below each test panel was then densified by vibroflotation, however, insertion spacings were varied for the different panels at ten, eleven, and twelve feet.

In order that the relative densities could be readily compared, a graph showing effective vertical stress vs. the uncorrected STP result was prepared (8). Superimposed on the graph were lines representing constant relative densities as corrected for overburden pressure. See Figure 23.

Several interesting facts were observed from the tests run at these test panels. As a result of the tests conducted before densification at the test panels, it was learned that the hydraulic backfill was deposited at an average relative density of 54 percent, however, individual tests indicated a range for D_r from ten to 105 percent. Even though the average relative density of the backfill was apparently very near the design general fill density, the range through which the test values varied indicated the need for the densification program. Of practical interest, the insertion spacing (through the small range investigated) had very little effect on the relative densities attained. Nearly all relative densities measured below the test panels were greater than eighty-five percent with many ranging well above 100 percent. Typical results of STP tests conducted before and after The numbers in parenthesis indicate the depth in feet at which the STP was conducted.

As a result of the tests conducted with the test panels, a twelve foot spacing was selected for the probe insertions. This increased spacing ultimately resulted in a savings of over $150,000 as credit against the original bid price based on ten foot spacings.

After vibroflotation of the sand backfill was complete, an array of eleven-four inch diameter ejection wells was installed around the perimeter of the backfill. The phreatic surface in the backfill was drawn down to approximately elevation - 6 MSL to allow dry excavation for footings, utility tunnels, etc.

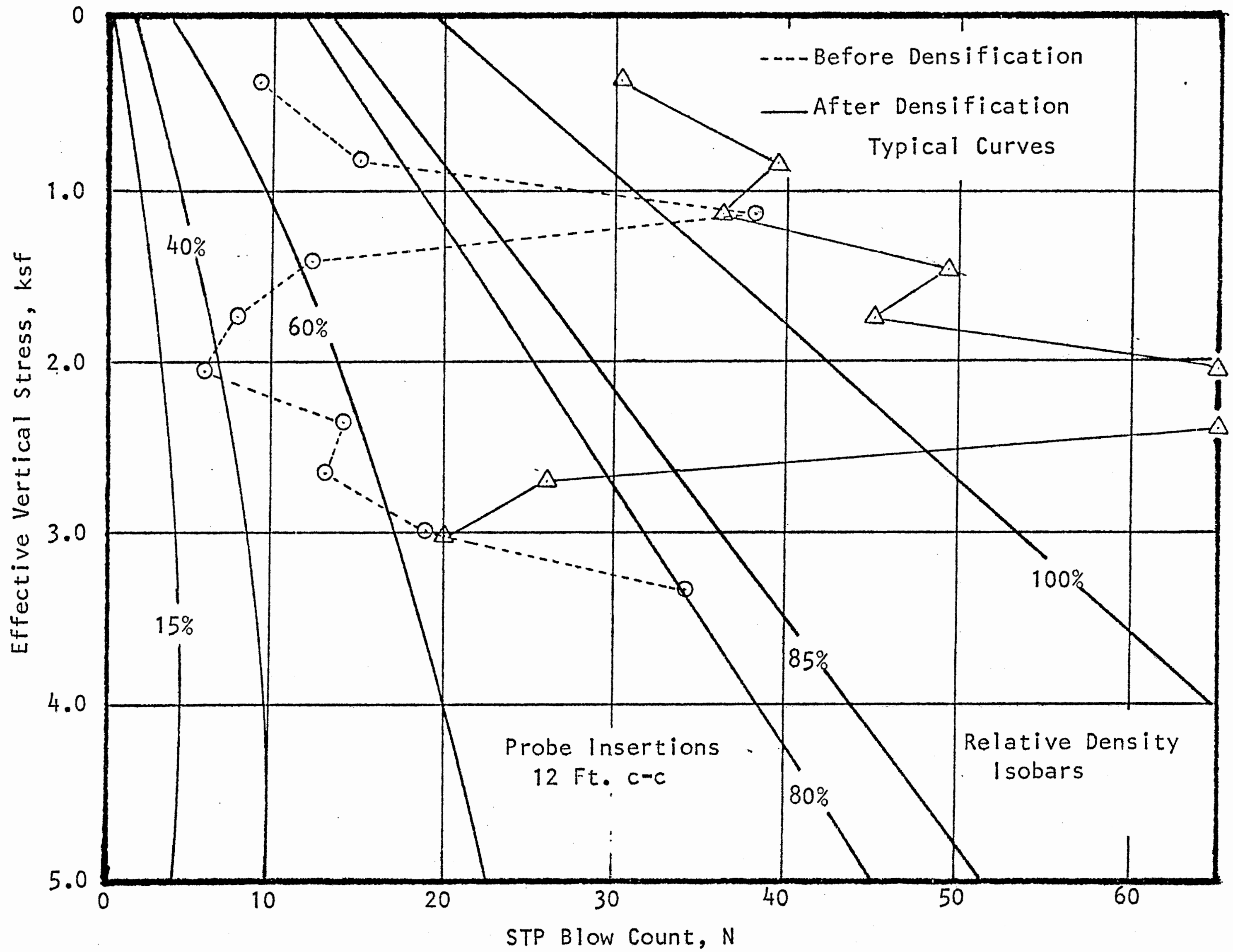


Figure 23. Comparison Of Density - Before And After Vibroflotation

There was some concern that this phreatic drawdown might adversely affect Building 8 by causing settlement of the compressible stratum to again occur. This concern, however, proved to be unwarranted, since no significant settlement of Building 8 was observed while the dewatering system was in operation. The most plausible explanation for the lack of accelerated settlement is that the surcharge fill preconsolidated this stratum to the extent that the increased effective stress due to the drawdown remained in the recompression range.

The effectiveness of the densification program was visually apparent when footing excavations in the densified sand were observed to remain open with near vertical slopes. At this time the settlement of any elements placed in the densified sand backfill, which could be attributed to elastic strains in the sand, have been well within design limitations.

Foundation Elements

Foundation elements for the structure and included equipment consisted of large spread footings and thin-shelled, mandrel-driven, cast-in-place concrete piling.

Spread footings were located within the limits of the sand backfill at a minimum depth of eight feet below the finished grade. The footings were dimensioned according to Equation 5., however, a maximum allowable bearing capacity of 8,000 pounds-per-square foot was adhered to. No bearing failures or excessive total or differential settlements (settlements greater than design limits) have been observed to date.

Cast-in-place concrete piles were designed by others using wave theory. Piles installed in the hydraulic backfill were driven to a penetration of sixty feet below "cutoff" and were assigned a design capacity of sixty tons (9). Subsequent pile load tests verified this design capacity with an apparent factor of safety of greater than two and one-half. Piles installed for auxillary structures in non-dredged but hydraulically backfilled areas were driven to similar penetration, but were assigned a design capacity of twenty tons to account for large anticipated downdrag forces resulting from consolidation of the soft, root-penetrated clay. No verification tests were conducted with these piling, however, no distress which could be attributed to foundation settlement has been observed for any of the auxillary structures.

CHAPTER VII

EPILOGUE AND CONCLUSIONS

Epilogue

All major site development work for the Georgetown Texas Steel Plant was completed during the last quarter of 1975. In the ensuing year construction of the plant superstructure, processing equipment and auxillary structures was completed. A "calibration melt" with the first of the two 100 ton furnaces was conducted on September 3, 1976, and the mill began full production in the first quarter of 1977. The duration of this project, from site option to full production, was just slightly over three years.

During the first six months of production the mill broke all records set by similar mills within the Korf Corporation, with production during this period equal to the rated annual production for the plant. Since the first production melt no major problems have occured which could be attributed to the design assumptions and concepts used in site development or foundation design. The minor problems which have occured can be traced to design changes which did not take into account the particular problems associated with the stratigraphy at this site.

Conclusions

It is hoped that the development of a steel mill at this site demonstrates the feasibility of economical marginal land development by the marriage of multiple design concepts and construction techniques. It is the author's belief that much marginal land suitable for industrial development has been and is presently being overlooked because consideration of "conventional" site and subsurface development schemes appear uneconomical. It is, however, appropriate to caution that the undertaking of a development scheme for marginal lands, such as the one reported herein, involves forseen and unforeseen risks in both design concepts and construction techniques. This fact must be clearly understood by all parties and construction funding must provide contingency monies to account for these risks.

Specific conclusions resulting form this study relate specifically to the accelerated settlement allowing the surcharged preparation of the Building 8 location and the hydraulic excavation and backfilling of the main plant location. Several conclusions can be drawn from the test embankment-surcharge fill portion of the site development:

1. The presence of permeable tree roots in a compressible clay stratum significantly affects the time-settlement characteristics of the stratum.
2. On this site it was appropriate to characterize the time-settlement of the root drains by a single parameter, the characteristic drainage distance, h', that replaces the drainage distance term in the various equations for settlement rate predictions.
3. For each of the rate prediction methods evaluated to improve the settlement rate predictability, it was possible to evaluate the characteristic drainage distance by surcharging a small area to a small height. This procedure, however, requires that the vegetative environment be the same at the test embankment site and at the point(s) of application of the constructed surcharge(s). Care must be taken to account for differences in

the coefficient of consolidation resulting from dissimilar magnitudes of effective stress between the prototype and surcharge fills. Other variables which must be taken into account are areal variations in the properties of the clay matrix which affect compressibility.

4. At this site, rate predition methods based on unmodified Terzaghi theory, in which the compressible stratum is modeled as a system of sub-layers, each 2h' thick and separated by infinitely thin drainage layers, provided the best predictions. At this site, the measured c_v varied with depth and was similarily varied in the rate prediction model.

5. Similar studies at other locations with different inherent vegetative environments are desirable.

With regard to the hydraulic excavation and backfilling of the main plant area, no new principles or techniques were employed. However, the planning and control during this phase of the site preparation resulted in a very economical site development. It is thought that the specific conditions for the economical use of hydraulic replacement, as demonstrated here, are:

1. A site to be modified in this manner should be river-dredge accessable. It is believed that had a landlocked, portable dredge been necessary for use at this site, the mass hydraulic replacement could not have been economically undertaken.

2. An area of sufficient capacity to contain the dredged spoil must be within pumping distance of the site. This area must have subsurface and peripheral characteristics suitable for use as a spoil disposal area.

3. If hydraulic backfilling is anticipated, a source of suitable sand must be available within pumping distance of the site.
4. Precise control of the backfilling operation is imperative if techniques such as vibro-densification are to be used to modify the backfill.
5. Concepts such as this may be valid for preparation of sites where "good" subsurface conditions exist if heavy areal ground loading or very limited settlement tolerances are stipulated.

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VITA 2

Buford Kent Merritt

Candidate for the Degree of

Master of Science

Thesis: INDUSTRIAL DEVELOPMENT OF MARGINAL LAND: A CASE STUDY

Major Field: Civil Engineering

Biographical:

Personal Data: Born in Fort Bragg, North Carolina, November 29, 1943, the son of Mr. and Mrs. B.A. Merritt.

Education: Graduated from Sulphur HIgh School, Sulphur, Oklahoma, in May, 1961; enrolled in Bachelor of Science degree program at Oklahoma State University, 1961; received Bachelor of Science in Civil Engineering degree from the University of Oklahoma in 1967; took graduate course work in Civil Engineering from the University of Kansas in 1969 and 1971, from the University of Missouri, Kansas City Campus in 1970; enrolled in Master's program at Oklahoma State University in 1972; took graduate course work in Civil Engineering from Lamar University, 1976; completed requirements for the Master of Science degree at Oklahoma State University in July, 1978.

Professional Experience: Rotational Training Engineer, State Highway Commission of Kansas, 1967-68; Manager, Soil Mechanics Laboratory, State Highway Commission of Kansas, 1968-73; Staff Geotechnical Engineer, Southwestern Laboratories, 1973-74; Geotechnical Project Manager, Southwestern Laboratories, 1974-77; Project Manager, Woodward-Clyde Consultants, 1978-.