# Investigation of the Failure of a Hydraulically Placed Embankment

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**A study was performed to determine the causes of differential subsidence of a hydraulically constructed sand embankment in a fresh water swamp. Among other test methods, an electric cone penetration test (ECPT) was used to determine the cause(s) for embankment, and thus pavement, distress. Because very little historical design and construction information was available to the authors for analysis, the approach used was of a forensic nature rather than that of a well-defined research study. The electronic cone penetrometer, in combination with other tests, proved to be an invaluable tool in determining the condition of the sand embankment. Consolidation test results and embankment and subgrade profiles obtained by ECPT and borings eliminated the possibility of continuing settlement of subgrade soils. The problem is within the hydraulically placed sand fill. It was determined that the sand embankment when hydraulically placed was not sufficiently densified and that in later years the top 6 ft of the embankment, which is above the water table, experienced various degrees of densification under traffic loadings. Remedial options were studied and recommendations were made for field evaluations of these methods.**

A 4.3-mile section of route I-10 between Baton Rouge and New Orleans, Louisiana, has been experiencing continuing, excessive pavement distress in the form of differential subsidences of 2–8 in. along its length.

This section of I-10 crosses a fresh water swamp. It was constructed during the period September 1967–July 1970 by removing approximately 15–20 ft of soft, alluvial organic clays overlying competent Pleistocene deposits and by replacing the soft organic clays with hydraulically placed sands pumped from sand bars of the Mississippi River. The sand for the fill was pumped about 15 mi from a dredge located on the river. The specifications for the sand required that it contain no more than 3 percent organic material, with 75–100 percent passing the No. 40 screen and 0–10 percent passing the No. 200 screen. The solid-to-water ratio of the pumped slurry was reported to be 14–18 percent by volume (*1*).

Following the pumping of the sand, a sand surcharge of approximately 200 psf was applied for a period of 3 yr to eliminate primary consolidation of any remaining soft layers and to minimize secondary settlement. A sand embankment about 15–25 ft thick was thus formed to provide the foundation for this section of the highway.

The highway profile consists of two roadways (Figure 1), each having a two-lane pavement 24 ft wide, with paved shoulders on the outside and inside, 10 ft and 6 ft wide respectively. The highway has a 52-ft depressed median between the roadways (64 ft between travel lanes).

The pavement, constructed in accordance with the AASHTO rigid pavement design method, consists of 12 in. of compacted shell placed over the sand embankment as subbase, with a 4-in.-thick black base and an 8-in.-thick continuously reinforced concrete pavement (CRCP). During recent years, the CRCP surface has exhibited signs of excessive vertical displacement.

In April 1985 the Louisiana Department of Transportation and Development (LDOTD) and Louisiana State University (LSU) initiated a study to investigate the causes and possible remedies and rehabilitation schemes to save the embankment.

This paper presents the method of investigation used to determine the causes for differential subsidence and the conclusions derived from the study.

## LITERATURE

In spite of the fact that hydraulic fills have been used for construction of dams, roadways, and structural foundations, there is little useful information on design, construction, or postconstruction procedures availablein the archival journals (*2*).

Cases cited in the literature indicate that when clean sand is used for hydraulic fill, relative densities obtained without mechanical compaction vary from 40 to 80 percent (*2–4*, *5*,p.402). However, relative densities obtained without compaction in excess of 70 percent are rare, which may result in the subsidence of the embankment (*2–4*, *6–7*). This is particularly true when ponding is used (i.e., the pumped sand is deposited under water without allowing the water to drain).

Practitioners disagree concerning the critical nature of the presence of fines in pumped sand (*2*, *4*). There is also disagreement on the use of relative densities for controlling hydraulic fills (*3*). The lack of sensitivity and scatter in the numerical values of relative density measurements render such measurements less desirable for quality control (*8*).

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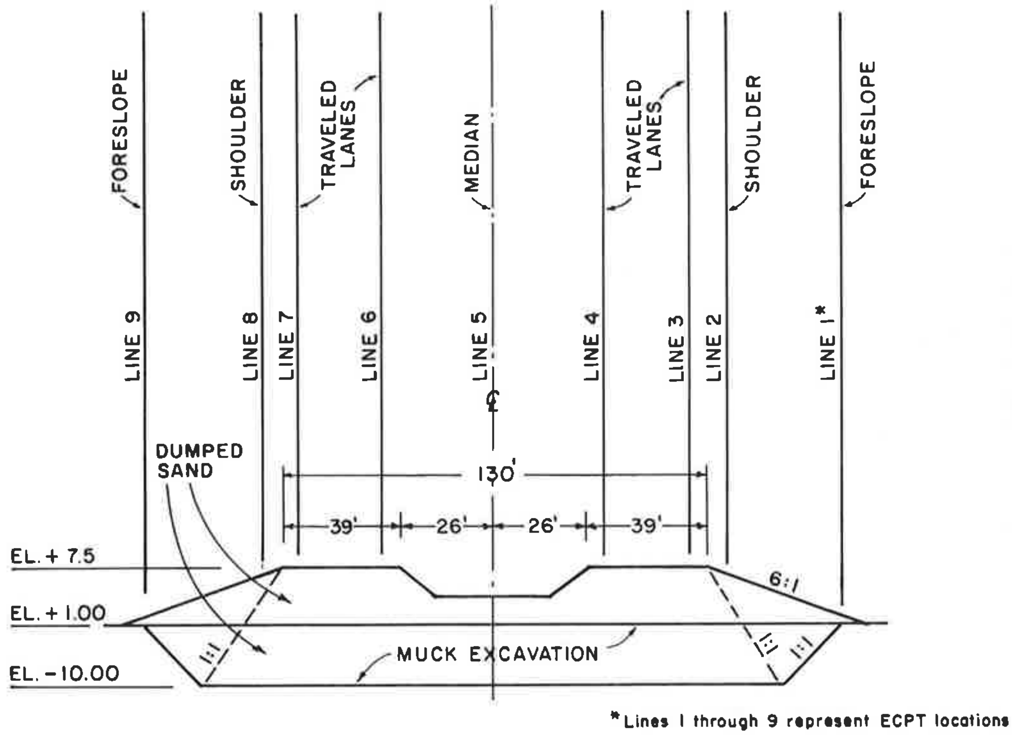


**FIGURE 1 Embankment profile and ECPT locations.**

Reitz et al. (*3*) state that "numerical data reported for this (hydraulic sand fill) should not be based on correlations from Standard Penetration Testing N values. . . . The writers strongly urge that where fill quality can be tested as fill is being placed, the more reliable determinations of relative densities by physical testing be used."

## METHODOLOGY

Upon review of historical testing and construction data on the section of roadway under study, it was determined that the baseline information on subsidence vs. time and original pavement (or natural ground) elevations was either very scant or totally unavailable, as were other data from monitoring instruments that would be essential for such an investigation. The available information was fragmented and incomplete. Virtually no design information was available on the embankment or on the pavement, except as indicated in general terms in this paper. The LDOTD Research Division files were made available to the authors so the historical data could be analyzed. The project engineers of this investigation and the LDOTD Rehabilitation Advisory Group also provided much valuable data and information.

Nevertheless, because of missing links in the data and because of the lack of baseline information, the nature of the methodology used in this investigation was more that of forensic engineering than that of a research project.

The following sets of data were made available and studied in connection with this investigation:

- Subgrade soil borings that were performed by LDOTD in 1985 (Figure 2)
- Standard penetration tests (SPT) (Figure 3)
- Dutch cone penetrometer tests
- Embankment material classification and gradation tests
- Electric cone penetration tests (ECPT)
- Inclinometer observations
- Aerial photographs
- Water table data
- Consolidation tests performed on clay representing soils underneath the embankment

Additionally, interviews concerning construction procedures, maintenance problems, field observations, etc., were conducted with the LDOTD and FHWA representatives.

The authors requested the LDOTD to contract Fugro International, Inc., to perform electric cone penetrometer tests (*9*) at three different sites selected by LDOTD and the investigators. Two of the selected sites were in the distressed portion of the subject project. A third, reference site was selected in a section of I-10 about 1 mi east of the subject embankment. This latter section of the embankment was constructed immediately following the completion of the subject section, also using excavation of the organic clays and hydraulically pumped sand from the Mississippi River. In spite of the fact that the third site investigated has been subjected to the same traffic and static loads, it has experienced relatively insignificant pavement or embankment distress.

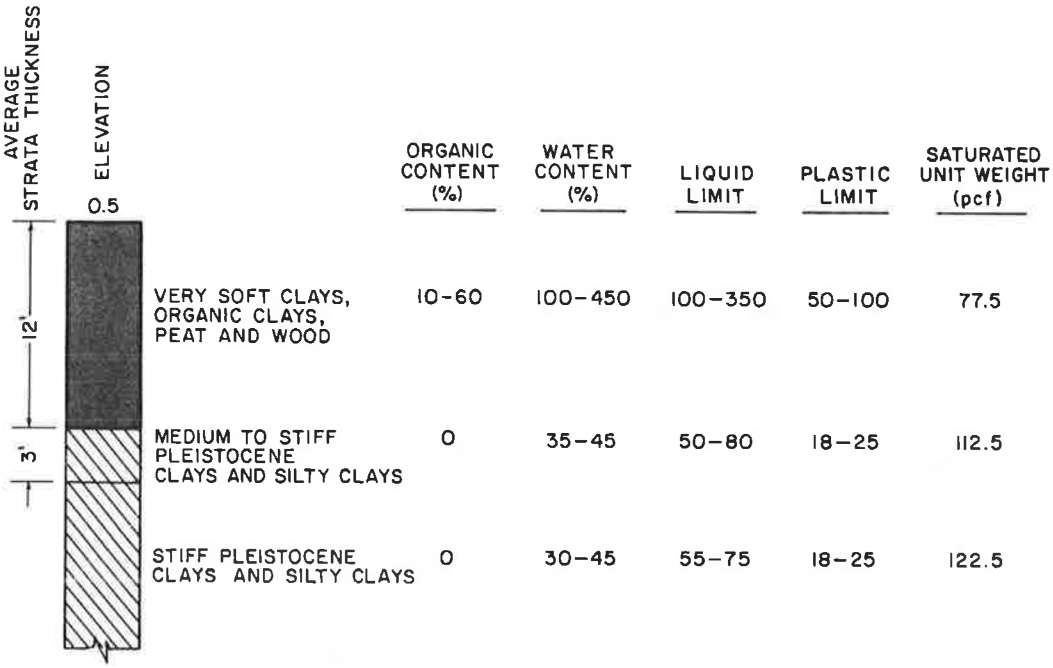


**FIGURE 2 Typical subgrade soil profile.**

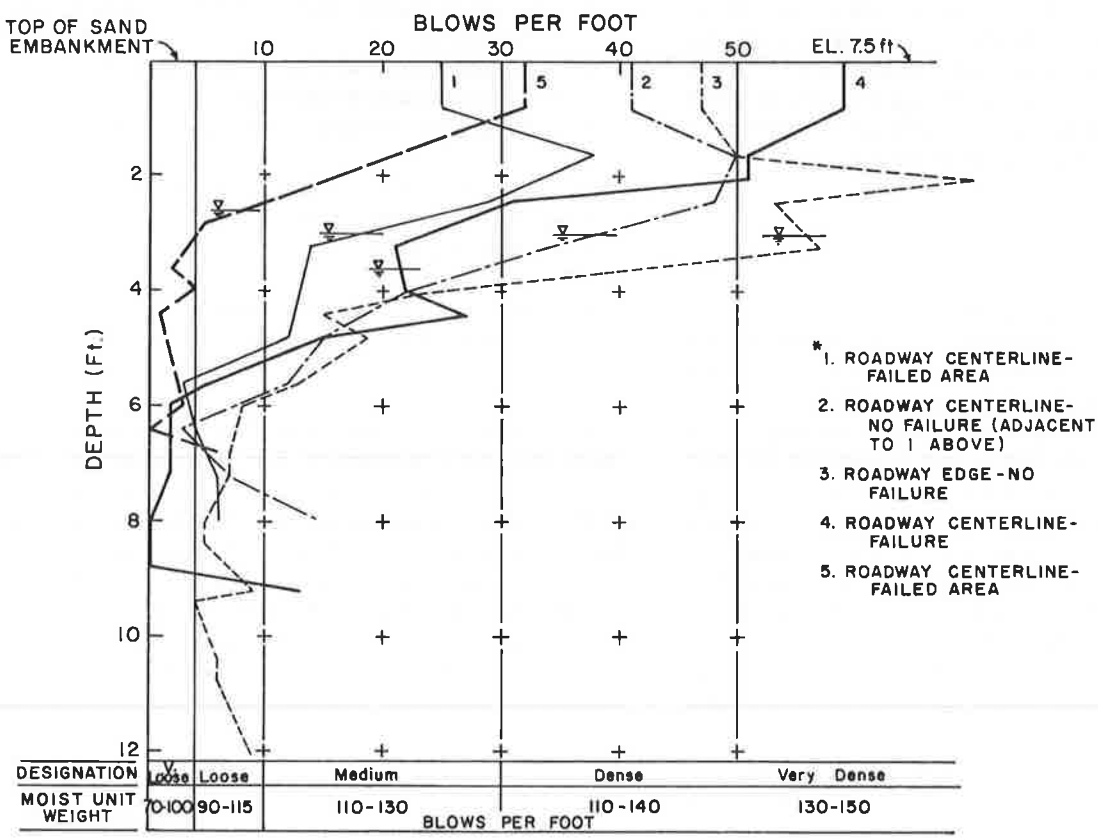


**FIGURE 3 Standard penetration tests.**

Approximately 180 electric cone penetrometer soundings were performed, concentrated about equally on the three selected sites.

The results of all observations, discussions, and tests were correlated and analyzed, along with additional statistical and numerical analysis performed to determine stress distribution, consolidation, and densification patterns. Scanning micrographs of sand samples were also taken to determine anomalies, if any, in the sand particles (Figures 4 and 5). Embankment rehabilitation methods were reviewed and further field evaluations were recommended.

## TEST RESULTS

Soil borings and SPTs performed by LDOTD show that, in general, the profile of the embankment and the underlying material is relatively uniform. The embankment consists of fine silty sand of medium density from 0 ft (datum: the bottom of the pavement structure or the top of the embankment) to a depth of 6 ft, underlain by a loose, fine silty sand to a depth of 9 ft. From 9 to about 18 ft, the embankment was composed of the same silty sand; however, its density was low. The underlying material from 18 to 21 ft is composed of a very-soft-to-stiff silty clay with scattered pockets of soft clays.

The number of blows obtained from SPT tests varies from an average of about 35 in the top 6 ft of sand embankment to 6 blows in the 6- to 9-ft depth and 2–0 blows below 9 ft (Figure 3). Typical sand gradations show that the sand is uniform in size with about 95 percent retained on No. 200 screen and with negligible percentages of clays and colloids. Field exploration identified the location of the water table at about 6 ft below the top of the embankment.

Inclinometers installed through the shoulders and anchored in the stiff Pleistocene clay layer did not show any lateral displacement throughout the approximate 1-yr observation period. During the same time, however, pavement distress continued and extended to new areas.

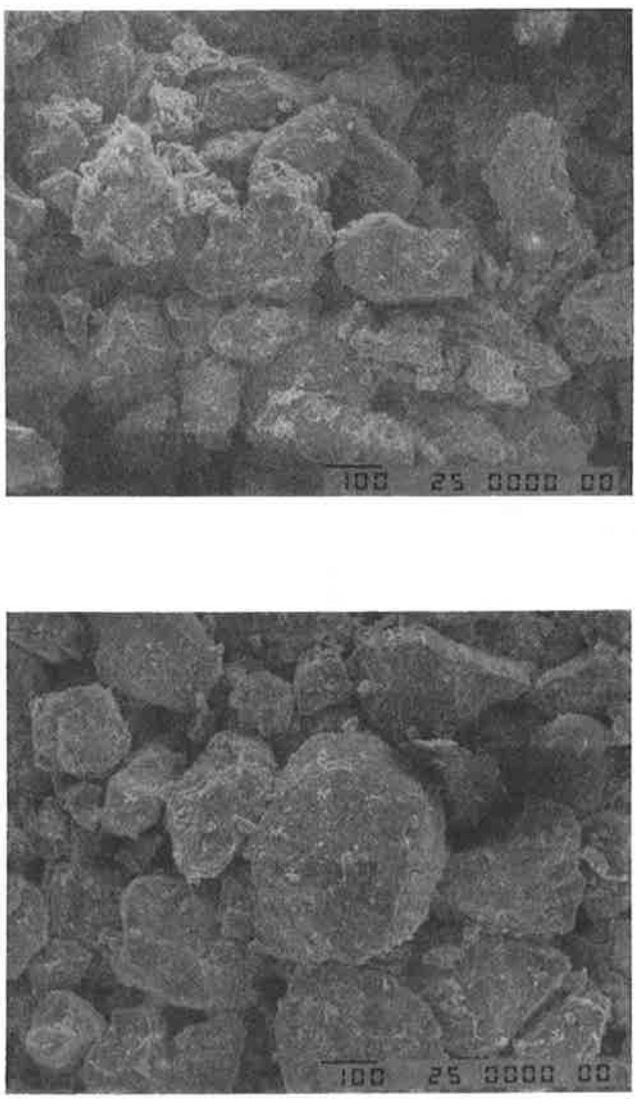

**FIGURE 4 Scanning micrographs of sand (100 magnification; scale shown represents 100 microns).**

## DENSITY TESTS

Density tests performed in the median showed that the dry unit weight of the sand embankment above the water table is in the vicinity of 100 lbs per cubic ft, or about 84 percent of standard proctor density for this sand (Table 1). The water level at the median is shown to be 4.2 ft below the surface, which in turn is about 2 ft below the top of the embankment section under the roadway pavement. These data place the water table in the embankment at about 8 ft below the pavement surface, or 6 ft below the top of the embankment. The moisture content of the sand in the top 3 ft of the median indicates the presence of capillary moisture; consequently, pseudo-cohesion induced by these capillary forces exhibited itself in the form of relatively higher strengths at the surface layers (Figure 5).

The relative densities, based on *N*-values, vary from 80 to 50 percent within the top 6 ft, to from 30 percent to less than 20 percent from 6 ft below the top of the embankment to the bottom of the sand fill. Experience in the area has shown that these polished and relatively rounded sands are unstable at low relative densities because they do not possess interlocking capabilities. Additionally, the micrographs indicate that the size of sand particles used in this embankment vary from one site to another.

## SCANNING MICROSCOPY

Scanning electron micrographs of sand specimens obtained from a representative site (Figure 4) show the rounded and/or polished nature of the sand typical of the

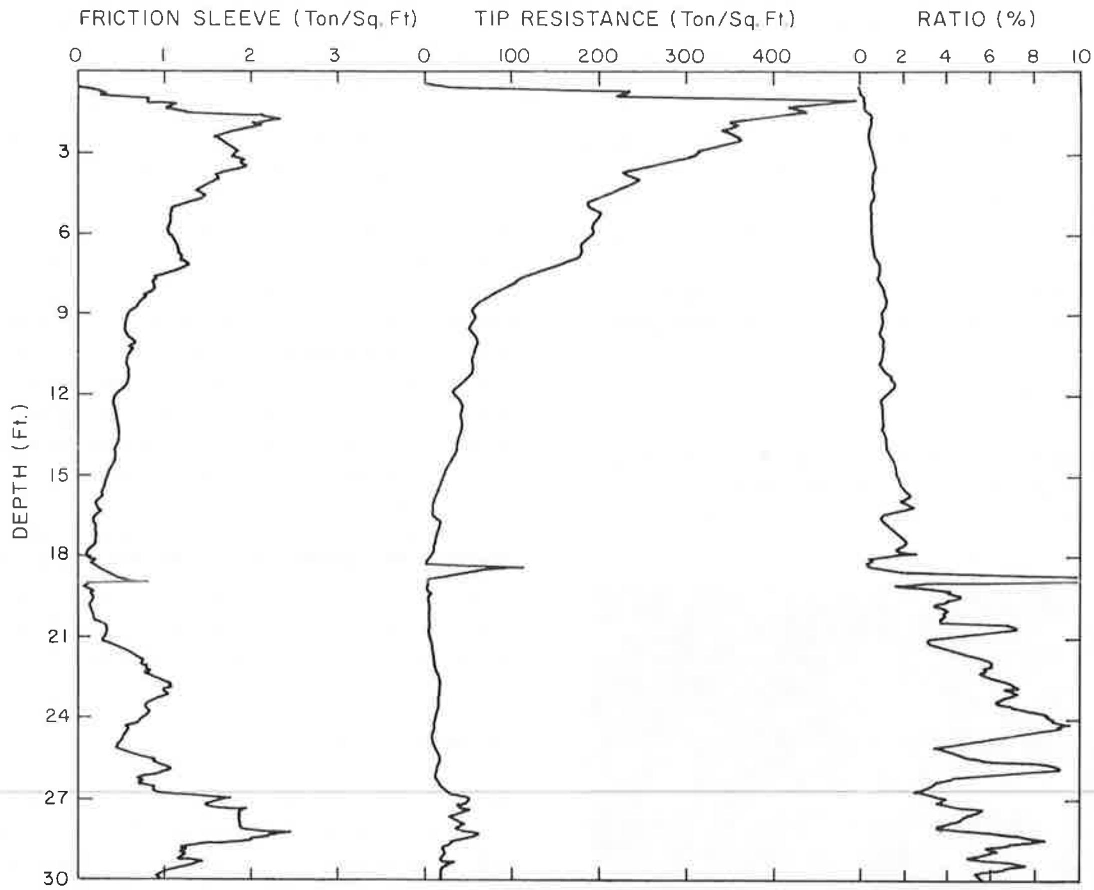


**FIGURE 5 Typical ECPT results for the median.**

TABLE 1 DENSITY OF IN SITU SAND IN MEDIAN

| | Depth 0–1.3′ | Wet Density Clay Material | Dry Density Removed | Moisture |
|---|---|---|---|---|
| | 1.3′–2′ | 105.6 lb/ft$^3$ | | |
| S-1 | 2′–2.7′ | 111.4 | 98.9 lb/ft$^3$ | 6.8% |
| S-2 | 2.7′–3′ | 116.1 | 100.9 | 10.4% |
| S-3 | 3′–3.3′ | 121.6 | 104.0 | 11.6% |
| S-4 | 3.3′–4′ | 128.6 | 101.8 | 19.5% |
| S-5 | 4.2′ | Water Level; No Test | 105.4 | 22.0% |

NOTE: Standard (ASTM D698) density of this sand 118 #/cu.ft.

Mississippi River sands of this area. A note of caution should be incorporated into this latter discussion: Specimens used to perform scanning electron microanalysis are not necessarily representative specimens, because the area or the sample size viewed is very minute. These images should be considered indicators rather than definitely representative specimens.

## ELECTRONIC CONE PENETROMETER (ECPT) TESTS

Electronic cone penetrometer tests were performed on three roadway sections as discussed earlier. Each section covered both roadways from foreslope to foreslope (200 ft) for a length of 400 ft. Approximately 60 soundings, each 30 ft deep, were performed in each of three test sections. Fugro International, Inc., provided the plots of each sounding for analysis (Figures 5 and 6).

The summary of ECPT results fitted with third-order polynomial curves (Figures 7, 8, and 9) shows that in sites 1 and 2, the density of the sand in the embankment, and thus the dimensional stability, varies substantially based on the location across the roadway. Test results also show that the sand in the embankment under the shoulders (identified as lines 2 and 8, Figure 1) is of medium density in the 0- to 3-ft depth, is loose from 3 to 6 ft, and is very loose from 6 to 15 ft.

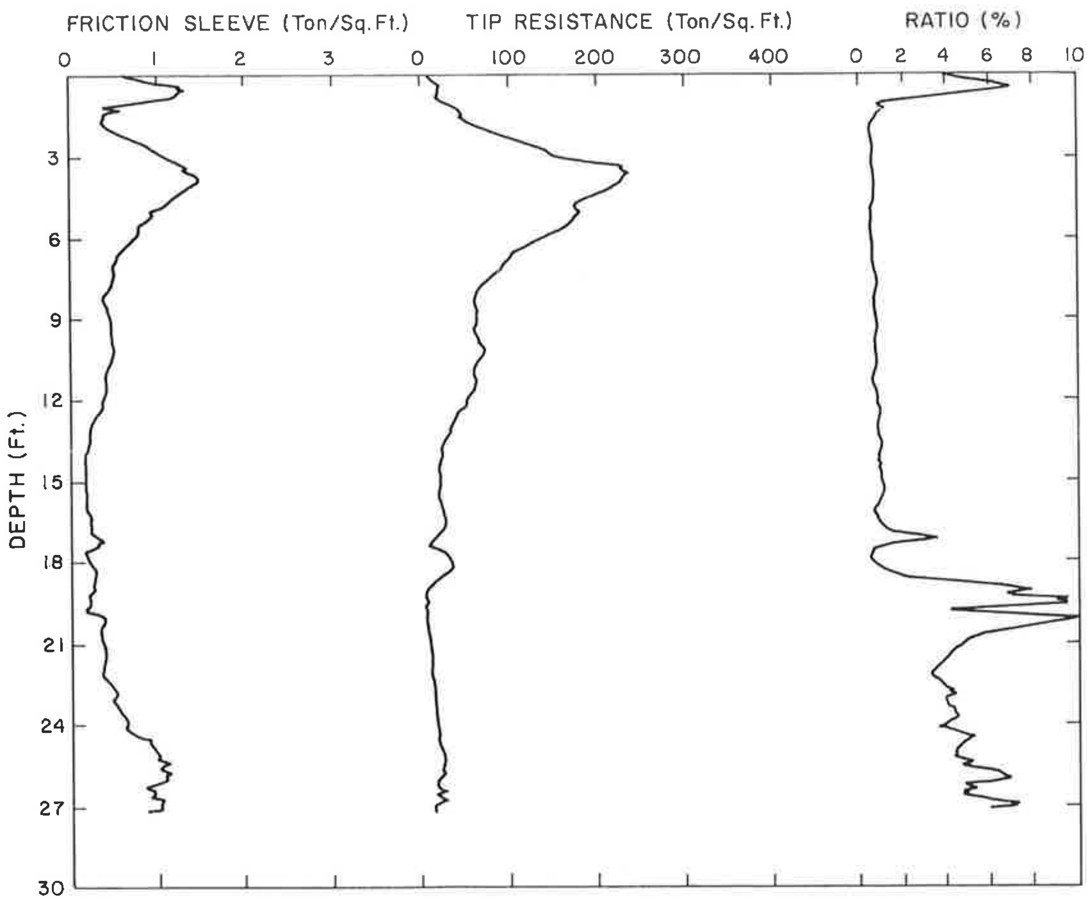


**FIGURE 6 Typical ECPT results for traveled lanes.**

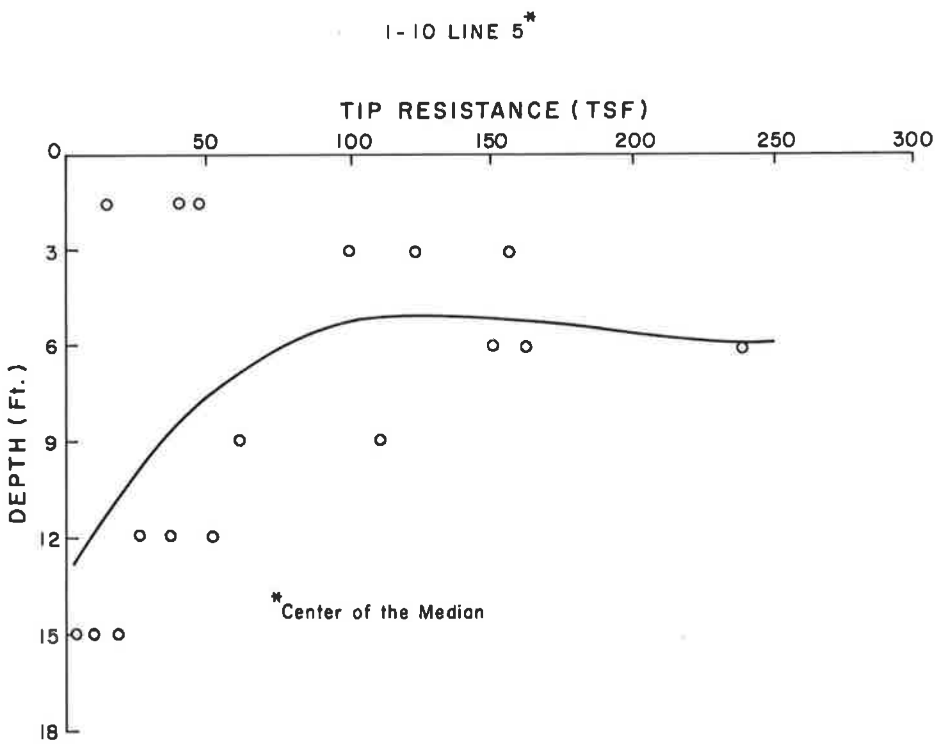


**FIGURE 7 Summary of ECPT results for the median.**

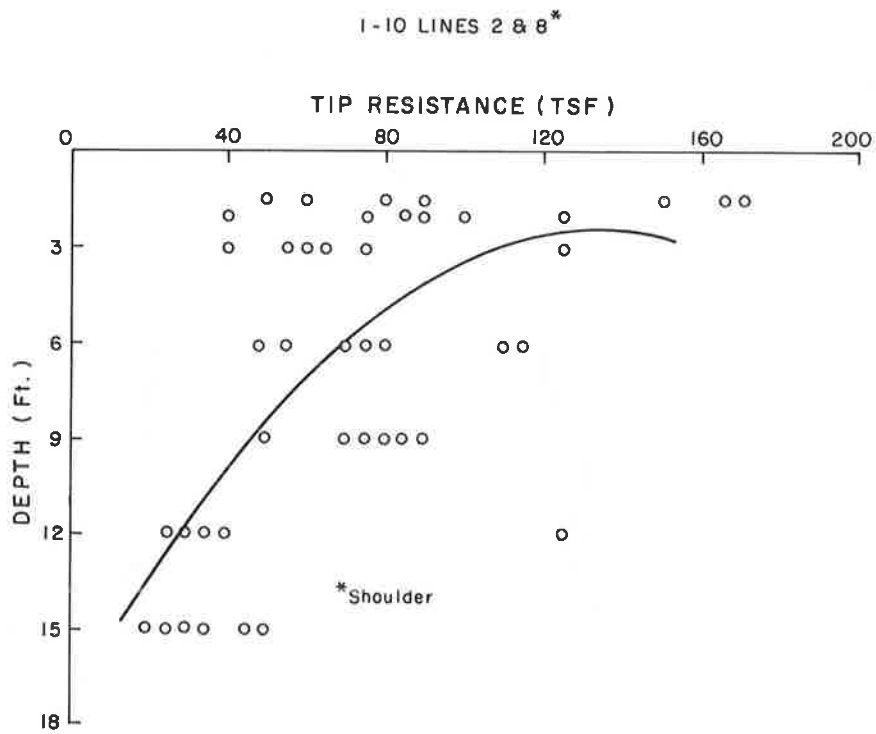


**FIGURE 8 Summary of ECPT results for the shoulder.**

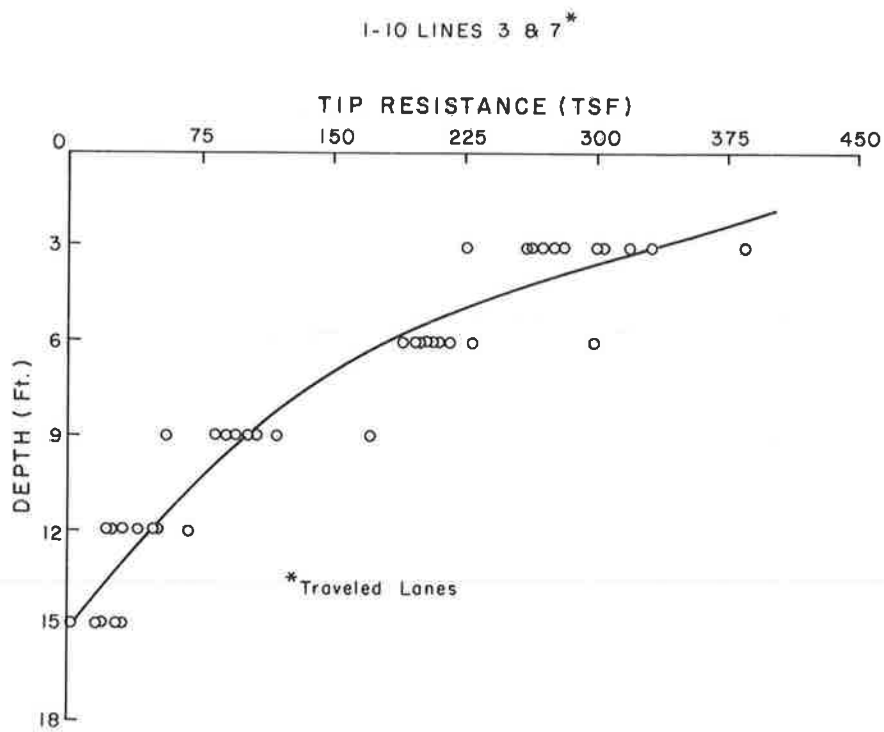


**FIGURE 9 Summary of ECPT results for traveled lanes.**

In contrast to the above, the sand under traveled lanes (lines 3, 4, 6, and 7, Figure 1) appears to be very dense to dense from 0 to 6 ft, after which the density appears to drop drastically to medium to loose. Typical ECPT tip resistance in the top 3 ft of the embankment at the shoulder is about 140 TSF, while under the traveled lanes (lines 3, 4, 6, and 7) it is about 450 TSF or higher.

The tip resistances under traveled lanes are about 320 percent of those at shoulders.

The ECPT results of line 5 (Figure 7), located at the center of the median, indicate the presence of loose-to-medium to dense sands at depths of 0–6 ft.

Typical tip resistances in the top 6 ft of the median vary from 50 TSF to about 150 TSF, similar to those at shoulders (Figure 8). The top 3 ft of the shoulder material appear to be slightly denser than the comparable layers at the centerline of the median.

Third-order polynomial curves developed high coefficients of correlations (95 to 98 percent) for data under the traveled lanes. However, under the centerline of the median and shoulders, because of the scatter of the data, the correlation coefficients were in the order of 50 percent.

The ECPT results of site 3 (section showing no distress) indicated that the sand was densified to a greater depth and more uniformly across the roadway sections. Sand in the embankment at site 3 appeared to be very dense to dense to a depth of about 9 ft, with a medium density to a depth of about 12 ft, and decreasing to a loose sand below that elevation.

Typical tip resistance at the centerline of the median was measured to be about 175 TSF at 0–6 ft. At similar depths under the traveled lanes (line 3) of site 3, typical tip resistance was about 400 TSF, or about 230 percent of that at the median.

In general, the ECPT results show the presence of pockets of soft clay under the sand embankment varying in thickness from 4 to 9 ft at sites 1 and 2, and from 1 to 5 ft at site 3.

## CONSOLIDATION TESTS

Samples of the soft clay representing the layer under the sand embankment were obtained from nonloaded locations near the embankment. These samples were assumed to be representative of original soft clay layers existing under the embankment, since soil classification and elevation of the soils were similar to those under the embankment. The LDOTD Research Division tested the samples.

An analysis of the test results was performed by these investigators. Results showed that under the worst case (assuming a 10-ft-thick layer of soft clay, single drainage, and 0 TSF original overburden), the settlement of the soft clay should have been completed in approximately 5 yr, with no substantial secondary consolidation expected after 9 yr. The embankment has been in place for about 15 yr (3 yr of which were under surcharge), and the pavement is still experiencing subsidence.

## ANALYSIS OF TEST RESULTS

The following analysis is presented based on results of investigations of the data and observations.

Test results show that there are pockets of soft clay under the embankment, the thicknesses of which vary from 4 to 9 ft at sites 1 and 2 and from 1 to 5 ft at site 3. A plot of the thickness contours of the soft clay layers at sites 1 and 2 shows that, in general, pockets of the soft clays are only 1–3 ft thick under the roadway sections and deeper only at very isolated locations, generally away from traveled lanes. Yet an investigation of the site 1 eastbound roadway showed signs of extensive maintenance patching of subsidence areas and of new and continuing subsidence throughout the entire test site (including areas containing no soft clay pockets).

Tests at site 3 also showed the presence of 1–3 ft of soft material under the roadway, with a few isolated pockets of deeper soft clay layers. It is significant to emphasize and recognize that site 3 has not, thus far, experienced damaging subsidence.

Considering the above evidence and the results of consolidation tests and analysis, along with the fact that the original embankment was surcharged following its placement, it was concluded that the soft clay was not the cause of continuing subsidence.

Liquefaction as a possible cause for the failure was suggested. A review of test results shows that saturated loose sands are at an elevation of about 8 ft below the surface of pavement, under an overburden pressure of about 1,000 lbs/sq ft. It is highly unlikely that shock waves from traffic loading could develop significant enough stresses in the pore water to overcome such an overburden pressure and develop a quick condition. Thus, liquefaction has been ruled out as a cause of continuing failures.

A review of the density, SPT, and ECPT tests at the median shows that the density of the sand varies from one test site to the other, indicating that deposition or placement rather than controlled compaction at these locations has affected the density and that the sand was not densified originally in a uniform manner. Construction information available to these authors shows that compaction or densification of the fill was not required by the specifications. While a review of the centerline of the median shows the densification is in a narrower and controlled band, the scatter of the ECPT data from test line 5 (and lines 2 and 8) indicates lack of controlled densification. The similarity of tip resistance values at sites 1, 2, and 3 leads one to conclude that the median is representative of as-placed density of the sand embankment. It is also concluded that the presence of moisture at the median has contributed to the tip resistances near the surface by developing capillarity-induced pseudo-cohesion.

A review of the ECPT results of sites 1 and 2 shows that the sand embankment under traveled lanes is denser up to a depth of about 5 ft than at other locations across the roadways. It also shows that all densities at sites 1 and 2 drop substantially at about 6 ft below the top of the embankment, at an elevation coinciding with that of the water table. The sand at site 3 appears to have been densified originally to a greater depth and to a higher degree. However, densities under traveled lanes at site 3 are also higher.

Interviews of LDOTD personnel indicate that the embankment represented by sites 1 and 2 was constructed by pumping sand into an excavated ditch filled with ground water, while the construction of the section of the road represented by site 3 was achieved by placing the pumped sand inside a construction levee system that was kept partially pumped. The placement of the fine sand slurry under water in the case of sites 1 and 2 did not allow the dissipation of pore pressures, which would have allowed the sand to settle to a denser state, nor did it allow the capillary forces resulting from the seepage of water out of the pores to assist in further densification.

Data which would clearly identify detailed construction procedures used on sites 1, 2 and 3 were not available to these investigators. Therefore, the reasons for differences in the quality of the embankments among sites 1 and 2 and site 3 are based on the interpretations of test and interview results only. It is clear, however, that the embankment represented by site 3 was densified during the construction to a more uniform, higher density and to a greater depth than that at sites 1 and 2.

## CONCLUSIONS

The variations of densities under the traveled lanes and at locations away from the traveled lanes in the top layers of the embankment represented by sites 1 and 2 lead to the conclusion that the sand, as placed in this embankment, initially had a relatively low density throughout the layer above the water table, and additional densification has taken place under the wheel loadings.

Considering the polished and rounded shape of Mississippi River sand particles and the evidence that adequate densification was not obtained during placement, it is concluded that sand under travel lanes densified gradually under heavy traffic loads and impacts, causing serious pavement distress.

Because of the higher degree and greater depth of original densification of the sand, the section of the road (site 3) on the east has not experienced similar subsidence and distress. However, lower layers of the sand in that embankment are also loose.

The electronic cone penetrometer proved to be an excellent, indispensable, and economic tool to identify the condition of the embankment and to reconstruct the physical events leading to the distress of this embankment.

Evidence shows that originally the hydraulic fill was placed partially under water. This procedure did not allow the submerged portion of the sand embankment to densify (below 6 ft) into a stable mass. The relatively loose sand under travel lanes above the water table densified under traffic loading, causing displacement of sand particles and thus a reduction of volume associated with densification. The sand embankment has subsided differentially due to loading by traffic and is continuing to subside.

## RECOMMENDATIONS

The original plans and specifications for the placement of the embankment should have required at least partial dewatering of the excavation prior to and during the placement of the hydraulic embankment. With the polished and fine sands of the lower Mississippi River, it is particularly necessary that water be allowed to escape from the pores following placement so that pore pressures can dissipate, thus allowing relatively uniform densification to take place aided by seepage-induced capillary forces. The dewatering should have been followed with a controlled compaction procedure, preferably using vibrators.

Considering the quality of the sand deposits available in most parts of the lower Mississippi River Basin and other alluvial areas, it is recommended that such sands not be used for hydraulic embankment construction without a thorough analysis of their fundamental properties. Properties such as particle shape, surface characteristics, and gradation must be considered carefully.

Placement procedures and quality control requirements for hydraulic embankments should be considered and specified prior to design and construction. Routine quality control procedures, for example, do not apply to this type of construction.

Construction procedures should specify a means of dewatering the sand after it is placed, particularly where the water table is high.

An electric cone penetrometer should be used for quality control of this type of construction.

## REMEDIAL RECOMMENDATIONS

A number of soil improvement methods have been considered for possible remedial procedures for this project. Such methods as dynamic compaction, vibroflotation, vibroreplacement, and blasting are among the methods considered.

When dynamic compaction is applied to a strip of sand located adjacent to very soft material, the sand, under dynamic loading, may migrate out of its boundaries, infringing into the soft soils. Furthermore, densification by dynamic compaction results in change in the volume of the embankment, which requires the addition of a considerable amount of new fill material over the existing embankment. Vibroflotation and vibroreplacement will also result in reduction of volume, presenting problems similar to that caused by dynamic compaction.

Densification by blasting is a chancy process practiced by very few specialty contractors. The results are not readily predictable. If blasting works, it requires the addition of new embankment material, and it is expected that in this case it would be uneconomical.

None of the above, in situ improvement methods were considered practical for the solution of this problem.

The nature of the problem does not lend itself to a ready-made, simple recommendation. The following solutions have been considered:

- Abandon the existing roadway and build an elevated highway at an approximate cost of $56 million. This alternative is the most drastic and expensive solution; it does not appear to be necessary or feasible at this time.
- Remove the pavement and the embankment and reconstruct both for about $52 million. This alternative presents serious problems in terms of traffic safety and economic impact upon Baton Rouge and New Orleans, resulting from the partial removal from service of the main artery between these two cities.
- Rehabilitate the embankment and replace the CRCP pavement at a cost of about $14 million. This alternative appears to be the most reasonable. If it can be accomplished, disruption to traffic can be minimized, existing materials can be reused, and the cost to the state can be drastically reduced. An embankment densification or stabilization scheme such as densification by grout (lime-fly ash or portland cement) injection, installation of lime columns, or vibroflotation may be used. It is the opinion of these investigators that the rehabilitation of this embankment using densification by injection grouting can be achieved with minimum disruption to traffic and in a relatively short period of time.

At the time of the preparation of this paper, plans were underway to perform field studies on the effects of densification and/or stabilization of the sand embankment by grouting. A section of the embankment will also be tested to determine the effects of dewatering of the embankment materials.

## ACKNOWLEDGMENT

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